Christchurch City Council

LDRP 507 Stopbank Risk Assessment
Pages Rd to Bridge St

April 2021

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1. Introduction

1.1 Readers Guide

This report presents the work and findings in relatively high level of technical detail. For readers seeking an executive summary level of reporting, this has also been produced as a standalone summary report.

The key finding of this report, as presented in section 4.3, is that risks to life associated with failure within the assessed portion of the stopbanks, as they are currently designed, and within the scope of their intended function, are clearly better than guideline values for tolerable risk to life, as shown on Figure 5-1 in section 5.1. This reflects the sound design practices which have been adopted by Council.

The residual risk, associated with infrequent and extreme high sea level events that would overtop the assessed portion of the stopbanks, are presented in section 5.1. These risks exceed guideline values for tolerable risk to life. Council have not historically managed this risk and these events exceed the current design level of service.

Some readers may prefer to begin their reading with sections 1, 5.1, 8 and 9 before exploring the other sections and Appendices which have more technical detail.

1.2 Background

Post the 2011 earthquake sequence, Council variously redressed damage in different sections of the Avon stopbanks. By 2015 a philosophy was adopted to design temporary stopbanks with an intention that they would be suitable for a 20-year design life from 2015 to 2035.

The design standard for temporary stopbank crest levels was established following a staff report to the Infrastructure, Transport and Environment Committee (05 May 2016). In section 5.18 of that report set the design crest level at the 100 year ARI flood level plus 300mm freeboard. Following revision of the tidal statistics by Goring in 2018, the previous design crest level was re-evaluated as being approximately equivalent to the higher of 100 year ARI with zero freeboard and 50 year ARI with 200mm freeboard when using the new sea level statistics.

ARI is short for Annual Recurrence Interval which means how many years (on average) is expected between recurring events. A 100 year ARI event means an event that would be expected to recur such that on average there is a 100 year interval between successive events. This concept is closely linked to AEP which is short of Annual Exceedance Probability. AEP means the probability of an event happening in any particular annual period (year). For example a 100 year ARI event can equivalently be described as a 1% (or 1/100) AEP event. This report uses both terms in different applications according to international practice.

GHD completed an ANCOLD[1] based stopbanks risk assessment in 2016 along the full length of the stopbanks. Findings at the time pointed out that risks to life exceeded ANCOLD guidelines for circumstances where the stopbank top level would be overtopped by an over-design event.

Two GHD reports were produced. “Temporary Stopbank Management Options” Webb and Dasler Rev 2, Sep 2016 and “Stopbank Levees Risk Assessment” Barker and Williams, Rev 0, Sep 2016.

Council asked for further analyses of the risks other than overtopping. These highlighted opportunities for improvement other than raising the crest level, many of which have since been implemented. In response to the identified risk in the 2016 work, there has been significant

[1] Australian National Committee of Large Dams

investment and improvements in the temporary stopbanks, including repairs, rebuild, tree removals and some raising of crest levels.

In 2017/18 several high sea level events prompted a revised analysis of extreme sea level risks. This was completed in 2018 by Derek Goring and resulted in substantial increases in estimated extreme sea levels at Bridge St of circa 150-200 mm for an ARI of 10-100 yrs. This change significantly increased the estimated risks of stopbank overtopping. This increased risk occurs most in the downstream reaches of the river. The stopbanks were still providing significant benefits reducing flood risks to many residences; however, the level of service was reduced by this new assessment.

On 29 August 2019, Council considered a staff report on earthquake issues in Southshore and South New Brighton. At that meeting Council resolved for staff to undertake an update of previous investigations into the safety risk of flooding from a breach.

Accordingly, GHD have been instructed to revisit the ANCOLD risk assessment process in the most downstream portion of the river, from Pages Road to Bridge Street, most affected by the increased design tide levels.

1.3 ANCOLD introduction

The ANCOLD guidelines used to guide this assessment, rely on professional risk assessments for individual and societal risks to life, and provide guidelines as to what level of risk is considered tolerable. While the guidelines were developed initially for dams, stopbanks have a similar function and under the guidelines stopbanks are considered to be a type of dam.

The guideline tolerable limit for individual risk is an annual frequency of 1 in 10,000. Individual risk means the additional risk imposed by any floodwater which is not retained by the stopbank on any individual in the community. This is assessed within typical groups in the community but not on a specific individual or household level.

For societal risks, there is a recognition that events that would cause a single or small number of deaths are tolerable with a higher frequency than events that would cause a large number of deaths. For example, in the guidelines an event that would be expected to cause 100 deaths has a tolerable annual frequency of 1 in 100,000. This tolerable limit is shown on all of the societal risk plots presented in this report as the purple line (bi-linear).

ANCOLD societal and individual risk is assessed as incremental risk against a background risk context associated with a scenario that the stopbanks always retains all water (ie: an idealised stopbank with infinite height and strength).

While compliance with the guidelines is not mandatory in NZ (and risk assessments against the guidelines are not mandatory either) it is common practice in Australia for territorial authorities to aim for compliance with the guidelines for dams (and stopbanks) within their jurisdiction. Council have not adopted these guidelines in any formalised manner.

The above tolerability limits apply to existing stopbanks (meaning stopbanks that have existed for some historic period as distinct from new built stopbanks).

1.4 Scope and Purpose

The primary purpose of the work described in this report is to revisit and update (reflecting new circumstances, in particular the revised tidal statistics) parts of the Avon River Stopbank risk assessment done in 2015 by GHD. Within that general objective the scope is specifically limited in the following ways:

- The updated risk analysis work is limited to a downstream river reach from Pages Road to Bridge Street (approx. 2km in length)

- The revisions excluded consideration of the geotechnical improvements achieved through remedial works in 2018/19 (as preliminary evaluation indicated that while some risks would be reduced by these remedial improvements these were minor in relation to the primary risks from overtopping)
- Only the design stopbank levels were considered for simplicity. Assessing the additional detail of “as built” stopbank levels would provide little benefit as there are typically only minor variations from the design levels and these will change in time with settlement and intermittent topping up activities
- The assessment was also limited to current day conditions and does not look forward to expected slow change future conditions such as risks associated with expected higher future sea levels and with continuing land subsidence under the stopbanks. Accordingly future development is not considered in the assessment.
- The assessment excludes risks associated with wave action and with potential future tectonic movement (liquefaction and subsidence movement and damage are considered but not the more profound phenomenon of wide scale tectonic plate uplift or downthrust).
- The effects of seismically induced deformation, which encompasses settlement combined with lateral spread was a particular challenge for in this study because there are no accepted methods for reasonably estimating deformations from lateral spread. A pragmatic methodology was adopted, as described in section 3.2.3.

The process for the risk assessment has been completed in accordance with ANCOLD 2003 Risk Guidelines for Dams, which also complies with the requirements of ISO 31000-2018 (Risk Management Guidelines).

1.5 Geographic limits

While the Council instruction was to focus on the area from Pages Road to Bridge St, the study area was extended upstream to midway between Pages Road and Wainoni Road which contains a natural low lying depression area which would ‘flood first’ in the event of a stopbank failure between Pages Road and Bridge St. This was necessary for technical merit to mitigate impacts on findings which would have occurred if the requested study area boundary had been adopted. The extended study area for stopbank risks and ANCOLD risk assessment is illustrated in Figure 1-1 below.

Figure 1-1 Avon River Stopbank risk assessmentstudy area geographic limits

1.6 Limitations

This report has been prepared by GHD for Christchurch City Council and may only be used and relied on by Christchurch City Council for the purpose agreed between GHD and the Christchurch City Council as set out in section 1.4 of this report.

GHD otherwise disclaims responsibility to any person other than Christchurch City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in the various sections of this report (with key assumptions summarised in section 1.7 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Christchurch City Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

1.7 Key Assumptions

Assumptions in this work are described throughout respective sections of the report. Some of the key assumptions include:

1. The geographic limitations are clearly reasonable by inspection of topography for the lower flood levels. At higher flood levels it is possible for areas within the study area to be impacted by stopbank failure outside of the study area. It is also possible for stopbank failure inside the study area to impact areas outside this study area. Both risks are assumed to be negligible in order to practically enable this study.
2. Flooding from stopbank failure (breach or overtopping) is assumed to result in flood levels outside the stopbanks equalling the peak tide level (bathtub flooding). This neglects flow constraints in the upper estuary, along the river channel over the stopbanks or through a stopbank breach. Some work to test this assumption with respect to flow constraints over the stopbanks was completed and described in Appendix A.
3. The use of Jonkman 2008[2] life risk model, with life risks limited to flooding > 0.3m above floor levels, is appropriate for the slow moving relatively shallow depths of inundation experienced with overtopping or failure of the stopbank
4. Evacuation during the day can be achieved with 40% of the population at risk evacuated while at night there is no evacuation. Sensitivity analyses have been completed to evaluate the potential for higher evacuation rates applicable to a well-defined and applied Emergency Action Plan.
5. The stopbank levels are uniform along the reaches with no low-lying sections likely to overtop or breach prematurely, notwithstanding piping failure modes that can occur at any tidal level above the stopbank base level.
6. Council preparedness to reinstate stopbanks after an earthquake damage event will ensure that normal function is restored within two months after the earthquake damage (See Section 3.2.3).

   Based on our experience from the 2011 seismic event, we estimate the length of time for the recovery of the stopbanks to operational crest level after a large seismic event with multiple sections of the stopbanks affected as likely to be between 6 to 8 weeks. This provides time to mobilise or divert a contracting team from key recovery work elsewhere in the city and the limitations on transport connections through the city for fully laden truck and trailer units bringing material to the site. Selection and supply of suitable material from the local quarries will also play a part in timing, however, this lesson has generally been learnt.
7. In accordance with section 1.4 we have made no allowance for wave setup, wave overtopping or risks from wave margin erosion. Accordingly, the assessed risks associated with section 1 will be slightly underestimated but there would be negligible impact on the total assessed risks. It will remain prudent for Council to allow for a slightly greater crest level in the area at risk from wave overtopping, above our recommendations for general stopbank levels in the absence of wave risks.

   We understand that there can be significant wave action during high tide levels in the broad lower reach of river upstream of Bridge Street, with potential fetch distances of up to 1 km with a southwest or northeast wind direction. We also understand that this was

[2] Jonkman S.N, Vrijling J. K, Vrouwenvelder A. C. W. M., Article in Natural Hazards · September 2008 DOI: 10.1007/s11069-008-9227-5 "Methods for the estimation of loss of life due to floods: a literature review and a proposal for a new method"

the reason Council increased the stopbank top level to 11.4 m along the left bank stopbank as shown in Figure 3-1.

8. This report is based on a stopbank situation where crest levels are at their design level and that the condition of the stopbanks has not deteriorated since our 2015 assessment. Recent survey and field inspection has reported that the stopbanks do not presently meet these conditions. The findings of this risk assessment will be applicable after currently planned short-term maintenance work is completed.

1.8 Reliance on Information Provided

Information provided by Council and relied on for this work includes:

- A collection of asset data, LiDAR ground levels and flood observational data used to form the Avon hydraulic model
- Goring 2018 extreme sea level data[3] and by inference the Council historic measured level data used to develop those values
- Council's floor level database
- Census data from 2013 (people per household)

[3] Extreme Sea Levels at Christchurch Sites: EV1 Analysis, Mulgor Consulting Ltd, 24-Jul-2018

2. Risk Assessment Methodology

The Risk Assessment approach, as shown on Figure 2-1 was similar to the 2015 methodology with the exception that only the tides and seismic events were evaluated (not river flooding) and, as stated in Section 1.2, the site selection was limited to the downstream area of the stopbank from near Pages Road to Bridge St. Furthermore, some of the failure modes previously evaluated were dismissed as being minor contributions to the risk owing to the recent upgrade work.

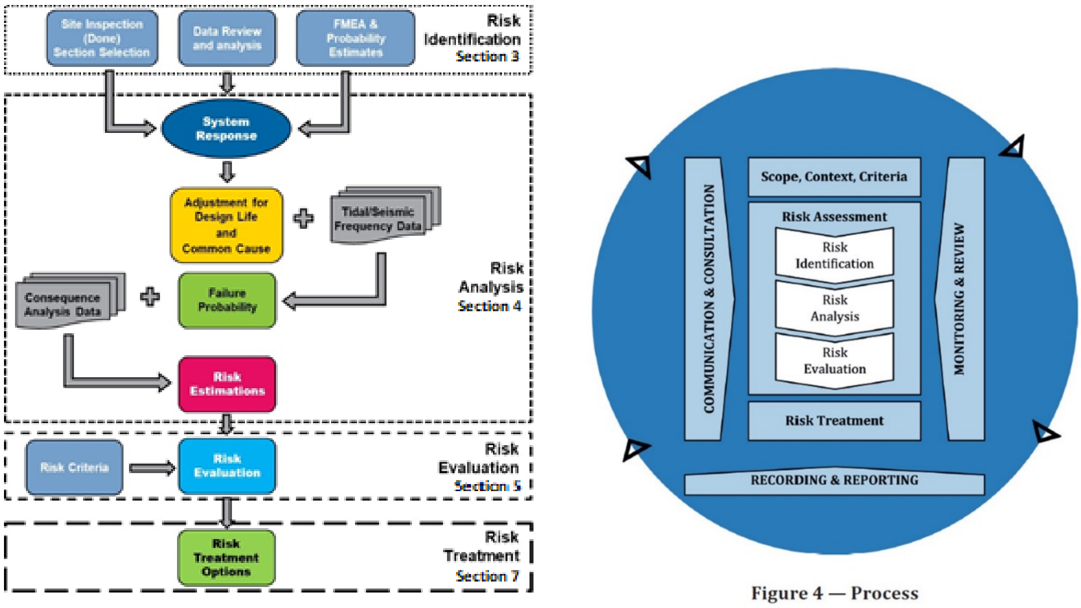


Figure 2-1 Avon Stopbank risk assessment process (left) and ISO 31000-2018 process (Risk Management Guidelines) (right)

The risk assessment was competed using the following process which can be compared directly with the ISO31000-2018 process shown on Figure 2-1.

- Discussions with Council to confirm the scope and identify the sections to be analysed in the risk assessment for the lower stopbank area
- Assess the possible failure modes for each section considered in the risk assessment
- Quantify the seismic and tidal loading conditions
- Develop event trees for each failure mode
- Determine the probability of each event in the event trees using the piping toolbox and various other available tools from which to assess the probability of stopbank failure for each section
- Make adjustments for the failure probabilities to account for the common cause failure resulting from the seismic or tidal events
- Estimate the population at risk (PAR) and potential life loss (PLL) in the event of a breach or overtopping for each section with consideration of tidal events with or without seismic events
- Calculate the life risk as the product of the annual probability of failure and the PLL for the current temporary stopbank levees for the tide and seismic loading
- Evaluate the risk, based on current ANCOLD risk guidelines, and
- Evaluate options for risk reduction.

3. Risk Identification

3.1 Analysis Area

The analysis was limited to the area of interest in the lower Avon River, as shown on Figure 3-1, which included the cross sections shown on Table 3-1 taken from the previous 2015 ANCOLD risk assessment.

Figure 3-1 Avon Stopbank risk assessment area of interest

Table 3-1 shows the crest level and length for each Stopbank section, and the number of analysis sections used for evaluating the likelihood of stopbank failure occurring at more than one location along the length of each stopbank section. The number of analysis sections was evaluated assuming an equivalent length of 500 m for analysis of the stopbank failure modes.

Table 3-1 Avon Stopbank lower reach analysis cross sections

Bank	Section Number	Crest Level (m)	Length (m)	Number of Analysis Sections (n)
Left	1	11.4	1418	3
Left	2	11.2	656	1
Left	3	11.2	217	1
Left	4	11.2	518	1
Right	17	11.2	901	2
Right	18	11.2	1862	4

The number of analysis sections was used to evaluate the likelihood of failure for each stopbank section using the following formula:

$Pf_{Stopbank\ section} = 1 - (1-Pf_{500m\ section})n$

Where:

$Pf_{Stopbank\ section}$ is the probability of failure for the stopbank length

$Pf_{500m\ section}$ is the probability of failure calculated for each failure mode, and

n is the number of analysis sections shown on Table 3-1.

3.2 Hazard Analysis

3.2.1 Hazard Screening and Shortlisting

There are a number of hazards that could affect the operation and potential failure of the Stopbanks, as shown on Table 3-2. These have been screened for inclusion in the risk analysis using the criteria shown below the table. Hazards shown highlighted in green have been included in the risk analysis.

Table 3-2 Avon Stopbank hazard analysis

Hazard/Failure Initiating Events	Screening Criteria	Comments
Aircraft Impact	5	No major flight paths directly over Stopbank
Avalanche	6	None in area
Chemical Reaction	6	No carbonates identified
Earthquake		Stopbank settlement, cracking and slope failure
		Lateral spreading
	7	Tectonic Seismic movement (Refer Section 1.1)
	1	Seiche loading
Fire	5	No affect on the Stopbank
Hail	5	No affect on the Stopbank
Human Error	2	Lack of maintenance to repair issues (Rabbit burrows, ant workings, vandalism etc) or people interfering with the Stopbank
Hydrological / Flood	2	Site specific, not included in present study
Ice	6	No ice at this location
Lightning	6	Affects electrical components and noe present that will affect the risk
Tidal level fluctuation		Tidal level causes piping or overtopping failure
Temperature	5	No affect on the stopbank
Material Deterioration	5	Embankment and foundation material unlikely to have deterioration in long term of significance to the risk (ie strain softening or creek not significant)
Meteor Strike	2	
Pore pressures	2	Increased pore pressures resulting from tidal variation included in stability analyses and show very small likelihood of causing failure
Rabbits	5	Burrows reduce the seepage pathway for piping analysis asnd increases probability
Reservoir Level Fluctuations	6	Not applicable to the Stopbank but tidal levels ar included
Reservoir Rim Slope Failure	6	No reservoir rim
Terrorism	2	Unlikely to have terror attacks on the Stopbank
Termites	6	Termites unlikely to penetrate to the core zone
Toxic Gas	3	No effect on dam
Transportation Accident	6	No transport on the dam
Trees		Trees on the Stopbank sides and roots included in analysis
Vandalism	2	Vandalism unlikely on the Stopbank
Volcanic Activity	2	None in area
Wind	4	Wind action leading to trees being uprooted has been included with Trees root piping analysis
	7	Wave action resulting from wind overtopping and failing the stopbank during high tides not included. (Refer Section 1.1 and Section 1.5)

Screening Criteria		
	1	The event is of equal or lesser damage potential that the events for which the Stopbank is designed. Design Significantly exceeds requirement.
	2	The event has a significantly lower mean frequency of occurrence than other events with similar uncertainties and could not result in worse consequences than those events
	3	The event cannot occur close enough to the Stopbank to affect it.
	4	The event is included in the definition of other event(s)
	5	The event is judged to have an insignificant effect on the Stopbank
	6	Not an initiator
	7	Excluded from current study

Based on the hazard analysis, the hazards included in the present study were tides (high sea levels) and seismic events. The potential for trees blowing over or for roots allowing piping to initiate have been included in the foundation piping failure mode.

3.2.2 Tidal Hazards

Tidal hazards in the study area are associated with extreme high-water levels in the estuary at Bridge Street. Extreme tide levels are predominantly caused by combinations of high astronomical tides, barometric uplift (low pressure) and south to south-westerly wind setup across the estuary. Risks of specific high-water levels also increase over time as sea levels rise.

In this analysis we use the conclusions for Bridge Street derived by Derek Goring in 2018 and subsequently adopted by Council as their official guidance. Further work by GHD and HKV[4] to improve these findings is nearing completion and if adopted by Council would replace the Goring levels. Accordingly, some sensitivity testing has been completed to assess any impact from the different tidal hazard risks.

The tidal data obtained from Goring are shown on Figure 3-2 which also shows the tidal levels used for evaluation of the stopbank risk. These levels have been selected to ensure that the tidal events adequately capture the levels surrounding the potential overtopping failures.

The risks in this assessment have been grouped into eight AEP[5] ranges (groupings) as shown on Figure 4-9. We consider these groupings provide adequate resolution to reasonably represent the risks assessed.

The Goring tidal data has been extrapolated to the level 11.8 m to estimate the low frequency of this event. Typical groupings use the average of risk to life from the endpoints of their range. This introduces some conservatism compared to if a more highly resolved calculation was carried out, but the difference is negligible in relation to these findings. The >10,000-yr grouping uses risks from the 10,000-yr calculation and the 50,000-yr calculation (11.817 m) as shown Figure 3-2, for Goring.

Prior to Goring's tidal data update in 2018, the best available tidal risk data was Goring's similar work produced in 2011. For the 200-yr ARI, the tide level increased from 10.958 m to 11.214 m in 2018, an increase of 0.256 m. Of similar importance the tide level difference between ARI's with a ratio of ten (i.e., one order of magnitude difference) increased from 0.090 m to 0.251 m. This later change greatly increases the predicted frequency of overtopping events with higher sea levels such as 11.4 m.

4 Joint Risks of Pluvial and Tidal Flooding Report, Rev 0, February 2021, GHD and HKV
5 Refer to section 1.2 for a definition of AEP (and the related term ARI)

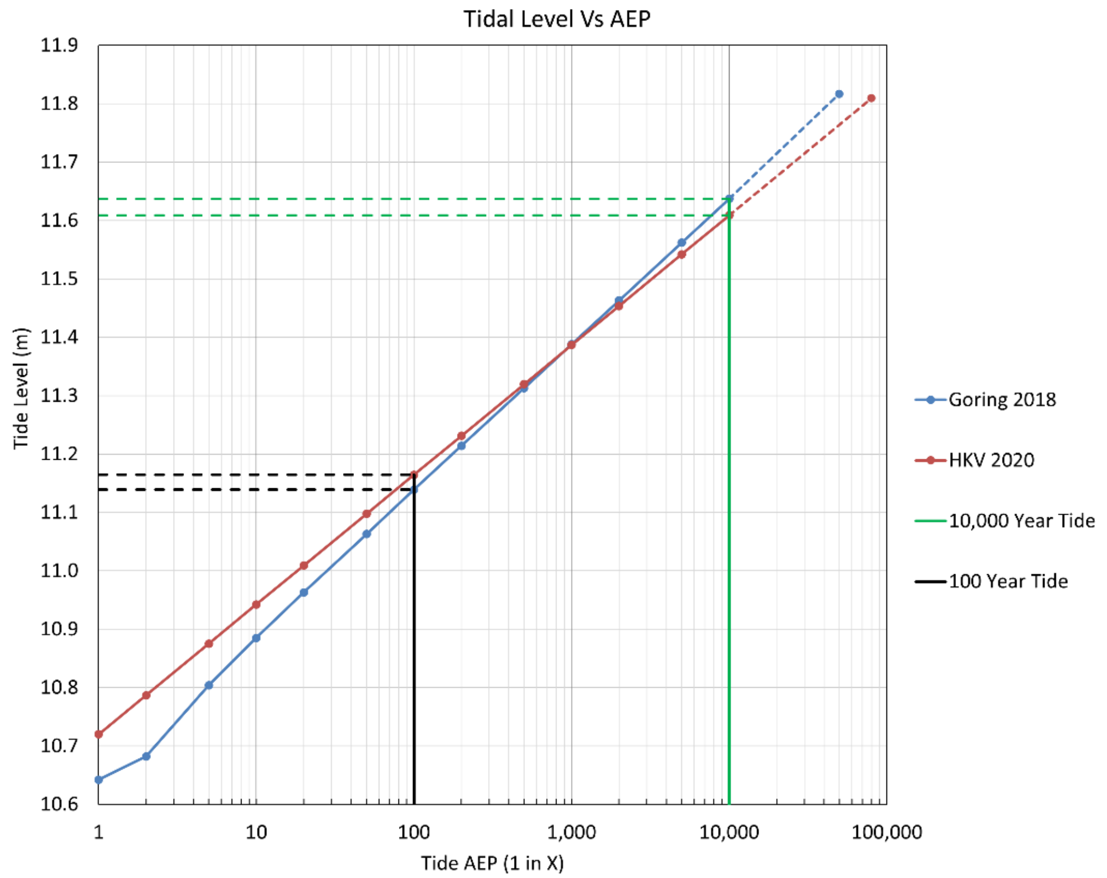


Figure 3-2 Avon Stopbank tidal level versus AEP

It is clear from Figure 3-2 that the HKV tidal levels are higher for the majority of the events up to the 1 in 1000 AEP event and then marginally lower for events up to the 1 in 10,000 AEP tide. The effect of this is the calculated risk for the HKV data being higher than the risk calculated using the Goring data.

As noted on Table 3-2, no allowance has been made for wave setup and wave overtopping nor risks from wave margin erosion. Accordingly, the risks associated with Section 1 will be slightly underestimated but there would be negligible impact on the total assessed risks. It will remain prudent for Council to allow for a slightly greater crest level in the area at risk from wave overtopping, above our recommendations for general stopbank levels in the absence of wave risks.

We understand that there can be significant wave action during high tide levels in the broad lower reach of river upstream of Bridge St, with potential fetch distances of up to 1 km with a southwest or northeast wind direction. We also understand that this was the reason Council increased the stopbank top level to 11.4 m along the left bank stopbank as shown in Figure 3-1.

3.2.3 Seismic Events

Seismic loading for the risk assessment was adopted from the 2015 risk analysis, as shown in Table 3-3 below. There has been no update to Table 3-3 data since the 2008 version quoted below. Peak ground acceleration (pga) values for a spectral acceleration of 0 seconds were adopted for the seismic loading considered in the risk assessment following standard geotechnical practice (as ground rarely exhibits any dynamic elastic response). This data is shown on Table 3-4 and Figure 3-3 below and was used to estimate stopbank crest settlement for each of the seismic events. As discussed in Section 4.1, the settlement data was evaluated using the pga values rather than response spectral data for each Stopbank section.

It has been assumed that, in the event of earthquake damage to Stopbanks, the Council preparedness would enable repairs to be made within two months after the damage. The likelihood of a seismic event and a tidal event occurring within a two month period before repairs could be made to the stopbank was calculated using the following formula (Ang and Tang 1975) and is shown in Table 3-4.

$$Pt = 1 - e^{-Pa.t}$$

Where

Pt= probability of failure over t years

Pa=probability of failure per annum, and

e=Euler number (base of natural logarithms).

Table 3-3 Christchurch PGA vs return period - adopted from Stirling et al (2008)

Table 1. **Location-specific PGA (Period T = 0.0 sec), and response spectral acceleration (T = 0.075 to 3.0 sec) and MMI (last row) for various return periods (see column 1), and for class C (shallow soil) site conditions. The centres are listed in alphabetical order.**

Christchurch
Latitude 43.53S Longitude 172.64E

T(s)	0.00	0.075	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.50	0.75	1.00	1.50	2.00	3.00
20 yrs	0.07	0.10	0.12	0.14	0.17	0.17	0.17	0.16	0.16	0.13	0.09	0.05	0.04	0.03	0.01
50 yrs	0.11	0.18	0.21	0.26	0.31	0.30	0.29	0.27	0.26	0.22	0.14	0.09	0.07	0.05	0.03
75 yrs	0.14	0.23	0.27	0.32	0.37	0.36	0.34	0.32	0.30	0.25	0.16	0.11	0.08	0.06	0.04
200 yrs	0.22	0.41	0.49	0.55	0.62	0.56	0.52	0.48	0.44	0.37	0.24	0.16	0.12	0.09	0.06
475 yrs	0.31	0.61	0.75	0.82	0.89	0.78	0.71	0.64	0.58	0.48	0.31	0.21	0.15	0.12	0.09
1,000 yrs	0.40	0.83	1.02	1.09	1.17	1.00	0.89	0.79	0.72	0.59	0.38	0.25	0.19	0.14	0.11
2,000 yrs	0.50	1.08	1.34	1.40	1.49	1.25	1.09	0.96	0.86	0.71	0.45	0.31	0.23	0.17	0.14
5,000 yrs	0.64	1.45	1.80	1.85	1.95	1.61	1.37	1.20	1.08	0.88	0.55	0.38	0.29	0.22	0.18
10,000 yrs	0.77	1.76	2.20	2.24	2.34	1.91	1.61	1.40	1.24	1.02	0.63	0.43	0.33	0.25	0.21
20,000 yrs	0.90	2.12	2.65	2.67	2.76	2.23	1.86	1.61	1.43	1.17	0.71	0.48	0.38	0.29	0.24

MMI 50 yrs = 6-7; 150 yrs = 7-8; 475 yrs = 7-8; 1,000 yrs = 8-9

Table 3-4 Avon Stopbanks risk assessment seismic hazard data

Code	Return period (years)	Annual Exceedance Probability (AEP)	Probability of earthquake and repairs not done within two months	Probability Interval per year	Peak ground acceleration (g)
EQ1	20	5.00E-02	8.30E-03	4.97E-03	0.07
EQ2	50	2.00E-02	3.33E-03	1.11E-03	0.11
EQ3	75	1.33E-02	2.22E-03	1.39E-03	0.14
EQ4	200	5.00E-03	8.33E-04	4.82E-04	0.22
EQ5	475	2.11E-03	3.51E-04	1.84E-04	0.31
EQ6	1,000	1.00E-03	1.67E-04	8.33E-05	0.40
EQ7	2,000	5.00E-04	8.33E-05	5.00E-05	0.50
EQ8	5,000	2.00E-04	3.33E-05	1.67E-05	0.64
EQ9	10,000	1.00E-04	1.67E-05	8.33E-06	0.77
EQ10	20,000	5.00E-05	8.33E-06	8.33E-06	0.90

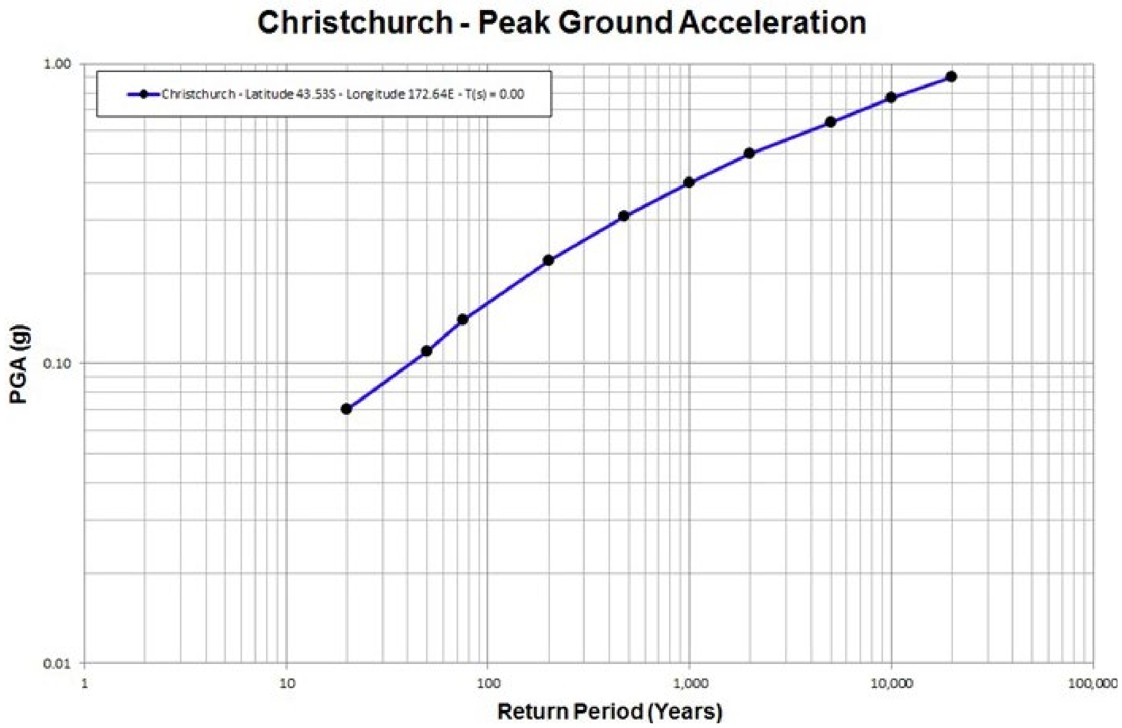


Figure 3-3 Christchurch PGA vs return period for T = 0s

The theory of liquefaction induced settlement assumes that all settlement is in the vertical direction. In reality, for the Avon River Corridor, it is known that liquefaction triggers both settlement and lateral spread. For the stopbanks this results in a combination of loss of crest height and deformation of the land towards the river. This deformation is expressed as large cracks.

Loss of crest height from lateral spread deformation is not considered in the vertical settlement modelling methodology. However, a combination of known factors of safety and comparison of modelling results with actual loss of crest heights from the 2011 earthquakes, provided the basis to consider that the vertical settlement modelling gives a reasonable estimate of the total loss of crest height (from settlement and lateral spread).

Accordingly, the loss of crest height has been modelled using propriety software and the modelled pga from Table 3-4 above. The quantum of lateral spread cracking damage has been estimated using the mapped crack data from the Canterbury Earthquake Sequence available on the NZ Geotechnical Database. In combination, the settlement and cracking estimates reasonably quantify the risk to the stopbanks from seismically induced liquefaction damage.

3.3 Failure Modes

The following five failure modes have been evaluated in the study. Note the abbreviation in brackets refers to code used for calculation and tabulated presentation of the results.

- Overtopping resulting from any one of the following initiating events
  - Seismic deformation loss of freeboard and overtopping (Earthquake Overtopping, EQ Otop)
  - Tides overtopping the stopbank (Embankment Overtopping, Emb Otop)
- Piping resulting from any one of the following initiating events:
  - Seismic cracking (Piping Earthquake, Piping EQ)
  - Cracks in embankment due to differential movement (Embankment Piping, Emb Pipe), and

- Through the sand foundation, including tree roots with consideration of narrowed section caused by trees blowing over (Foundation Piping, Fnd Pipe).

The following additional failure modes were identified for the risk analysis in 2015 but have been removed for the present study:

- Slope instability was evaluated and found to be significantly lower likelihood than the other failure modes and was dismissed for further analysis
- The sandbags present at Section 2 during the 2015 evaluation have since been replaced as part of the 2015 strengthening project and this failure mode is therefore not applicable for the revised study, and
- River floods have not been included in the present analysis, only tides. This was because in the section being studied the river levels are almost entirely driven by tide level and any river flow including flood flows make negligible difference to event water levels.

4. Risk Analysis

4.1 System Response

The system response of the stopbank, for each failure mode showing the likelihood of the piping failure versus the load causing failure was evaluated using the input data obtained from the 2015 risk analysis for which the relevant sections are shown in Appendix C. The input data for system response included factors affecting the strength and resilience of the stopbanks such as cross section shape, dry-side ground level, geotechnical characteristics and trees.

The above input data was not updated from 2015, despite the upgrade project removal of some trees and root balls because it was obvious that this change would have negligible effect on the main findings from this risk assessment. However, the stopbanks top level, tide level probabilities, population at risk and potential loss of life inputs and assumptions were revised in the current study.

Based on the 2015 analysis results, the following failure modes were found to be the most significant contributors:

- Overtopping from tides alone
- Overtopping following a seismic event, and
- Foundation piping.[6]

The system response curve for overtopping is shown on Figure 4-1. This illustrates how the probability of erosional failure increases with greater flow depth over the stopbank crest, assessed as being certain to fail for overtopping depths in excess of 0.5m.

[6] Foundation piping refers to piping beneath (not through) the stopbanks. This includes failure below stopbanks which are built on natural ground without foundations.

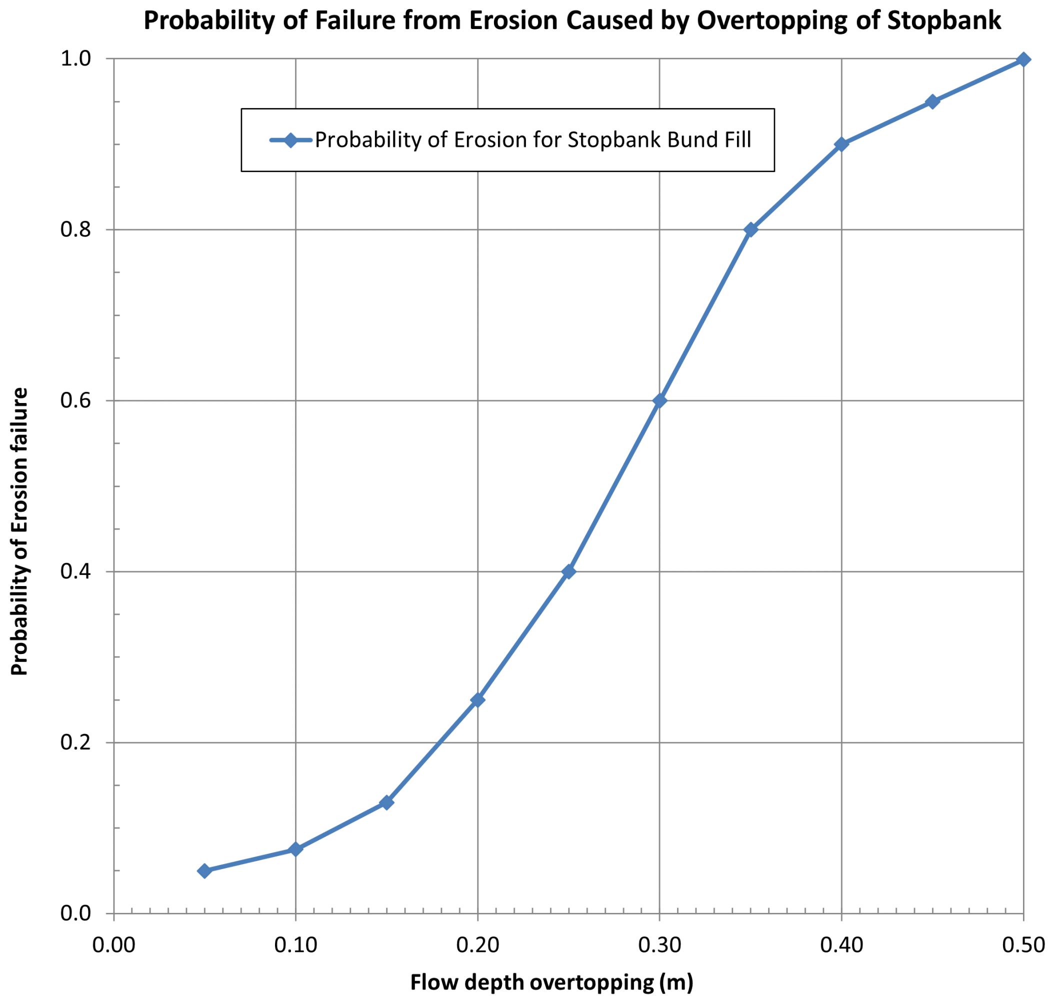


Figure 4-1 Stopbank failure probabilities versus overtopping depth

Figure 4-2 shows the relationship that was developed between the probability of failure and the ratio of the cross-sectional water head (river level above the stopbank foundation level “Head”) as a fraction of the critical head “Hc” calculated as being required to cause backward erosion (piping failure) in the foundation. The data was evaluated for various bund heights, crest widths and side slopes using a side slope of 1H:1V with the crest width of 1.0 m and 1.5H:1V for the crest width of 2 m. This relationship was used to evaluate the system response curves shown on Figure 4-3.

The tree foundation piping failure mode was evaluated using a similar process but with a longer seepage length, assuming that the tree roots will need to be connected to the upstream side of the stopbank and will provide a pathway for backward erosion to initiate.

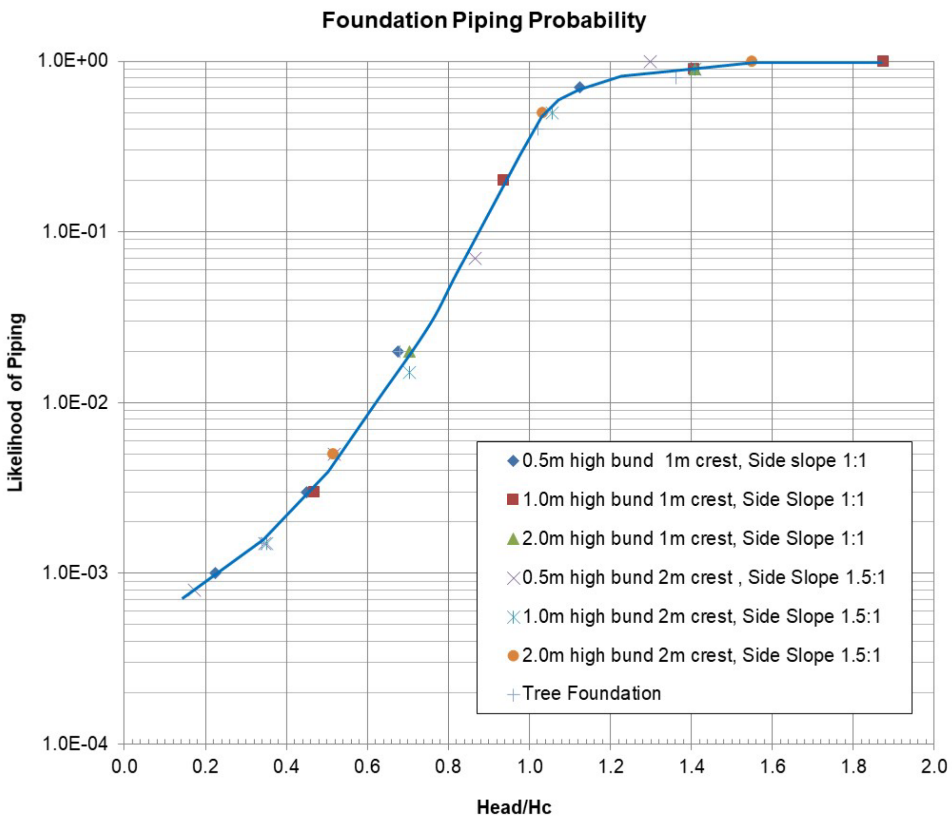


Figure 4-2 Estimated Probability of Foundation Piping versus ratio of head above foundation level to critical head

Curves for overtopping and foundation piping are shown on Figure 4-1 and Figure 4-3

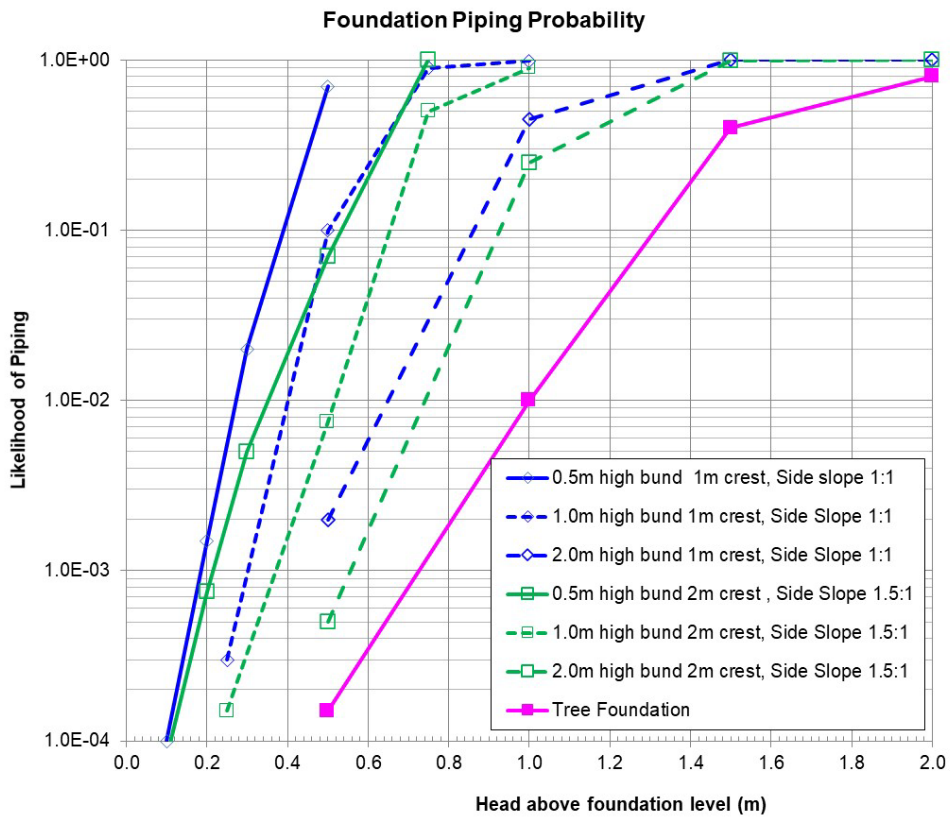


Figure 4-3 Estimated probability of foundation piping versus head above foundation level for various stopbank configurations

Seismic deformation analyses were completed in 2015 for each Stopbank section to assess the likely settlement resulting from a seismic event by extrapolating the settlement data from historical seismic data versus the pga values, as shown on Figure 4-4 and Table 4-1 taken directly from GHD report “Stopbank Levees Risk Assessment” Barker and Williams, Rev 0, Sep 2016.

The settlement data was used to evaluate the probability of the tidal level overtopping the stopbank within a two-month period following a seismic event, assuming that it would take 2 months before repairs could be completed to restore the original crest level.

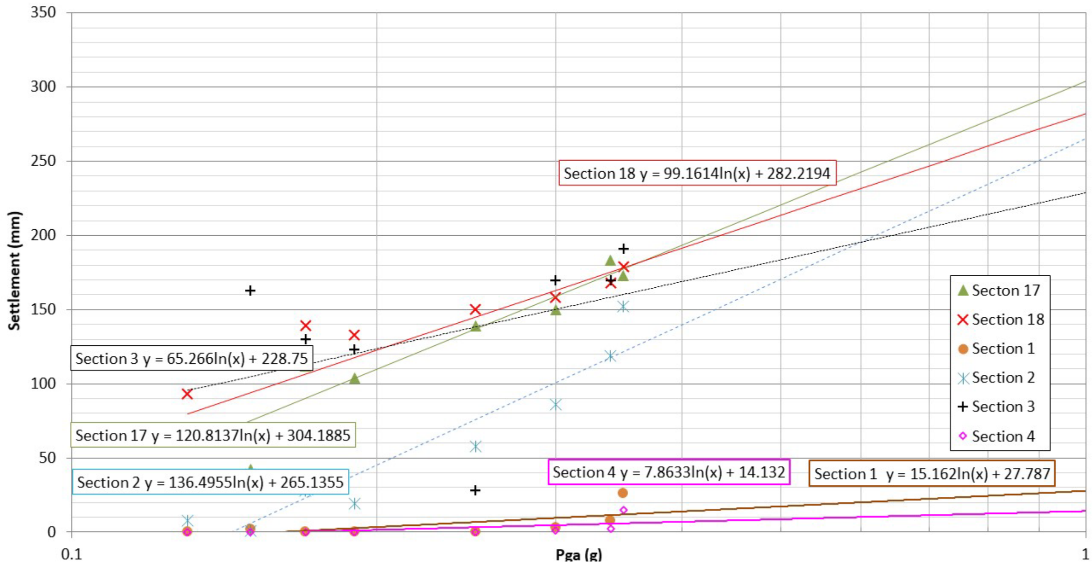


Figure 4-4 Stopbank settlement versus peak ground acceleration

Table 4-1 Estimated stopbank settlement from extrapolated historical seismic data (from GHD 2016)

Return period	PGA (g assumed) (mm)	Section 17 (mm)	Section 18 (mm)	Section 1 (mm)	Section 2 (mm)	Section 3 (mm)	Section 4 (mm)
20	0.07	0	19	0	0	55	0
50	0.11	38	63	0	0	85	0
75	0.14	67	87	0	0	100	0
200	0.22	121	132	5	58	130	2
475	0.31	163	166	10	105	152	5
1,000	0.40	193	191	14	140	169	7
2,000	0.50	220	213	17	171	184	9
5,000	0.64	250	238	21	204	200	11
10,000	0.77	273	256	24	229	212	12
20,000	0.90	291	272	26	251	222	13

4.2 Consequence Analysis

4.2.1 Population at Risk (PAR)

An early part of the ANCOLD risk assessment process is to identify geographically and quantify the number of people at risk in the event of a high river level and stopbank failure through either breach and/or overtopping and the resulting flooded community.

Since 2015 Council has developed a floor levels database which was used to improve on the prior life risk analysis. This database consists of both surveyed and estimated floor levels. The majority of the levels are estimated based on building age, where houses older than 1970 are assumed to have floor levels 400mm above ground and newer houses are assumed to have floor levels 150mm above ground. Ground levels are taken at building centroids from LiDAR survey data. From inspection we excluded records classified as “outbuildings” and those with null floor levels (typically old house outlines where house had since been demolished).

Council staff have verbally reported that they have observed a bias in the floor levels, suspected to be up to 100 mm on average, with true floor levels higher than results from the database estimation methodology. This bias has not been confirmed generally and is not specifically connected with the present study area. This potential positive bias is ignored through the main assessment in this report. The importance of such a potential bias is explored through sensitivity analysis in section 6.3. Other than in section 6.3 we have made no adjustment (applied no freeboard) to the floor levels data.

This database was cropped to the low-lying topography within the study area. Red zone houses were removed with the exceptions of 92 Bexley Rd and 9 Valsheda Street which remain occupied. This resulted in a dataset with 3180 buildings. This data was then analysed for a variety of flood levels to identify buildings at risk. Each building was assigned an expected occupancy number to evaluate the population at risk.

A key simplifying assumption for this analysis is that in the event of breach or overtopping failures that flooding level will match the predicted peak sea level. This means there is no consideration of the duration of high tide, credible flow rates and flooded volume. This assumption was tested as described in Appendix C.

From the dataset, residential buildings were initially recognised by their data classification. For all non-residential buildings we carried out desktop review mainly reliant on Google Maps, street view and zoning to identify the building type and reclassified some as being most probably residential. We similarly reviewed the lowest floor level residential buildings and reclassified some of those to non-residential.

For the purposes of analysis, it was appropriate to define day and night-time periods in order to differentiate what are typically quite different occupancy patterns as well as ability to evacuate and risks to life for the un-evacuated population. We adopted a daytime of 14 hours and night-time of 10 hours per day for both weekdays and weekend days. This night-time definition reflects the typical period in which residential population are most likely to be at home when it is dark outside and when occupants are less likely to be paying attention to news broadcasts or communication of requests for evacuation.

For the most significant (low lying and/or large) non-residential properties we carried out site specific estimates of peak and average daytime populations as tabulated below.

Table 4-2 Population assessment for larger non-residential properties

Type	Peak population	Day population	Population at risk	Floor level (m CDD)
Kidsfirst Kindergarten	40	15	5.25	10.85
New Brighton Pre school	40	15	5.25	10.97
New Brighton Police	40	32	11.2	11.07
Annabel Educare preschool	40	15	5.25	11.14
Montessori School	50	20	7	11.21

The above five properties are all on the left bank (east of the river) and in the upstream half of the study area, broadly proximate to the New Brighton commercial area. They were all assessed as having zero night-time population.

The research basis for the above numbers was limited to consideration of the size of buildings, general knowledge and online search discovered the student roll for the Montessori school as being 44. The day population was an average occupancy over the 98 hr/week daytime period, with consideration that in all cases the buildings would be occupied for only some fraction of that time, hence the reduction from peak population to day population. None of the above locations had a significant impact on the total results, with the overall life risks heavily dominated by the much larger residential population at risk.

For each building the daytime population was reduced by 40% reflecting an assumption that 40% of the population would be evacuated prior to a stopbank failure flood event. This assumption is based on a reasonable interpretation of Council's current emergency preparedness but is relatively uncertain and dependant on many factors on the day. Fortunately, the daytime life risk overall is minor in comparison to night-time risk and hence the results are not overly sensitive to this assumption.

The night-time population was left with no reduction, assuming nil evacuation. The resulting day and night-time post evacuation person counts were then averaged over a typical week to produce that buildings 'population at risk'.

In typical residential and commercial buildings, we assumed the following daytime and night-time population figures. The weighted population at risk accounts for the proportion of day and night and includes the 40%-day time evacuation.

Table 4-3 Population assessment for typical buildings

Type	Day population	Night population	Weighted population at risk (PAR)
Residential	1	2.3[7]	1.31
Commercial	3	0	1.05

Table 4-3 is provided as an illustrative example only. It applies to some census meshbocks on the left bank where there is an average population per household of 2.3. Population per household is different for every census meshblock area.

This initial data preparation was analysed for a variety of tide levels covering a wide range of tidal ARIs detailed in the following sections. For each tide level, the full database of building floor levels was assessed and depth of flooding above (or below) floor level evaluated.

[7] Normally resident population per building was assessed from 2013 meshblock census data supplied by Council. From this the normally residential population was estimated at 2.3 on the left bank and 2.8 on the right bank.

The population at risk for flood depths of less than 300 mm inundation above floor levels were not included in the estimates for potential life loss as this depth is considered to have a very low mortality rate and has been used as a cut-off for PAR in ANCOLD 2003 Guidelines on Risk Assessment.

The total PAR for the night-time is shown on Table 4-4 below.

Table 4-4 Night population at risk for different tide levels

Tide ARI	Goring Tidal level (m CDD)	Left bank PAR	Right bank PAR	Total PAR
1	10.64	0	56	56
2	10.68	0	59	59
5	10.80	0	78	78
10	10.89	14	92	106
20	10.96	25	118	143
50	11.06	74	165	239
100	11.14	124	190	315
200	11.21	235	221	456
500	11.31	396	260	656
1,000	11.39	582	305	887
2,000	11.46	782	364	1146
5,000	11.56	1024	442	1466
10,000	11.64	1242	501	1743
50,000	11.81	1635	720	2355

4.2.2 Potential Life Loss (PLL) from General Inundation

The report attached in Appendix A provides the background and detail for the evaluation of the fatality rates used to estimate the Potential Life Loss for each failure or overtopping event. This section and the following section summarise from that memo.

This appended report considers two causes of potential life loss;

- General inundation area (negligible water velocity), and
- Houses and buildings in close proximity to the stopbank (significant water velocity).

This section Potential Life Loss (PLL) from General Inundation 4.2.2 presents potential life loss from general inundation, which is characterised by having negligible water velocity and is applicable across the whole study area. Section 4.2.4 below presents potential life loss from houses immediately adjacent to the stopbank. These have additional risks due to significant water velocity from a stopbank breach failure. In the final assessment both life risks are added together to assess total potential life risk.

The approach selected for general tidal overtopping inundation was by Jonkman (September 2008), for which Figure 4-5 was developed by Jonkman to show the process for estimating the potential life loss.

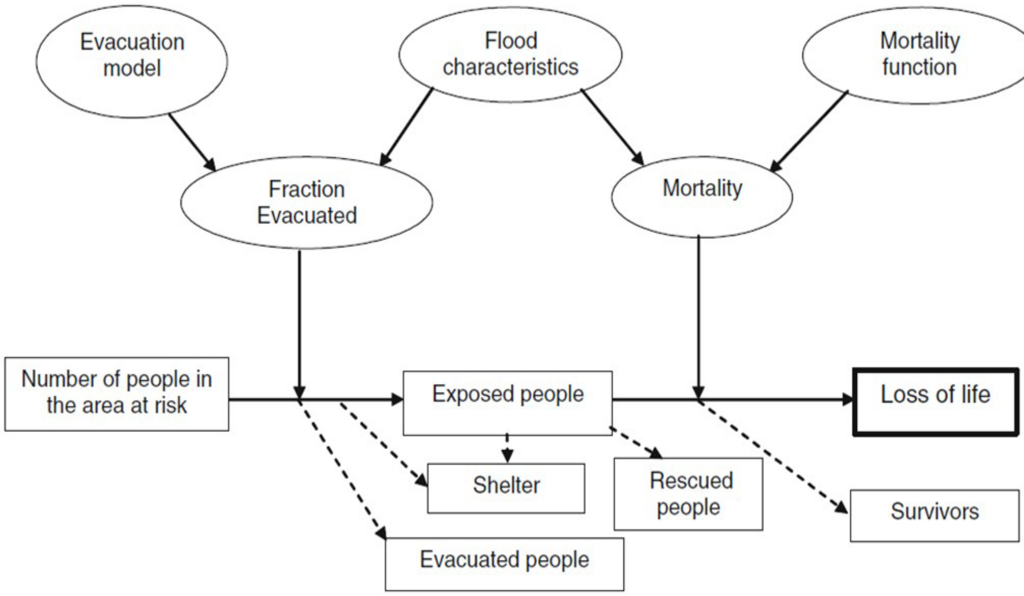


Figure 4-5 General model for evaluating Potential Life Loss (Jonkman September 2008)

The number of people exposed for the event N_{EXP} is evaluated as follows

$N_{EXP} = (1-F_E)(1-F_S)N_{PAR} - N_{RES}$

Where

N_{PAR} = population at risk

F_E = Evacuation Factor

F_S = Shelter Factor for people able to find shelter

N_{RES} = people rescued

For this study we consider the ability for people to find shelter and likelihood for them to be rescued are minimal and taken as zero. The evacuation factor however is material and we have adopted 40% for evacuation from a daytime event and nil evacuation from a night-time event (Jonkman Sep 2008).

The Mortality (Fatality) rate $F_D(h)$ applied to the people exposed for the event N_{EXP} was estimated using Jonkman's 2008 log normal function with the depth *h* (above estimated floor level), mean 7.60 and standard deviation 2.75 as follows:

$$F_D(h) = \Phi_N\left(\frac{\ln(h) - \mu_N}{\sigma_N}\right)$$

$$\mu_N = 7.60 \quad \sigma_N = 2.75$$

The Potential Life Loss was then calculated for each house as follows.

$PLL = F_D(h)*N_{EXP}$

Using the above formulae, the Potential Loss of Life (PLL) was calculated separately for each building and tidal (flood) level and then summed to produce total PLL for that tide level event, as shown on Table 4-5 and Figure 4-6.

Table 4-5 Potential Life Loss from General Inundation

Tide ARI	Goring tidal level (m CDD)	Night-time population at risk (PAR)	Left bank PLL*	Right bank PLL*	Total PLL*
1	10.64	56	0.0	0.1	0.1
2	10.68	59	0.0	0.2	0.2
5	10.80	78	0.0	0.2	0.2
10	10.89	106	0.0	0.3	0.3
20	10.96	143	0.1	0.3	0.4
50	11.06	239	0.2	0.5	0.7
100	11.14	315	0.3	0.6	0.9
200	11.21	456	0.6	0.7	1.4
500	11.31	656	1.1	0.9	2.0
1,000	11.39	887	1.7	1.0	2.7
2,000	11.46	1146	2.4	1.3	3.6
5,000	11.56	1466	3.3	1.6	4.8
10,000	11.64	1743	4.1	1.8	5.9
50,000	11.81	2355	5.9	2.7	8.6

*PLL is defined as the expected (average) loss of life from a single flood event of a particular level, where the timing of the peak tide is equi-probable across 24 hours and the risk is averaged across day or night-time events.

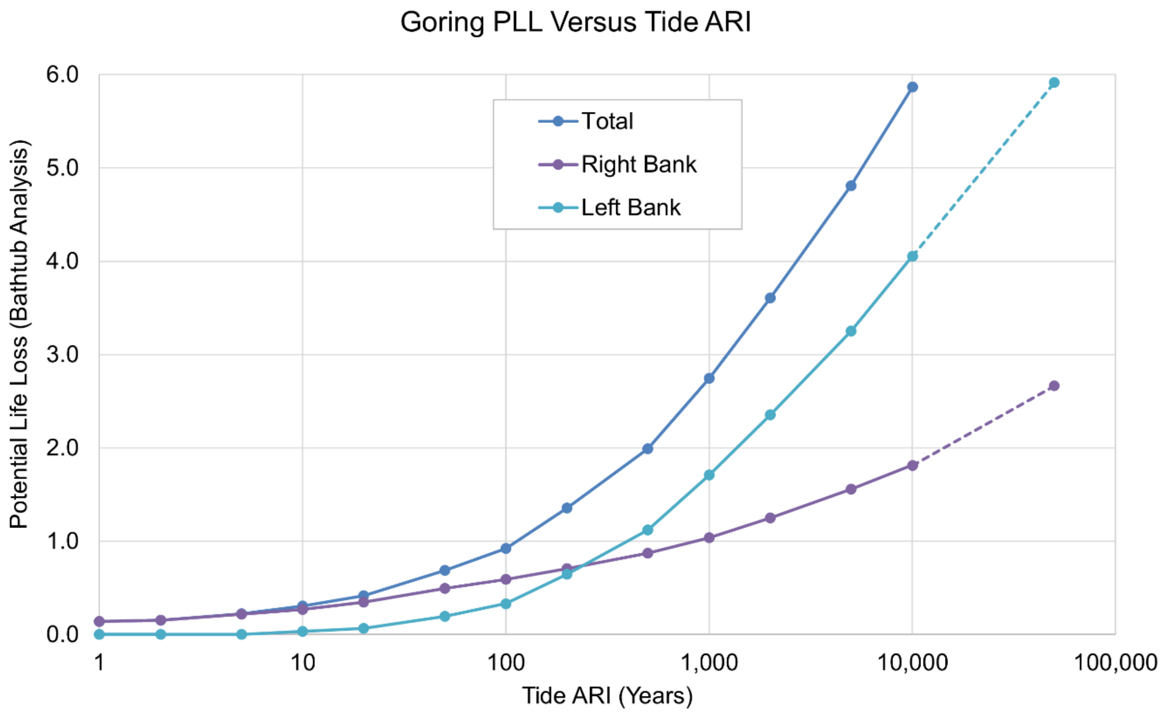


Figure 4-6 Potential Life Loss from General Inundation, 40% evacuation day and 0% evacuation night

Figure 4-6 shows the notable difference in the PLL profiles between the left and right banks) with the right bank containing most of the lowest floor levels below 10.5 m (for flood levels below 10.8 m). In the larger flood events, however, the left bank contained the much larger number of houses at risk with floor levels below 11.2 m, (for flood levels below 11.5 m).

4.2.3 General Inundation Example

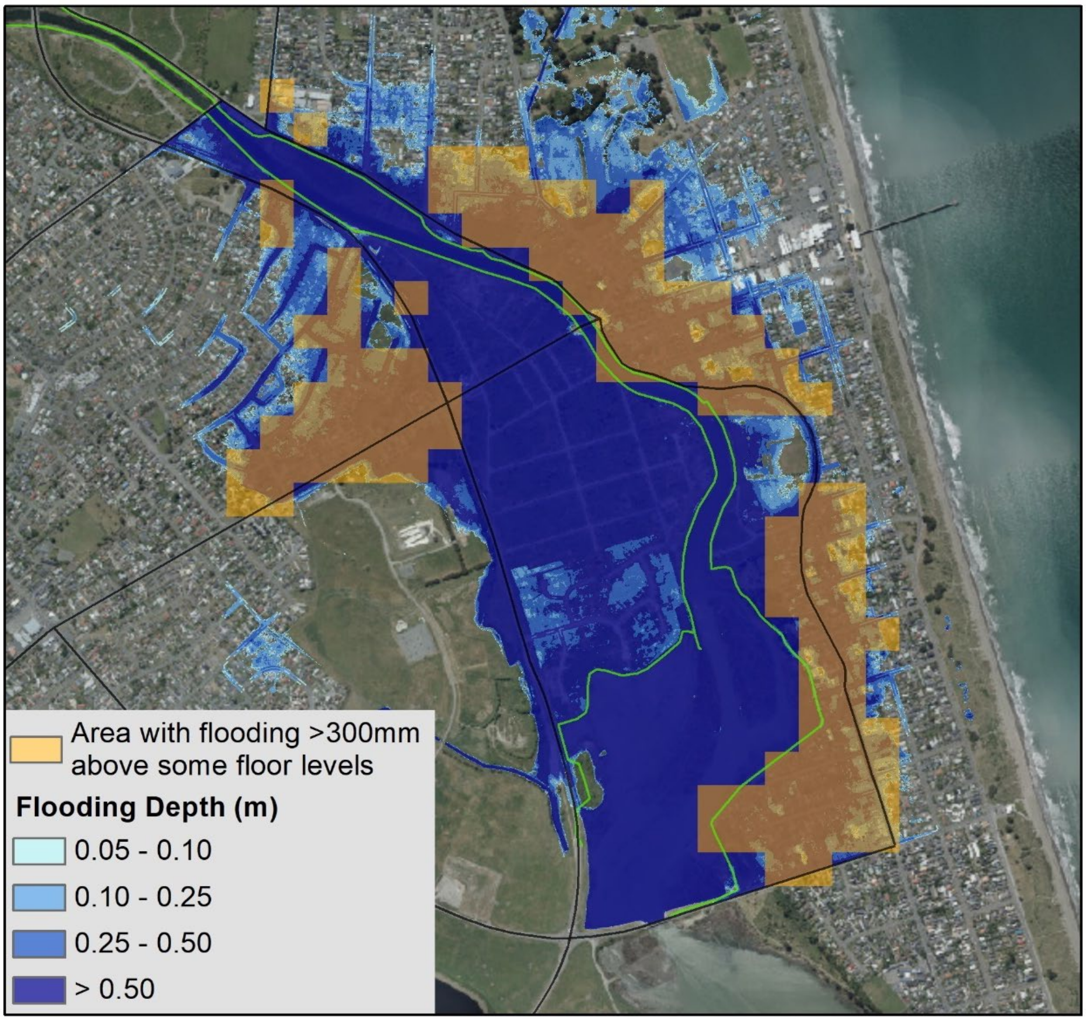


Figure 4-7 Map of risks associated with 11.4 m tidal flood event

Figure 4-7 shows, as an example, the flood depths within the study area caused by an 11.4 m tidal flood event (approximately a 1,000 year ARI sea level) overtopping design crest levels of 11.2m and assuming flow goes around (or breaks through) the stopbanks with 11.4m design crest levels. This illustrates the geographical character of the stopbank overtopping risks which are evaluated in this report.

The figure also shows the location of buildings with estimated floor levels below 11.1 m. These are the buildings where estimated flood levels are deeper than 0.3 m above floor level, which is the depth above which risks to life have been assessed for the illustrated flood event (as discussed in Appendix A, section 4.1). The location of buildings is simplified in that individual specific properties are not identified.

Note that this figure also illustrates the benefit of the stopbanks. For example in an 11.1m sea level event (approximately 70-year ARI), many buildings within the tan coloured ‘area with flooding above some floor levels’ would be flooded if there was no stopbank but are protected in that event by the stopbank.

4.2.4 PLL from Houses in Close Proximity

The breach zone adjacent to the stopbank is shown schematically on Figure 4-8.

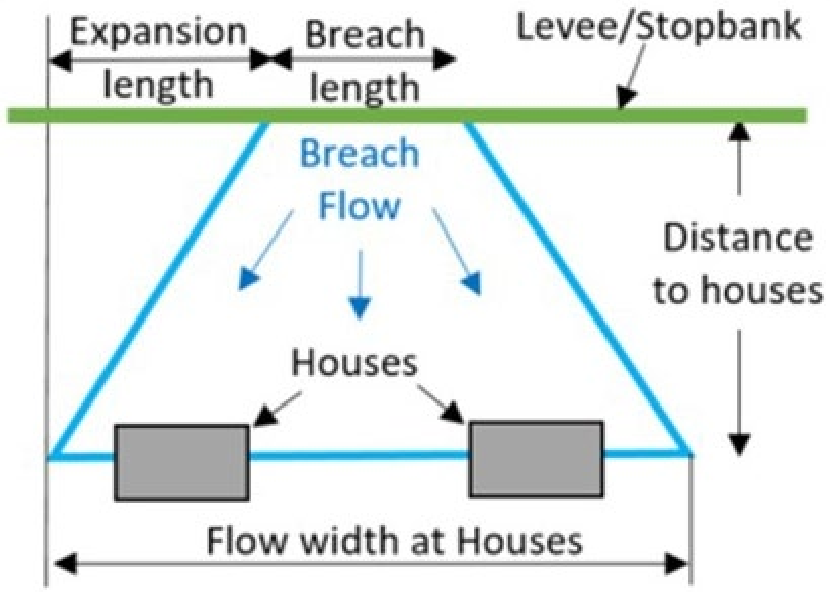


Figure 4-8 Schematic of the breach expansion from the stopbank to the closest houses

The breach through the stopbanks was evaluated for varying breach lengths and tidal levels from which it was found that the depths and velocities at the houses in close proximity to the stopbanks, while significant would be low due to the lower water height relative to global literature studies.

We considered therefore that the daytime risk to houses in close proximity was adequately covered by the general inundation risk described earlier. For the night, risks of overtopping failure and embankment failure were assessed. Assuming there is no warning, a fatality factor of 0.01 was used (as discussed in Appendix A, section 3.1.4 and 3.1.5). Section 3.1.3 of Appendix A provides details of the evaluation for the number of houses affected by a potential breach of the stopbank. It was assessed that five houses on the left bank were in close proximity to the stopbanks and they would have materially higher night time risk from overtopping failure with an embankment breach length of about 50 m. In the case of piping failure, it was estimated that breach length would be 10 m and three of the five houses would be affected.

The resulting PLL for the night with overtopping failure was calculated to be:

5 (houses) * 2.3 (PAR) * 0.01 (Fatality) = 0.12

The resulting PLL for the night with piping failure was calculated to be:

3 (houses) * 2.3 (PAR) * 0.01 (Fatality) = 0.07

Because there was no additional daytime PLL for the houses in close proximity the weighted PLL for day (14 hrs) and night (10 hrs) periods was calculated to be

0.12* 10/24 + 0 * 14/24 = 0.05 (overtopping)

0.07* 10/24 + 0 * 14/24 = 0.03 (embankment piping)

These PLL estimates were added to the general inundation area PLL for the left bank sections for events in which piping or overtopping occurred. The right bank houses and buildings are all a significant distance away from the stopbank and so there are no close proximity houses and no additional PLL for the close proximity risks.

4.2.5 Individual Risk Fatality Rates

According to the ANCOLD Risk Guidelines (2003), individual risk is defined as "*the increment of risk imposed by the existence of the stopbank on the person most at risk*". In order to estimate this, the fatality rate was estimated for the population in closest proximity to the stopbank as follows:

Fatality rate 0.01

Day evacuation 40%

Night Evacuation 0%

Weighted fatality rate = 14/24* 0.01*(100%-40%) + 10/24* 0.01 *100% = 0.0077

The Individual risk was calculated as the product of this fatality rate times the total annual failure probability the stopbank.

4.3 Failure Probabilities and Risk to Life

The results of the analysis are failure probabilities and risk to life for each section and failure mode. These results using the Goring tidal data with the 40% evacuation during the day and 0% evacuation during the nights are tabulated in the following sections and the details are provided in Appendix E.

4.3.1 Consequences - All Events

The Individual Risk was calculated using the product of the Individual Fatality rate and the annual failure probability on Table 4-6 below.

The Individual Risk was calculated as 7.7E-5 (0.77 in 10,000), as shown on Table 4-6.

The individual risk is better than (below) the ANCOLD limit of tolerability for individual risk of 10.0E-5 (1 in 10,000).

In Table 4-6, the headings are explained as follows:

- Failure Probability: is the annual probability of a stopbank failure event
- Annual Failure Probability: subdivides the probability into each failure mode and each stopbank section (the total below is a simple sum of the element probabilities)
- Percentage of total shows the importance of each failure mode and each stopbank section probability as a percentage of the total probability
- Percentage: shows the sum of percentages across the stopbank sections within each failure mode (highlights the most important failure modes), and
- Total: shows the sum of percentages across the failure modes within each stopbank section (highlights the most important sections).

Table 4-6 Summary of annual failure probabilities for each stopbank section and failure mode

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.4m Others at 11.2m				
Failure Mode	Stopbank Section	Failure Probability		
		Annual Failure Probability	Percentage of Total	Percentage
Piping Earthquake	Section 1	2.82E-08	0.000%	
	Section 2	1.41E-07	0.001%	
	Section 3	7.56E-09	0.000%	
	Section 4	6.62E-08	0.001%	
	Section 17	4.33E-07	0.004%	
	Section 18	6.38E-07	0.006%	0.013%
Foundation Piping	Section 1	1.24E-04	1.242%	
	Section 2	7.10E-05	0.710%	
	Section 3	2.35E-07	0.002%	
	Section 4	5.11E-05	0.511%	
	Section 17	2.11E-04	2.107%	
	Section 18	5.44E-04	5.439%	10.011%
Earthquake Overtopping	Section 1	3.63E-07	0.004%	
	Section 2	2.44E-06	0.024%	
	Section 3	1.03E-05	0.103%	
	Section 4	1.05E-06	0.010%	
	Section 17	1.37E-05	0.137%	
	Section 18	2.28E-05	0.228%	0.506%
Embankment Piping	Section 1	4.40E-10	0.000%	
	Section 2	1.12E-09	0.000%	
	Section 3	3.12E-12	0.000%	
	Section 4	2.93E-10	0.000%	
	Section 17	6.39E-09	0.000%	
	Section 18	4.41E-09	0.000%	0.000%
Embankment Overtopping	Section 1	3.65E-04	3.645%	
	Section 2	1.23E-03	12.258%	
	Section 3	1.23E-03	12.258%	
	Section 4	1.23E-03	12.258%	
	Section 17	1.94E-03	19.389%	
	Section 18	2.97E-03	29.663%	89.470%
Total	Section 1	4.89E-04	4.89%	
	Section 2	1.30E-03	12.99%	
	Section 3	1.24E-03	12.36%	
	Section 4	1.28E-03	12.78%	
	Section 17	2.16E-03	21.64%	
	Section 18	3.53E-03	35.34%	
Total all tidal events		1.00E-02	100.0%	100.0%
Individual Fatality Rate		0.0077		
Individual Risk		7.70E-05		

The individual fatality rate of 0.0077 shown on Table 4-6 was calculated as discussed in Section 4.2.5.

Note that in Table 4-6 the majority of the failure probability shown (89% of total probability) is associated with the embankment overtopping failure. This failure occurs with infrequent and extreme events that are significantly larger (higher sea level) than the stopbank is designed for.

The avoidable failure of the stopbank associated with more common events that are within the stopbanks design expectations are a higher priority concern for Council than more intrinsic failure probabilities of a rare event that exceeds the stopbanks design expectations. Those probabilities are shown in the top 4 items in Table 4-6, and of those four failure modes the dominant failure mode is foundation piping failure.

In Table 4-7, the headings are explained as follows:

- Risk (lives/Annum): is the expected average number of deaths per year, given the previously assessed probability of stopbank failure and the associated potential life loss.
- Risk: subdivides the annual deaths into each failure mode and each stopbank section (the total below is a simple sum of the element numbers), and
- Percentage of Total, Percentage and Total: are all analogous to their counterparts under failure probability.

Table 4-7 Summary of risk to life for each stopbank section and failure mode

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.4m Others at 11.2m				
Failure Mode	Stopbank Section	Risk (lives/Annum)		
		Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	1.31E-08	0.000%	
	Section 2	7.89E-09	0.000%	
	Section 3	5.00E-09	0.000%	
	Section 4	6.83E-09	0.000%	
	Section 17	7.88E-08	0.001%	
	Section 18	1.22E-07	0.001%	0.002%
Foundation Piping	Section 1	5.67E-06	0.039%	
	Section 2	4.13E-06	0.029%	
	Section 3	3.24E-07	0.002%	
	Section 4	2.59E-06	0.018%	
	Section 17	4.46E-05	0.308%	
	Section 18	1.30E-04	0.895%	1.291%
Earthquake Overtopping	Section 1	1.20E-06	0.008%	
	Section 2	3.46E-06	0.024%	
	Section 3	1.50E-05	0.104%	
	Section 4	2.19E-06	0.015%	
	Section 17	1.19E-05	0.082%	
	Section 18	1.79E-05	0.124%	0.357%
Embankment Piping	Section 1	1.39E-10	0.000%	
	Section 2	8.65E-11	0.000%	
	Section 3	1.85E-12	0.000%	
	Section 4	5.84E-11	0.000%	
	Section 17	1.16E-09	0.000%	
	Section 18	9.73E-10	0.000%	0.000%
Embankment Overtopping	Section 1	1.33E-03	9.208%	
	Section 2	2.58E-03	17.803%	
	Section 3	2.58E-03	17.803%	
	Section 4	2.58E-03	17.803%	
	Section 17	2.12E-03	14.656%	
	Section 18	3.05E-03	21.077%	98.351%
Total	Section 1	1.34E-03	9.26%	
	Section 2	2.59E-03	17.86%	
	Section 3	2.59E-03	17.91%	
	Section 4	2.58E-03	17.84%	
	Section 17	2.18E-03	15.05%	
	Section 18	3.20E-03	22.10%	
Total all tidal events		1.45E-02	100.0%	100.0%

The results clearly show that the overtopping of the Stopbank is the dominant failure mode followed by the foundation piping failure mode. The percentage contributions for the risk of overtopping for each stopbank section are shown on Figure 4-9. This figure shows the increase in risk with higher tidal levels up to the 1 in 2,000 to 1 in 10,000 AEP events followed by a reduction for the events greater than the 1 in 10,000 AEP event.

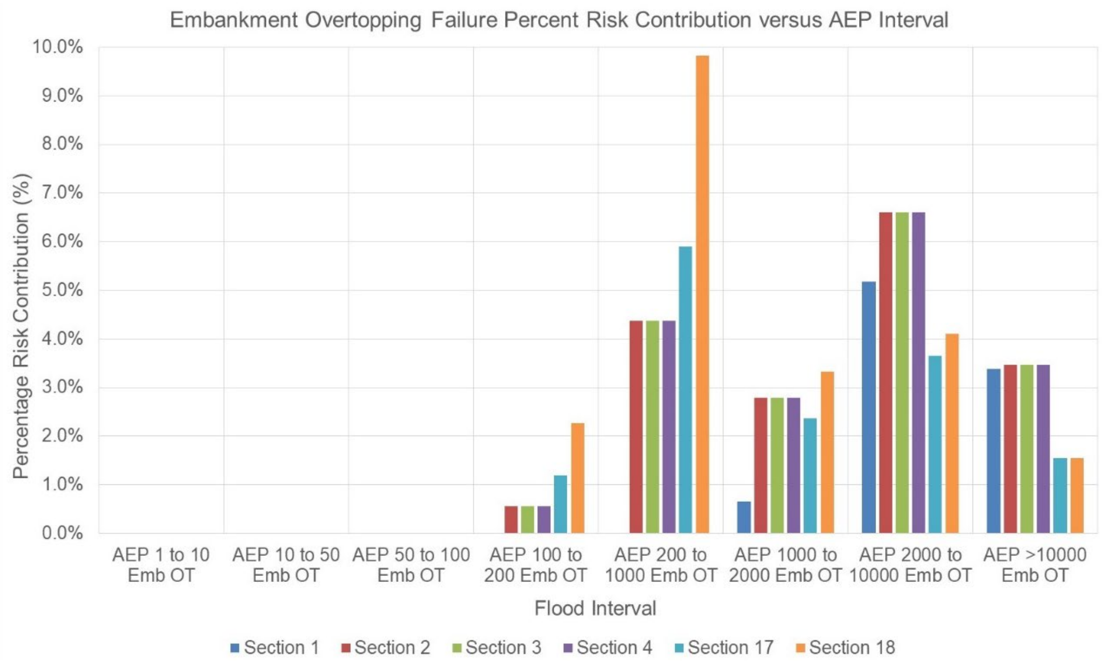


Figure 4-9 Embankment overtopping failure mode – risk contribution versus AEP interval for each stopbank section

This shows the effective absence of life risks (very low) up to the stopbank design crest level. The stopbanks are effective for the events they are designed to control. In more extreme overdesign events, the risks are reflective of flooding generally rather than the stopbank failing to perform to its design intention.

The relative importance of embankment overtopping failure mode is noteworthy in both failure probability and life risks. Given the design method used to set the stopbank levels this finding is to be expected. The reason that the embankment overtopping failure mode is even more dominant in the life risks assessment, is that the other failure modes collect a lot of their total failure risk from lesser ARI and lower tidal events. These lower flood levels have consequently less life risks if the failure occurs. By comparison if overtopping failure occurs then all those failure events involve extreme high-water levels all of which have higher life risks. This is why the overtopping failure mode is so heavily the dominant failure mode in the life risks assessment.

4.3.2 Consequences - Below 100-year ARI Tide Event

Council's first interest in the stopbanks is to ensure that they will perform their intended function with an acceptably low risk of failure and resulting low life risks. The stopbanks within this study area are there to protect the community from coastal inundation from high sea level events up to the 100 year ARI design standard, sea level (11.14m).

Accordingly, failure probabilities and life risks are presented numerically in Table 4-8 below for the below 100 year ARI tide event.

This table can be compared with results from Table 4-6 and Table 4-7 above.

Table 4-8 Summary of annual failure probabilities and life risk for each section and failure mode with existing stopbank levels for tidal levels below the 100 year ARI level

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.5m Others at 11.5m							
Failure Mode	Stopbank Section	Failure Probability			Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	1.99E-08	0.002%		2.03E-09	0.002%	
	Section 2	1.36E-07	0.014%		2.10E-09	0.002%	
	Section 3	4.44E-09	0.000%		4.75E-10	0.000%	
	Section 4	6.21E-08	0.006%		1.51E-09	0.001%	
	Section 17	4.23E-07	0.044%		6.97E-08	0.057%	
	Section 18	6.18E-07	0.064%	0.131%	1.05E-07	0.086%	0.148%
Foundation Piping	Section 1	1.21E-04	12.556%		1.51E-06	1.245%	
	Section 2	6.93E-05	7.179%		7.20E-07	0.592%	
	Section 3	2.96E-08	0.003%		6.63E-09	0.005%	
	Section 4	4.98E-05	5.163%		5.75E-07	0.473%	
	Section 17	1.99E-04	20.584%		3.16E-05	26.025%	
	Section 18	5.13E-04	53.132%	98.616%	8.27E-05	68.034%	96.374%
Earthquake Overtopping	Section 1	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 2	5.26E-07	0.054%		7.05E-08	0.058%	
	Section 3	1.72E-06	0.178%		3.04E-07	0.250%	
	Section 4	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 17	3.10E-06	0.321%		1.17E-06	0.965%	
	Section 18	6.73E-06	0.698%	1.252%	2.68E-06	2.202%	3.476%
Embankment Piping	Section 1	3.51E-10	0.000%		1.98E-11	0.000%	
	Section 2	1.07E-09	0.000%		2.16E-11	0.000%	
	Section 3	2.50E-12	0.000%		2.45E-14	0.000%	
	Section 4	2.56E-10	0.000%		1.05E-11	0.000%	
	Section 17	6.24E-09	0.001%		1.03E-09	0.001%	
	Section 18	4.17E-09	0.000%	0.001%	7.57E-10	0.001%	0.002%
Embankment Overtopping	Section 1	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 2	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 3	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 4	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 17	0.00E+00	0.000%		0.00E+00	0.000%	
	Section 18	0.00E+00	0.000%	0.000%	0.00E+00	0.000%	0.000%
Total	Section 1	1.21E-04	12.56%		1.51E-06	1.25%	
	Section 2	6.99E-05	7.25%		7.92E-07	0.65%	
	Section 3	1.75E-06	0.18%		3.11E-07	0.26%	
	Section 4	4.99E-05	5.17%		5.76E-07	0.47%	
	Section 17	2.02E-04	20.95%		3.29E-05	27.05%	
	Section 18	5.20E-04	53.89%		8.54E-05	70.32%	
Total < 1 in 100 AEP		9.65E-04	100.00%		1.21E-04	100.00%	

The results show that foundation piping failure risks are the dominant failure mode but that the risk to life associated with such failure is very low contributing <1% of the total life risk (comparing Table 4-8 against Table 4-7). Sections 17 and 18 have the highest probability of piping failure mainly due to the less-suitable loose sandy material underlying the stopbank but they are still comfortably within guideline risks.excel

This compares with the analysis for all events which showed failure probabilities and life risks heavily dominated by overtopping failure models (events).

5. Risk Evaluation

5.1 Societal Risk Below 100 year ARI Tide Event

Council's first interest in the stopbanks is to ensure that they will perform their intended function with an acceptably low risk of failure and resulting low life risks. Their intended function is to protect the community from inundation for sea level events below the 100 year ARI design standard.

We therefore present first the societal risks associated with below the 100-year ARI high tide events. These are shown on the Societal Risk (FN) plot on Figure 5-1.

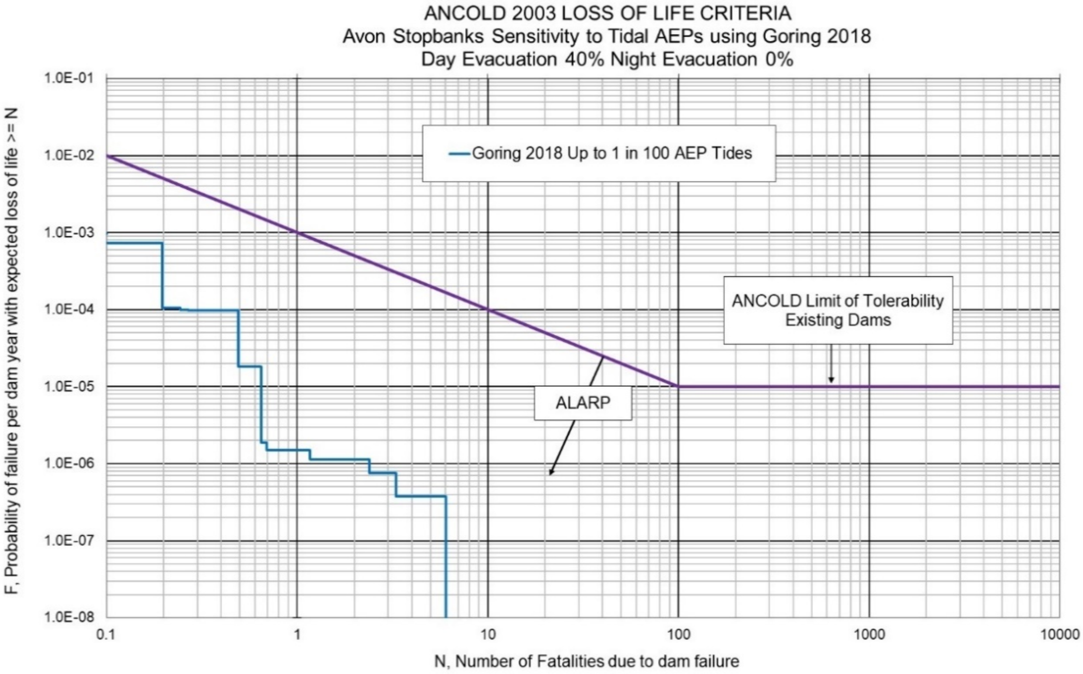


Figure 5-1 Avon Stopbanks societal risk – current stopbank levels – with risks up to 100-year ARI tide levels separated

It is noteworthy that the risks below the 100 year ARI tide are almost ten times better than (below) the ANCOLD guidelines limit of tolerability. This indicates that these risks are adequately managed and are in the area where any further risk reduction may only be motivated by the ALARP principles. Council and the community should have confidence that there is low-risk to life from the stopbanks in this study area in relation to their intended purpose to mitigate flood risks below the 100-year ARI tide level.

5.2 Societal Risk – All Tide Events

This section considers all tide events including those above the 100 year ARI design event.

A more extreme high sea level event that exceeds the design standard and overtops the stopbank, could be considered to be stopbank failure (ie: the stopbank has not contained the water and is likely to be heavily damaged) or not a failure since it was never intended to perform any functional service in such an event. In stopbank risk analysis overtopping water level events are conventionally classified to be failure. In the case here overtopping has a different character of expected failure different from the types of failure mechanisms and risks where the stopbank is not overtopped and is not expected to fail.

The Societal Risk (FN) plot, for all events including overtopping, as shown on Figure 5-2 is above the ANCOLD guideline limit of tolerability for existing dams. If Council decide to manage all risks, especially overdesign risks, and to adopt and comply with the ANCOLD guideline limit of tolerability then action would be needed to lower the risk to below the limit line using the As Low As Reasonably Practicable (ALARP) principle.

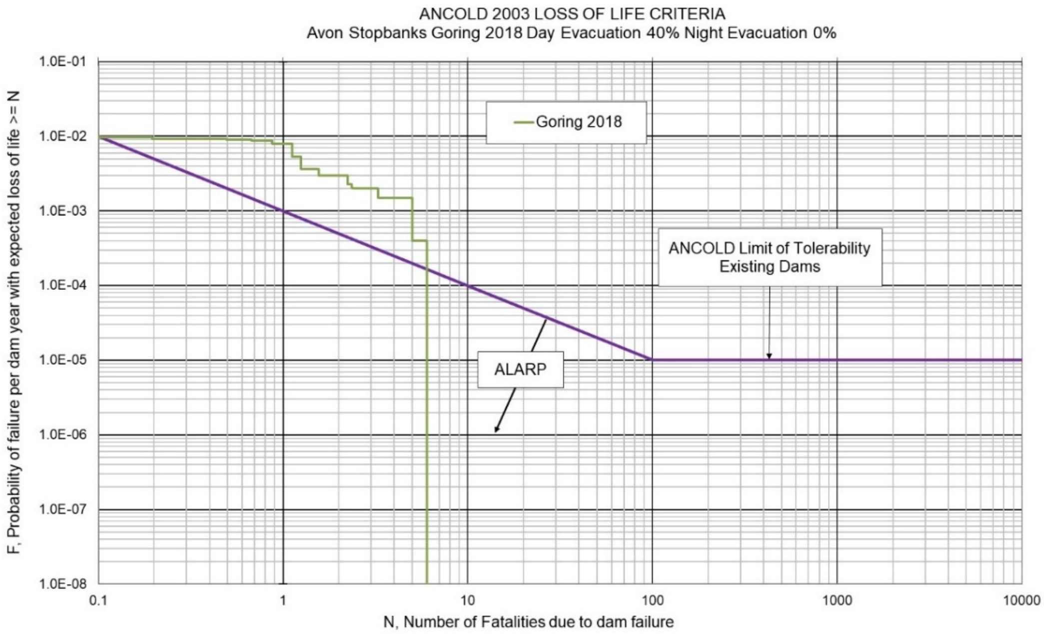


Figure 5-2 Avon Stopbanks societal risk – current stopbank levels (generally 11.2 m, Section 1 at 11.4 m)

6. Sensitivity Testing

6.1 Sensitivity to Fatality Rates and Extreme Sea Level risks

In order to understand the confidence around the above conclusions, we have chosen to vary two input variables. We have selected the variables of fatality rate and extreme sea level as we believe these have uncertainty which is most influential on the conclusions.

The two inputs were adjusted simultaneously by approximately 1SD (standard deviation). The joint variation informally approximates a 2SD confidence interval result.

6.1.1 Extreme Sea Level Confidence Interval

Understanding the confidence interval around extreme sea levels is a mathematical process. This work has been done and reported under a separate Council project LDRP-097 “Joint Risks - Pluvial and Tidal Flooding, Dec 2020”. That work reports a 95% (2SD) confidence interval around a newly determined extreme values trend relationship at Bridge St. This report takes the spread of sea levels from that work, halves the spread to approximate a 68% (1SD) confidence interval, and re-centres the interval on the Goring 2018 extreme values trend relationship at Bridge Street.

6.1.2 Fatality Rate Confidence Interval

Understanding the confidence around fatality rate requires professional judgement beyond guidance of published literature and interpretation and extrapolation from collections of reported life losses from actual flood events. Christchurch circumstances, particularly due to the low velocities, are milder than most events in the published literature, thus requiring a degree of extrapolation to evaluate expected values. The context of published literature however provides some basis on which to form a view of confidence in the predicted life risks.

From our evaluation of this literature, aiming at a 1SD confidence interval, we have evaluated the range in fatality rate to be between a factor of ten-fold reduction to a factor of 1.5 increase. Given the Christchurch circumstances the possibility of extremely low life risks cannot be discounted by the published literature and many of the case studies have reported zero loss of life from circumstances with more intrinsic danger (higher depth and velocity) than is expected from Christchurch stopbanks flooding events. Table 6-1 below shows our conclusions on the confidence intervals for fatality rate with varying depth.

Table 6-1 Fatality rate confidence interval

Depth (m)	Low Fatality Rate (div 10)	Fatality Rate (Baseline)[8]	High Fatality Rate (x 1.5)
0.3	0.0001	0.0007	0.0010
0.5	0.0001	0.0013	0.0019
1.0	0.0003	0.0029	0.0043
2.0	0.0006	0.0060	0.0090

[8] Fatality rate (baseline) is as discussed in this report section 4.2.2 “estimated using Jonkman’s 2008 log normal function”

We have compared these fatality rates, especially the above rate for 2m depth, compared to the DV = 10 ft2/sec (DV = 1 m2/s) on the RCEM plots shown as Figures 2-3 and 2-4 from GHD report “Stopbank Failure Loss of Life Review” by Malcolm Barker (see Appendix A).

6.1.3 Results

The results of the above defined sensitivity analyses, varying each of the two variables by 1SD each, are shown on either side of the baseline result on Figure 6-1 below.

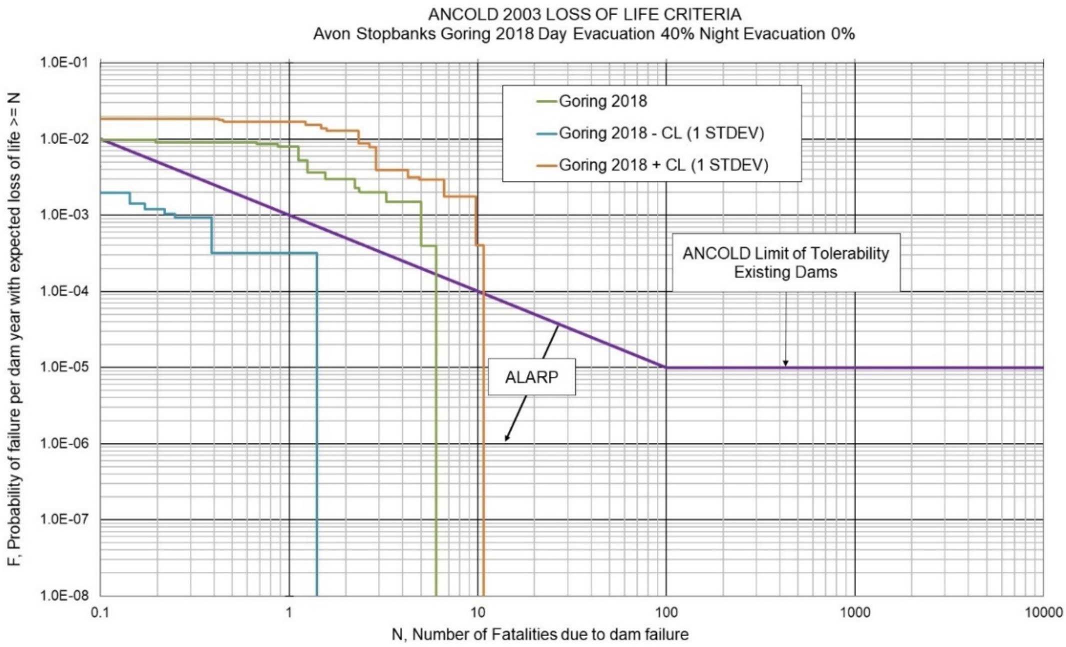


Figure 6-1 Avon Stopbanks societal risk – current stopbank levels – with high/low confidence intervals

6.1.4 Discussion

The assessment shows that uncertainty in the fatality rate is the dominant factor in the overall uncertainty. The fatality rate uncertainty results in a vertical offset of the societal risk curve. The low bound reduction by ten results in a full order of magnitude reduction. The upper bound increase by 1.5 times, results in 1/6th of an order of magnitude increase.

The remainder of the change shown above results from the uncertainty in the extreme sea levels. These tend to shift the societal risk curves vertically and horizontally. The impacts of this are most evident below the 1E-04 (1 in 10,000) probability of failure, where the expected number of fatalities is greatly reduced with the lower sea levels (low bound estimate) or conversely increased with the higher sea levels.

The above findings in Figure 6-1 can also be considered in terms of the annual probability of a flooding event with one life loss (the N=1 position on the above Societal Risk plot).

Table 6-2 Annual probability of flood event with one life loss

	Probability of failure with N=1	Approximate average year interval
Goring 2018 – lower bound	3.16E-04	3,000 years
ANCOLD Guideline	1.00E-03	1,000 years
Goring 2018	5.30E-03	200 years
Goring 2018 – upper bound	1.69E-02	50 years

6.2 Sensitivity to Newer Tidal Data

A sensitivity analysis was completed using the HKV tidal data for comparison with the Goring data for which the HKV analysis results are shown on Table 6-3 and a comparison of the Societal risk plots are shown on Figure 6-2.

The results of this sensitivity analysis show that there is very small reduction in the Societal risk using the HKV data, but the risk remains above the ANCOLD Limit of Tolerability.

Table 6-3 Summary of annual failure probabilities and risk for each section and failure mode with existing stopbank levels using HKV tidal data

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.4m Others at 11.2m							
Failure Mode	Stopbank Section	Failure Probability			Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	9.01E-08	0.001%		1.46E-08	0.000%	
	Section 2	1.72E-07	0.002%		8.31E-09	0.000%	
	Section 3	1.19E-08	0.000%		5.20E-09	0.000%	
	Section 4	9.03E-08	0.001%		7.76E-09	0.000%	
	Section 17	5.18E-07	0.005%		1.05E-07	0.001%	
	Section 18	7.88E-07	0.008%	0.017%	1.66E-07	0.001%	0.002%
Foundation Piping	Section 1	1.47E-04	1.505%		6.03E-06	0.046%	
	Section 2	7.86E-05	0.803%		3.39E-06	0.026%	
	Section 3	3.04E-07	0.003%		3.27E-07	0.002%	
	Section 4	5.87E-05	0.599%		2.23E-06	0.017%	
	Section 17	2.27E-04	2.314%		5.24E-05	0.400%	
	Section 18	5.95E-04	6.079%	11.304%	1.34E-04	1.023%	1.514%
Earthquake Overtopping	Section 1	2.78E-07	0.003%		8.24E-07	0.006%	
	Section 2	3.19E-06	0.033%		3.54E-06	0.027%	
	Section 3	1.19E-05	0.122%		1.51E-05	0.115%	
	Section 4	1.00E-06	0.010%		1.99E-06	0.015%	
	Section 17	1.85E-05	0.188%		1.38E-05	0.105%	
	Section 18	2.95E-05	0.302%	0.657%	2.07E-05	0.158%	0.427%
Embankment piping	Section 1	5.33E-10	0.000%		1.50E-10	0.000%	
	Section 2	1.47E-09	0.000%		9.09E-11	0.000%	
	Section 3	2.87E-12	0.000%		6.49E-13	0.000%	
	Section 4	5.69E-10	0.000%		6.63E-11	0.000%	
	Section 17	7.59E-09	0.000%		1.55E-09	0.000%	
	Section 18	5.77E-09	0.000%	0.000%	1.35E-09	0.000%	0.000%
Embankment Overtopping	Section 1	2.53E-04	2.580%		8.48E-04	6.464%	
	Section 2	1.17E-03	11.989%		2.34E-03	17.805%	
	Section 3	1.17E-03	11.989%		2.34E-03	17.805%	
	Section 4	1.17E-03	11.989%		2.34E-03	17.805%	
	Section 17	1.91E-03	19.456%		2.03E-03	15.503%	
	Section 18	2.94E-03	30.018%	88.022%	2.97E-03	22.675%	98.057%
Total	Section 1	4.00E-04	4.09%		8.55E-04	6.52%	
	Section 2	1.26E-03	12.83%		2.34E-03	17.86%	
	Section 3	1.19E-03	12.11%		2.35E-03	17.92%	
	Section 4	1.23E-03	12.60%		2.34E-03	17.84%	
	Section 17	2.15E-03	21.96%		2.10E-03	16.01%	
	Section 18	3.57E-03	36.41%		3.13E-03	23.86%	
Totals		9.79E-03	100.000%	100.000%	1.31E-02	100.000%	100.000%

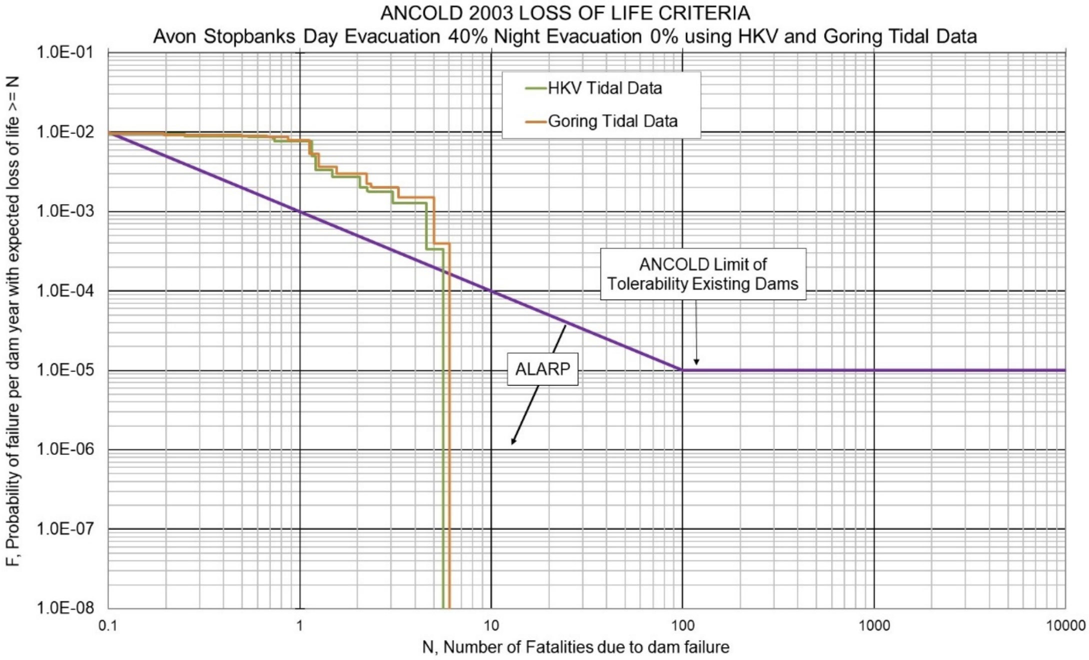


Figure 6-2 Avon Stopbanks comparison of societal risk using Goring and HKV tidal data with current stopbank levels (general levels at 11.2 m and Section 1 level 11.4m)

6.3 Sensitivity to Floor Level Uncertainty

In order to understand the importance of potential bias in the floor levels database (as described in section 4.2.1), some analyses have been repeated with an assumption of a 100 mm average bias existing in the database within the study area. In essence, for this evaluation the database floor levels have been raised 100 mm and the life risks recalculated.

Accordingly, life risk analyses have been repeated for:

1. the current stopbank levels (11.2, 11.4 m) and
2. hypothetical stopbank levels of 11.5 m.

The hypothetical levels were considered potentially interesting as the results fall close to the ANCOLD guideline limit for tolerable life risk. The results are presented in Figure 6-3 and Figure 6-4 below.

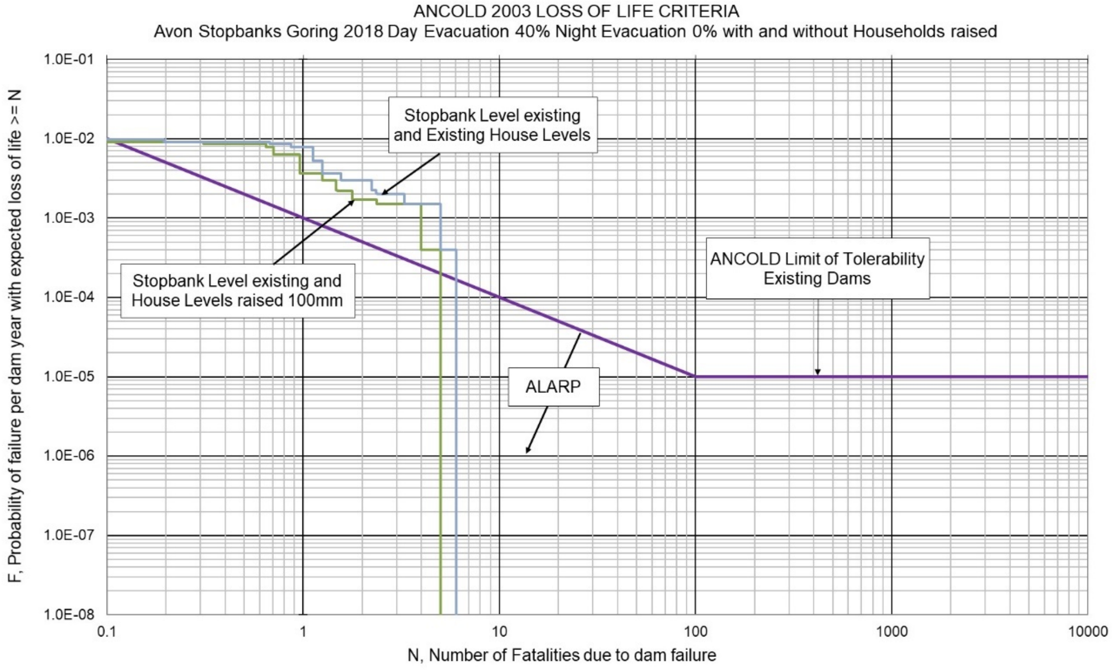


Figure 6-3 Avon Stopbanks comparison of societal risk with floor levels raised by 100 mm compared with the original analyses (Section 1 level at 11.4 m and others 11.2 m)

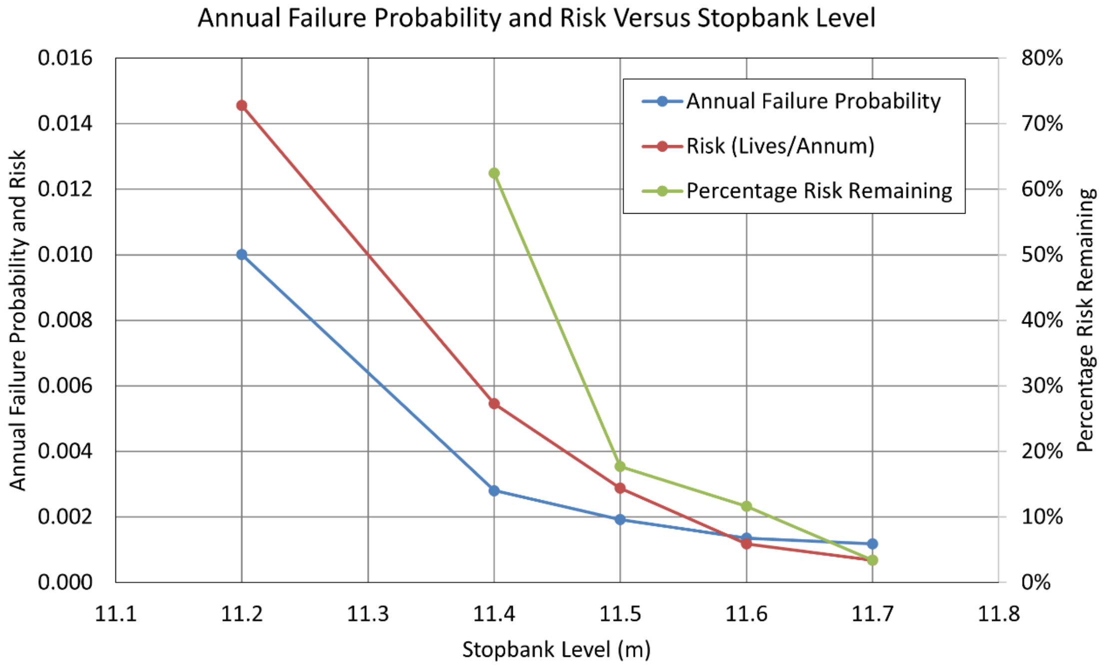


Figure 6-4 Avon Stopbanks comparison of societal risk with floor levels raised by 100 mm compared with the original analyses (all levels at 11.5 m)

These results show the modest sensitivity to a 100 mm assumed average difference in estimated floor levels from this study. The magnitude of change is not sufficient to change conclusions as to achieving the ANCOLD risk guidelines in either case presented in section 5.

7. Risk Treatment

The following options have been evaluated for lowering the risk from events in excess of the design level of service using the Goring Tidal Data:

- Raising the Stopbank Levels, and
- Increased Evacuation using the Emergency Action Plan.

Each of these are presented below. Some discussion is also provided on the alternative to address individual properties with the highest risks, for example through house raising.

All analyses in section 7 use the Goring 2018 central values for extreme sea levels, not the sensitivity testing scenarios.

7.1 Raising Stopbank Levels

The stopbank levee crest level was evaluated using levels of 11.4 m, 11.5 m and 11.6 m for the stopbanks. The results are shown on to Table 7-3 and Figure 7-1 to Figure 7-4 respectively.

Within this section 7.1 and as noted in section 3.2.2, no allowance has been made for wave setup, wave overtopping nor risks from wave margin erosion. As part of the 2015 ITE decision on stopbank upgrades, risks from wind and wave action was anticipated when the stopbanks were increased height to 11.4m in locations as shown on Fig 3-1. It will continue to be prudent for Council to consider and allow for these risks in future stopbank design. Our recommendations for stopbank levels are applicable in areas where there is negligible wave activity.

This data presented below for the risk reduction option of raising the stopbank level shows the following:

- The significance of the stopbank overtopping failure mode for all tidal levels
- A steady reduction in annual failure probability and risk for the stopbank level raised to RL 11.6 m followed by reduced change in risk reduction to RL 11.7 m, and
- The level of acceptance for the Societal risk according to ANCOLD risk tolerability is achieved with the Stopbank level raised to RL 11.6 m.

Table 7-1 Summary of annual failure probabilities and risk for each section and failure mode with stopbank levels raised to 11.4 m

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.4m Others at 11.4m							
Failure Mode	Stopbank Section	Failure Probability			Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	2.82E-08	0.001%		1.31E-08	0.000%	
	Section 2	7.91E-08	0.003%		5.88E-09	0.000%	
	Section 3	2.88E-09	0.000%		2.88E-09	0.000%	
	Section 4	1.64E-08	0.001%		4.68E-09	0.000%	
	Section 17	3.00E-07	0.011%		5.62E-08	0.001%	
	Section 18	3.84E-07	0.014%	0.029%	7.76E-08	0.001%	0.003%
Foundation Piping	Section 1	1.24E-04	4.432%		5.67E-06	0.104%	
	Section 2	7.10E-05	2.534%		4.13E-06	0.076%	
	Section 3	2.35E-07	0.008%		3.24E-07	0.006%	
	Section 4	5.11E-05	1.822%		2.59E-06	0.048%	
	Section 17	2.11E-04	7.514%		4.46E-05	0.821%	
	Section 18	5.44E-04	19.400%	35.710%	1.30E-04	2.385%	3.440%
Earthquake Overtopping	Section 1	3.63E-07	0.013%		1.20E-06	0.022%	
	Section 2	4.38E-07	0.016%		1.24E-06	0.023%	
	Section 3	1.79E-06	0.064%		5.14E-06	0.094%	
	Section 4	2.05E-07	0.007%		7.65E-07	0.014%	
	Section 17	2.47E-06	0.088%		3.28E-06	0.060%	
	Section 18	3.86E-06	0.138%	0.326%	4.71E-06	0.087%	0.301%
Embankment Piping	Section 1	4.40E-10	0.000%		1.39E-10	0.000%	
	Section 2	1.12E-09	0.000%		8.65E-11	0.000%	
	Section 3	3.12E-12	0.000%		1.85E-12	0.000%	
	Section 4	2.93E-10	0.000%		5.84E-11	0.000%	
	Section 17	6.39E-09	0.000%		1.16E-09	0.000%	
	Section 18	4.41E-09	0.000%	0.000%	9.73E-10	0.000%	0.000%
Embankment Overtopping	Section 1	3.65E-04	13.003%		1.33E-03	24.533%	
	Section 2	2.38E-04	8.477%		8.93E-04	16.430%	
	Section 3	2.38E-04	8.477%		8.93E-04	16.430%	
	Section 4	2.38E-04	8.477%		8.93E-04	16.430%	
	Section 17	3.24E-04	11.547%		5.58E-04	10.271%	
	Section 18	3.91E-04	13.954%	63.935%	6.61E-04	12.165%	96.257%
Total	Section 1	4.89E-04	17.45%		1.34E-03	24.66%	
	Section 2	3.09E-04	11.03%		8.99E-04	16.53%	
	Section 3	2.40E-04	8.55%		8.99E-04	16.53%	
	Section 4	2.89E-04	10.31%		8.97E-04	16.49%	
	Section 17	5.37E-04	19.16%		6.06E-04	11.15%	
	Section 18	9.39E-04	33.51%		7.96E-04	14.64%	
Total all tidal events		2.80E-03	100.0%	100.0%	5.44E-03	100.0%	100.0%

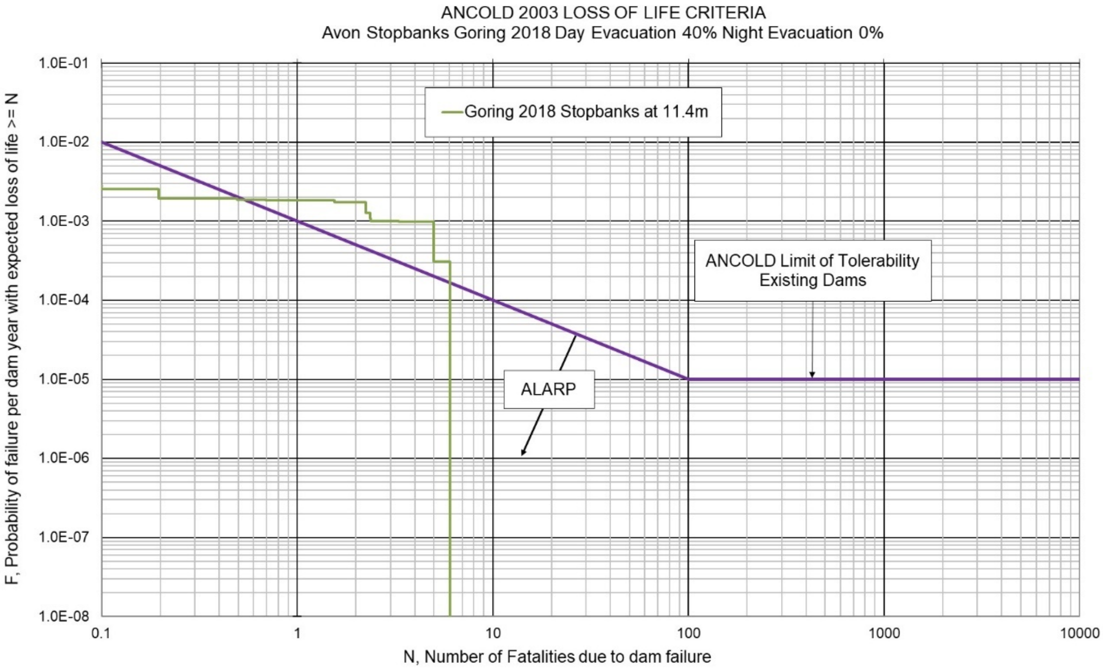


Figure 7-1 Avon stopbanks societal risk – all sections at 11.4m

Table 7-2 Summary of annual failure probabilities and risk for each section and failure mode with stopbank levels raised to 11.5 m

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.5m Others at 11.5m							
Failure Mode	Stopbank Section	Failure Probability			Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	2.05E-08	0.001%		1.07E-08	0.000%	
	Section 2	5.46E-08	0.003%		5.10E-09	0.000%	
	Section 3	1.45E-09	0.000%		2.12E-09	0.000%	
	Section 4	8.17E-09	0.000%		3.87E-09	0.000%	
	Section 17	2.49E-07	0.013%		4.73E-08	0.002%	
	Section 18	2.64E-07	0.014%	0.031%	5.72E-08	0.002%	0.004%
Foundation Piping	Section 1	1.24E-04	6.475%		5.67E-06	0.198%	
	Section 2	7.10E-05	3.702%		4.13E-06	0.144%	
	Section 3	2.35E-07	0.012%		3.24E-07	0.011%	
	Section 4	5.11E-05	2.662%		2.59E-06	0.090%	
	Section 17	2.11E-04	10.979%		4.46E-05	1.555%	
	Section 18	5.44E-04	28.345%	52.175%	1.30E-04	4.515%	6.513%
Earthquake Overtopping	Section 1	1.83E-07	0.010%		7.09E-07	0.025%	
	Section 2	2.09E-07	0.011%		7.26E-07	0.025%	
	Section 3	9.11E-07	0.047%		3.18E-06	0.111%	
	Section 4	8.56E-08	0.004%		3.32E-07	0.012%	
	Section 17	1.21E-06	0.063%		1.90E-06	0.066%	
	Section 18	1.70E-06	0.089%	0.224%	2.57E-06	0.089%	0.328%
Embankment Piping	Section 1	4.40E-10	0.000%		1.39E-10	0.000%	
	Section 2	1.12E-09	0.000%		8.65E-11	0.000%	
	Section 3	3.12E-12	0.000%		1.85E-12	0.000%	
	Section 4	2.93E-10	0.000%		5.84E-11	0.000%	
	Section 17	6.39E-09	0.000%		1.16E-09	0.000%	
	Section 18	4.41E-09	0.000%	0.001%	9.73E-10	0.000%	0.000%
Embankment Overtopping	Section 1	2.10E-04	10.944%		8.07E-04	28.118%	
	Section 2	9.92E-05	5.168%		3.81E-04	13.278%	
	Section 3	9.92E-05	5.168%		3.81E-04	13.278%	
	Section 4	9.92E-05	5.168%		3.81E-04	13.278%	
	Section 17	1.66E-04	8.628%		2.96E-04	10.294%	
	Section 18	2.40E-04	12.494%	47.569%	4.28E-04	14.908%	93.154%
Total	Section 1	3.34E-04	17.43%		8.14E-04	28.34%	
	Section 2	1.70E-04	8.88%		3.86E-04	13.45%	
	Section 3	1.00E-04	5.23%		3.85E-04	13.40%	
	Section 4	1.50E-04	7.83%		3.84E-04	13.38%	
	Section 17	3.78E-04	19.68%		3.42E-04	11.92%	
	Section 18	7.86E-04	40.94%		5.60E-04	19.52%	
Total all tidal events		1.92E-03	100.0%	100.0%	2.87E-03	100.0%	100.0%

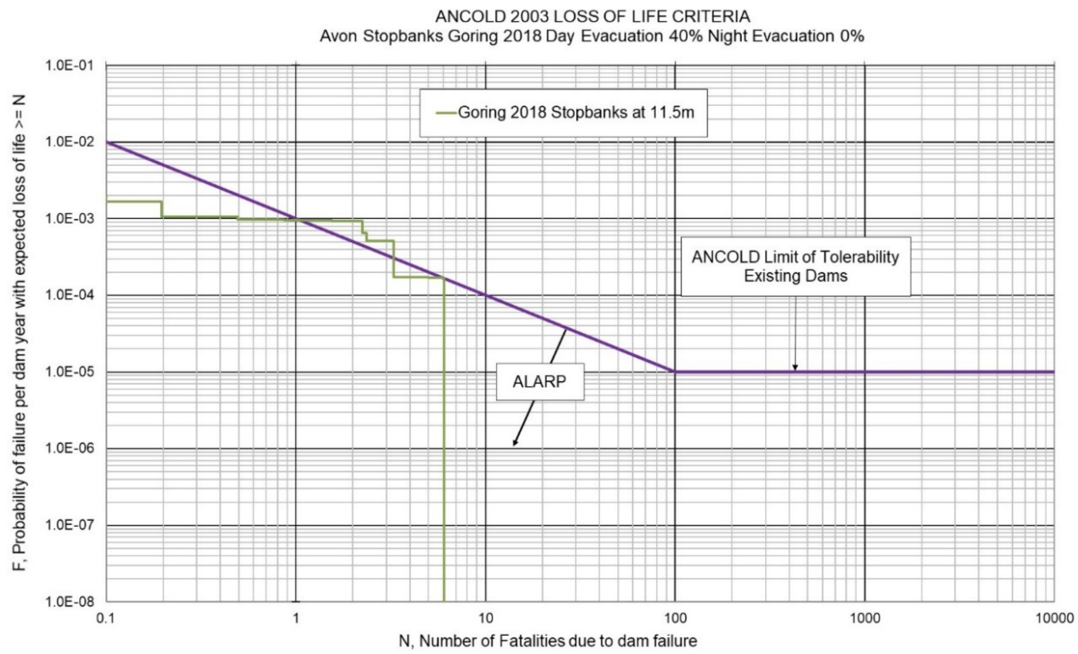


Figure 7-2 Avon Stopbanks societal risk all sections 11.5 m

Table 7-3 Summary of annual failure probabilities and risk for each section and failure mode with stopbank levels raised to 11.6 m

Day Evacuation 40% Night Evacuation 0% Section 1 at 11.6m Others at 11.6m								
Failure Mode	Stopbank Section	Failure Probability				Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Total	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	1.45E-08	0.001%		15.49%	8.63E-09	0.001%	
	Section 2	4.21E-08	0.003%		7.60%	4.36E-09	0.000%	
	Section 3	8.88E-10	0.000%		2.37%	1.55E-09	0.000%	
	Section 4	5.85E-09	0.000%		6.11%	3.16E-09	0.000%	
	Section 17	2.03E-07	0.015%		20.08%	3.92E-08	0.003%	
	Section 18	1.95E-07	0.014%	0.034%	48.34%	4.43E-08	0.004%	0.009%
Foundation Piping	Section 1	1.24E-04	9.213%			5.67E-06	0.482%	
	Section 2	7.10E-05	5.267%			4.13E-06	0.352%	
	Section 3	2.35E-07	0.017%			3.24E-07	0.028%	
	Section 4	5.11E-05	3.787%			2.59E-06	0.221%	
	Section 17	2.11E-04	15.620%			4.46E-05	3.797%	
	Section 18	5.44E-04	40.327%	74.231%		1.30E-04	11.026%	15.905%
Earthquake Overtopping	Section 1	7.59E-08	0.006%			2.94E-07	0.025%	
	Section 2	8.44E-08	0.006%			3.18E-07	0.027%	
	Section 3	3.64E-07	0.027%			1.40E-06	0.119%	
	Section 4	2.72E-08	0.002%			1.06E-07	0.009%	
	Section 17	4.92E-07	0.036%			8.55E-07	0.073%	
	Section 18	8.26E-07	0.061%	0.139%		1.43E-06	0.122%	0.374%
Embankment Piping	Section 1	4.40E-10	0.000%			1.39E-10	0.000%	
	Section 2	1.12E-09	0.000%			8.65E-11	0.000%	
	Section 3	3.12E-12	0.000%			1.85E-12	0.000%	
	Section 4	2.93E-10	0.000%			5.84E-11	0.000%	
	Section 17	6.39E-09	0.000%			1.16E-09	0.000%	
	Section 18	4.41E-09	0.000%	0.001%		9.73E-10	0.000%	0.000%
Embankment Overtopping	Section 1	8.46E-05	6.272%			3.25E-04	27.659%	
	Section 2	3.14E-05	2.325%			1.21E-04	10.254%	
	Section 3	3.14E-05	2.325%			1.21E-04	10.254%	
	Section 4	3.14E-05	2.325%			1.21E-04	10.254%	
	Section 17	5.94E-05	4.407%			1.06E-04	9.026%	
	Section 18	1.07E-04	7.941%	25.596%		1.91E-04	16.264%	83.711%
Total	Section 1	2.09E-04	15.49%			3.31E-04	28.17%	
	Section 2	1.03E-04	7.60%			1.25E-04	10.63%	
	Section 3	3.20E-05	2.37%			1.22E-04	10.40%	
	Section 4	8.25E-05	6.11%			1.23E-04	10.48%	
	Section 17	2.71E-04	20.08%			1.52E-04	12.90%	
	Section 18	6.52E-04	48.34%			3.22E-04	27.42%	
Total all tidal events		1.35E-03	100.0%	100.0%		1.18E-03	100.0%	100.0%

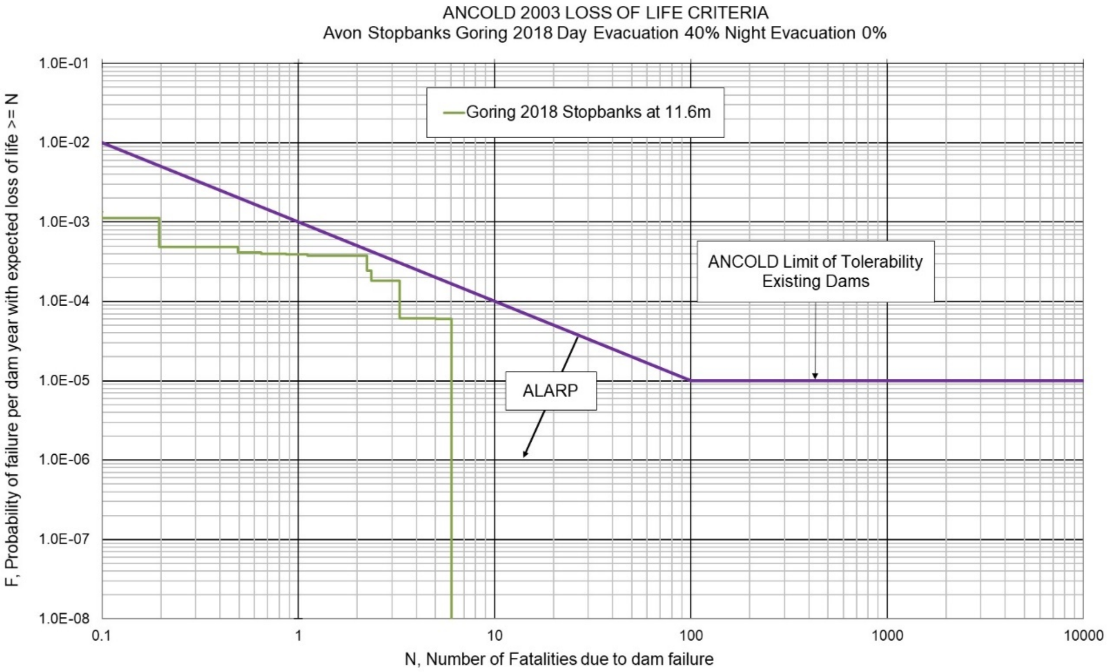


Figure 7-3 Avon Stopbanks societal risk all sections 11.6 m

The total annual failure probability and risk to life for each Stopbank raised level, together with the risk remaining (reduced) in percent for each Stopbank level relative to the existing condition are shown on Figure 7-4 and Table 7-4.

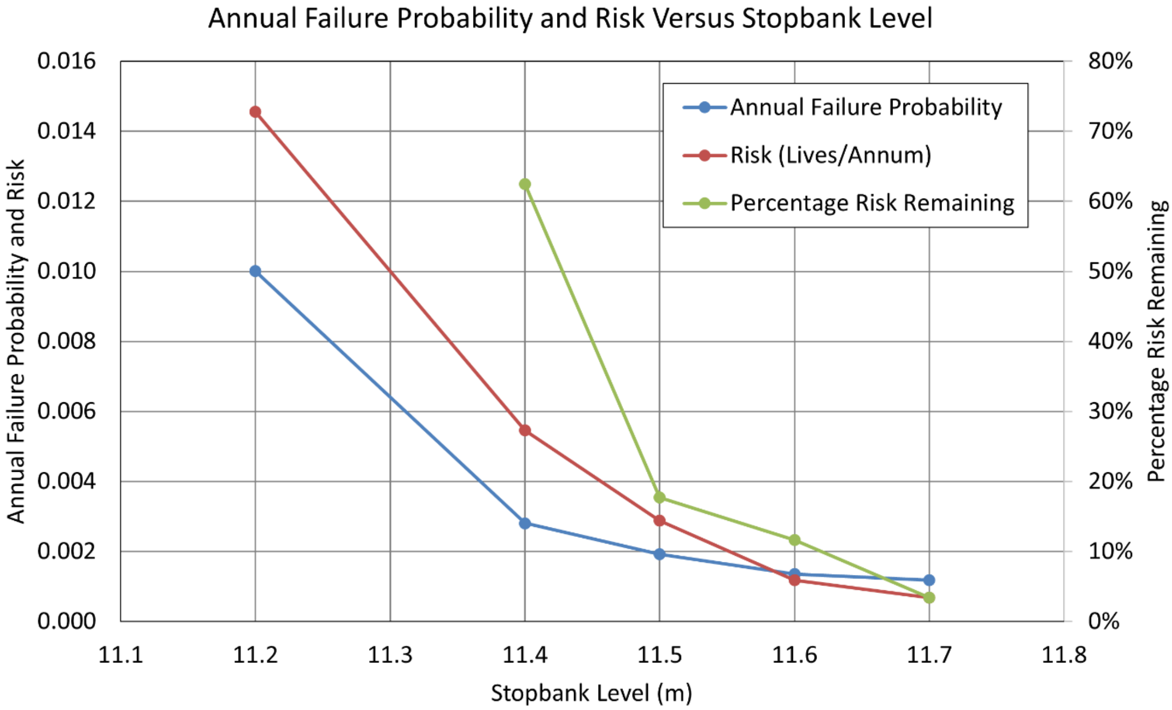


Figure 7-4 Avon Stopbanks annual failure probability and risk versus stopbank level

Table 7-4 Avon Stopbanks annual failure probability and risk reduction data for varying stopbank raised levels

Stopbank level (m)	Annual failure probability	Risk (lives/annum)	Risk remaining (reduced)
11.2	10.0E-03	14.50E-03	
11.4	2.81E-03	5.46E-03	62.3%
11.5	1.92E-03	2.88E-03	17.8%
11.6	1.35E-03	1.18E-03	11.7%
11.7	1.18E-03	0.68E-03	3.4%

7.2 Improved Evacuation of the Population at Risk

An analysis was completed for the potential life loss, assuming that 95% evacuation of the population at risk is possible day or night. By comparison the standard evacuation outcome used in our main conclusions are for 40% evacuation during the day and nil evacuation at night.

The population at risk and to be evacuated is presented on Table 4-4 (page 21). The resulting Potential Life Loss with evacuations are shown on Figure 7-5. The 95% evacuation outcome produced an acceptable risk to life whereas a 90% evacuation outcome still faileds the ANCOLD criteria. The data shown extended using the dashed line is calculated from our extrapolated tide level beyond the Goring published 1 in 10,000 AEP tide event.

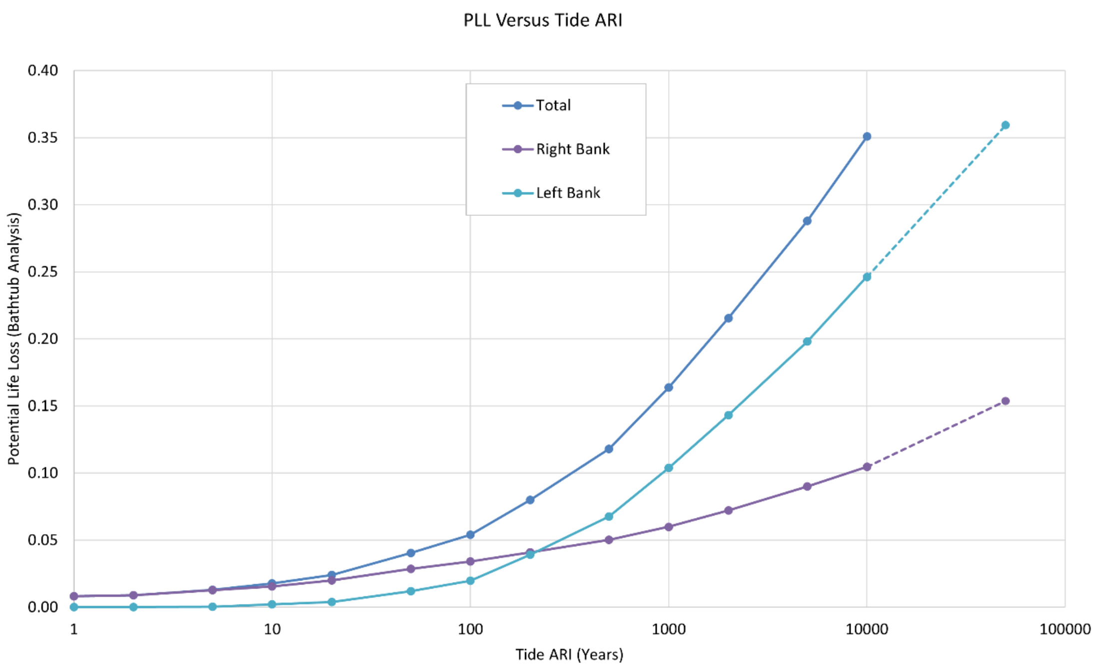


Figure 7-5 Potential Life Loss given tidal events with 95% evacuation day and night

The risk analysis was evaluated using this reduced PLL and the results are shown on

Table 7-5 and Figure 7-6, which show that the risk is below the Tolerable Limit and within the zone requiring ALARP consideration for further reduction in risk using an evacuation rate of 95% of the population at risk.

Table 7-5 Summary of annual failure probabilities and risk for each section and failure mode with stopbank levels section 1 at 11.4 m and remaining sections at 11.2 m with 95% evacuation of PAR

Day Evacuation 95% Night Evacuation 95% Section 1 at 11.4m Others at 11.2m							
Failure Mode	Stopbank Section	Failure Probability			Risk (lives/Annum)		
		Annual Failure Probability	Percentage of Total	Percentage	Risk	Percentage of Total	Percentage
Piping Earthquake	Section 1	2.81E-08	0.000%		7.26E-10	0.000%	
	Section 2	1.41E-07	0.001%		4.22E-10	0.000%	
	Section 3	7.51E-09	0.000%		2.78E-10	0.000%	
	Section 4	6.62E-08	0.001%		3.71E-10	0.000%	
	Section 17	4.33E-07	0.004%		4.54E-09	0.001%	
	Section 18	6.38E-07	0.006%	0.013%	7.03E-09	0.001%	0.002%
Foundation Piping	Section 1	1.24E-04	1.258%		2.66E-07	0.032%	
	Section 2	7.09E-05	0.719%		1.96E-07	0.024%	
	Section 3	2.32E-07	0.002%		1.85E-08	0.002%	
	Section 4	5.09E-05	0.516%		1.07E-07	0.013%	
	Section 17	2.10E-04	2.133%		2.54E-06	0.306%	
	Section 18	5.36E-04	5.431%	10.058%	6.63E-06	0.799%	1.175%
Earthquake Overtopping	Section 1	3.10E-07	0.003%		6.03E-08	0.007%	
	Section 2	2.44E-06	0.025%		2.08E-07	0.025%	
	Section 3	1.03E-05	0.104%		9.03E-07	0.109%	
	Section 4	1.03E-06	0.010%		1.29E-07	0.016%	
	Section 17	1.37E-05	0.139%		6.88E-07	0.083%	
	Section 18	2.28E-05	0.231%	0.512%	1.03E-06	0.124%	0.364%
Embankment Piping	Section 1	4.39E-10	0.000%		7.76E-12	0.000%	
	Section 2	1.12E-09	0.000%		4.67E-12	0.000%	
	Section 3	2.88E-12	0.000%		5.54E-14	0.000%	
	Section 4	2.93E-10	0.000%		3.23E-12	0.000%	
	Section 17	6.39E-09	0.000%		6.70E-11	0.000%	
	Section 18	4.41E-09	0.000%	0.000%	5.60E-11	0.000%	0.000%
Embankment Overtopping	Section 1	2.94E-04	2.978%		6.43E-05	7.746%	
	Section 2	1.21E-03	12.239%		1.52E-04	18.261%	
	Section 3	1.21E-03	12.239%		1.52E-04	18.261%	
	Section 4	1.21E-03	12.239%		1.52E-04	18.261%	
	Section 17	1.94E-03	19.646%		1.22E-04	14.728%	
	Section 18	2.97E-03	30.075%	89.416%	1.76E-04	21.202%	98.459%
Total	Section 1	4.18E-04	4.24%		6.47E-05	7.79%	
	Section 2	1.28E-03	12.98%		1.52E-04	18.31%	
	Section 3	1.22E-03	12.35%		1.53E-04	18.37%	
	Section 4	1.26E-03	12.77%		1.52E-04	18.29%	
	Section 17	2.16E-03	21.92%		1.26E-04	15.12%	
	Section 18	3.53E-03	35.74%		1.84E-04	22.13%	
Totals		1.97E-02	200.000%	100.000%	1.66E-03	200.000%	100.000%

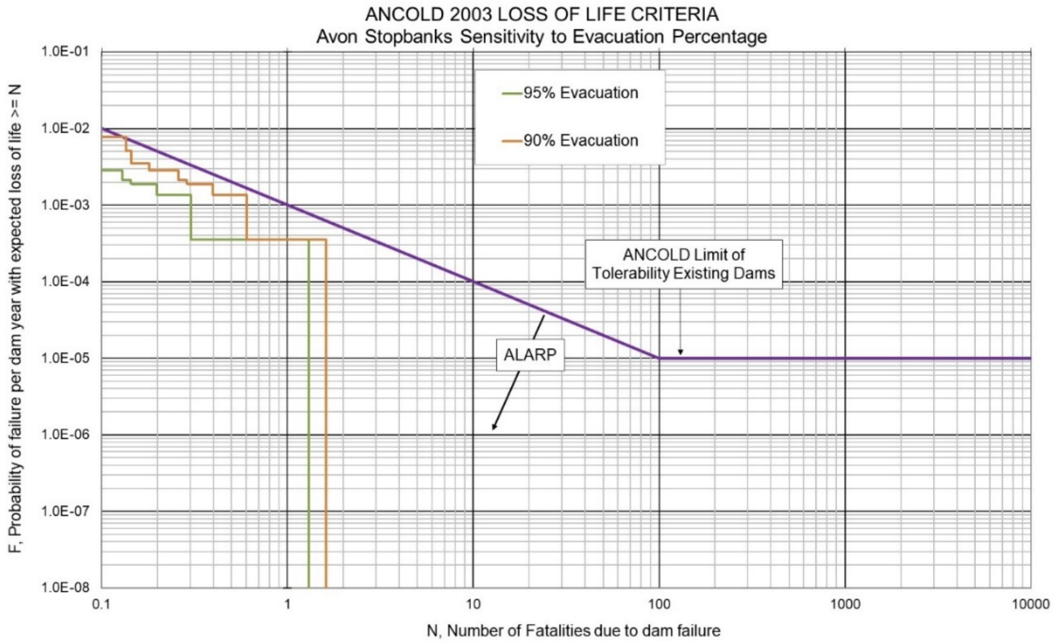


Figure 7-6 Avon Stopbanks societal risk section 1 level at 11.4 m and others 11.2m with 90% and 95% evacuation of PAR

Given lack of natural urgency to avoid dangers associated with relatively modest flooding, reliably achieving a 95% night-time evacuation outcome would be a significant challenge for Council. This would require significant improvements to evacuation planning and associated communications, responsibilities and action trigger levels.

Demonstrating that enhanced evacuation planning could reasonably be expected to achieve the required outcome would require experienced professional assessment using advanced techniques such as LifeSim. Such an outcome is technically achievable but may not be feasible give real life constraints.

7.3 Addressing Properties at Risk

Addressing specific properties at risk of flooding, by removing the people or raising the floor levels, is a potential risk treatment. Either approach would be expected to have a greater capital cost than improving the stopbanks and there also many social and community factors to consider. This approach may be worth considering in areas where large lengths of stopbank protect small numbers of houses. However no such options have been analysed in this report.

8. Conclusions

An appropriate risk assessment has been carried out on the section of stopbanks between Pages Rd and Bridge St. This assessment updated previous work reported in 2016. The update is primarily motivated by a new understanding of tidal risks produced by Goring in 2018.

The risks associated with tides and seismic deformation (including lateral spread, settlement and cracking) have been included in the analysis.

The following potentially relevant risks have been excluded by assumption and/or necessity, these are

- Risks associated with wave action, (which Council have addressed separately to date), and the
- Macro risk of land uplift or down-thrust caused by tectonic plate movements.

There is currently some short-term maintenance work that is planned. The findings of this risk assessment will be applicable after this is completed.

The stopbanks are designed to manage flooding up to a 100 year ARI event and the societal risk for tidal levels below this is about an order of magnitude below (better than) the ANCOLD limit of tolerability and is, therefore, currently acceptable. This means that current risks to life would have to worsen by approximately ten times in order to exceed ANCOLD guidelines for limits on tolerable risk to life.

The findings in relation to total risk, including events greater than the stopbank design standard, such as a 1,000 year ARI sea level at 11.39m, show that the current day Societal Risks to life would not be acceptable according to the ANCOLD guidelines for acceptable risk to life. This finding is reflective of broader flood risks beyond events that the stopbanks are designed to manage.

Uncertainties in the assessment have been characterised through sensitivity analyses and these show that the uncertainties are significant and that the actual societal risks to life might be above or below the ANCOLD guidelines for tolerability.

The Individual Risk was calculated to be 6.5.0E-5 (6.5 in 100,000), which is acceptable given that it is below the ANCOLD limit of tolerability for Individual Risk of 10.0E-5 (10 in 100,000).

Three risk treatments have been discussed and two of those evaluated to determine how the Societal Risk situation might be improved to achieve compliance with the ANCOLD guidelines on tolerable risk.

The ANCOLD guideline is used to help inform Council decision making. Further assessments of practicality, cost and community preference for risk treatment options may be motivated if Council prefer to further reduce risks.

9. Recommendations

From this assessment, GHD make the following recommendations for Council's consideration

- Set guidelines as to what relevance ANCOLD risk tolerance should have in driving Council decisions in relation to decisions on stopbank design performance and risk assessment.
  - Such a guideline may imply further stopbank improvements and further evaluation to choose the best form of improvements, undertaking implementation works and confirming that the resulting risks are acceptable
  - As part of the decision for choosing stopbank improvements, Council should also consider predictable future risks so as to understand the longevity of improvements
- Such a guideline may also motivate ANCOLD risk assessment of other sections of the Avon stopbanks system. There have been material changes in physical conditions, modelling improvements, changes in risks and improvements in risk methodology since the 2015 risk assessment. It would be timely to reassess the full length of the Avon stopbanks system.
- Design of any future stopbank replacement, improvement or upgrades, should ensure that resulting risks are acceptable to Council through reasonable future design scenarios for a given design life

10. References

[2] Jonkman S.N, Vrijling J. K, Vrouwenvelder A. C. W. M., Article in Natural Hazards · September 2008 DOI: 10.1007/s11069-008-9227-5 "Methods for the estimation of loss of life due to floods: a literature review and a proposal for a new method"

[3] Extreme Sea Levels at Christchurch Sites: EV1 Analysis, Mulgor Consulting Ltd, 24-Jul-2018

[4] Joint Risks of Pluvial and Tidal Flooding Report, Rev 0, February 2021, GHD and HKV

GHD “Stopbank Levees Risk Assessment” Barker and Williams, Rev 0, Sep 2016.

GHD “Temporary Stopbank Management Options” Webb and Dasler Rev 2, Sep 2016

Piping Toolbox “A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping” Draft Guidance Document dated August 21, 2008, (US Army Corps of Engineers et al).

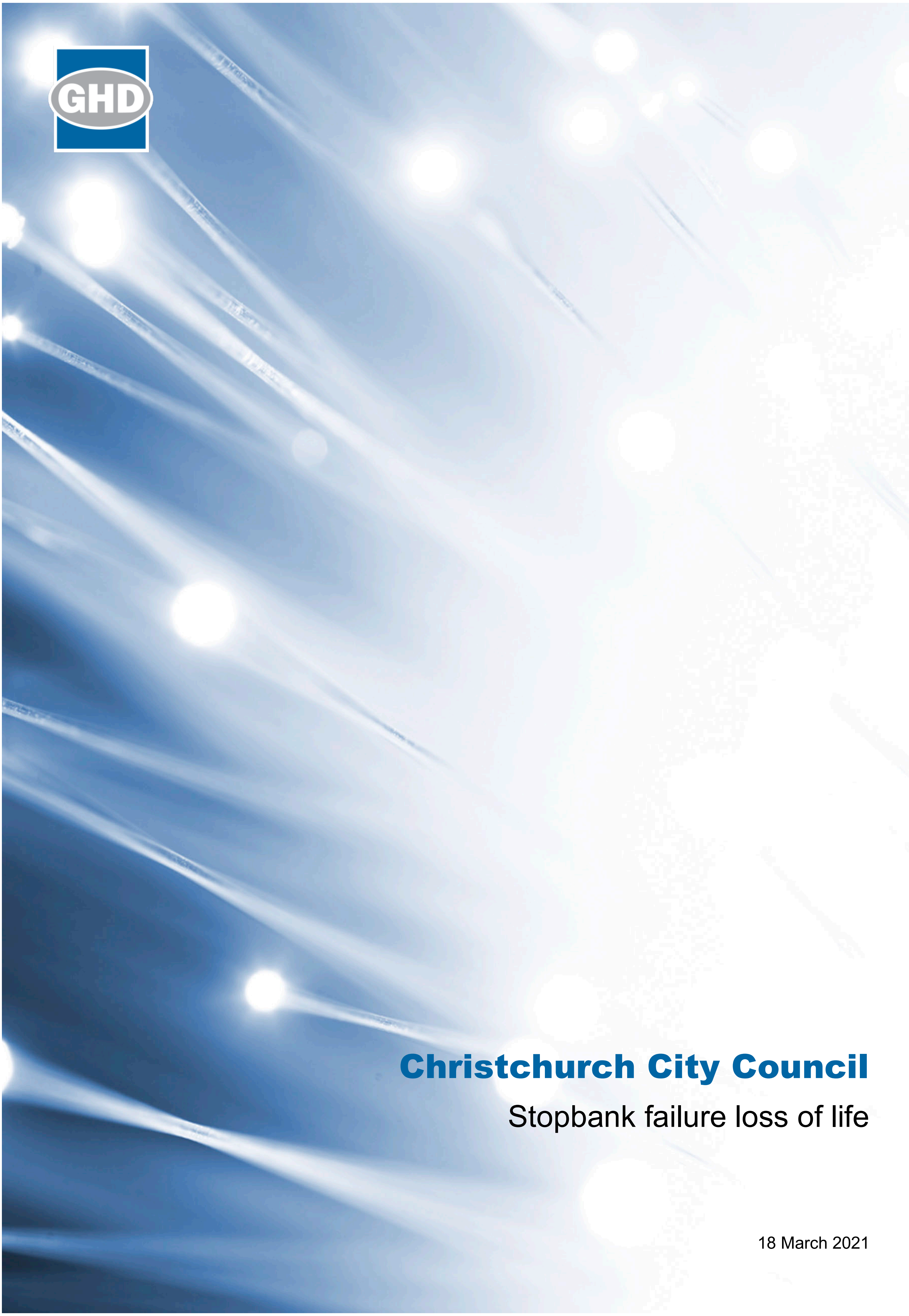
Ang, A.H.-S. and Tang, W.H. (1975). Probability Concepts in Engineering Planning and Design, Vol. 1, Basic Principles, John Wiley, New York.

Bowles et al (2001): "On the Art of Event Tree Modelling for Portfolio Risk Analysis" Hill, Bowles et al., NZSOLD/ANCOLD Conference 2001.

Appendices

Appendix A – Stopbank Failure Loss of Life Review report

"Stopbank Failure Loss of Life Review" report by Malcolm Barker, Rev4



Christchurch City Council

Stopbank failure loss of life

18 March 2021

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1. Background

The potential for the Avon stopbank failure to result in life loss is one of the two factors in evaluating the risk associated with the operation of the stopbank when dealing with tidal fluctuations and floods. There is no universal approach for evaluating the fatality rates associated with stopbank failures.

Figure 1-1, as presented by Jonkman (2008) provides an overview of the life loss estimating methods currently in use. The complexity of the methods increases along the horizontal scale as the level of detail increases on the vertical scale. The most complex mechanistic approaches (BC Hydro LSM, and LifeSim) provide real time visualisation of the vehicular movement and potential life loss as the flood wave travels through the areas of population at risk. These methods are appropriate where there are large populations at risk in close proximity to a dam or stopbank or where high life losses are likely to occur. In the case of the Avon Stopbank system, the flood magnitude and rate of flooding would not warrant the sophistication of these models.

The empirical approaches are presented in this memorandum with the intention of providing a way forward for estimating the life loss resulting from the failure of the Avon stopbank during tidal events when piping and overtopping failure modes could occur.

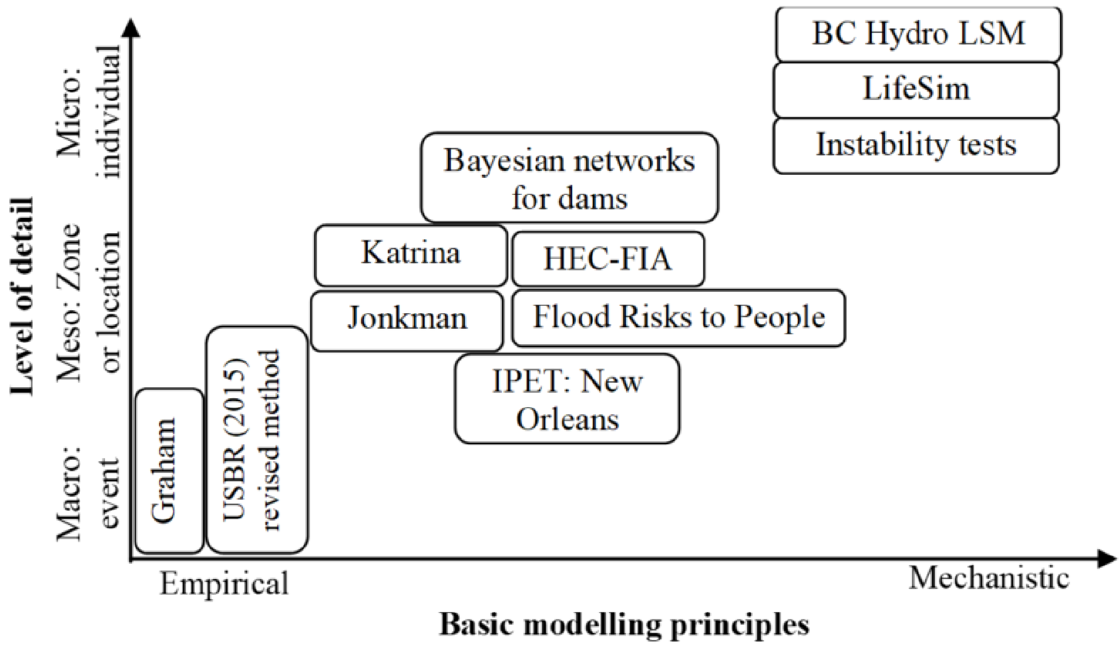

Figure 1-1 Methods for estimating loss of life (Jonkman 2008)

2. Empirical methods for estimating life loss

2.1 Sources reviewed

The following references have been reviewed:

- ANCOLD Risk Guidelines (2003 and 2020 draft version under review)
- Australian Rainfall and Runoff 2013 and 2016
- Lee et al 1986: Institute of Water Resources, U.S. Army Corps of Engineers
- Dekay and McClelland 1993
- Wayne Graham USBR 1999: A Procedure for Estimating Loss of Life Caused by Dam Failure DSO-99-06
- Reclamation Consequence Estimating Methodology (RCEM 2014/15)
- DEFRA Guide: UK Department for Environment Flood and Rural Affairs (DEFRA) Guide to risk assessment for reservoir safety management Volume 2: Methodology and supporting information Report – SC090001/R2
- DEFRA Supplementary 2008: UK Department for Environment Flood and Rural Affairs (DEFRA) Flood and Coastal Defence Appraisal Guidance Social Appraisal Supplementary Note to Operating Authorities Assessing and Valuing the Risk to Life from Flooding for Use in Appraisal of Risk Management Measures, May 2008
- Jonkman Mar 2008: Jonkman S.N. and Vrijling J.K. - Flood Risk Management Journal (Mar 2008)
- Jonkman Sep 2008: Jonkman S.N, Vrijling J. K, Vrouwenvelder A. C. W. M., Article in Natural Hazards · September 2008 DOI: 10.1007/s11069-008-9227-5

2.2 ANCOLD Risk Guidelines

The ANCOLD Guidelines (from 2003 and the 2020 draft version under review) do not specifically address the PLL estimates for low height embankments or stopbanks; however, the following references are provided for the PLL from flood events where dam breach has not occurred.

2.2.1 Fatality rates from Dartmouth Floods Observatory

NSW Dams Safety Committee (2005) and Hill et al (2007) describe analysis of fatality rates inferred from the databases of Dartmouth Flood Observatory (DFO) in New Hampshire, USA, which collates information on large floods from around the world (www.dartmouth.edu/~floods). Hill et al (2007) derived indicative fatality rates as a function of severity of flooding and warning time in a manner which is consistent with DSO-99-06 (Graham 1999), as shown on Table 2-1.

Table 2-1 Recommended indicative fatality rates for natural flooding (Hill et al 2007 after Graham 1999)

Flood severity	Warning time (minutes)	Flood severity understanding	Fatality rate
Medium	No warning	Not applicable	0.03
	15-60	Vague	0.01
		Precise	0.005
	More than 60	Vague	0.005
		Precise	0.002
Low	All	All	0.0002

2.2.2 Reclamation Consequence Estimating Methodology

The Reclamation Consequence Estimating Methodology (RCEM 2014/15) approach provides upper and lower bound fatality rates as a function of depth and velocity (DV) and so one reasonable approach is to adopt the lower bound to estimate PLL for non-failure scenarios. If RCEM is being used to estimate the PLL for the failure scenarios, then one benefit of this approach is that the resulting values for the no failure scenarios will be consistent with the values for the failure scenarios which, should result in plausible values of incremental PLL. The RCEM figures are reproduced below in metric units, as presented by HARC (North Pine Dam - Options Analysis and Concept Design, Detailed Consequence Assessment, 10/1/2019). The figures also include the data from Graham DSO-99-06 together with the arithmetric and graphical midpoint of the RCEM range. Further discussion on the RCEM data is provided below in section 2.5.

As shown on the figure for little/No warning below, the fatality rates have been truncated horizontally at the level where the original REM curves were stopped as the Consultant concluded that there was no data to substantiate the trend line continuing downwards. This is an important conclusion to make given that the DV values for stopbank failures can be lower than the value of 1.0 as shown in the below.

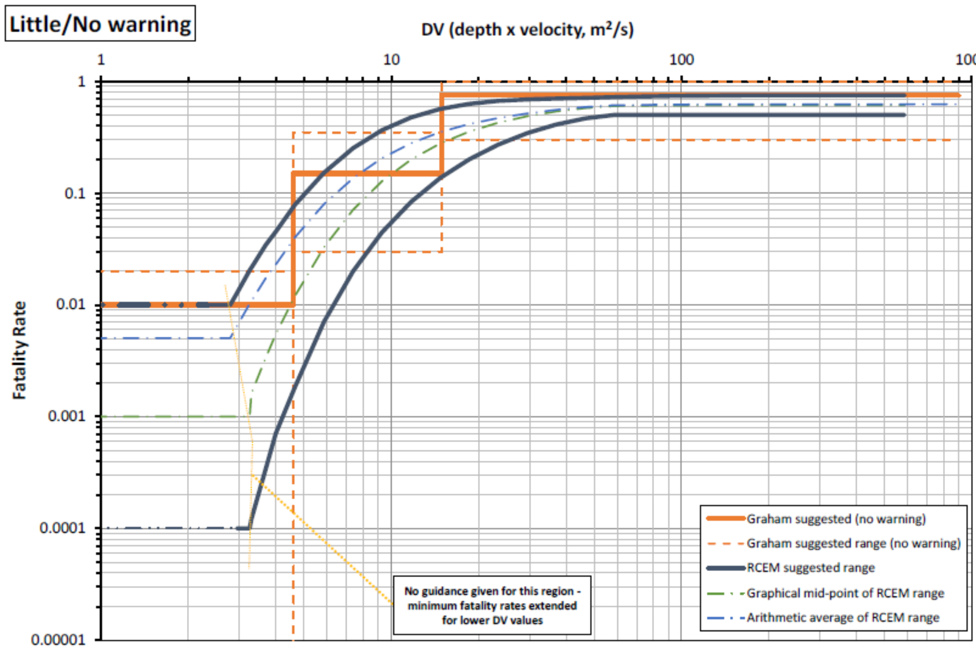

Figure 2-1 RCEM Fatality Rates with Little or No Warning

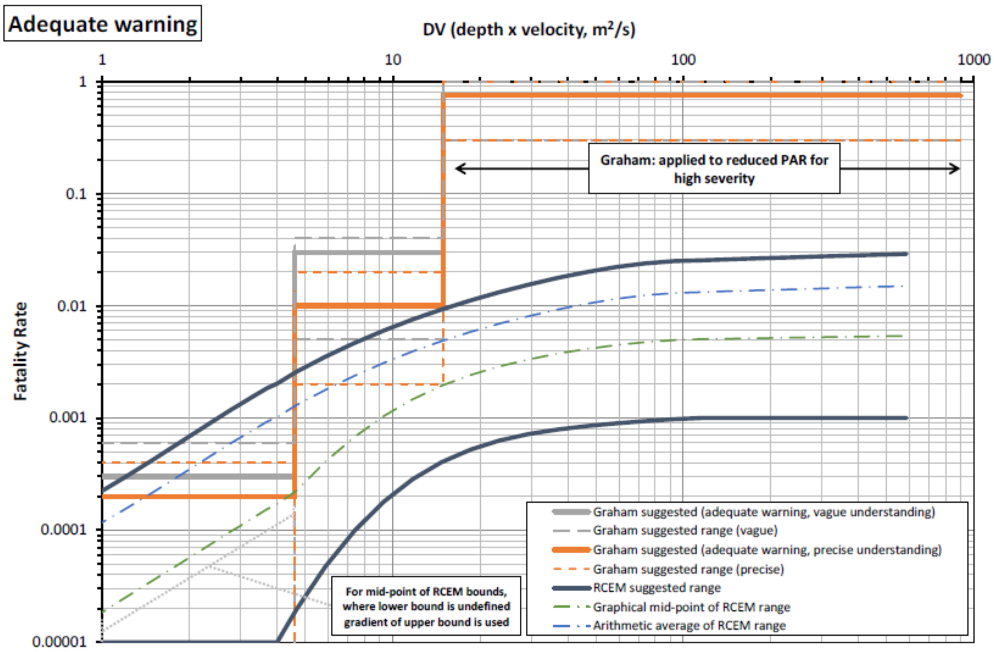

Figure 2-2 RCEM Fatality Rates with Adequate Warning

2.3 Australian Rainfall and Runoff

2.3.1 2013 Chapter 6 Safety Design Criteria - People Stability (AR&R 2010)

The data presented below can be used to evaluate the likelihood of hazardous conditions for people affected by flood waters from which a fatality rate can be assessed using the available data from available sources.

Determining safety criteria for people requires an understanding of the physical characteristics of the subjects along with the nature of the flow.

The best measure of physical attributes for human stability is the parameter H.M (mkg), the product of subject height (H; m) and mass (M; kg) (Cox et al., 2010). The measure of flow attributes is the parameter D.V (m^2/s), the product of flow depth (D, m) and flow velocity (V, m/s).

In order to define safety limits, which are applicable for all persons, hazard regimes are defined based on H.M for representative population demographics. Each classification is based on laboratory testing of subject stability within floodwaters. The following suggested classifications, after Cox et al., 2010) are:

- Adults, where H.M > 50 mkg
- Children, where H.M is between 25 and 50 mkg, and
- Infants and very young children, where H.M < 25 mkg.

These hazard regimes for tolerable flow conditions (D.V) as related to the individual's physical characteristics (H.M) are presented in Table 2-2 and Figure 2-3.

Table 2-2 Flow Hazard Regimes for People (Cox et al, 2010)

DV (m^2s^{-1})	Children (H.M = 25 to 50)[1]	Adults (H.M > 50)
0	Safe	Safe
0 – 0.4	Low hazard if depth < 0.5 m and velocity < 3m/s otherwise extreme hazard	Low hazard if depth < 1.2 and velocity < 3m/s otherwise extreme hazard
0.4 – 0.6	Significant hazard Dangerous to most if depth < 0.5m and velocity < 3m/s otherwise extreme hazard	
0.6 – 0.8	Extreme hazard, dangerous to all	Moderate hazard, dangerous to some[2] if depth < 1.2m and velotity < 3m/s otherwise extreme hazard
0.8 – 1.2		Significant hazard; danterous to most[3] if depth < 1.2m and velocity < 3m/s otherwise extreme hazard
> 1.2		Extreme hazard; dangerous to all

[1] More vulnerable community members such as infants and the elderly should avoid exposure to floodwater. Flood flows are considered extremely hazardous to these community members under all conditions

[2] Working limit for trained safety workers or experienced and well equipped persons (D.V < 0.8 m2/s), and

[3] Upper limit of stability observed during most investigations (D.V > 1.2 m^2/s).

Maximum depth stability limit of 0.5m for children and 1.2m for adults under good conditions.

Maximum velocity stability limit of 3m/s for both adults and children.

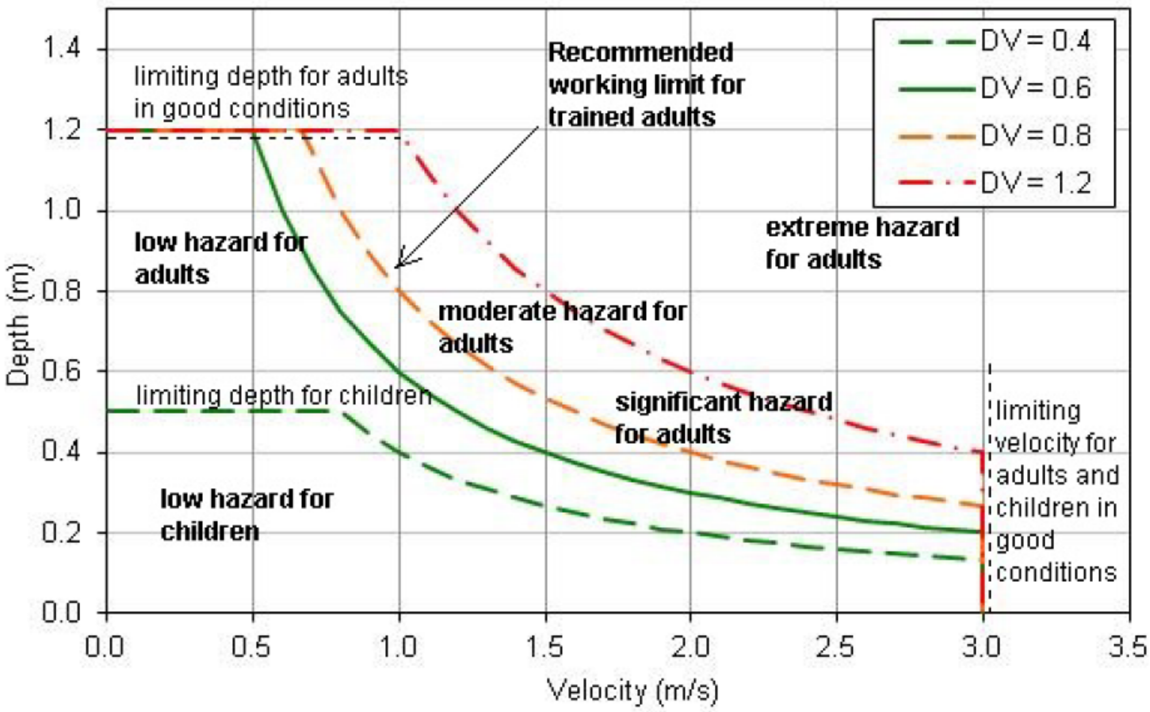

Figure 2-3 Safety criteria for people in variable flow conditions (after Cox et al, 2010)

2.3.2 Australian rainfall and runoff (ARR) guidelines (Ball, et al., 2019)

Further guidance on categorisation of the flood severity is provided in the Australian Rainfall and Runoff (ARR) guidelines as shown below (Ball, J., Babister, M., Nathan, R., Weeks, W., Weinmann, P., Retallick, M., & Testoni, I. (2019). Australian Rainfall and Runoff - A Guide to Flood Estimation. Commonwealth of Australia)

Flood severity is based on the work from 2010 and is described by six hazard classifications, based on the maximum flood depth, velocity and depth-velocity product (DV) at a given location. It is noted that the time of the maximum DV may not coincide with the time at which the maximum inundated depth or velocity occurs.

The adopted hazard classifications are presented in Figure 2-4 and summarised in Table 2-3 and can be used as an aid to assess the potential fatality rate.

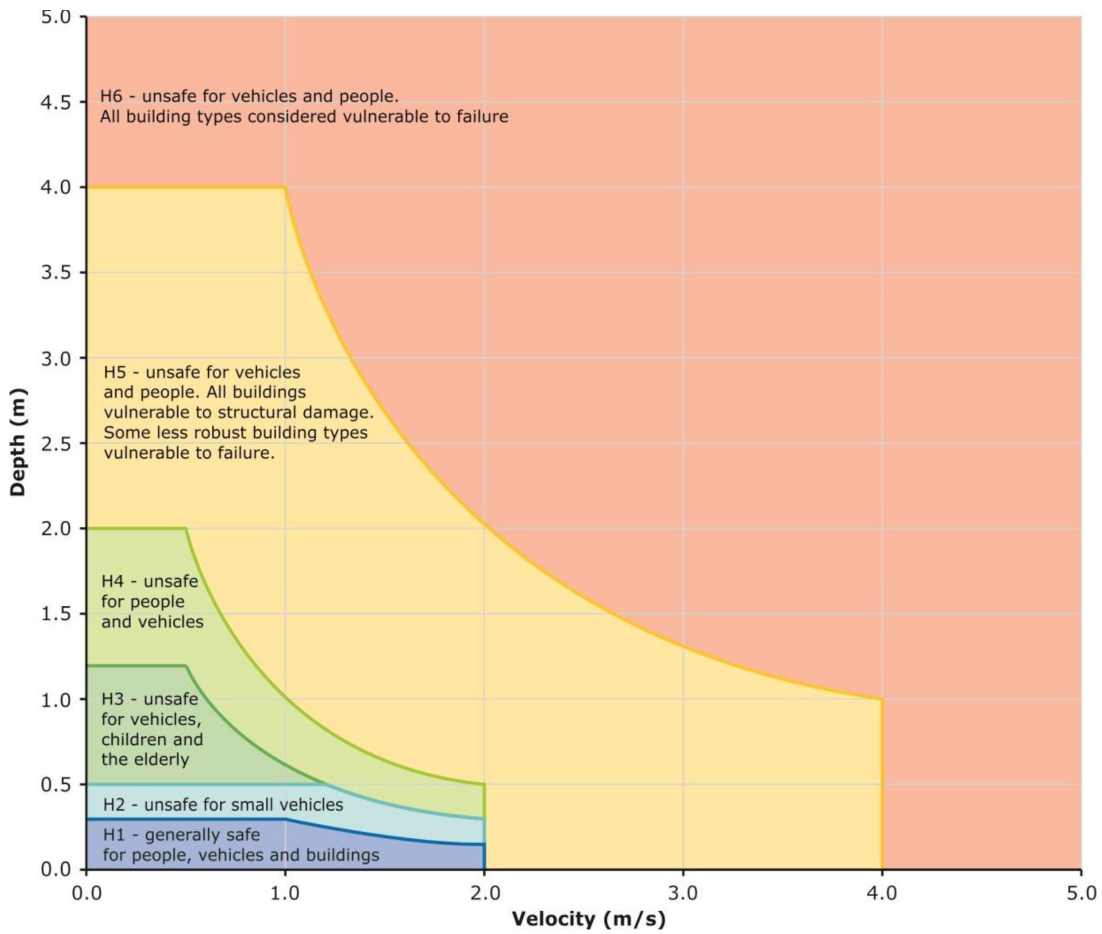

Figure 2-4 Combined flood hazard curves (Smith, Davey, & Cox, 2014)

Table 2-3 Vulnerability thresholds classification limits of flood hazard curves (Smith, Davey, & Cox, 2014)

Hazard vulnerability classification	Description of hazard classification	Depth-velocity product (DV) (m^2/s)	Limiting still water depth (m)	Limiting velocity (m/s)
H1	Generally safe for vehicles, people and buildings	≤ 0.3	0.3	2.0
H2	Unsafe for small vehicles	≤ 0.6	0.5	2.0
H3	Unsafe for vehicles, children and the elderly	≤ 0.6	1.2	2.0
H4	Unsafe for vehicles and people	≤1.0	2.0	2.0
H5	Unsafe for vehicles and people. All buildings vulnerable to structural damage. Some less robust buildings subject to failure	≤ 4.0	4.0	4.0
H6	Unsafe for vehicles and people. All building types considered vulnerable to failure	> 4.0		

2.4 Lee et al 1986

Lee et al. Institute of Water Resources, U.S. Army Corps of Engineers, (1986) at the Oak Ridge National Laboratory, U.S. Department of Energy, prepared three methods for predicting loss of life from floods. Their focus included flash floods and dam failures but was not limited to catastrophic events. The authors compiled additional information shedding light on the mechanisms resulting in life loss. For example, summarizing a variety of studies, they suggested the following circumstances for life loss:

1. Being trapped in a structure by rising water
2. Being swept out of a structure
3. Being in a structure that fails
4. Attempting to cross flood waters
5. Being caught in flood water while in the floodplain
6. Attempting to rescue others in flood waters
7. Attempting to drive across a flood-way, and
8. Attempting to boat or raft on flood waters.

To these were added four reasons people drown:

1. The flood stage is life-threatening
2. People receive inadequate warning time
3. They respond too slowly, and
4. They do the wrong thing.

For their own regression equation, Lee et al. assembled a data set consisting of 47 floods, most of which resulted in loss of life, and all of which occurred in the United States between 1963 and 1985.

The final equation that was considered to be most appropriate was equation L-4 as follows:

$$\frac{L}{P} = \frac{\exp\left\{-6.2 + 3.1\left[\frac{1}{1+Wt}\right] - 0.00034(Wt * P)^{0.5} - 0.0077P^{0.5} + 1.4E + 0.0039D\right\}}{1 + \exp\left\{-6.2 + 3.1\left[\frac{1}{1+Wt}\right] - 0.00034(Wt * P)^{0.5} - 0.0077P^{0.5} + 1.4E + 0.0039D\right\}} \quad \text{(L-4)}$$

Where:

- L = loss of life
- P = Population at risk (PAR)
- Wt = warning time in minutes (Lee et al. used W)
- E = experience with floods in the last 10 years (1 = yes; 0 = no)
- D = depth of flooding at peak stage (feet above flood stage), and
- U = denotes an urbanized area (1 = urban area with pop. ≥ 10,000; 0 = otherwise).

Table 2-4 provides the fatality and PLL estimates for various depths of inundation and populations at risk.

Table 2-4 Lee et al fatality rate and PLL for various depths of inundation

PAR	3	5	10	50	100	1000
Wt (mins)	10	15	15	120	120	120
E	1	1	1	1	1	1
D (1 ft)	2	2	2	2	2	2
U	0	0	0	0	0	0
L/P	0.01071	0.00977	0.00969	0.00779	0.00753	0.00589
Life Loss	0.03	0.05	0.10	0.39	0.75	5.89
PAR	3	5	10	50	100	1000
Wt (mins)	10	15	30	120	120	120
E	1	1	1	1	1	1
D (1 ft)	1	1	1	1	1	1
U	0	0	0	0	0	0
L/P	0.01067	0.00973	0.00878	0.00776	0.00750	0.00587
Life Loss	0.03	0.05	0.09	0.39	0.75	5.87
PAR	3	5	10	50	100	1000
Wt (mins)	10	15	30	120	120	120
E	1	1	1	1	1	1
D (0.5 ft)	0.5	0.5	0.5	0.5	0.5	0.5
U	0	0	0	0	0	0
L/P	0.01065	0.00971	0.00876	0.00774	0.00749	0.00586
Life Loss	0.03	0.05	0.09	0.39	0.75	5.86

As shown in Table 2-3 the formula is not sensitive to the depth of inundation and results in fatality rates varying from 0.01 to 0.006.

The methodology had a number of shortcomings, the most significant being as follows (McClelland and Bowles, IWR USACE July 2002):

- The model uses warning time rather than excess evacuation time.
- The equations have a built-in bias to underestimate when loss of life is large and to overestimate when loss of life is small
- Since the events were treated globally, and since the equations are nonlinear with respect to population, estimates of life loss will be different when summed over subpopulations and will depend on how the global population is divided, and
- Current definitions of warning time do not describe the average warning time, the extent to which a warning is propagated, the effectiveness of the message at mobilizing a timely evacuation, informal types of warnings like sensory clues and shouts from neighbours, or the time required to evacuate.

2.5 Dekay and McClelland 1993

In 1993 DeKay and McClelland published "Predicting Loss of Life in Cases of Dam Failure and Flash Flood" in the publication Risk Analysis. The events used by DeKay and McClelland were the same as those used by Brown and Graham. The DeKay and McClelland procedure demonstrated that loss of life is related to the number of people at risk in a nonlinear fashion. They also found that loss of life is greater in situations where the flood waters are deep and swift. DeKay and McClelland have a separate equation for high and low force conditions.

Their equation for high force conditions, i.e., where 20% or more of flooded residences are either destroyed or heavily damaged is:

$$\text{Deaths} = \frac{\text{PAR}}{1 + 13.277(\text{PAR}^{0.440})e^{[2.982(\text{WT}) - 3.790]}}$$

Their equation for low lethality conditions, i.e., where less than 20% of flooded residences are either destroyed or heavily damaged is:

$$\text{Deaths} = \frac{\text{PAR}}{1 + 13.277(\text{PAR}^{0.440})e^{[0.759(\text{WT})]}}$$

Where:

- PAR is the number of people at risk, and
- WT is warning time in hours.

Warning time (WT), as used by Dekay and McClelland, is the time in hours from the initiation of dam failure warning until the dam failure floodwater reaches a community or other group of people. Warning time must therefore consider the time it takes for flood water to reach the community or group of people. When dam failure warnings do not precede the arrival of dam failure flooding in an area, WT would be zero. A negative warning time should not be used in these equations.

Table 2-5 provides fatality rates and estimated PLL for various populations at risk and warning times using the low lethality conditions.

Table 2-5 Dekay and McClelland Fatality rate data examples

PAR	3	5	10	50	100	1000
WT (mins)	15	15	15	15	15	15
WT (hrs)	0.25	0.25	0.25	0.3	0.3	0.3
Deaths/PAR	0.037	0.030	0.022	0.011	0.008	0.003
PLL	0.11	0.15	0.22	0.55	0.81	2.97

PAR	3	5	10	50	100	1000
WT (mins)	60	60	60	60	60	60
WT (hrs)	1.0	1.0	1.0	1.0	1.0	1.0
Deaths/PAR	0.021	0.017	0.013	0.006	0.005	0.002
PLL	0.06	0.09	0.13	0.31	0.46	1.68

PAR	3	5	10	50	100	1000
WT (mins)	120	120	120	120	120	120
WT (hrs)	2.0	2.0	2.0	2.0	2.0	2.0
Deaths/PAR	0.010	0.008	0.006	0.003	0.002	0.001
PLL	0.03	0.04	0.06	0.15	0.22	0.79

As shown on Table 2-5, the fatality rates for low PARs vary from about 0.03 to 0.01 with warning times from 15 minutes to 2 hours while the fatality rates for larger PARs vary from about 0.003 to 0.001 with warning times from 15 minutes to 2 hours.

There are a number of shortcomings of the methodology, which is no longer in use (Duane M. McClelland and David S. Bowles, USACE Institute for Water Resources July 2002, Estimating Life loss for dam safety risk assessment – A review and new approach, IWR Report 02-R-3).

The most significant shortcomings are as follows (McClelland and Bowles, IWR USACE July 2002):

- The model uses warning time rather than excess evacuation time
- There is no distinction between day and night, although this could be included by varying the warning time
- Neither author had a background in fields related to dam safety, hydraulics, hydrology, or emergency management. In their words, "our approach is primarily data-driven rather than theory driven
- They treated the individual as the unit for regression, causing events with large populations to dominate the results
- Current definitions of warning time do not describe the average warning time, the extent to which a warning is propagated, the effectiveness of the message at mobilizing a timely evacuation, informal types of warnings like sensory clues and shouts from neighbours, or the time required to evacuate
- Since the events were treated globally, and since the equations are nonlinear with respect to population, estimates of life loss will be different when summed over subpopulations and will depend on how the global population is divided, AND
- The equations can misestimate by a large margin, even within the original data set.

2.6 Wayne Graham USBR 1999

The method developed by Graham (A Procedure for Estimating Loss of Life Caused by Dam Failure DSO-99-06) is now considered by many to be outdated and replaced by the RCEM approach discussed below, which incorporated the data from the Graham approach. Notwithstanding this, the Graham approach serves as a basis for some of the other approaches, as discussed further in this document. Furthermore, based on personal communications with Dr David Bowles, who is a world leader in risk assessment, he has indicated that the RCEM approach is not well founded and is not used by the US Army Corps of Engineers, who prefer to use the LifeSim approach (17 July 2020).

The basis for the DSO-99-06 method uses the data shown on Table 2-6 with the following factors.

2.6.1 Severity

1) Use low severity for locations where no buildings are washed off their foundation.

2) Use medium severity for locations where homes are destroyed but trees or mangled homes remain for people to seek refuge in or on.

3) Use high flood severity only for locations flooded by the near instantaneous failure of a concrete dam, or an earthfill dam that turns into "jello" and goes out in seconds rather than minutes or hours.

In determining whether flooding is low severity or medium severity, use low severity if most of the structures will be exposed to depths of less than 10 feet and medium severity if most of the structures will be exposed to depths of 10 feet or more. Low severity was also defined where DV is less than 50 ft^2/s (< 4.6 m^2/s)

2.6.2 Warning Time

The warning time for a particular area downstream from a dam should be based on when a dam failure warning is initiated and the flood travel time.

2.6.3 Flood Severity Understanding

The flood severity understanding categories are as follows:

1) Vague Understanding of Flood Severity means that the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.

2) Precise Understanding of Flood Severity means that the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others.

3) Flood severity understanding does not apply when there is no warning.

The method allows the use of a range of values to estimate the upper and lower bound estimates and the suggested "best estimate" fatality rate that is applied to the population at risk. The range of fatality rates for the method are presented on the RCEM figures shown in Section 2.2. In the case of Stopbank failures, the Low flood severity fatality rates would be applicable with fatality rates varying from 0.01 to 0.0002, depending on the warning and flood severity understanding.

Table 2-6 Recommended fatality rates for estimating loss of life resulting from dam failure (Table 7 DSO-99-06)

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

2.7 Reclamation Consequence Estimating Methodology

The Reclamation Consequence Estimating Methodology (RCEM) approach was developed in 2014/15 by the United States Bureau of Reclamation (USBR) using case history data for dams with flood severity defined as follows:

- Low severity where DV is less than 50 ft^2/s (< 4.6 m^2/s)
- Medium severity for DV greater than 50 ft^2/s (> 4.6 m^2/s), AND
- High severity for DV greater than 160 ft^2/s (14.9 m^2/s) combined with rate of rise of at least 10 feet (3 m) in five minutes.

The RCEM approach was developed using 60 dam failure and flood event case histories. Some of the 60 case histories were sufficiently detailed to have information at multiple locations and 80 sets of DV and fatality rate were estimated. Two cases (two points) have DV values less than the minimum axis value of 10 ft^2/s (0.9 m^2/s) and were not used for the plots. Therefore, 78 data points were used to generate points on the RCEM plots. Of the 78 total data points, 42 were judged to have little or no warning, 11 were judged to have partial warning, and 25 were judged to have adequate warning.

The following plots are taken from the RCEM approach for varying warning times, defined as follows.

2.7.1 Warning time

The amount of warning that the PAR would be expected to receive in the event of dam failure. Specifically, the amount of time between receiving the warning and the advent of the threatening flood flows. There are two different warning categories considered in RCEM described below.

Adequate warning

An undefined amount of time that would allow most of the PAR to understand the threat posed by dam failure, to take reasonable actions to leave the inundation plain and to successfully move to a safe location. However, even if given adequate warning, there are a multitude of reasons that people may choose not to leave or are unable to leave.

“Adequate” cannot be defined as an exact amount of time because adequate warning is very dependent on-site specific conditions. For example, 30 minutes may be an adequate warning for residents of a small town to evacuate; but it may take many hours of warning to enable a large city to evacuate.

Little or no warning

A limited (but undefined) amount of time that essentially results in most or much of the PAR receiving an inadequate notification (reflecting quality and timeliness of the warning) of an impending failure and a resulting inability to get out of the inundation plain (or seek adequate shelter from flooding).

The low severity dams in the database have all been reviewed, as shown on Table 2-7. The yellow highlighted data shown on this table did not have any DV data available and so were not included on the RCEM figures showing the fatality rate versus DV. The data for each of the events on Table 2-7 have been shown on the figures from the RCEM manual for the cases with cases with little or no warning and partial warning (Figure 2-5) and adequate warning and partial warning (Figure 2-6). The fatality rates highlighted in yellow for the locations where there was no DV estimate have been added as oval plots on the figures with the DV ranging up to 50 ft^2/s (4.6 m^2/s).

Table 2-7 RCEM data for low severity flood events

RCEM Table	Dam	Height (ft)	PAR	Fatality	Fatality Rate	DV (ft2/s)	Distance to PAR (Miles)	Warning Time	Understanding	Breach formation time (hrs)	Volume (acre-ft)	Comment
49	Dongkoumiao Dam – Failed June 2, 1971 (Lijiayuan and Huangxikou Villages)	71	3,500	154	0.044	11 to 15	0.9 to 1.2	Nil	U/K	U/K	2067	
52	Meadow (Bergeron) Pond Dam – Failed March 13, 1996	32	25	1	0.04	7	0.8		N/A	Fairly fast	282	
49	Dongkoumiao Dam – Failed June 2, 1971 (Jiyi Village)	71	1,200	32	0.027	26 to 48	0.3 to 0.6	Nil	U/K	U/K	2067	
40	Lee Lake Dam – Failed March 24, 1968	25	80	2	0.025	10 to 80	0 to 5	No formal	N/A	U/K	300	
42	Texas Hill Country Flood - August 1-3, 1978	N/A	2,070	27	0.013	10 to 80	N/A		Vague	N/A	N/A	Flash flood
41	Austin, Texas Flood – May 24-25, 1981	N/A	1,180	13	0.011	10 to 70	N/A		Vague	N/A	N/A	Flash flood
51	Cyclone Xynthia, France – Coastal Flooding February 28, 2010	N/A	3000	29	0.0097	11 to 32	Varied	Nil	N/A	U/K	N/A	Seawall failure
48	Brush Creek Flash Flood – September 12, 1977	N/A	2,380	25	0.008	10 to 50	N/A	Some	Vague	N/A	N/A	Flash flood
39	Mohegan Park (Spaulding Pond) Dam – Failed March 6, 1963	20	1,000	7	0.007	10 to 80	0 to 2		Unknown	U/K	138	
50	Hurricane Katrina at New Orleans – Coastal Flooding August 29, 2005 (Lower 9th Ward)	13.1	14,000	73	0.0052	73	<0.1		U/K	N/A	N/A	Hurricane
38	Allegheny County, Pennsylvania Flash Flooding, 1986	N/A	2,200	8	0.004	Unknown	N/A		N/A	N/A	N/A	Flash flood
50	D.M.A.D. Dam – Failed June 23, 1983	34	500	1	0.0025	10 to 15	9 to 15		Precise	12mins	16000	
50	Hurricane Katrina at New Orleans – Coastal Flooding August 29, 2005 (Metro Bowl)	13.1	255,900	260	0.001	260	<0.1		U/K	N/A	N/A	Hurricane
50	Hurricane Katrina at New Orleans – Coastal Flooding August 29, 2005 (East Bowl)	13.1	69,290	68	0.001	Unknown	<0.1		U/K	N/A	N/A	Hurricane
44	Great Flood of 1993, Upper Midwestern United States, April to October 1993	N/A	150,000	32	0.0003	Unknown	N/A		Precise	N/A	N/A	Regional Flood
43	Kansas River Flood – July, 1951	N/A	58,010	11	0.0002	Unknown	N/A		Precise	N/A	N/A	Regional Flood
45	Hurricane Agnes Floods- June/July 1972	N/A	250,000	117	0.0002	Unknown	N/A		Precise	N/A	N/A	Regional Flood
60	South Platte River Flood – June 16, 1965	N/A	10,000	1	0.0001	10 to 40	N/A	Adequate	Precise	N/A	N/A	Flash flood
61	Passaic River Basin Flood – April 1984	N/A	25,000	3	0.0001	Unknown	U/K	Adequate	Precise	N/A	N/A	River basin flood
36	South Davis County Water Improvement District, Reservoir No. 1 Dam	15	80	0	0	10 to 25	100		N/A	U/K	4.4	Small turkeys nest reservoir
37	Seminary Hill Reservoir No. 3 – Failed October 5, 1991	17	150	0	0	10 to 80	1/4		N/A	U/K	10.7	Turkeys nest
49	Quail Creek Dike - Failed January 1, 1989	28	1,500	0	0	29	16		Precise	2	40000	
51	Bushy Hill Pond Dam – Failed June 6, 1982	29	300	0	0	20 to 30	1.6	3hrs	Precise	U/K	616	
46	Phoenix Area Flood – February, 1980	N/A	6,000	0	0	10 to 50	N/A		Precise	N/A	N/A	Storm flood
47	Prospect Dam – Failed February 10, 1980	45	100	0	0	4	U/K		Precise	>1hr	5850	

1

Note: The yellow highlighted data shown on this table did not have any DV data available and so were not included on the RCEM figures showing the fatality rate versus DV

The data for the low severity dams indicate that the fatality rates for dams could be as follows with low severity for DV values between 1 to 5.

- Adequate warning in the range from 0.001 to 0.00001 (zero fatality), and
- Little or no warning 0.03 to 0.00001 (zero fatality).

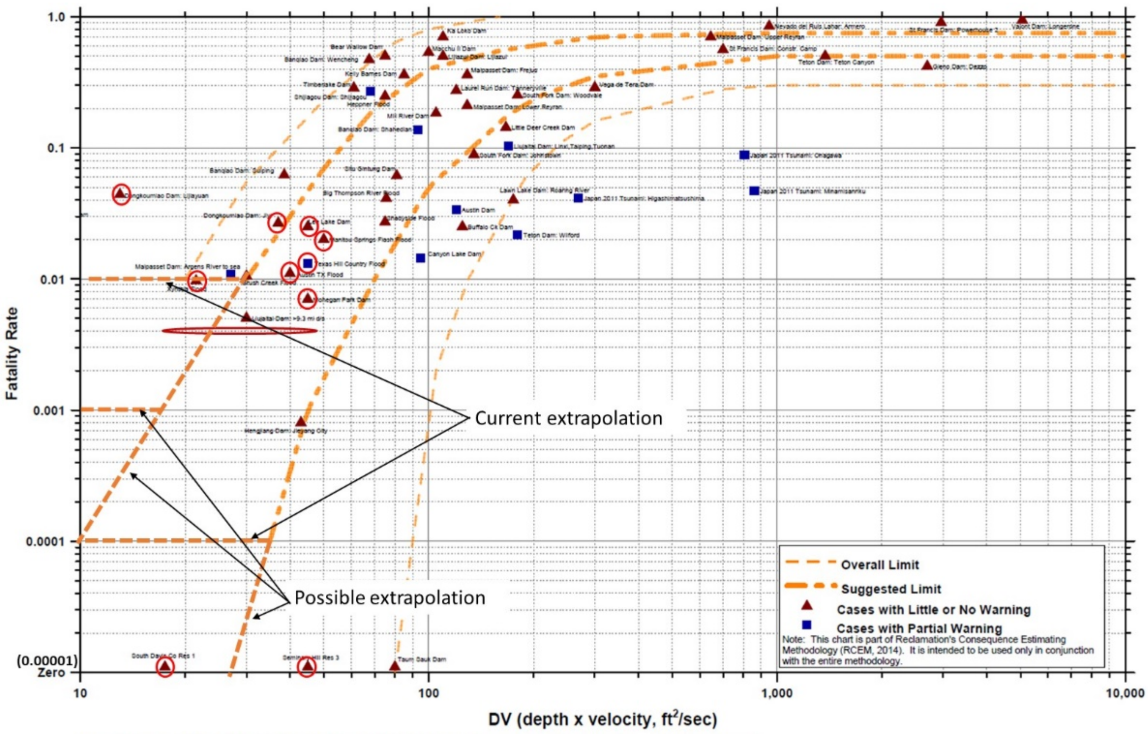

Figure 2-5 RCEM Figure 1 - Fatality rate vs DV for cases with little or no warning and partial warning (Showing existing and potential extrapolations to the original lines)

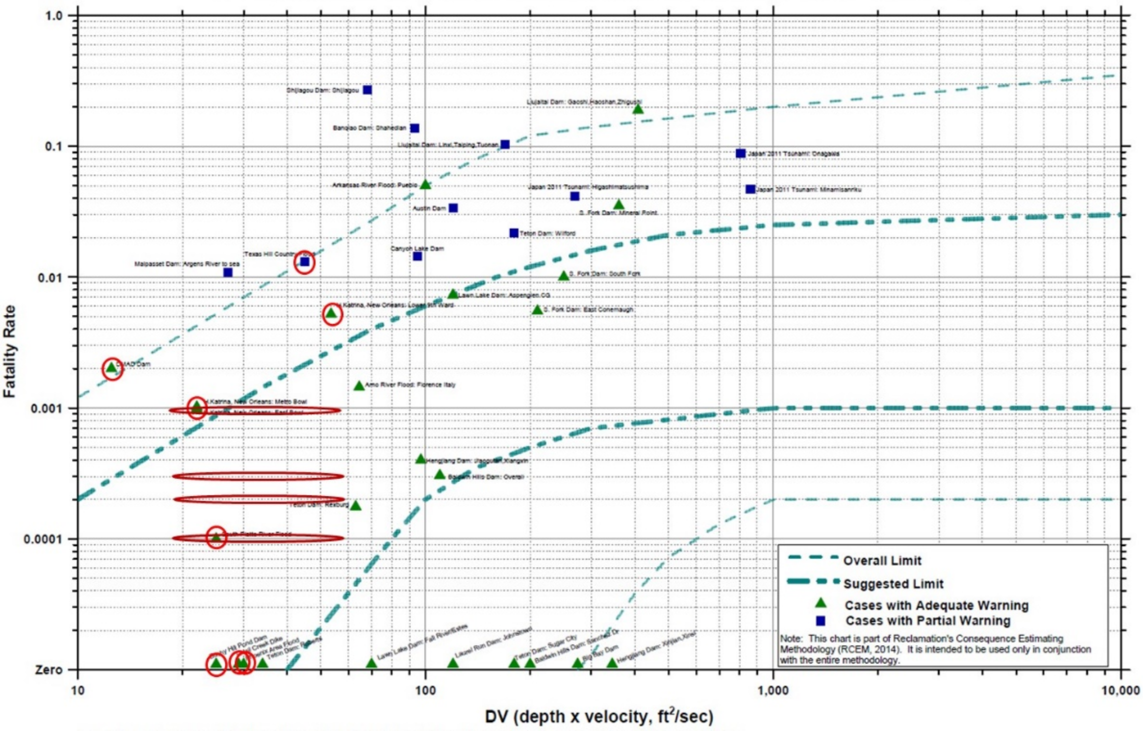

Figure 2-6 RCEM Figure 2 - Fatality rate vs DV for cases with adequate warning and partial warning

2.8 DEFRA Guide

UK Department for Environment Flood and Rural Affairs (defra) Guide to risk assessment for reservoir safety management Volume 2: Methodology and supporting information Report – SC090001/R2. Section 9.2.1 of the report deals with risk to people within the inundation area.

Adults are unable to stand in still floodwater with a depth of about 1.5 m or greater (although this depends on the height of the person). The depth of flowing water in which people are unable to stand is much less. Some people will be at risk when water depth is only 0.5 m if the velocity is 1 m/s. If this is increased to 2 m/s, some will only be able to stand in 0.3 m of water. Most people will be unable to stand when the velocity is 2 m/s and the depth is 0.6 m.

Use the average water velocity and depth in each reach appropriate to the property group identified (which may include subdivision for position across the inundated area) to calculate average societal life loss, individual risk and property damage.

Assess the hazard to an individual life from the hydraulic parameters of velocity and depth (that is, chance of death given dam failure) expressed as fatality rate. Read off the fatality rates from the graph in Figure 2-7, noting that the measure of forcefulness is the total discharge divided by the flooded width.

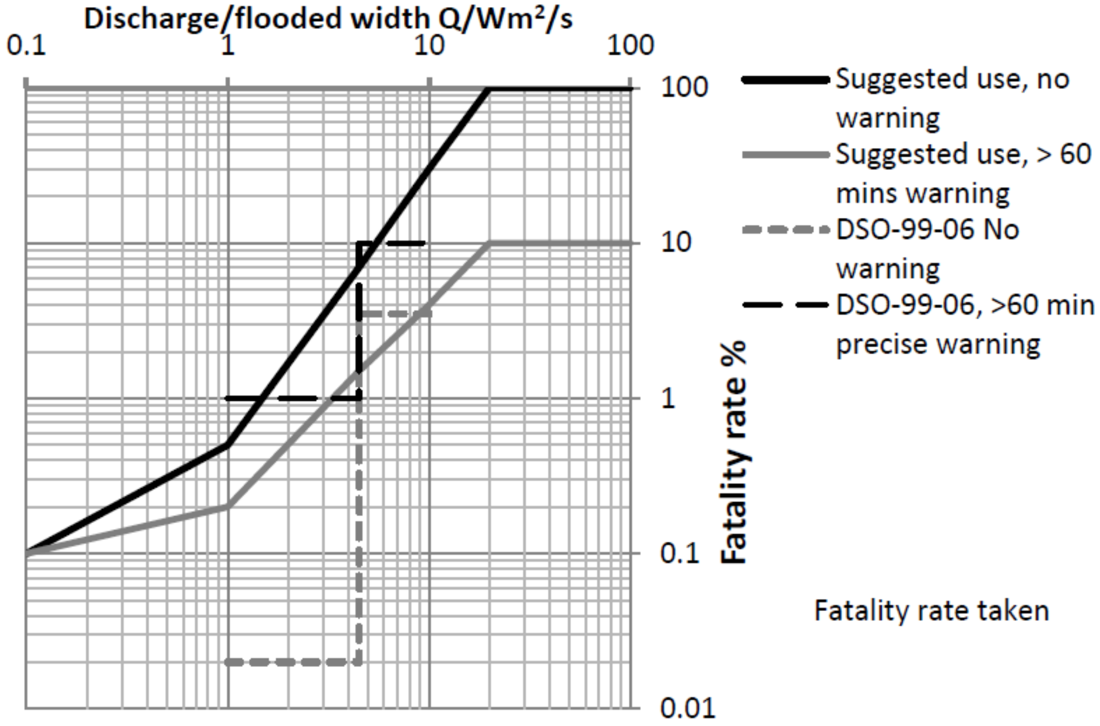

Figure 2-7 Suggested relationship of fatality rate to force of water – Defra report Figure 9.1

The figure above provides fatality rates for people at risk where the DV values are lower than 1 and is useful for evaluating the failures resulting from stopbanks where the DV values are in this order.

2.9 DEFRA Supplementary 2008

UK Department for Environment Flood and Rural Affairs (DEFFRA) Flood and Coastal Defence Appraisal Guidance Social Appraisal Supplementary Note to Operating Authorities Assessing and Valuing the Risk to Life from Flooding for Use in Appraisal of Risk Management Measures, May 2008

This guidance is from the UK and is supplementary to the Flood and Coastal Defence Project Appraisal Guidance. It provides a new method for the valuation of the risk to life associated with flood risks. Its main purpose is to enable the risk of fatalities to be assessed as part of a more comprehensive flood risk appraisal where the social benefits associated with any reduction in this risk are also taken into account when considering options for risk management.

The "Risks to People – Phase 2" (R2P) research project completed in March 2006 developed and demonstrated a method for estimating and mapping serious injury or fatalities from flooding which may occur during, or in the immediate aftermath, of a flood event. The Risks to People (R2P) method is nested within a 'Source – Pathway – Receptor' (S-P-R) model, predominately dealing with a key component of the receptors (e.g. people).

2.9.1 Overview

The number of fatalities is calculated using the following equation:

N(F) = f(N(Z), HR, AV, PV).

Where:

- N(F) is the possible number of fatalities
- N(Z) is the population within the zone at risk of flooding
- HR is the Flood Hazard
- AV is the Area Vulnerability, and
- PV is the People Vulnerability.

The variables used in the methodology are:

Flood Hazard

Flood Hazard describes the flood conditions in which people are likely to be swept over in a flood with the possibility of drowning, and is a combination of flood depth, velocity and the presence of debris.

- Depth of flood water (m)
- Velocity of flood water (m/s)
- Debris factor (score)

The Flood Hazard rating is calculated using the following equation:

$$HR = d \times (v + 0.5) + DF$$

Where HR = (flood) hazard rating:

- d = depth of flooding (m)
- v = velocity of floodwaters (m/sec), and
- DF = debris factor calculated using Table A.1 (debris factor depends on probability that debris will lead to a significantly greater hazard).

Table 2-8 Table A.1 Guidance on debris factors for different flood depths, velocities and dominant land uses (from DEFRA Supplementary 2008)

Depths	Pasture/Arable	Woodland	Urban
0 to 0.25 m	0	0	0
0.25 to 0.75 m	0	0.5	1
d>0.75 m and/or v>2	0.5	1	1

Ref: FD2321/TR1 Table 3.1

Experimental work from Abt (1989) and RESCDAM (2000) was reviewed. Figure 3-1 plots the results from these two experiments with (a) and indication of the typical height times mass for different ages based on UK Department of Health figures and (b) some thresholds indicating the relative hazard associated with different depth-velocity combinations.

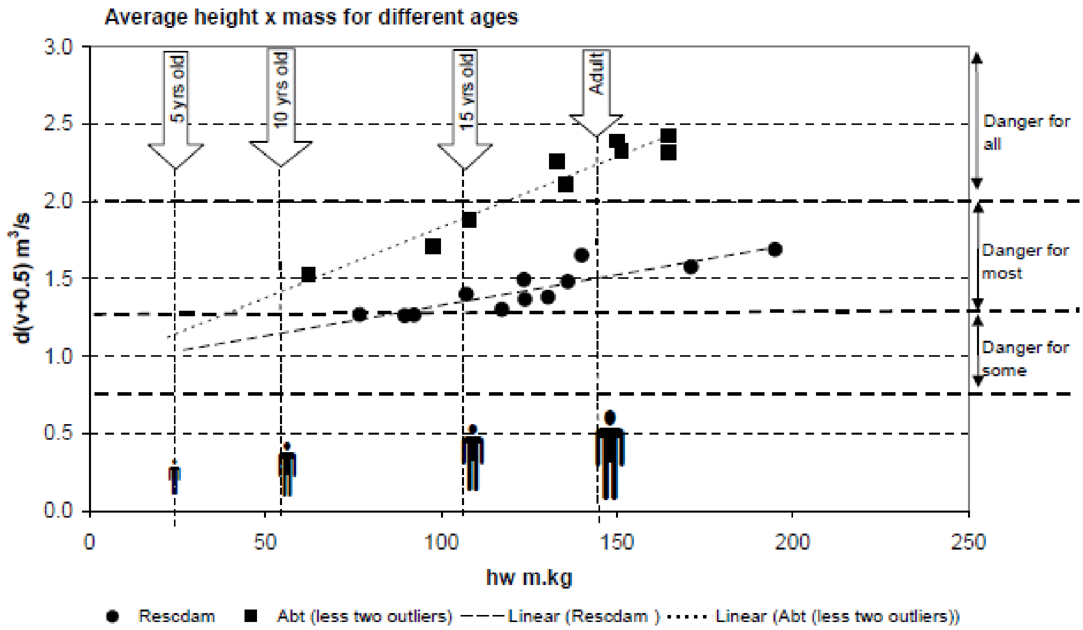

Figure 2-8 The interpretation of the data sets to derive flood hazard thresholds (from DEFRA Supplementary 2008)

Area Vulnerability

Area Vulnerability describes the characteristics of an area of the floodplain that affect the chance of being exposed to the flood hazard

- Flood warning: including % of at-risk properties covered by the flood warning system; % of warnings meeting the two-hour target; and % of people taking effective action (score)
- Speed of onset of a flood (score), and
- Nature of area: multi-storey apartments; typical residential/commercial/industrial properties; bungalows, mobile homes, campsites, schools, etc. (score).

The Area Vulnerability is calculated using Table A.2.

Table 2-9 Table A.2: Area Vulnerability (from DEFRA Supplementary 2008)

Parameter	Low risk area Score =1	Medium risk area Score =2	High risk area Score=3
Speed of onset	Onset of flooding is very gradual (many hours)	Onset of flooding is gradual (an hour or so)	Rapid flooding
Nature of area	Multi-storey apartments	Typical residential area (2-storey homes); commercial and industrial properties	Bungalows, mobile homes, busy roads, parks, single storey schools, campsites, etc.
Flood warning Score	Indicative score for England use 2.15 Indicative score for Wales use 2.23		
Area Vulnerability (AV) = sum of scores for 'speed of onset', 'nature of area' and 'flood warning'			

Ref: FD2321/TR1 Table 4.4 and Table 4.3

Note: The flood warning scores quoted above are indicative values. Their use is appropriate for most loss of life calculations. However, if significant factors influence flood warning and response in the project area then a specific Flood Warning Score can be calculated using the method in Report FD2321/TR1

People Vulnerability

People Vulnerability describes the characteristics of the people affected by flooding and their ability to respond to ensure their own safety and that of their dependants during a flood.

- % residents aged 75 years or over, and
- % residents suffering from long term illness.

The People Vulnerability score (expressed as a percentage) is simply:

PV = %residents suffering from long-term illness + %residents aged 75 or over.

Example provided

Step 1. Calculate Flood Hazard Rating (HR)

The flood hazard is calculated using the formula given above for zones of different hazard in the floodplain. It is therefore necessary to divide the floodplain into zones of different hazard. In the example below, the floodplain has been divided into strips of different hazard based on the distance from the river/coast. Refer to Table A1 for the Debris Factor.

Distance from river/coast (m)	Typical depth, d(m)	Typical velocity, v (m/sec)	Debris factor (DF)	**Hazard rating =d(v+0.5) + DF**
0-50	3	2	1 - possible	**8.5**
50-100	2	1.8	1 - possible	**5.6**
100-250	1	1.3	1 - possible	**2.8**
250-500	0.5	1.2	1 - possible	**1.85**
500- 1000	0	0	0 - unlikely	**0**

Ref: FD2321/TR1 Table 6.1

Step 2. Calculate Area Vulnerability (AV)

Calculate the Area Vulnerability using Table A.2.

Distance from river/coast (m)	Flood warning	Speed of onset	Nature of area	**Sum = Area Vulnerability**
0-50	2.15	3	2	**7.15**
50-100	2.15	2	1	**5.15**
100-250	2.15	2	3	**7.15**
250-500	2.15	1	2	**5.15**
500-1000	2.15	1	2	**5.15**

Ref: FD2321/TR1 Table 6.2

Step 3. Calculate those exposed to the flood

This Area Vulnerability score is simply multiplied by the Hazard Rating derived above to generate the value for X (the % of people exposed to risk). Should the score exceed 100, this is simply taken as 100. Whilst this is not a true percentage, it provides a practical approach to the assessment of flood risk. X is multiplied by the number of people in each zone to determine the number of people exposed to the flood.

Distance from river/coast (m)	N(Z)	Hazard rating (HR)	Area vulnerability (AV)	X = HR x AV (as %); $0 \le X \le 100\%$	**N(ZE) = X x N(Z)**
0-50	25	8.5	7.15	61%	**15**
50-100	50	5.6	5.15	29%	**14**
100-250	300	2.8	7.15	20%	**60**
250-500	1000	1.85	5.15	10%	**95**
500-1000	2500	0	5.15	0%	**0**

Note: N(Z) is the population in each hazard zone
N(ZE) is the number of people exposed to the risk in each hazard zone
Ref: FD2521/TR1 Table 6.3

Step 4. Calculate People Vulnerability (PV)

Distance from river/coast (m)	Factor 1 (% very old i.e. >75 years)	Factor 2 (% Disabled or infirm)	**PV**
0-50	15%	10%	**25%**
50-100	10%	14%	**24%**
100-250	12%	10%	**22%**
250-500	10%	15%	**25%**
500-1000	15%	20%	**35%**

Ref: FD2321/TR1 Table 6.4

Step 5. Calculate the numbers of possible fatalities

The number of possible fatalities is assumed to be proportional to the People Vulnerability and the Hazard Rating. The number of people exposed to the risk (N(ZE)) is multiplied by 2Y x 2HR (as a percentage) to obtain the number of fatalities.

Distance from river/coast (m)	N(ZE)	PV (as %) from Step 4	HR from Step 1	2PV x 2HR (as %)	**No. of fatalities (rounded)**
0-50	15	25%	8.5	8.5%	**1**
50-100	14	24%	5.6	5.4%	**1**
100-250	60	22%	2.8	2.5%	**1**
250-500	95	25%	1.85	1.9%	**2**
500-1000	0	35%	0	0%	**0**
All	185				**5**

Ref: FD2321/TR1 Table 6.5

2.10 Jonkman March 2008

Jonkman S.N. and Vrijling J.K. - Flood Risk Management Journal (Mar 2008)

The study analysed various floods for which the averages presented in Table 2-9 were derived and provide very general indications of the order of magnitude of the overall event mortality. These provide a rough but useful first estimate for mortality for an event type. However, the variation in event mortality remains large due to variations in circumstances between events.

Table 2-10 Order of magnitude of average event mortality for different flood types (Table 4 from Jonkman & Vrijling 2008)

Flood type	Severity of impacts	Evacuation	Mortality (order of magnitude)
Tsunamis	Severe	Difficult	
Dam breaks			0.1
Flash floods			
Coastal floods			0.01
River floods			10^{-3}
Drainage floods	Less severe	Possible	10^{-4}

2.11 Jonkman September 2008

Jonkman S.N, Vrijling J. K, Vrouwenvelder A. C. W. M., Article in Natural Hazards · September 2008 DOI: 10.1007/s11069-008-9227-5

2.11.1 General Approach

The title of the article is "Methods for the estimation of loss of life due to floods: a literature review and a proposal for a new method."

In the article, the authors present the general approach for the estimation of loss of life due to flooding, as shown on Figure 2 6, which is appropriate for the Stopbank failure or overtopping flooding. The approach is similar in some respects to the programme LifeSim, which was originally developed by Dr David Bowles of RAC Engineers and now has now been commercially developed by USACE.

The analysis starts with the total number of people at risk before the event in the threatened area. By analysing the possibilities for evacuation, shelter and rescue, the total number of people exposed to the floodwaters can be estimated. Consequently, the mortality (fatality) in the exposed population (FD) can be estimated using a mortality (fatality) rate.

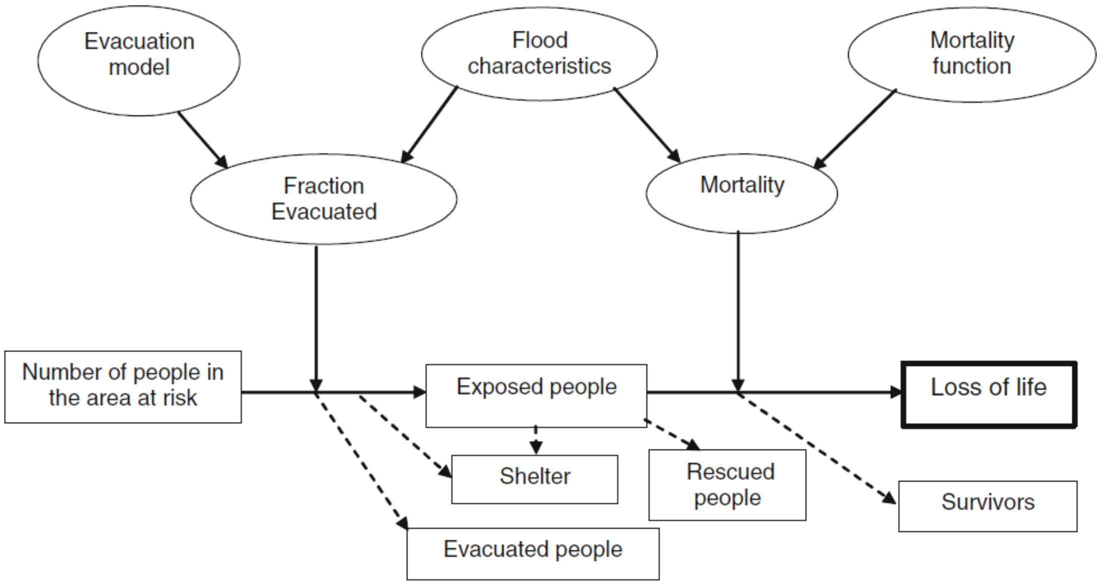

Figure 2-9 General approach for the estimation of loss of life due to flooding

2.11.2 Calculation process

The number of fatalities – Loss of Life (N) can be estimated as follows:

$N = F_D N_{EXP}$

Where:

F_D = the mortality (fatality rate) of the exposed population, and

N_{EXP} = the total number of people exposed to the floodwaters.

The number of people exposed to the floodwaters (N_{EXP}) is estimated based on the following elements:

- The number of people at risk before the event: N_{PAR}
- The fraction of the population that is evacuated out of the area before the flood: F_E
- The fraction of the (remaining) population that has the possibility to find shelter: F_S, and
- The number of people rescued: N_{RES}.

The number of people exposed equals:

$N_{EXP} = (1-F_E)(1-F_S)N_{PAR} - N_{RES.}$

The analysis of the four elements in this formula applicable to shallow depth, low velocity flood or breach waters are as follows.

Population at risk (NPAR)

The number of people at risk includes all of the individuals in the affected area before the event.

Evacuation (F_E)

Evacuation was defined in the study as ‘‘the movement of people from a (potentially exposed area to a safe location outside that area before they come into contact with physical effects’’. In general the possibilities for successful evacuation will depend on (a) the time available until the arrival of the floodwater in an area and (b) the time required for evacuation.

For analysis of flood evacuation in the Netherlands, a macro-scale traffic model was been developed (van Zuilekom et al. 2005). The model accounts for the number of inhabitants in the area, the capacities of the road network and the exits, the departure time distribution of evacuees and the effects of traffic management. An example is shown on Figure 2-10.

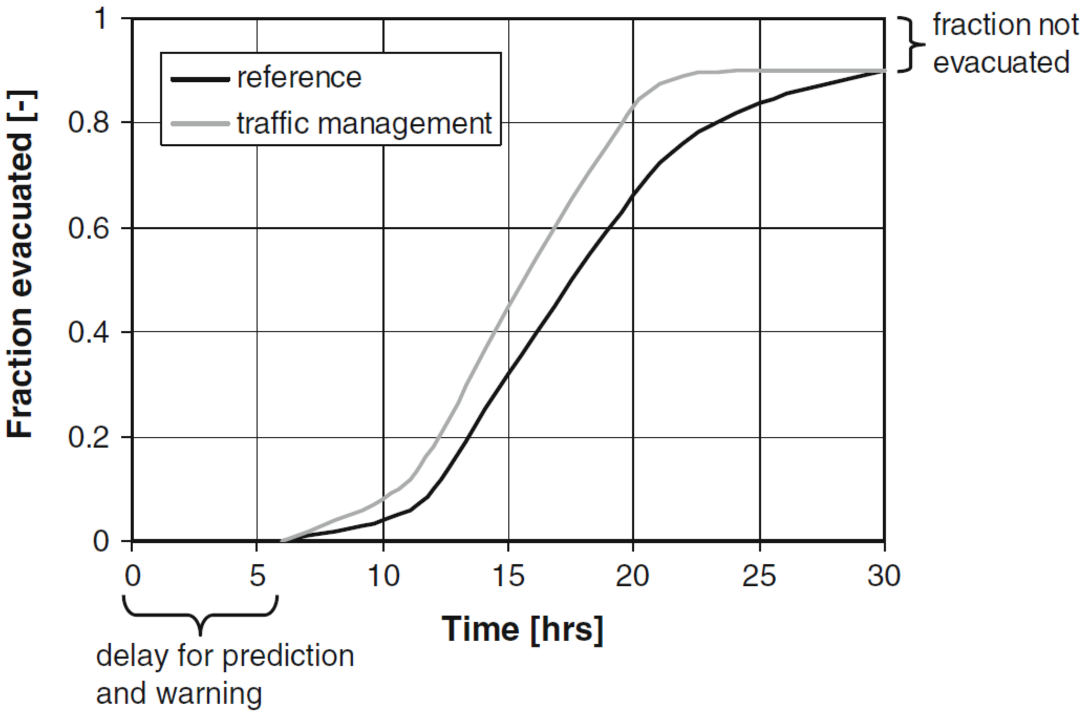

Figure 2-10 Example of estimation of the time required for evacuation for the area

Based on the results from modelling, the evacuated fraction was estimated at $F_E = 0.5$ if traffic management is used and 0.4 if no traffic management is used, e.g. in the case of an unorganized evacuation.

Shelter (F_S)

Within the flooded area people may find protection within shelters. These are constructed facilities in the exposed area, which offer protection.

In the Netherlands, where the study was completed, the fraction of people living in higher buildings could range between 0 in rural areas to 0.2 in cities. The possibilities to reach shelter depend on the level of warning, and also on the rise rate and depth of the water. For specific cases the presence of formal shelters and/or high grounds can be discounted.

For the purpose of the present evaluation, the likelihood of finding protection in the houses along the stopbank has been taken to be 0.

Rescue (N_{RES})

Rescue concerns the removal of people from an exposed area either by professionals or other affected people. A general estimate of the number of people rescued (NRES) can be obtained based on the capacities of rescue services with boats and helicopters. In addition, the delays in the initiation of rescue need to be accounted for.

In the case of the Avon stopbanks, the rescue would likely be ad hoc based on the ability of the emergency services to mobilise. Furthermore, the depths of flow and velocities are such that the rescue would likely be owing in the large part to local community involvement rather than emergency services.

Mortality (F_D)

To estimate mortality and loss of life more accurately for one event, case-specific circumstances (flood characteristics, possibility of warning and evacuation) have to be taken into account. The following approach was developed for loss of life estimation.

Breach Zone Mortality

FD = 1 if hv>7 m^2/s and v ≥ 2 m/s

Where h is the depth of water and v is the velocity.

Mortality in the zone with rapidly rising water

The following mortality function was established for the lognormal distribution

$$F_{\mathrm{D}}(h) = \Phi_{\mathrm{N}}\left(\frac{\ln(h) - \mu_{\mathrm{N}}}{\sigma_{\mathrm{N}}}\right)$$

$$\mu_{\mathrm{N}} = 1.46 \quad \sigma_{\mathrm{N}} = 0.28$$

Where:

- Φ_N is the cumulative normal distribution
- μ_N is the average of the normal distribution, and
- σ_N is the standard deviation of the normal distribution.

If (depth h ≥ 2.1 m and rate of rise w ≥ 0.5 m/h) and (hv < 7m^2/s or v < 2 m/s).

The function for the zone with rapidly rising water is only used when it gives higher mortality fractions than the function for the remaining zone, ie for water depths larger than 2.1 m.

The mortality function for the zone with rapidly rising water can be corrected for improved building quality to current standards. For a first-order estimate of this effect, the following constants can be assumed in the lognormal mortality function: μ_N = 1.68, σ_N = 0.37.

Mortality in the remaining zone

The remaining zone in Jonkman 2008 Fig 16 (our Figure 2-11 below) means the area of the chart not described by the other areas with special risks. The following mortality function was established for the lognormal distribution.

$$F_{\mathrm{D}}(h) = \Phi_{\mathrm{N}}\left(\frac{\ln(h) - \mu_{\mathrm{N}}}{\sigma_{\mathrm{N}}}\right)$$

$$\mu_{\mathrm{N}} = 7.60 \quad \sigma_{\mathrm{N}} = 2.75$$

If (rate of rise w < 0.5 m/h or (w ≥ 0.5 m/h and depth h < 2.1 m)) and (hv < 7 m^2/s or v < 2 m/s)

The range of flood conditions for which the above mortality functions can be applied are indicated in Figure 2-7. For clarity, a distinction was made between situations with rise rates below and above the threshold value of w = 0.5 m/h.

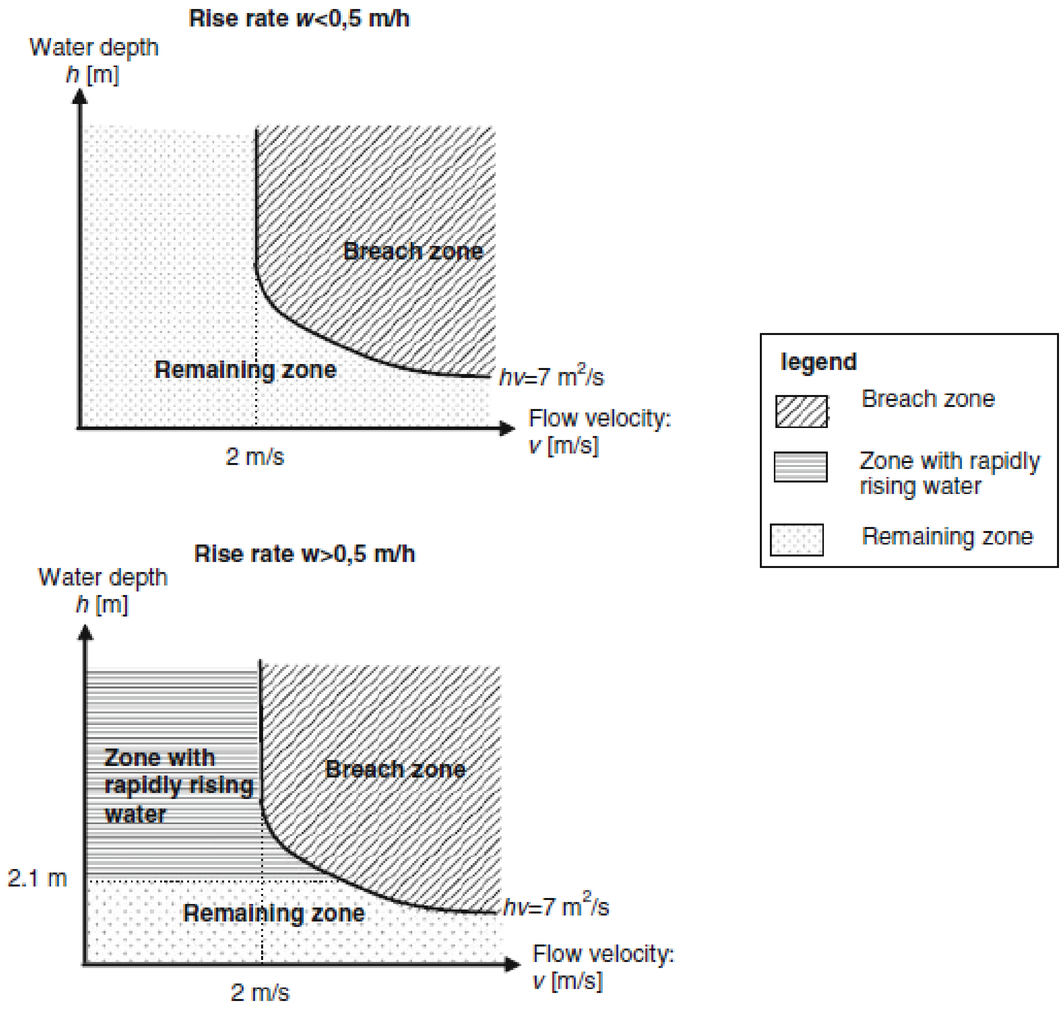

Figure 2-11 Area of application of mortality functions, as a function water depth, rise rate and flow velocity (Jonkman Sep2008 Figure 16)

The results for the method using inundation depths from 0.1 m to 2 m are shown on Table 2-10, which indicates that the fatality rate can vary from 0.0002 for very small depth of about 0.1 m to 0.003 for depths of about 1 m. These fatality rates are relatively high compared with the other methods and will be applied to the houses and buildings in close proximity to the stopbank, together with the other approaches discussed above where higher fatality rates have been estimated.

Table 2-11 Jonkman (Sep2008) mortality in the remaining zone with varying depths of inundation

h	ln (h)	F_g (h)
0.1	-2.30	0.0002
0.3	-1.20	0.0007
0.5	-0.69	0.0013
0.75	-0.29	0.0021
1	0.00	0.0029
2	0.69	0.0060

3. Fatality rates for houses close to the stopbank failure area

3.1 Breach characteristics

3.1.1 Breach discharge

The breach zone adjacent to the stopbank will have a breach depth of approximately 0.5 times the depth of water from the ground level at the stopbank to the tidal level and the discharge can be calculated per unit length using a broad crested weir equation as follows:

$$q = CLH^{1.5}$$

Where

- q = unit discharge $m^3/s/m$
- C= coefficient of discharge = 1.45
- L = length of breach = 1m, and
- H = head of water = depth of water from the ground level of the stopbank at the breach area to the tidal level.

3.1.2 Breach Flow Velocity at stopbank

Given the input discharge, the velocity can be calculated using the following formula.

$$V = \frac{q}{(H/2)}$$

3.1.3 Depth and Velocity at closest houses

The depth and velocity of flow at houses 25 m away from the stopbank was then calculated, assuming that the ground level is flat within 25 m beyond the stopbank and that the water flows out at an angle of 45 degrees on both sides of the breach, as shown on Figure 3-1.

The slope of the energy gradient from the stopbank to the houses was taken to be 1 in 1000 and the Mannings “n” value for the overland flow was assumed to be 0.02. The DV data was then calculated for varying widths of breach, as shown on Figure 3-2.

The tide (flood water) depth above stopbank base versus depth at houses for various stopbank breach lengths was developed, as shown on Figure 3-3.

This DV data was used together with daytime fatality rates from Figure 2-7 (defra method), to develop the tide depth versus daytime fatality rate at the houses as shown on Figure 3-4. The fatality rate using Jonkman “Bathtub” approach was also calculated for the depth at the stopbank, as shown on Figure 3-4. As can be seen, the Jonkman approach results in higher fatalities than the defra method and will be used for the close proximity houses and buildings.

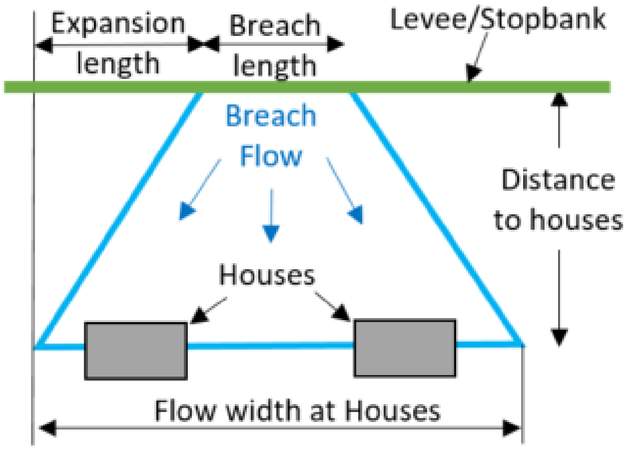

Figure 3-1 Schematic of the breach expansion from the stopbank to the closest houses

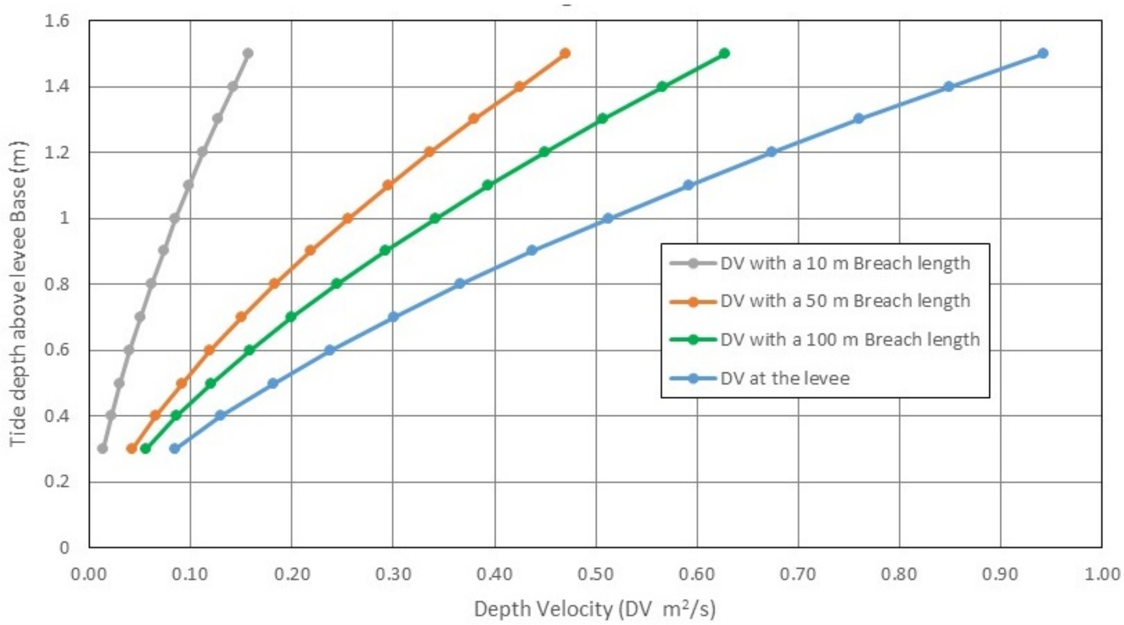

Figure 3-2 Tide depth above stopbank base versus DV at the stopbank and 25m from the stopbank with various breach lengths

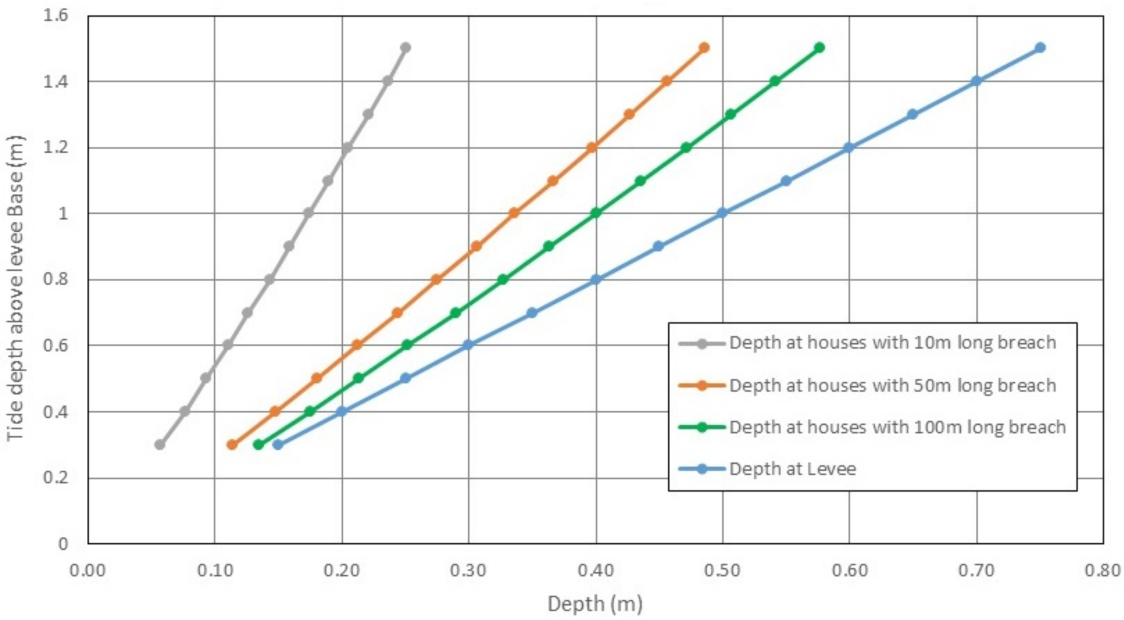

Figure 3-3 Tide depth above stopbank base versus depth at the stopbank and 25m from the stopbank with various breach lengths

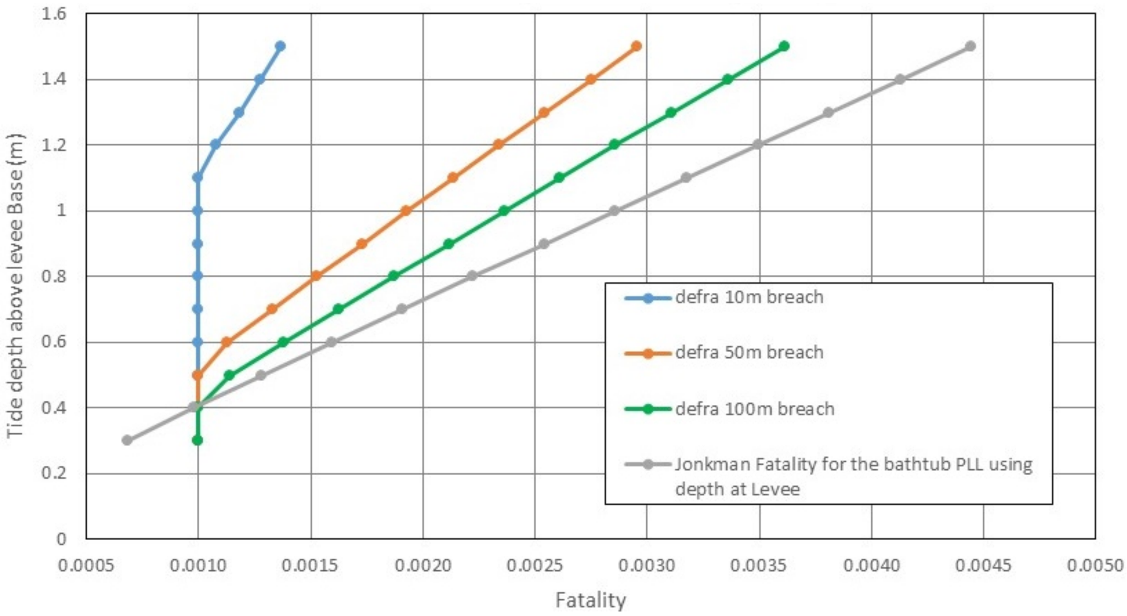

Figure 3-4 Tide depth above stopbank base versus fatality using "defra" Figure 2-7 and Jonkman "Bathtub"

3.1.4 Summary of approaches

The fatality rate for the houses in close proximity to the stopbank can be estimated using each approach described above, as shown on Table 3-1.

Table 3-1 Fatality rates for houses in close proximity to the stopbank failure

Method	Day fatality (adequate warning)	Night fatality (no warning)
Graham 1999	0.0003	0.01
RCEM 2015	0.0002	0.01 upper limit 0.005 arithmetic mean
Hill et al 2007	0.0002	Not defined
Defra Report Figure 9.1	0.001 to 0.002	0.001 to 0.005
Jonkman & Vrijling 2008	0.0001	Not defined
Jonkman, Vrijling & Vrouwenvelder 2008	0.0007 to 0.0044 using varying tidal depth at stopbank	Not defined
Recommended fatality rates	**Jonkman, Vrijling & Vrouwenvelder 2008 using tidal depth at stopbank**	**0.01**

3.1.5 Breach width, houses affected, fatality rate and PLL estimate for close proximity houses

For the estimation of the PLL, it was assumed that the daytime risk to houses in close proximity was adequately covered by the general inundation risk described earlier. For the night, risks of overtopping failure and embankment failure were assessed.

Assuming there is no warning, a fatality factor of 0.01 was used. It was assessed that five houses on the left bank were in close proximity to the stopbanks and they would have materially higher night time risk from overtopping failure with an embankment breach length of about 50 m.

In the case of piping failure, it was estimated that breach length would be 10 m and three of the five houses would be affected.

Based on the above evaluation, the breaching of the embankment owing to failures prior to overtopping the stopbank will be based on the following parameters for evaluating the number of houses, PLL for piping and overtopping and weighted PLL for day and night.

Table 3-2 Parameters for evaluation of PLL

Description	Piping failure	Overtopping failure
Breach width	10 m	50 m
Distance to houses	20 m from stopbank centreline to building centroid	20 m from stopbank centreline to building centroid
Spread on both sides of the breach	1:1	1:1
Flow width at houses	60 m	100 m
Typical House width	20 m	20 m
Number of houses impacted	3	5
Day Population at risk (1 person per house)	3	5
Night population at risk (2.3 people per house)	6.9	11.5
Evacuation of PAR	Day 40%	Night 0%
Fatality rate to be applied		
Day (14 hours)	Jonkman, Vrijling, Vrouwenvelder 2008 using tidal depth at Stopbank	
Night (10 hours)		0.01
PLL Estimate Day (included in general inundation PLL and not included for close proximity)	0	0
PLL Estimate Night	6.9 * 0.01 = 0.07	11.5 * 0.01 = 0.12
Weighted PLL Estimate for day and night	0.07* 10/24 + 0 * 14/24 = 0.03	0.12* 10/24 + 0 * 14/24 = 0.05

These PLL estimates for the piping and overtopping failure were added to the general inundation area PLL for the left bank sections for events in which piping or overtopping occurred. The right bank houses and buildings are all a significant distance away from the stopbank and so there are no close proximity houses and no additional PLL for the close proximity risks.

4. Conclusions

The following approaches are recommended for estimating the potential life loss for the general population at risk exposed in the “bathtub area” and for those in close proximity to the stopbank at the time of a potential stopbank failure.

4.1 Critical depth for fatality

The breach zone adjacent to the stopbank will have a breach depth of approximately 0.5 times the depth of water from the ground level at the stopbank to the tidal level. The discharge can be calculated per unit length using a broad crested weir equation as follows:

$$q = CLH^{1.5}$$

Where

- q = unit discharge $m^3/s/m$
- C= coefficient of discharge = 1.45
- L = length of breach = 1 m, and
- H = head of water = depth of water from the ground level of the stopbank at the breach area to the tidal level.

Given the input discharge, the velocity can be calculated using the following formula.

$$V = \frac{q}{\left({}^{H}/_{2}\right)}$$

Table 4-1 Breach velocity and DV

Tidal depth above stopbank base (m)	Breach depth (m)	Unit discharge ($m^3/s/m$)	Velocity (m/s)	DV (m^2/s)
0.3	0.15	0.08	0.56	0.08
0.6	0.30	0.24	0.79	0.24

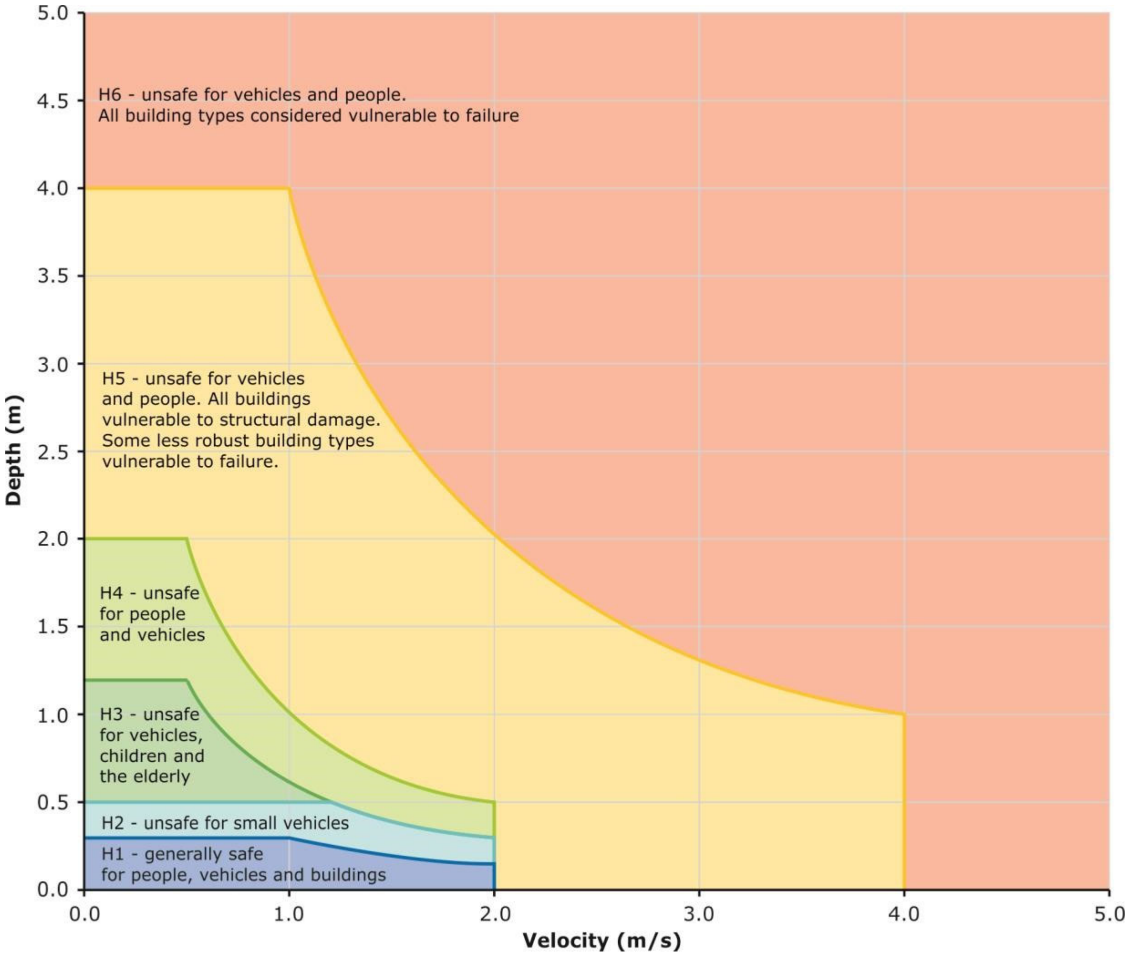

Figure 4-1 Combined flood hazard curves (Smith, Davey, & Cox, 2014)

Based on this approach, given a tidal depth of flow above the base of the stopbank, and velocity, it is evident that a depth of 0.3 m can be taken as being a “Safe” depth for the stopbank flooding, given the relatively low velocity of flow from the breach.

Assumption: No fatalities for depths of flow less than 0.3 m

4.2 Warning levels

We interpret the no warning and adequate warning results into night / day conditions, on the basis that the public are typically disconnected from environmental news while asleep and during early morning and late evening, especially in hours of darkness. On the other hand during daylight, waking hours the public may be more readily alerted to risks through news media, social media, personal observation, friends and neighbours.

For the purposes of this assessment we define day conditions as existing for 14 hours and night conditions (no warning) as existing for 10 hours per day.

4.3 Bathtub Analysis

4.3.1 Little or no warning fatality rate (Night – Overtopping and Piping, Day – Piping)

The Jonkman et al (Sep 2008) approach for evaluating the fatality owing to the “bathtub” flooding, as discussed in 22, will be used for the analysis. The fatality rates will be calculated using the lognormal function and the depth of flooding at each building and it will be assumed that there is no evacuation or rescue of people in the flooded area.

$N = F_D(1-F_E)(1-F_S)N_{PAR} - N_{RES}$

$F_E = 0$

$F_S = 0$

N_{PAR} = people exposed to the flood water

$N_{RES} = 0$

$N_{EXP} = N_{PAR}$

Fatality F_D Table 2-10	$F_D(h) = \Phi_N\left(\frac{\ln(h) - \mu_N}{\sigma_N}\right)$ $\mu_N = 7.60 \quad \sigma_N = 2.75$

4.3.2 Adequate Warning (Day – Overtopping)

The Jonkman (Sep 2008) et al approach for evaluating the fatality owing to the "bathtub" flooding, as discussed in Section 2.11, will be used for the analysis. The fatality rates will be calculated using the lognormal function and the depth of flooding at each building and it will be assumed that there is 40% evacuation of people at risk.

$N = F_D(1-F_E)(1-F_S)N_{PAR} - N_{RES}$

$F_E = 0.4$

$F_S = 0$

N_{PAR} = people exposed to the flood water

$N_{RES} = 0$

$N_{EXP} = 0.6*N_{PAR}$

Fatality F_D Table 2-10)	$F_D(h) = \Phi_N\left(\frac{\ln(h) - \mu_N}{\sigma_N}\right)$ $\mu_N = 7.60 \quad \sigma_N = 2.75$

4.4 Properties in close proximity to the stopbank

The population exposed to the breach flow and the fatality rate for people in houses or buildings in close proximity to the stopbank will be estimated as described in Section 3 using the details shown on Table 3-1 It should be noted that the approach for the day period is already included in the "Bathtub" approach and the potential Life Loss will not be added to the bathtub calculated data.

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Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
		(Revision 2 was issued in memo format)				
2	Malcolm Barker	Jon Williams		Martin Dasler		11/7/2020
3	Malcolm Barker/ Tim Preston	Sam Webb		Martin Dasler		18/03/2021
4	Malcolm Barker/ Tim Preston	Sam Webb		Martin Dasler		12/04/2021

Appendix B – Stopbank Weir Overtopping Calculations Memo

By Tim Preston

Memorandum

18 March 2021

To	Jo Golden		
Copy to	Tom Parsons		
From	Tim Preston	Tel	+64 3 378 0913
Subject	Overtopping weir calculations	Job no.	12/51/9849

1 Introduction

GHD are presently updating a risk to life assessment for the Avon stopbanks between Pages Rd and Bridge St (detailed report in progress “”LDRP 507 Stopbank Risk Assessment - Pages Rd to Bridge St”). A key assumption in this assessment is that bathtub flooding is a reasonable approximation to real flood risks. Bathtub flooding means that the flood event tide level (as defined at Bridge St) propagates fully across the flood plain, with flow restrictions through Bridge St, along the river channel and over the stopbanks providing negligible influence on peak flood levels outside the stopbanks.

The additional analysis presented in this memo is intended to explore the above assumption, for a scenario of overtopped stopbank levels but with no breach failure and to study, in particular, the capacity of weir flows across the stopbank crest level to fill the adjacent floodplain volume within the timing of a high tide cycle. Prior expectations were, that for some large depth of overtopping (eg: 0.5 m) there will be a huge flow capacity across the stopbank crest and the floodplain volume will fill quickly. This study is designed to focus on small and medium depths of overtopping and identify what overtopping depth is required to approximately reach bathtub flooding.

2 Methodology

2.1 Geography

The geography of study area presented in the main report was preserved for this study. In particular the upstream limits for the stopbank lengths and floodplain volume were set at Wainoni Road and Bower Ave and the downstream limits set at Breezes Road and Bridge St.

The extent of the floodplain volume was set to the centreline of the stopbank position at the river and was always defined by high ground with distance from the river. In the key flood levels of interest high ground also defined the floodplain volume upstream and downstream, however the most extreme high flood events considered floodplain volume upstream and downstream was artificially constrained by the study area boundaries.

The stopbank design crest level of 11.2 m was used for the analysis. Stopbank crest length was measured within the study area, but excluded the higher portion on the left bank downstream of Admirals Way. This resulted in crest lengths of 1,850 m and 3,100 m on the left and right banks respectively.

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2.2 Preparation

Key input analytical inputs were

1. time varying sinusoidal tide pattern
2. weir crest flow formulation
3. floodplain depth / volume relationship

The time varying sinusoid was taken from Goring 2011, 200 yr time-series. The time-shape of this peak tide was typical and the results are not expected to be sensitive to subtle differences of any particular tidal time-shape.

The weir crest formulation was represented as broad crested with Q= 1.67*LH^1.5 and free overflow conditions by assumption. Clearly, as the downstream floodplain level approaches the river/tide level this assumption is violated, and the analysis becomes invalid. However, the methodology is valid to approximate conditions where the floodplain remains materially (say 0.1 m) below the peak river level.

The river level was set to the tide level with no consideration of longitudinal flow or surface slope, nor lateral flow and surface slope toward the stopbanks.

The floodplain depth and volume relationship was determined through GIS calculations with a resolution of 0.1 m vertical slices and separate calculations for left and right banks.

2.3 Calculations

For a given peak tide level, the sinusoid tide pattern was shifted up or down to match the peak tide.

For a given bank side (left or right) a time varying xls calculation used 15 minute time steps to calculate overtopping depth and flow rates. These were time integrated to produce a total volume of overtopping flow from the peak tide cycle for that tide level and bank side.

This volume was matched to the floodplain depth volume relationship to determine the resulting floodplain depth.

The process was repeated across a range of tide levels and separately for each of left and right bank flooding.

When floodplain depth results exceeded the peak tide level the findings were disregarded. When floodplain depth results were below the peak tide level the findings are considered useful.

3 Results

In summary the results showed (all values below are presented left bank, then right bank)

- The bathtub volume under 11.2 m RL = 402006, 971983 m^3
- Duration of overflow with 11.35 m peak water level = 90 minutes
- Estimated overflow rates at peak water level = 180, 300 m^3/s
- Estimated overflow volumes = 648768, 1087124 m^3
- Estimated flood levels 11.35 m, 11.35 m
- Duration of overflow with 11.32 m peak water level = 90 minutes
- Estimated overflow rates at peak water level = 128, 215 m^3/s
- Estimated overflow volumes = 396314, 664093 $m^{3,}$ and
- Estimated flood levels 11.10 m, 10.90 m.

4 Conclusions

For a peak tide event of 11.35 m, the overflow volume on both bank sides exceeds the 11.2 m flood level volume, so the real outcome (with anticipation of what would become the submerged downstream condition) would be closely approximated to a bathtub assumption.

For a peak tide event of 11.32 m, the overflow volume on both bank sides is less than the 11.2 m flood level volume, so the free overflow calculation assumption is valid and the predicted flood level is less than bathtub is a reasonable realistic estimate.

The left bank area is faster to fill due to the right bank having the larger floodplain volume below 11.2 m, (despite the left bank having the shorter overflow length).

Within this study area, for depths of overtopping between 0-120 mm (11.32 m RL) realistic flood levels, and risks to life, would be materially better than the bathtub flood level assumption. For depths of overtopping above 150 mm (11.35 m RL) realistic flood levels, and risks to life, will be approximated well by the bathtub flood level assumption.

With respect to the broad range of peak tide levels considered in this study, the benefits in terms of lower flood level and reduced life risks from more sophisticated estimates of flood level associated with stopbank overtopping but not breach failure flood events are considered sufficiently minor that they can be neglected in the analysis.

Regards

Tim Preston
Senior Water Engineer

Appendix C – Stopbank Piping and Overtopping Failure Input Data

Christchurch City Council

Stopbank Piping and Overtopping Failure Input Data

LDRP 507 Stopbank Risk Assessment Appendix C

April 2021

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1. Definition of Stopbanks and Appurtenant Structures

1.1 Stopbank Geometry

The geometry and arrangement of the Avon River stopbank levees varies along the alignment of the river on both the left and right banks. A generalised schematic section of the River – Levee interface is shown in Figure 1-1 below.

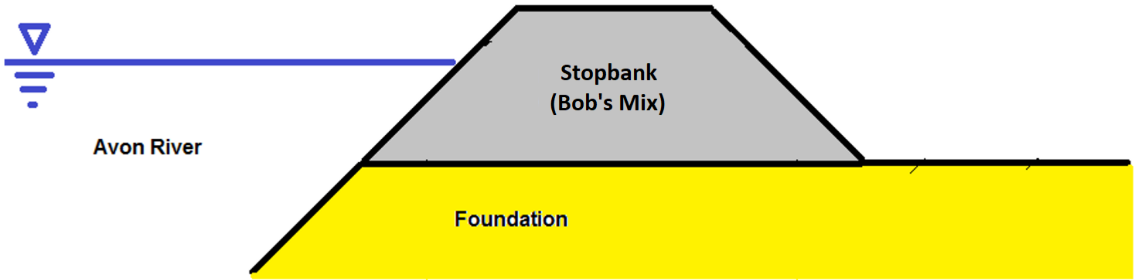


Figure 1-1 Generalised Schematic Section of River – Stopbank Interface

1.1.1 Stopbank Configuration

During temporary stopbank construction, it was agreed with Council that for ease and rapid rate of construction, the standard stopbank configuration would be constructed as follows:

- Minimum crest elevation of RL 11.2 and 11.4
- Trapezoidal cross section, crest width of 2.5 m and side slopes of 1:4 (V:H)
- Cutoff trench typically of depth 0.3 m to 1.5 m and 2.0 m wide to be taken into the original stopbank or founding material, and
- With material comprising silty gravel with maximum particle size 200 mm and containing approximately 15% fines. The material was reasonably well graded and was easily compacted. The gravel/cobble component comprised rounded or sub-rounded material
  - The material was sourced from a number of quarries and was blended at the Fulton Hogan's yard at Breezes Road. The material was placed and compacted to approximately 95% of maximum modified dry density, and
  - The permeability of this material as measured in the laboratory and an in-situ measure was carried out and ranged from 10^{-9} m/s to 10^{-6} m/s.

Typical gradings of the material used for the stopbank construction are shown on Figure 1-2.

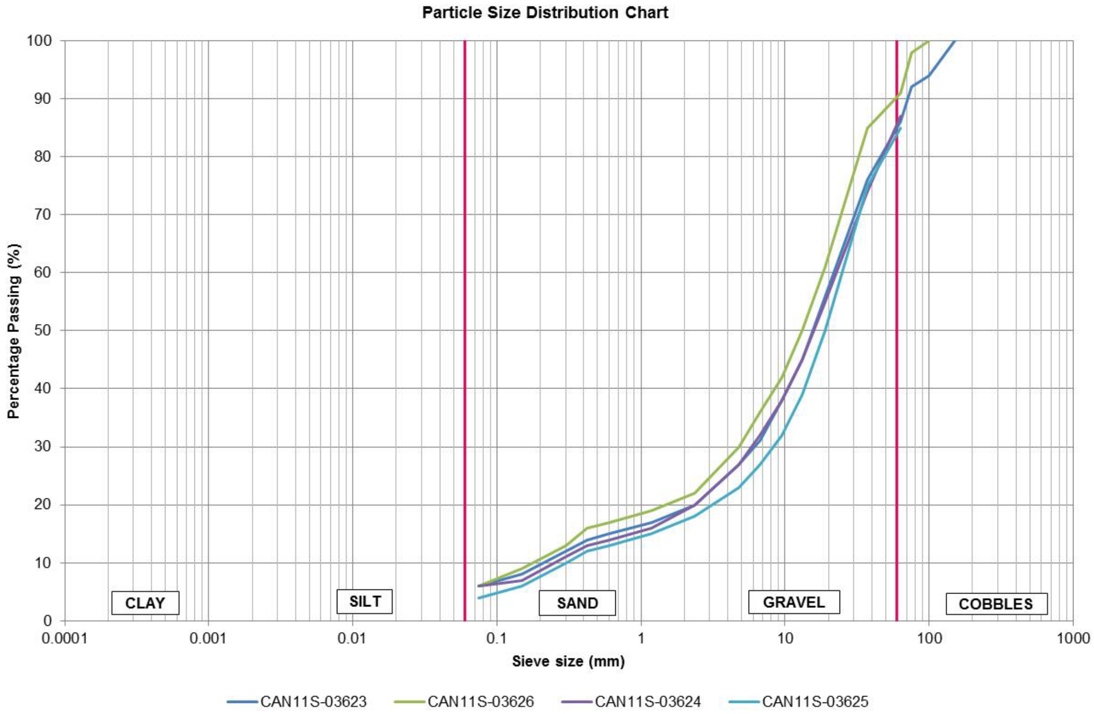


Figure 1-2 Avon Stopbank material gradings

1.2 Stopbank Data Evaluation and Analysis

1.2.1 Stability Analysis

Slope stability analyses had been completed for the 2011 emergency stopbank repairs; however, these did not cover the range of loads required for the risk analysis. Further slope stability analyses were, therefore, undertaken on five sections. Analysis was undertaken using SlopeW of the Geostudio 2012 software package. The following information was obtained for the analysis:

- Cross sectional profiles provided by survey undertaken by Davie Lovell-Smith Ltd on the 25 August 2015
- Soil profile provided by sonic boreholes to 105 m below ground level (bgl) with standard penetration tests (SPT NZS4402 Test 6.5.1 – 1988) at 1.5 m centres, and
- Particle size distribution and plasticity index tests on samples retrieved from sonic boreholes.

Stability Cases Considered

- Static – No seismic load applied and groundwater table at 1 m bgl.
- High water level – No seismic load applied and water at top of stopbank.
- Seismic – Seismic load of 0.15 g applied to slope, based on 0.5 x the pga (0.3 g) of the 23 December 2011 earthquake (USACE 1984).
- Seismic equilibrium – Seismic load applied to slope that generates a Factor of Safety of 1.

Material Parameters

The material parameters for the various zones were evaluated using the available CPT data together with the gradings and indicator test results and judgement for zones where no data was available. The parameters used for each section are shown on Table 1-1.

Table 1-1 Avon Stopbank Slope Stability analysis material parameters

Soil type	Friction angle Φ (Degrees)	Effective strength Cohesion c' (kPa)	Density (kN/m^3)
Dirty pit run	30	1	18
Gabion Foundation Fill	30	1	19
Gabion	90	500	15
Sandy SILT	22	0	17
Clayey SILT	20	2	16
SILT	21	1	17
Loose silty fine to medium SAND	26	0	17
Loose fine to medium SAND	28	0	17
Medium Dense fine to medium SAND	30	0	18

Analysis results

The results for the slope stability analyses are presented on Table 1-2 and clearly show that the Stopbank sections are unlikely to fail under static or high-water level conditions but have low factors of safety under seismic loads. This is indicative of deformation occurring, which is evidenced from past performance.

Table 1-2 Stopbank factors of safety for selection sections

Section location	Load cases			
	Static	High water table	Seismic 0.15 g (0.5 x 0.3 g)	Seismic equilibrium pga (FoS = 1)
Section 2	1.59	1.94	1.01	0.15 g
Section 17	1.32	1.71	0.76	0.07 g
Section 18	0.91	0.96	0.75	Not found

Based on the slope stability results, the failure modes associated with normal and high-water tables were dismissed for inclusion in the risk analysis as their contribution to the risk was expected to be significantly lower than the other failure modes.

1.2.2 Seismic Deformation Assessment using Historical Data

Seismic deformation analyses were completed for each Stopbank section using the available data and section geometry.

The raw data for the CPT's has been obtained from the construction report and the recent geotechnical investigations. Additional cone penetrometer tests (CPT's) including raw data near each selected Stopbank section were also obtained from the Canterbury Geotechnical Database.

Liquefaction assessment was done using CLiq (CPT Liquefaction Assessment Software) with the Boulanger and Idriss 2014 method.

Assumptions made for the analysis process were as follows:

- Importance Level 2, 50-year design life, giving peak ground accelerations (PGA's) of:
  - 0.35 g for Ultimate Limit State (ULS), and
  - 0.13 g for Serviceability Limit State (SLS)
- Earthquake Magnitude 7.5, and
- Groundwater levels at 0.0 m bgl.

Table 1-3 Historical seismic events considered in the assessment (Sections 15, 16, 17, 18 & 2 only)

Earthquake	Magnitude	PGA
4-Sep-10	7.1	0.17
22-Feb-11	6.2	0.34
13-Jun-11	6	0.25
16-Apr-11	5	0.15
23-Dec-11	5.9	0.3
SLS	7.5	0.13
ULS	7.5	0.35
MCE	6	0.19

The deformation analysis results obtained, as shown on Table 1-4 and Figure 1-3.

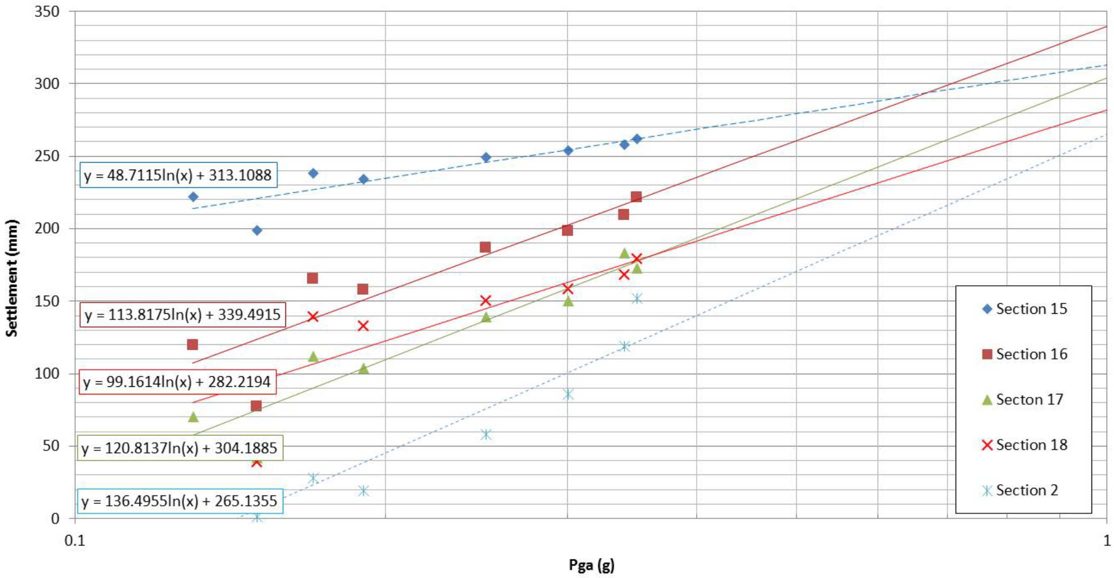


Figure 1-3 Avon Stopbanks typical deformation analysis results

The deformation results were used to estimate the likely crest settlement at each selected cross section from which to evaluate the overtopping potential given tidal fluctuations

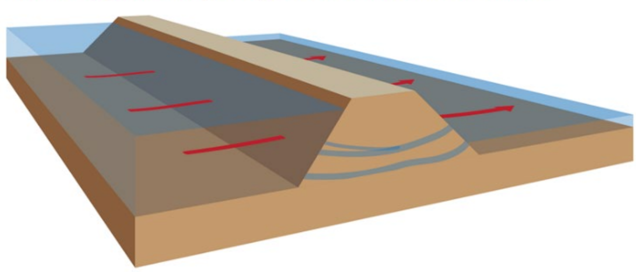
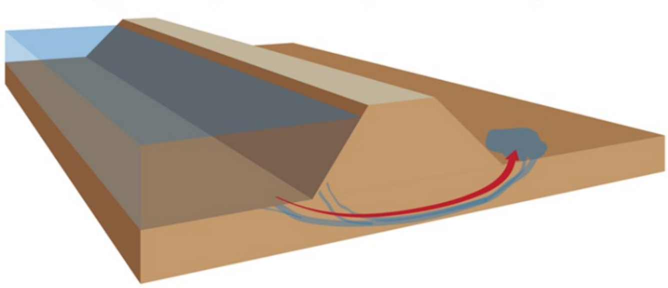
Table 1-4 Estimated stopbanks Deformations from Extrapolated Historical Seismic Data

Return period	PGA (g assumed)	Expected deformation					
		Section 17	Section 18	Section 1	Section 2	Section 3	Section 4
20	0.07	0	19	0	0	55	0
50	0.11	38	63	0	0	85	0
75	0.14	67	87	0	0	100	0
200	0.22	121	132	5	58	130	2
475	0.31	163	166	10	105	152	5
1,000	0.40	193	191	14	140	169	7
2,000	0.50	220	213	17	171	184	9
5,000	0.64	250	238	21	204	200	11
10,000	0.77	273	256	24	229	212	12
20,000	0.90	291	272	26	251	222	13

1.3 Stopbank Piping for Flood or Tidal events

1.3.1 General

The applicable failure modes are illustrated in Figure 1-4 below

Event	Initiating Event	Generalized Schematic Diagram
Piping		
Seepage through stopbank	*Tide*	
Seepage through foundation sands	*Tide*	

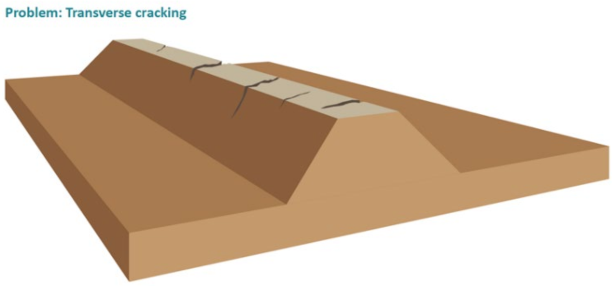
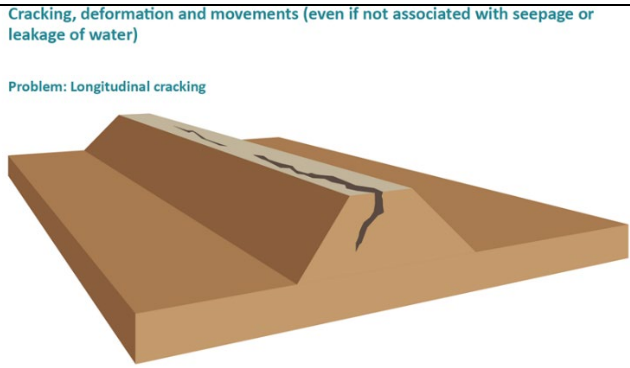
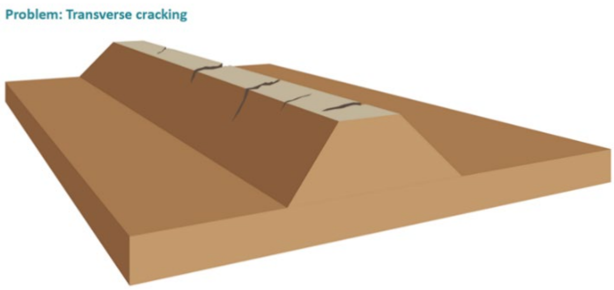
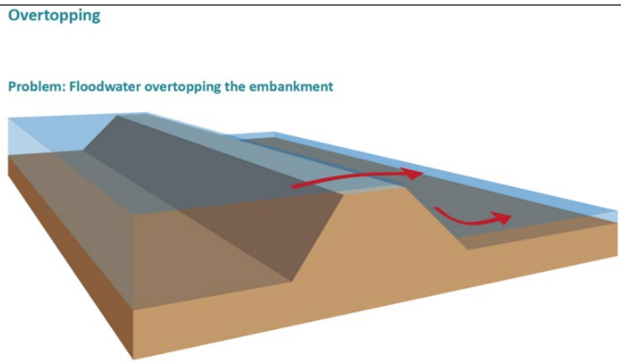
Event	Initiating Event	Generalized Schematic Diagram
Transverse cracking of the wall - Differential foundation conditions	*Earthquake / Tide*	
Longitudinal cracks - Translation (Lateral Spreading)	*Earthquake*	
Transverse cracking of the wall - Slope failure through weak foundation layers	*Earthquake*	
Overtopping		
Loss of Freeboard -	*Earthquake*	
Loss of Freeboard - Slumping (stopbank or foundation)	*Tide*	
Overtopping during tide - Settlement	*Tide*	
Longitudinal cracks - Translation (Lateral Spreading)	*Earthquake / Tide*	

Figure 1-4 Summary of Applicable Failure Modes

Failures associated with internal erosion (piping) were assessed using the Piping Toolbox (USACE et al 2008). Other probabilities in the event trees were assigned using subjective engineering judgement and the probability data provided in Table 1-5 together with engineering analysis of the failure modes.

Table 1-5 Mapping Scheme after Barneich et al (1996) ANCOLD 2003 Table 8.1

Description of condition or event	Order of magnitude probability assigned
Occurrence is virtually certain	1
Occurrences of the condition or event are observed in the database	10^{-1}
The occurrence of the condition or event is not observed, or is observed in one isolated instance, in the available database; several potential failure scenarios can be identified.	10^{-2}
The occurrence of the condition or event is not observed in the available database. It is difficult to think about any plausible failure scenario; however, a single scenario could be identified after considerable effort.	10^{-3}
The condition or event has not been observed, and no plausible scenario could be identified, even after considerable effort.	10^{-4}

1.3.2 Stopbank Piping Failure Mode Sequence

The evaluation of the piping failure modes were mostly based on the generic sequence of events presented in Figure 1-5. The process depicted in this figure is specific to flood loading but is also applicable to seismic loading as the tidal water level of the river could be at any level at the time of seismic loading. The events are described in further detail below.

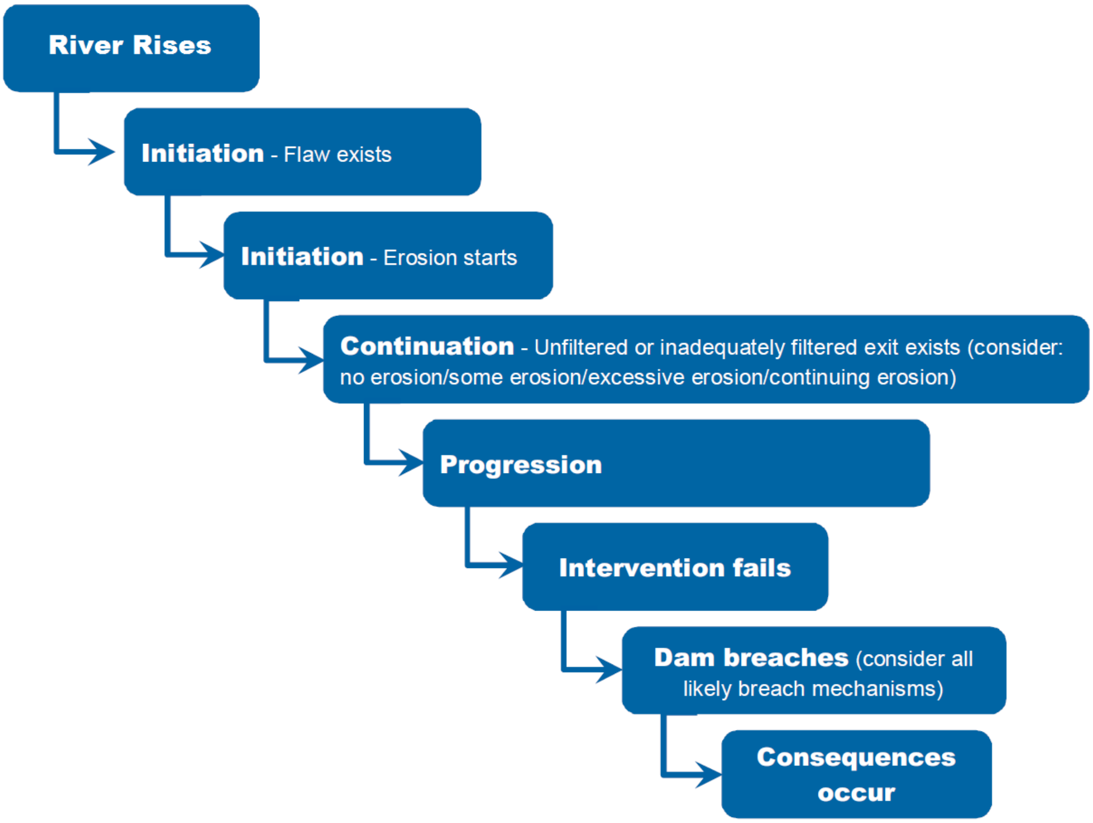


Figure 1-5 Generic Sequence of Events for Piping Failure Modes Analyses

1.3.3 Initiation

Initiation is the first phase and considers the existence of a flaw in the stopbank or the foundation. The potential flaws within the stopbank include a continuous crack or poorly compacted layer in which a concentrated leak may form. Flaws at the foundation comprise open defect or gaps within the in-filled defects or silty sands which can be prone to internal erosion under higher hydraulic gradients.

If a flaw exists, erosion must start to initiate for internal erosion to develop. There are several processes by which erosion can initiate in the stopbank or foundation as follows:

- Concentrated leak erosion. Erosion can commence from the walls of a crack within the soil or within a poorly compacted layer
- Scour at the stopbank – foundation contact. Erosion of the soil may occur where it is in contact with seepage passing through the foundation either through a coarse-grained soil or open joints in rock. In the case of the Avon Stopbanks, there is no rock foundation, and the foundation is not coarse grained
- Backward erosion. Backward erosion involves the detachment of soils particles when the seepage exits to a free unfiltered surface. The detached particles are carried away by the seepage flow and the process gradually works its way towards the upstream side of the stopbank or its foundation until a continuous pipe is formed, and
- Suffusion. This is a form of internal erosion which involves selective erosion of fine particles from the matrix of coarser particles (coarse particles are not floating in the fine particles). The fine particles are removed through the constrictions between the larger particles by seepage flow, leaving behind an intact soil skeleton formed by the coarser particles.

The potential for piping through the stopbank has considered concentrated leak erosion and backward erosion estimated using the Piping Toolbox.

The Piping Toolbox initiating mechanisms were screened as follows.

Table 1-6 Piping Toolbox Initiating Mechanisms

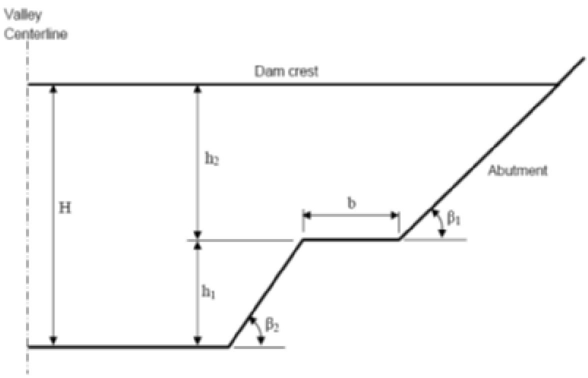
Transverse Cracks - Upper Parts of Embankment			
Initiating Mechanism	**Exclusions**	**Excludable (Yes/No)**	**Refer to**
IM1 - Transverse cracking due to cross valley differential settlement	No exclusions	No	IM1
IM2 - Transverse cracking due to differential settlement adjacent to a vertical cliff at the top of the embankment	Exclude if; (1) There is no vertical cliff with the embankment OR (2) A wide bench is present at the base of the cliff (Wb/Hw)>2.5 OR (3) The abutment slope below the cliff is gentle ($B1<25^0$) From Dimensions Entered: Excludable	Yes	IM2
IM3 - Transverse cracking due to cross valley arching	Exclude if; Width of valley to dam height ratio (Wv/Hw)>2 From Dimensions Entered: Excludable	Yes	IM3
IM4 - Transverse cracking resultant on cross section settlement	Exclude if; (1) The dam is zoning type homogeneous earthfill, earthfill with filter drains or zoned earthfill. OR (2) Evidence from relative settlements of core and shoulders that the materials have a similar modulus OR (3) Finite Element Analyses have demonstrated that stresses are such that hydraulic fracture is very unlikely.	Yes	IM4
IM5 - Transverse cracking due to differential settlements In the foundations beneath the core	Exclude if there is no compressible soil in the foundation below the core.	no	IM5
IM6 - Transverse cracking due to differential settlements due to embankment staging	Exclude if the embankment construction was not staged	Yes	IM6
IM7 - Cracking in the crest due to desiccation by drying	No exclusions	Yes	IM7
IM8 - Cracking on seasonal shutdown layers during construction and staged construction due to desiccation by drying	Exclude: (1) if the reservoir stage being considered is below the level of the seasonal shutdown surface. OR (2) This mechanism only applies above the level of saturation of the core. Below that any desiccation cracks should have swelled and closed. OR (3) This mechanism only applies where there has been a seasonal shutdown during construction, or the embankment has been staged. OR (4) Very good control and clean up practices used - desiccated layers removed from the embankment and replaced with new soil or adequately reworked to specified MC.	Yes	IM8
IM13 - Cracking due to earthquake	No exclusions *IM13A - Earthquake Hazard and Damage Class Rating* *IM13B - Probability of Transverse Cracking*	No	**IM13** IM13A IM13B

The following failure modes were evaluated for stopbank piping, which included the Piping toolbox mechanisms together with the failure mechanisms associated with trees in the Stopbanks.

- Piping through cracks in stopbank resulting from cross valley settlement and differential foundation settlement (Piping Toolbox IM1 and IM5)
- Piping through seismic induced cracks (Piping Toolbox IM13)
- Piping through rotted tree roots, and
- Piping through stopbank narrowed section caused by trees blowing over.

1.3.4 Piping Toolbox Base Data

The use of the piping toolbox requires levee geometry to evaluate cross valley arching, transverse cracking due to differential foundation movements, hydraulic fracture, etc. While the stopbanks are not major structures, nevertheless, the foundation geometry, as shown by the riverbed long section could result in differential movement and cracking through the stopbank. This was considered as follows.



Benches in the Foundation

h1	0.9
h2	
b	
B1	0.4
B2	0.4
H	0.9
b/h2	
H2/h1	
b/H	

Figure 1-6 Piping Toolbox Figure 5.1 for benching

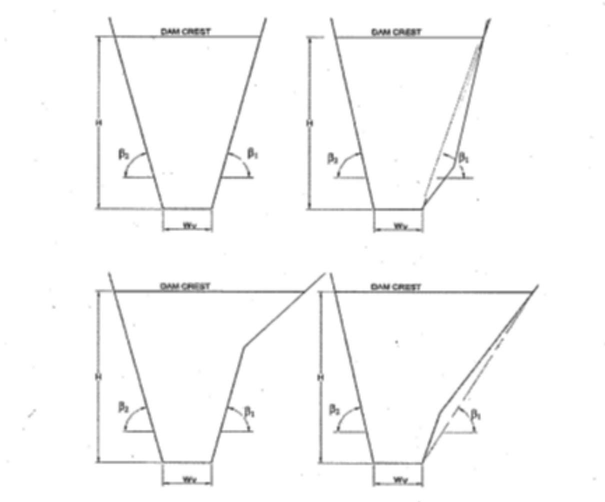


Figure 5.3 – Longitudinal profiles of the dam showing the definition of terms for cross valley arching.

Cross Valley Arching of the fill

B1	0.4
B2	0.4
Wv	20
H	0.9
Wv/H	22.22222

Figure 1-7 Piping Toolbox Figure 5.3 for cross valley arching

1.3.5 Crack formation

Cracking within the stopbank may be the result of differential movements or settlement within the foundation or cracking due to seismic deformation.

Initiating mechanism IM1

This initiating mechanism was used for evaluating the piping potential through the stopbank material.

The probability of cracks being present for IM1 was estimated according to Table 1-7 as follows for the cracks above or below the Pool of record. Pool of record is the highest historic recorded water level on the river side of the stopbank. In this assessment pool of record information was not readily available and so we substituted for this with a typical river water depth of 0.75m above the ground level outside the stopbank. This approach only influences embankment piping failure which is a minor contributor to total risk and this approximation therefore has no material influence on the conclusions.

Table 1-7 Probability of Cracking for IM1 from Piping Toolbox

IM1 Transverse Cracking Due to Cross Valley Differential Settlement *(Table 5.2, 5.3 in book)*

Excludable:	No
Dam	
b/h2=	0.00
h2/h1=	0.00
***B1*=**	0.40
Dam Height=	0.90

			Likelihood Factor			
			Less Likely	Neutral	More Likely	Much More Likely
Factor	**Relative Importance Factor (RF)**	**Rating (1-4)**	**1**	**2**	**3**	**4**
Cross valley profile under embankment core	3	2	Uniform abutment profile without benches. Narrow bench very low in the abutment. b/h2<0.5 h2/h1>1.5	Wide bench, low in the abutment. b/h2>1 h2/h1>1	Wide bench in upper half to one third of the abutment. b/h2>1; 0.5<h2/h1<1 Or narrow bench in upper half to one third of the abutment. b/h2>0.5; h2/h1<0.25	Wide bench near the crest in the abutment. b/h2>1 0<h2/h1,0.5
Slope of abutments under embankment	2	1	Gentle abutment slope *B*1<30°	Moderate abutment slopes 30°<*B1*<45°	Steep abutments 45°<*B*1<60°	Very steep abutments *B*1>60°
Height of embankment	1	1	Dams less than 15 m high.	Dams 15 to 30 m	High dams 30 to 60 m	Very high dams >60 m (for dams higher than 120 m assign a likelihood factor of 5)
RF x LF		9				

Probability vs. RF*LF:

0.00001	0.00005	0.00015	[0.0005]	0.005	0.02	Below POR
0.0001	0.0005	0.002	[0.007]	0.05	0.2	Above POR
6	9	11	13	18	24	RF x LF

Probability:

Below POR	0.00005
Above POR	0.0005

Initiating mechanism IM5

This initiating mechanism was used for evaluating the piping potential through the foundation in the event that trees fall over.

Typical scenarios which may lead to differential settlement in foundations are shown in Figure 1-8 below.

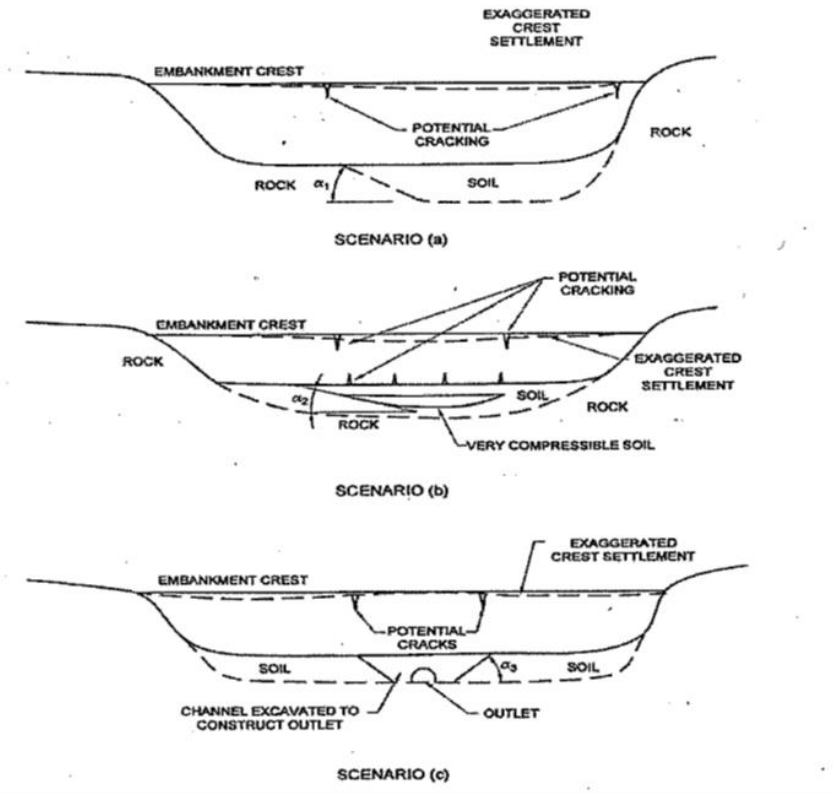


Figure 1-8 Typical scenarios which may lead to differential settlement in foundations

The probability of cracks being present for IM5 was estimated as follows for the cracks above or below the Pool of record.

IM5 Likelihood of Transverse Cracking Due to Differential Settlements in Soil in the Foundation

(Section 5.2 in book)
(Table 5.9, 5.10 in book)

Excludable: no
Dam Geometry:
α= See dia. below
H= 0.9

Factor	Relative Importance Factor (RF)	Rating (1-4)	Likelihood Factor: Less Likely 1	Neutral 2	More Likely 3	Much More Likely 4
Foundation geology and geometry	3	3	Rock foundations or uniform soil foundations.[1]	Shallow soils or soils with gradual variation in depth and compressibility sufficient to cause differential settlement of less than 0.2% of the embankment height.	Moderate depth of compressible soil in the foundation sufficient to cause differential settlement of 0.3 to 0.5% of the embankment height.	Deep compressible soil in the foundation[2] sufficient to cause differential settlement of >0.5% of the embankment height.
Slope of the sides of the compressible zones	2	1	Gentle α <30°	Moderate 30° < α <45°	Steep 45°< α < 60°	Very steep α > 60°
Height of embankment	1	1	Dams less than 15 m high.	Dams 15 to 30 m	High dams 30 to 60 m	Very high dams >60 m
RF x LF		**12**				

Probability vs. RF*LF:

negligible negligible 0.00005	0.0002	[0.0005]	0.003	0.02	Below POR
negligiblenegligible 0.0005	0.002	[0.007]	0.03	0.2	Above POR
6 8 9	11	13	18	24	RF x LF

Note: "POR" refers to the Pool of Record level + 1 foot.

Notes:
[1] If there is no compressible soil in the foundation this mode does not apply.
[2] Including soils which collapse on saturation and which have not been treated or removed during construction.

Probability:
Below POR 0.00035
Above POR 0.0035

Initiating Mechanism IM13

The initiation of piping for seismic events was completed as follows:

- Evaluate damage class for peak ground accelerations and magnitudes.

  This (Table 1-8) was developed based on our knowledge of observed damage from the Canterbury Earthquake Sequence supported by background understanding from Piping Toolbox. We found that observed damage aligned reasonably well with understanding from the Piping Toolbox and that there was no adjustment or other reconciliation required between them. Accordingly, the damage class for peak ground accelerations with representative magnitudes was evaluated for a range of events using Table 1-8 and the results are shown on Table 1-9.

Table 1-8 Avon Stopbanks Seismic loading and damage class

Earthquake peak ground acceleration	Representative earthquake magnitude	Damage class (0-4) (from figures below)
0.07	5	0
0.11	5.5	0
0.14	6	0
0.22	6.5	1
0.31	7	2
0.4	7	3

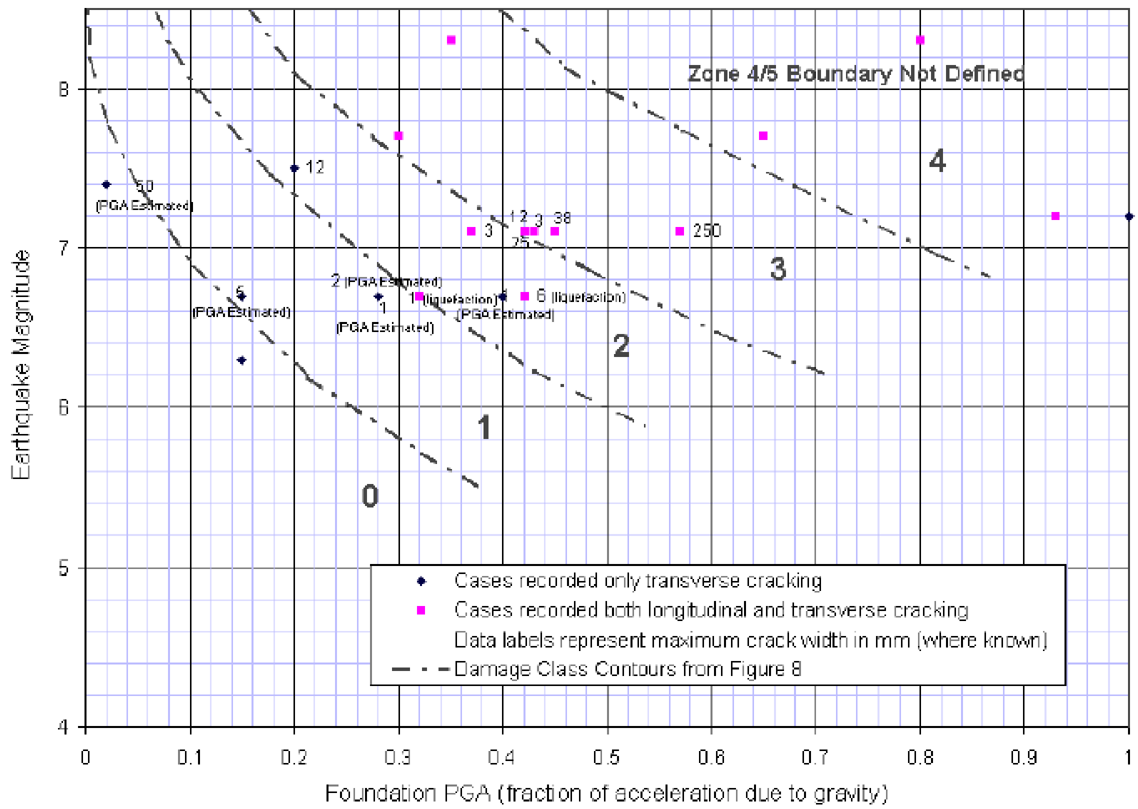


Figure 1-9 Incidence of transverse cracking versus seismic intensity and damage class contours for earthfill dams (Piping Toolbox Fig 5.8)

- Evaluate probability of cracks forming and crack widths at the Stopbank crest level.

  The probability of crack formation and estimated maximum likely crack widths for each of the representative seismic events was evaluated using Table 5.39 of the Piping Toolbox as shown in Table 1-9 and the results are shown on Table 1-10.

Table 1-9 Probability of transverse cracks in an stopbank caused by a Seismic event (Piping Toolbox Table 5.39)

Damage class	For cases where cross valley or cross section cracking assessment is in lower three "boxes" i.e. Rf x If ≤ 12	
	Probability of transverse cracking	Maximum likely crack width
0	0.001	5
1	0.01	20
2	0.05	50
3	0.2	100
4	0.5	150

Table 1-10 Avon Stopbanks Probability of transverse cracks and likely maximum crack width for selected seismic events

Failure mechanism	Rf*If	Earthquake peak ground acceleration	Damage class (0-4)	Probability of transverse cracking	Maximum likely crack width at crest (mm)
(IM1) / (IM5)	9 / 12	0.07	0	0.001	5
		0.11	0	0.001	5
		0.14	0	0.001	5
		0.22	1	0.01	20
		0.31	2	0.05	50
		0.4	3	0.2	100

The crack width at the crest was used to estimate the cracks at depth. Given the likely level of cracks and widths of cracks, the potential for piping was calculated using the hydraulic gradient at each level for tidal events with the material parameters appropriate to the stopbank material.

1.3.6 Cracking Factor

The cracking factor for adjusting the cracking potential was evaluated to be 1.0 using the following table taken from the piping toolbox.

Table 1-11 Cracking Factors from the Piping Toolbox

Factor	Influence on Likelihood			
	Less Likely	**Neutral**	**More Likely**	**Much More Likely**
Cracking observed in test pits to the top of or into the core	No cracking observed when large areas of the top of the core are exposed.	No test pits	Transverse cracks persistent across the top of the core and/or, extensive, open longitudinal cracking	Transverse cracks which pits show persist across the core, and extend below reservoir water level in the reservoir level partition being considered
Cracking Factor (A)	0.5 to 0.1 depending on the extent of exposure and how relevant the exposure is to the possible mechanism of cracking	1.0	5 to 100 depending on width(2) of cracking and whether they are in locations in which cracking might be expected	Probability of transverse crack = 1.0
Cracking in the surface of the crest, no test pits	No cracking observed, core exposed on the surface, careful inspection for cracking	No cracking observed, core covered with road pavement or other granular material	Narrow (<10mm) transverse cracks persistent across the crest and/or, extensive, narrow longitudinal cracking	Transverse cracks which persist across the crest and/or, extensive, wide longitudinal cracking.
Cracking Factor (B)	0.5 to 0.2 depending on the quality of exposure and whether they are in locations in which cracking might be expected	1.0	2 to 5 depending on and whether they are in locations in which cracking might be expected	2 to 20 depending on the width(2) of cracking and whether they are in locations in which cracking might be expected

Notes: (1) Apply either Cracking Factor (A) or Cracking Factor (B), whichever gives greatest probability of cracking
(2) The greater the crack width the more likely it represents cracking in the core.

1.3.7 Settlement Factor

The settlement factor for adjusting the cracking potential was evaluated to be 1.0 using the following table taken from the piping toolbox.

Table 1-12 Settlement factor guidelines from the Piping Toolbox

Factor		Influence on Likelihood			
		Less Likely	Neutral	More Likely	Much More Likely
Observed maximum settlements as percentage of embankment height					
- Core settlement during construction		< 1.5%	1.5% to 3%	3% to 4%	> 4%
- Post construction crest settlement at 10 years after construction dams with poorly compacted shoulders		<0.5%	0.5% to 1.0%	1.0% to 1.5%	> 1.5%
- Post construction crest settlement at 10 years after construction other dams		<0.25%	0.25% to 0.5%	0.5% to 1%	> 1%
- Long term settlement rates(% per log time cycle in years) dams with poorly compacted shoulders		< 0.15%	0.15% to 0.4%	0.4% to 0.7%	> 0.7%
- Long term settlement rates(% per log time cycle in years)-other dams		< 0.1%	0.1% to 0.25%	0.25% to 0.5%	> 0.5%
Settlement multiplication factors for cracking or hydraulic fracture in the upper part (a) of the embankment based on observed maximum settlements	**Dams with poorly compacted rockfill (b)**	0.05 to 0.2	0.2 to 0.5	1.0	2 to 5
	All other dams	0.2 to 0.5	1.0	2 to 10	10 to 20
Settlement multiplication factors for cracking or hydraulic fracture in the middle and lower parts (c)(d) of the embankment	**Dams with poorly compacted rockfill (b)**	0.2	0.2 to 0.5	1.0	2 to 5
	All other dams	0.5	1.0	2 to 5	5 to 10

Notes:
(a) Multiplication factors to be applied to Probabilities from Sections 5.2.1, 5.2.2 and 5.2.3.
(b) Includes dumped rockfill, and rockfill and other granular zones compacted by tracking with bulldozers and by small rollers in thick layers
(c) To be applied to probabilities from Sections 5.3.1, 5.3.2 and 5.3.3
(d) Multiplication factors assumed to be half those for cracking in the upper part.

A summary of the crack formation for the initiation mechanisms IM1 and IM5 is shown on Table 1-13.

Table 1-13 Crack summary for piping initiating mechanisms IM1 and IM5

Initiation mechanism	Partition	Pc (unfactored)	Settlement factor	Cracking factor	Probability of crack (pcrack)
IM1 - Transverse cracking due to cross valley differential settlement	1.00	0.00005	1	1	5.00E-05
	1.25	0.00005	1	1	5.00E-05
	1.50	0.00005	1	1	5.00E-05
	1.75	0.0005	1	1	5.00E-04
	2.00	0.0005	1	1	5.00E-04
IM5 - Transverse cracking due to differential settlements in the foundations beneath the core	1.00	0.00035	1	1	3.50E-04
	1.25	0.00035	1	1	3.50E-04
	1.50	0.00035	1	1	3.50E-04
	1.75	0.0035	1	1	3.50E-03
	2.00	0.0035	1	1	3.50E-03

1.3.8 Stopbank Crack depth and size

Given the potential crack, the size of the crack was evaluated for Initiation mechanisms IM1 and IM5 using table 5.24 of the Piping toolbox as shown on Table 1-14. The theoretical maximum likely crack width was adjusted to the assumed width based on site observations.

Table 1-14 Avon Stopbank crack width at crest for Initiating mechanisms IM1 and IM5

Crack Formation Mechanism		RL*LF	Maximum likely crack width at the dam crest relative to RL*LF (mm)					Assumed Max likely Crack Width at Crest (mm)	Theory Max likely Crack Width at Crest (mm)
			6-9	9-11	11-13	13-18	18-24		
IM1	Cross valley differential settlement	9	1	20	50	75	100	1	1
IM5	Differential settlement of the foundations	12	1	20	50	100	150	10	35

The likely crack width at depth was then calculated using Table 5.25 of the Piping Toolbox for which the cracks widths were estimated as shown on Table 1-15 and Table 1-16.

Table 1-15 Avon Stopbank crack width at depths below crest for Initiating mechanisms IM1 and IM5

Crack formation mechanism		Depth below crest level (m)				
		1.00	0.75	0.50	0.25	0.0
		Average crack width (mm)				
IM1	Cross valley differential settlement	0.1	0.2	0.3	0.4	0.5
IM5	Differential settlement of the foundations	1.0	2.0	3.0	4.0	5.0

Table 1-16 Avon Stopbank crack width at depths below crest for Initiating mechanism IM13

Maximum crack width at crest	Depth below crest level (m)				
	1.00	0.75	0.50	0.25	0.0
	Average crack width (mm)				
5	1.0	2.0	3.0	4.0	5.0
20	7.3	10.5	13.7	16.8	20.0
50	25.0	31.3	37.5	43.8	50.0
100	70.0	77.5	85.0	92.5	100.0

Given the likely level of cracks and widths of cracks, the potential for piping was calculated using the hydraulic gradient at each level with the material parameters appropriate to the stopbank material.

1.3.9 Hydraulic Gradient for Stopbank Piping

The hydraulic gradients used to assess the likelihood of piping through the stopbank where cracks are initiated were calculated for a range of partition levels. Following the seismic events, cracks were observed at various locations along the levee alignment on both the left and right banks. These cracks were mapped and can be found in New Zealand Geotechnical Database. Transverse cracks were generally observed to be diagonal to axis of the levee rather than perpendicular hence the seepage length was taken as three times the transverse width (perpendicular to the axis of the levee). The estimated piping initiation level was taken as the levee crest level after settlement (initiated by seismic loading) minus half of the original height of the levee. This information is shown schematically in Figure 1-10 below.

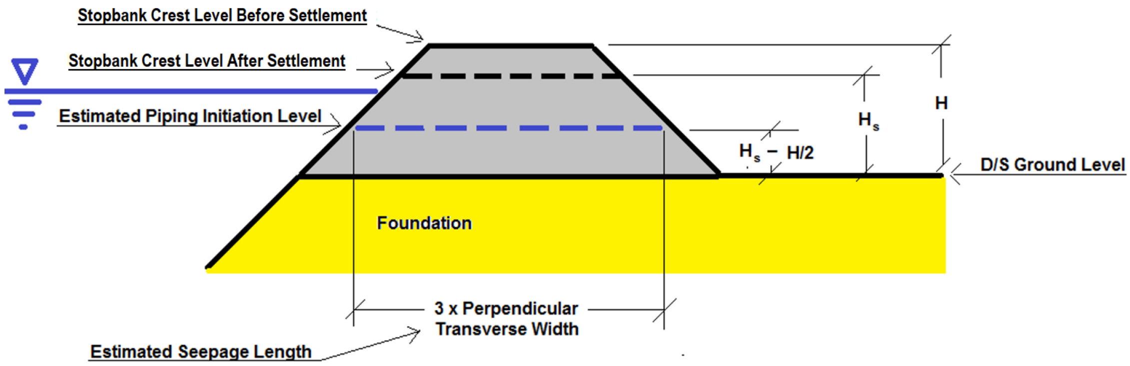


Figure 1-10 Schematic section showing the estimation of Hydraulic Gradient Initiating Piping

The hydraulic gradients were calculated for various core widths and defect levels as shown on Table 1-17.

Table 1-17 Avon Stopbanks hydraulic gradients for stopbank piping

Defect level (m)	Core width (m)	Hydraulic gradient across core when river level is at a specific level			
		0.25	0.5	0.75	1
0.1	3.00	0.08	0.17	0.25	0.33
0.25	2.50		0.10	0.20	0.30
0.50	2.00			0.13	0.25
0.75	1.50				0.17
1.00	1.00				

These hydraulic gradients were used for estimating the initiation probabilities.

1.3.10 Piping Initiation Probability Estimates

The probability of piping initiating in a crack through the stopbank given an average hydraulic gradient was estimated for the cracks at various depths within the stopbanks using Table 5.29 of the Piping toolbox for a ML or SM soil with <30% fines copied below as Table 1-18.

Table 1-18 Estimation of probability of initiation in a crack for ML or SM with <30% fines soil types (Adopted from Table 5.29 USACE (2008) and extrapolated)

Estimated crack width (mm)	Probability of initiation of erosion for different seepage gradients							
	Average hydraulic gradient							
	0	0.01	0.1	0.25	0.5	1	2	5
0	0	0	0	0	0	0	0	0
0.5	0	0.00005	0.025	0.1	0.3	0.475	0.5	0.5
1	0	0.0001	0.05	0.2	0.6	0.95	1	1
2	0	0.001	0.1	0.6	0.9	1	1	1
5	0	0.005	0.6	1	1	1	1	1
10	0	0.01	0.9	1	1	1	1	1
25	0	0.1	1	1	1	1	1	1
50	0	0.15	1	1	1	1	1	1
75	0	0.2	1	1	1	1	1	1
100	0	0.5	1	1	1	1	1	1

The probability of piping initiation given the cracks for the failure initiating mechanisms IM1 and IM5 were estimated, as shown on Table 1-19.

Table 1-19 Avon Stopbank Probability of Piping initiation for Initiating mechanisms IM1 and IM5

Initiation Mechanism	Height above Base (m)	1m Crest Width		1.5m Crest Width		2m Crest Width	
		Initiation given crack P(Init)	P(Crack) *P(Init)	Initiation given crack P(Init)	P(Crack) *P(Init)	Initiation given crack P(Init)	P(Crack) *P(Init)
IM1	0.0	0.00E+00	1.00E-08	0.00E+00	1.00E-08	0.00E+00	1.00E-08
	0.25	3.53E-03	1.77E-07	2.87E-03	1.44E-07	2.38E-03	1.19E-07
	0.50	1.22E-02	6.11E-07	1.02E-02	5.12E-07	8.76E-03	4.38E-07
	0.75	3.00E-02	1.50E-05	2.29E-02	1.14E-05	1.86E-02	9.31E-06
	1.00	5.83E-02	2.92E-05	4.64E-02	2.32E-05	3.75E-02	1.88E-05
IM5	0.0	0.00E+00	1.00E-07	0.00E+00	1.00E-07	0.00E+00	1.00E-07
	0.25	3.58E-02	1.25E-05	2.93E-02	1.03E-05	2.44E-02	8.53E-06
	0.50	2.18E-01	7.62E-05	1.83E-01	6.40E-05	1.57E-01	5.48E-05
	0.75	5.06E-01	1.77E-03	4.54E-01	1.59E-03	4.03E-01	1.41E-03
	1.00	7.78E-01	2.72E-03	7.14E-01	2.50E-03	6.67E-01	2.33E-03

The probabilities of piping initiating through the cracks resulting from seismic deformation for mechanism IM13 were calculated using the crack widths and depths from Table 1-16Table 1-18 and the data shown on Table 1-19.

1.3.11 Piping Continuation

Continuation is the phase where the relationship of the particle size distribution between the base (core or infill materials within the foundation) and the filter controls determines whether or not erosion will continue. No filter materials make up the fill of the levee bunds and therefore, a probability of 1 was assigned to the occurrence of this event.

1.3.12 Piping Progression

Progression is the third phase of internal erosion, where hydraulic shear stresses within the eroding soil may or may not lead to the enlargement of the pipe. Increases of pore pressure and seepage occur. The main issues are whether the pipe will collapse and whether upstream zones may control the erosion process by flow limitation or crack filling. The likelihood of progression was evaluated using Table 11.1 of the Piping Toolbox copied below as Table 1-20.

Table 1-20 Probability of a soil being able to support a roof to an erosion pipe (Piping Toolbox Table 11.1)

Soil Classification	Percentage Fines	Plasticity of the Fines	Moisture Condition	Likelihood of Supporting a Roof
Clays, sandy clays (CL, CH, CL-CH)	> 50%	Plastic	Moist or saturated	1.0
ML or MH	>50%	Plastic or non-plastic	Moist or saturated	1.0
Sandy clays, Gravely clays, (SC, GC)	15% - 50%	Plastic	Moist or Saturated	1.0
Silty sands, Silty gravels, Silty sandy gravel (SM, GM)	> 15%	Non plastic	Moist Saturated	0.7 to 1.0 0.5 to 1.0
Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)	5% to 15%	Plastic	Moist Saturated	0.5 to 1.0 0.2 to 0.5
Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)	5% to 15%	Non plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils, (SP, SW, GP, GW)	< 5%	Non plastic Plastic	Moist and saturated Moist and saturated	0.0001 0.001 to 0.01

Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e. not rolled), and upper bound for well compacted materials.
(2) Cemented materials give higher probabilities than indicated in the table. If soils are cemented, use the category that best describes the particular situation.

Given the granular nature of the stopbank material, the probability was assessed to be 0.001 while for the foundation soils, the continuation was taken to be 0.5, as shown in Table 1-21.

Table 1-21 Avon Stopbank piping continuation probabilities

Stopbank piping area	Height (m)			
	0.25 m	0.50 m	0.75	1.00
	Continuation probability			
Stopbank	0.001	0.001	0.001	0.001
Soil foundation (trees)	0.5	0.5	0.5	0.5

Consideration can also be given to the duration of the flood event that causes the piping initiation to determine whether the river level is sustained for the time required to progress the failure mode towards failure. At the present stage of the analysis, it has been conservatively assumed that the flood or tidal events have sufficient duration to progress the failure.

1.3.13 Piping Intervention fails

Failure to intervene is the fourth phase of the failure pathway and this considers whether the internal erosion failure mechanism will be detected and whether intervention and repair will successfully stop the failure process. Given the rapid response to the previous seismic events, the likelihood of not intervening was taken to be 0.5 for the smaller seismic and flood events to 0.9 for the larger events.

1.3.14 Piping Related Breach

StopbankBreach is the final phase of internal erosion and the following four phenomena were considered:

- Gross enlargement of the pipe (which may include the development of a sinkhole from the pipe to the crest of the stopbank)
- Slope instability of the downstream slope
- Unravelling of the downstream face, and
- Overtopping (e.g. due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the stopbank).

No differentiation has been made with respect to the breach mechanism for the risk analysis, however, given the low height of the Stopbank and construction material, the most likely breach mechanism is expected to be sloughing or unravelling for which the likelihood was evaluated using table 13.12 copied from the Piping Toolbox as Table 1-22 below. This indicates that the Probability could be between 0.1 to 1, depending on the amount of seepage that is likely to pass through the stopbank zone. The probability of breach has, therefore, been taken to be 0.5 for the low flood events to 0.9 for the largest flood event.

Table 1-22 Likelihood for Breach Mechanism (table 13.12 from Piping Toolbox)

Table 13.12			Likelihood Factor			
			Less Likely	Neutral	More Likely	Much More Likely
Factor	**Relative Importance Factor (RF)**	**Rating (1-4)**	1	2	3	4
Material in downstream zone	3	2	Cohesive Soils	Sandy Gravels (<20% fines)	Silty sand, silty sandy gravel, 20%-50% non plastic fines	As for more likely, but uncompacted materials
Freeboard at the time of incident	2	4	>4 m	3 m	2 m	1 m
Downstream Slope of the Embankment	1	4	3H:1V or flatter	2.5H:1V	2H:1V	Steeper than 1.8H:1V
RF x LF		**18**				

a. For internal erosion in the embankment, soil foundations and from embankment into foundation.

1.0	1.0	1.0	1.0	1.0	1.0	(CE)
0.01	0.05	0.1	0.5	0.9	1.0	(EE)
0.001	0.003	0.01	0.05	0.1	0.5	(SE)
6	9	11	13	18	24	

Note: Select the probability scale corresponding to the filter erosion condition being considered on the event tree.
CE = Continuing Erosion branch, EE = Excessive Erosion branch, and SE = Some Erosion branch.

1.4 Foundation Piping

The foundation was assessed for piping through the following:

- Silty Sands
- Rotted tree roots, and
- Stopbank that has been narrowed by trees blowing over.

1.4.1 Piping through Silty Sands

Piping through the silty foundation material is possible as the hydraulic gradient increases with higher tidal levels, particularly when the tide level is above any historical high level.

Sellmeijer et al. (2011) method was used to determine a critical hydraulic gradient for piping through the foundation for a range of applicable partition levels.

Water levels were adopted from the flood and tidal levels under consideration in the risk assessment. Levee geometry varied along the Avon river and was determined for each section under consideration. Figure 1-11 shows the general levee geometry and water levels used to estimate the critical hydraulic gradient required to initiate piping.

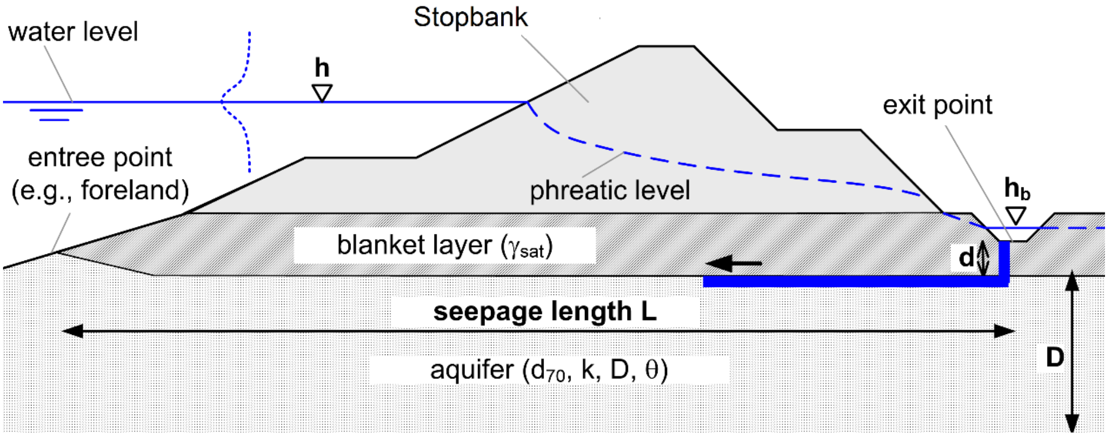


Figure 1-11 Geometry of backward erosion piping model

The formula used for evaluating the critical hydraulic gradient is shown below.

$$\frac{H_c}{L} = \frac{1}{c} = F_R\, F_S\, F_G$$

$$F_R = \eta\, \frac{\gamma'_p}{\gamma_w} \tan \vartheta \left(\frac{RD}{RD_m}\right)^{0.35}$$

$$F_S = \frac{d_{70}}{\sqrt[3]{\kappa\, L}} \left(\frac{d_{70m}}{d_{70}}\right)^{0.6}$$

$$F_G = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1} + 0.04}$$

Notations

H_c	H_c critical head over the levee [m]
γ'_p	Volumetric underwater weight of particles [kN/m^3] $\gamma'_p = \gamma_p - \gamma_w$
γ_w	Volumetric weight of water [kN/m^3]
θ	Bedding angle [°]
η	White's coefficient (= 0,25) [-]
κ	Intrinsic permeability of the sand layer [m^2] $\kappa = \nu{*}k/g = 1{,}35{*}10^{-7}{*}k$
k	Darcy permeability [m/s]
ν	Kinematic viscosity [m^2/s]
g	Gravity acceleration (g = 9,81) [m/s^2]
d_{70}	Grain size at 70-percent cumulative weight [m]
d_{70m}	Mean d_{70} of small-scale tests (d_{70m} = 2,08*10^{-4}) [m]
D	Thickness of the sand bed [m]
L	Seepage length [m]

The critical hydraulic gradient was calculated using various seepage lengths appropriate to the bund height and crest width using the data shown on Table 1-23.

Table 1-23 Avon Stopbank input data for analysis of critical seepage gradient for initiation of piping in the foundation

Description	Factor
n Whites coefficient	0.25
Particle density	2.6
Water density	1
Friction angle (degrees)	30
d70 (m)	1.00E-04
d70m (m)	2.08E-04
Permeability (m/s)	3.00E-04
Intrinsic Permeability (m/s)	4.05E-11
Layer Thickness D (m)	3
Seepage Length (m)	Varies

The probability of piping was assumed to be 0.4 with the critical hydraulic gradient ration of Head/Hc of 1.0. The relationship of the head to critical hydraulic gradient (Head/Hc) was then evaluated, as shown on Figure 1-12. This relationship was then used for evaluating the probability of piping through the stopbank sand foundations using 20% of the differential head from the river level to the ground level on the land side of the Stopbank. The factor of 20% allows for headloss through the foundation.

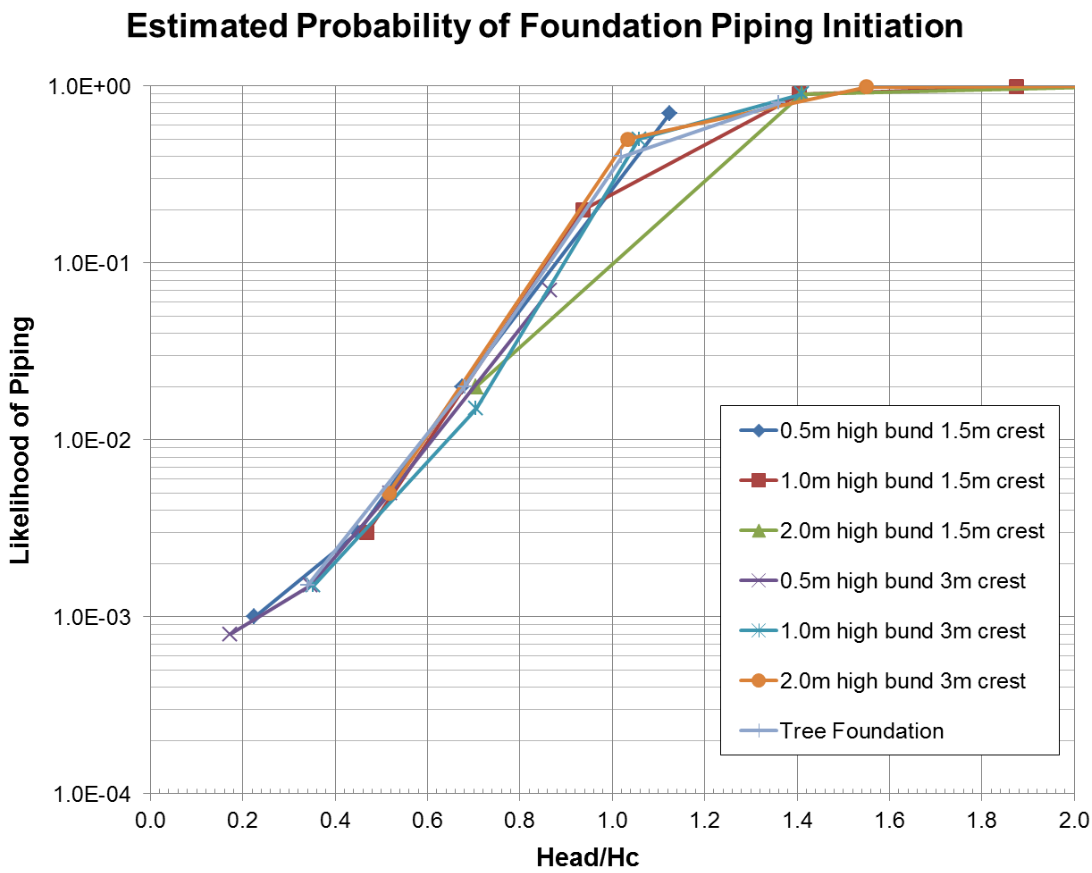


Figure 1-12 Estimated probability of foundation piping initiation for several bund geometries

The Stopbank sections where alluvial sands are present through which piping could occur were evaluated using the interpolation of the foundation probability with the head of the river above the bank level.

1.5 Overtopping Failure

1.5.1 General

This failure mode is applicable whenever the river water level exceeds the crest level of the stopbank under consideration and has been assessed for all loading conditions including the following.

- Seismic deformation loss of freeboard and overtopping
- Tides overtopping the stopbank

The failure modes were evaluated for the Stopbanks as follows.

Gravel Fill

The Avon stopbank levees have been constructed with gravel fill material (Figure 1-13), which is erodible hence with sufficient depth and velocity of overtopping flow, erosion of the levee could occur.

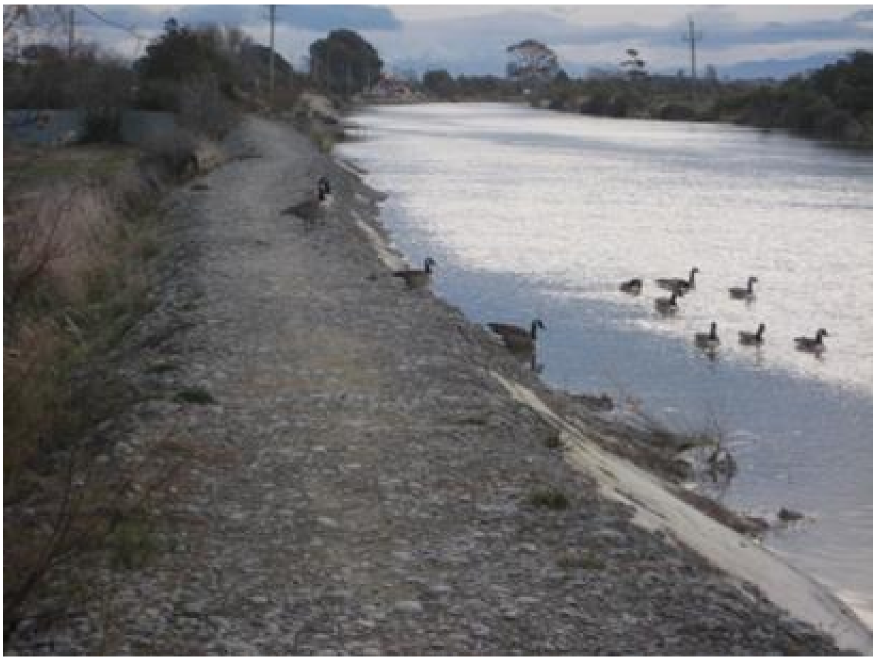


Figure 1-13 Section 17 right bank – typical gravel fill stopbank

1.5.2 Overtopping Failure Probability Analysis

Overtopping failures were assessed where the water level in the Avon River exceeded the crest height of the stopbank levee under consideration. Overtopping flow up to 500 mm flow depth was assessed as this was close to the maximum caused by the flood events under consideration in this risk assessment.

The potential for overtopping erosion failure was evaluated using data from "The International Levee Handbook", (CIRIA 2013) as follows.

The critical velocity that would likely cause erosion of the levee crest was evaluated using the data shown on Table 8.10 and Table 8.11 of the Levee handbook copied below.

Table 1-24 CIRIA Levee handbook Tables 8.10 and 8.11 critical depth velocity and correction factors

Table 8.10 Critical depth averaged velocities for loose granular material in water depth of 1 m

Material	Sieve size, *D* (mm)	Critical velocity *V* (m/s) for *h* = 1 m
Very coarse gravel	200–150	3.9–3.3
	150–100	3.3–2.7
Coarse gravel	100–75	2.7–2.4
	75–50	2.4–1.9
	50–25	1.9–1.4
	25–15	1.4–1.2
	15–10	1.2–1.0
	10–5	1.0–0.8
Gravel	5–2	0.8–0.6
Coarse sand	2–0.5	0.6–0.4
Fine sand	0.5–0.1	0.4–0.25
Very fine sand	0.1–0.02	0.25–0.20
Silt	0.02–0.002	0.20–0.15

Table 8.11 Velocity correction factors for water depths in range 0.3 m to 3 m

Depth, *h* (m)	0.3	0.6	1.0	1.5	2.0	2.5	3.0
Ki (-)	0.8	0.9	1.0	1.1	1.15	1.2	1.25

The data from the Levee handbook was then extended down to a depth of 0.05 m, as shown on Figure 1-14.

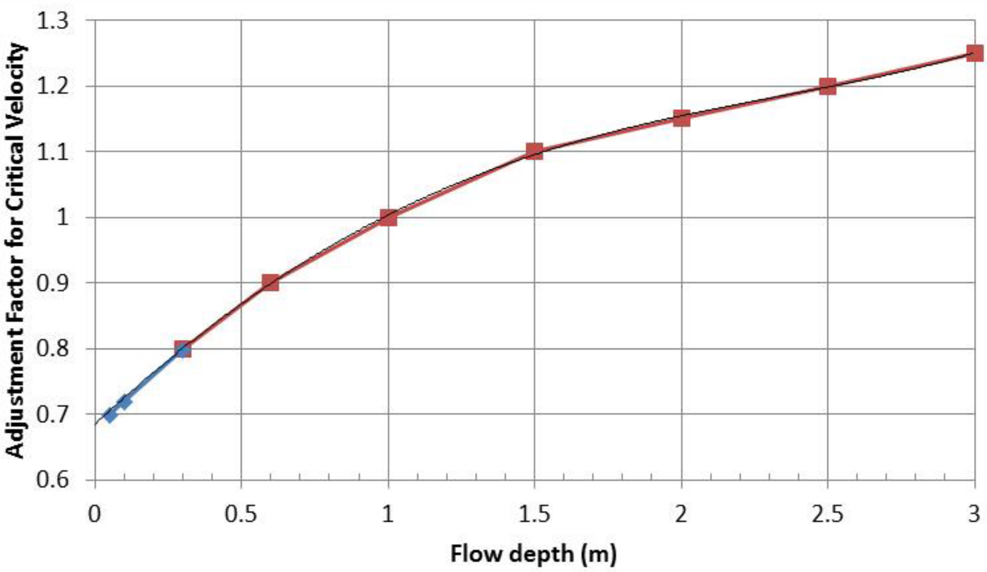


Figure 1-14 Adjustment factor for critical velocity of flow

The critical velocity of flow for each of the Stopbank material types was evaluated using the data from Table 8.10 of the Levee handbook as shown in Table 1-25.

Table 1-25 Avon Stopbank critical velocities for material types and 1 m depth of flow

Stopbank material zone	Critical erosion velocity (m/s)
Gravel 50-25 mm	1.5

Weir flow discharge for various flow depths from 0.05 m to 0.5 m over the Stopbank crest was calculated from which the critical depth and velocity were calculated using the following formula.

For a rectangular channel $Q = qb$, $B = b$ and $A = by$, and taking $\alpha = 1$ this equation becomes

$$y_c = \left(\frac{q^2}{g}\right)^{1/3}$$

as $V_c\, y_c = q$

$$V_c = \sqrt{g y_c}$$

Equation 1.18

The allowable critical velocity was estimated for each of the Stopbank material types for the flow depths varying from 0.05 m to 0.5 m and compared with the actual critical velocity from which the probability of erosion failure was assessed, as shown on Table 1-26 and Figure 1-15.

Table 1-26 Critical erosion velocities used to estimate probability of overtopping failure of levee bund fill material and sandbags

Flow depth (m)	Discharge (l/s/m)	Critical depth (m)	Critical velocity (m/s)	Levee bund fill material and sandbag material		Deteriorated sandbag material	
				Levee bund fill material and sandbag critical erosion velocity	P (Erosion)	Regular sandbag material critical erosion velocity	P (Erosion)
0.05	16.2	0.03	0.54	1.05	0.050	0.35	0.999
0.10	45.9	0.06	0.77	1.08	0.075	0.36	0.999
0.15	84.2	0.09	0.94	1.11	0.130	0.37	0.999
0.20	129.7	0.12	1.08	1.14	0.250	0.38	0.999
0.25	181.3	0.15	1.21	1.17	0.400	0.39	0.999
0.30	238.3	0.18	1.33	1.20	0.600	0.40	0.999
0.35	300.2	0.21	1.43	1.23	0.800	0.41	0.999
0.40	366.8	0.24	1.53	1.25	0.900	0.42	0.999
0.45	437.7	0.27	1.63	1.28	0.950	0.43	0.999
0.50	512.7	0.30	1.71	1.30	0.999	0.43	0.999

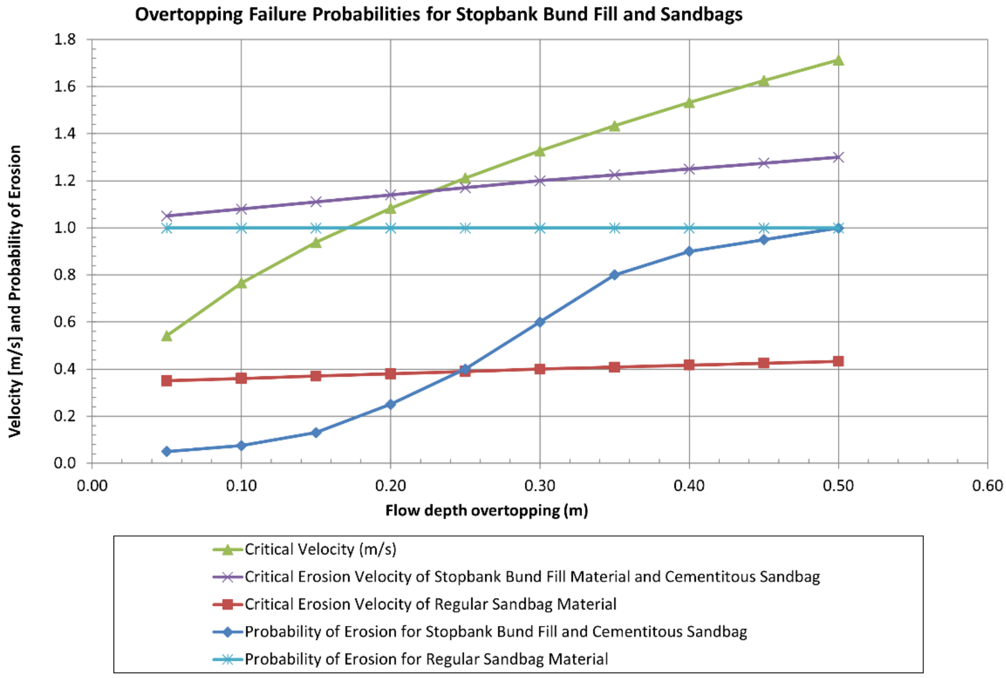


Figure 1-15 Estimated Probability of Overtopping Failure for Range of Overtopping Flow Depths

The depth of overtopping for each Stopbank section was calculated using the tidal levels with or without seismic deformation and the flood levels without seismic deformation. The depth was then used to interpolate the probability of overtopping erosion failure for the material type appropriate to each Section.

1.6 Common Cause Adjustment

The common cause adjustment described below was applied to the lifetime failure probabilities rather than the individual failure modes for which it is commonly used. This was owing to expediency and simplification of the analysis process. Common cause adjustment is required where a flood or seismic event may cause multiple sections to fail with the same event.

The lifetime (1, 5,10, 20 years) failure probabilities for the various sections associated with the same seismic, flood or tidal event were, therefore, adjusted using the uni-modal bounds theorem (Ang and Tang, 1984) (de Morgan's rule).

The conditional probabilities for the failure modes that are not mutually exclusive can be adjusted for common cause occurrence by using the uni-modal bounds theorem. The unimodal bounds theorem (Ang and Tang, 1984) states that for k positively correlated failure modes, with conditional branch failure probabilities (system response probabilities), p_i, the system (total) branch failure probability, p_f, lies between the following upper (u) and lower (l) bounds:

$$max_i[p_i] \leq p_f \leq 1 - \prod_{i=1}^{k}(1 - p_i)$$

$$p_f^l \leq p_f \leq p_f^u$$

While the uni-modal bounds theorem provides an approach to bounding the total branch failure probability, it does not provide a direct means of bounding individual failure mode probabilities. This latter adjustment is normally needed because the consequences associated with each failure mode or section may differ. In the case of the Stopbank levees, the combined risk for each section with the Seismic and Flow or Tidal events have been adjusted rather than the individual failure modes.

While there is no unique approach to adjusting each system response probability, the following approach is proposed by Bowles et al (2001) was used to adjust the seismic, flood and tidal hazard data. The upper bound (u) was used to adjust the failure probabilities for each of the Stopbank lifetime failure probabilities, using the following formula:

$$p_i^u = p_i(p_f^u / p_f)$$

where p_f is the total probability of failure without the application of the uni-modal bounds theorem i.e. the total of the failure modes derived by addition. The adjustment was made simultaneously for all Stopbank sections for each lifetime and the resulting adjusted values used for the failure probability estimation for each lifetime.

2. References

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Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
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1	Malcolm Barker/ Tim Preston	Sam Webb		Martin Dasler		18/03/2021
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Appendix D – Stopbank Piping and Overtopping Risk Analysis Results using Goring Tidal Data

Section 1

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	0.00E+00	0.00E+00	1.23E-05	2.30E-05	3.35E-05	5.80E-05	6.86E-05	1.68E-04
	0.00E+00	0.00E+00	1.23E-05	2.30E-05	3.35E-05	5.80E-05	6.86E-05	1.68E-04
	0.00E+00	0.00E+00	1.23E-05	2.30E-05	3.35E-05	5.80E-05	6.86E-05	1.68E-04
	0.00E+00	0.00E+00	1.18E-04	2.14E-04	3.08E-04	5.28E-04	6.23E-04	6.94E-04
	0.00E+00	0.00E+00	2.78E-04	4.18E-04	5.56E-04	8.76E-04	1.01E-03	1.08E-03
	0.00E+00	0.00E+00	5.08E-04	6.77E-04	8.44E-04	1.23E-03	1.40E-03	1.47E-03
	0.00E+00	0.00E+00	1.14E-03	1.28E-03	1.41E-03	1.73E-03	1.87E-03	1.92E-03
	0.00E+00	0.00E+00	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03
	0.00E+00	0.00E+00	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03
	0.00E+00	0.00E+00	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03	2.43E-03
	0.00E+00	0.00E+00	3.80E-07	5.53E-07	7.24E-07	1.12E-06	1.29E-06	2.09E-06
P(Failure)	0.00E+00	1.52E-08	4.67E-09	3.19E-09	3.69E-09	6.02E-10	6.75E-10	2.09E-10
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	5.41E-10	9.14E-10	2.06E-09	4.13E-09	1.42E-09	2.20E-09	1.05E-09
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	7.13E-05	1.59E-04	2.23E-04	2.50E-04	2.77E-04	3.40E-04	3.67E-04	1.03E-03
P(Failure)	1.04E-04	1.53E-05	2.36E-06	1.32E-06	1.23E-06	1.77E-07	2.79E-07	1.03E-07
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	5.43E-07	4.63E-07	8.52E-07	1.38E-06	4.16E-07	9.07E-07	5.17E-07

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.67E-01	9.80E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.73E-01	9.84E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.43E-01	1.79E-01	9.87E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.43E-01	1.83E-01	9.89E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.43E-01	1.88E-01	9.91E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.43E-01	1.92E-01	9.92E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.43E-01	1.95E-01	9.93E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	2.38E-05	1.43E-04	8.18E-04
P(Failure)	0.00E+00	0.00E+00	0.00E+00	0.00E+00	4.75E-08	4.16E-08	1.92E-07	8.18E-08
PLL	0.000	0.036	0.196	0.647	1.187	2.421	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.64E-08	1.01E-07	6.38E-07	4.17E-07
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	7.22E-11	2.72E-10	3.49E-09	5.52E-09	7.54E-09	1.25E-08	1.46E-08	2.21E-08
P(Failure)	1.55E-10	1.51E-10	4.50E-11	3.27E-11	4.00E-11	6.77E-12	7.33E-12	2.21E-12
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	5.36E-12	8.82E-12	2.11E-11	4.48E-11	1.59E-11	2.39E-11	1.11E-11

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.60E-01	9.75E-01
P(Failure)	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	4.00E-05	2.27E-04	9.75E-05
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	9.42E-05	7.53E-04	4.97E-04

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	0.00E+00	5.96E-06	1.41E-05	1.76E-05	2.10E-05	3.72E-05	4.76E-05	8.43E-05
	0.00E+00	5.96E-06	1.41E-05	1.76E-05	2.10E-05	3.72E-05	4.76E-05	8.43E-05
	0.00E+00	5.96E-06	1.41E-05	1.76E-05	2.10E-05	3.72E-05	4.76E-05	8.43E-05
	2.52E-09	7.79E-05	1.51E-04	1.83E-04	2.14E-04	2.29E-04	2.31E-04	2.38E-04
	2.81E-05	1.74E-04	2.81E-04	3.26E-04	3.60E-04	3.60E-04	3.60E-04	3.60E-04
	1.14E-04	2.90E-04	4.19E-04	4.75E-04	4.90E-04	4.90E-04	4.90E-04	4.90E-04
	3.51E-04	4.95E-04	6.00E-04	6.40E-04	6.40E-04	6.40E-04	6.40E-04	6.40E-04
	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	5.92E-08	1.90E-07	3.22E-07	3.78E-07	4.26E-07	5.54E-07	6.33E-07	9.10E-07
P(Failure)	1.12E-07	2.05E-08	3.50E-09	2.01E-09	1.96E-09	2.97E-10	3.09E-10	9.10E-11
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	7.28E-10	6.86E-10	1.30E-09	2.20E-09	6.99E-10	1.00E-09	4.58E-10
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	5.29E-05	8.21E-05	1.03E-04	1.13E-04	1.22E-04	1.42E-04	2.21E-04	1.77E-03
P(Failure)	6.08E-05	7.42E-06	1.08E-06	5.85E-07	5.28E-07	9.07E-08	3.98E-07	1.77E-07
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	2.64E-07	2.11E-07	3.78E-07	5.91E-07	2.14E-07	1.30E-06	8.91E-07

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	6.12E-02	3.89E-01	6.86E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	5.00E-02	9.62E-02	5.73E-01	8.37E-01	9.99E-01
	0.00E+00	0.00E+00	5.00E-02	6.45E-02	1.40E-01	7.12E-01	9.03E-01	9.99E-01
	0.00E+00	0.00E+00	5.00E-02	8.55E-02	2.13E-01	8.17E-01	9.34E-01	9.99E-01
	0.00E+00	0.00E+00	5.86E-02	1.23E-01	3.05E-01	8.84E-01	9.67E-01	9.99E-01
	0.00E+00	0.00E+00	7.12E-02	1.74E-01	3.80E-01	9.17E-01	9.92E-01	9.99E-01
	0.00E+00	0.00E+00	9.01E-02	2.25E-01	4.59E-01	9.39E-01	9.99E-01	9.99E-01
	0.00E+00	0.00E+00	8.99E-06	2.42E-05	8.16E-05	4.24E-04	6.39E-04	8.32E-04
P(Failure)	0.00E+00	3.59E-07	1.66E-07	2.65E-07	1.01E-06	2.66E-07	2.94E-07	8.32E-08
PLL	0.000	0.036	0.263	0.714	1.187	2.421	3.319	5.093
Risk	0.00E+00	1.28E-08	4.37E-08	1.89E-07	1.20E-06	6.43E-07	9.77E-07	4.24E-07
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	9.07E-11	1.74E-09	3.38E-09	4.10E-09	4.81E-09	6.45E-09	7.16E-09	9.65E-09
P(Failure)	8.24E-10	2.05E-10	3.74E-11	2.23E-11	2.25E-11	3.40E-12	3.36E-12	9.65E-13
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	7.29E-12	7.33E-12	1.44E-11	2.52E-11	8.01E-12	1.09E-11	4.85E-12

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.21E-01	4.52E-01	9.99E-01
P(Failure)	0.00E+00	0.00E+00	0.00E+00	1.25E-04	5.42E-04	1.68E-04	2.90E-04	9.99E-05
PLL	0.000	0.036	0.196	0.647	1.187	2.421	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	8.08E-05	6.44E-04	4.07E-04	9.63E-04	5.09E-04

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	1.57E-06	7.76E-06	2.21E-05	3.51E-05	1.01E-04
	0.00E+00	0.00E+00	2.10E-08	4.01E-06	1.02E-05	2.46E-05	4.24E-05	1.08E-04
	0.00E+00	0.00E+00	1.39E-07	5.31E-06	1.15E-05	2.76E-05	4.64E-05	1.12E-04
	0.00E+00	0.00E+00	1.34E-05	6.98E-05	1.26E-04	2.27E-04	2.31E-04	2.44E-04
	0.00E+00	0.00E+00	7.61E-05	1.58E-04	2.39E-04	3.60E-04	3.60E-04	3.60E-04
	0.00E+00	0.00E+00	1.68E-04	2.68E-04	3.66E-04	4.90E-04	4.90E-04	4.90E-04
	0.00E+00	0.00E+00	3.93E-04	4.74E-04	5.54E-04	6.40E-04	6.40E-04	6.40E-04
	0.00E+00	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	0.00E+00	8.14E-08	1.55E-07	2.56E-07	4.51E-07	5.63E-07	1.06E-06
P(Failure)	0.00E+00	3.26E-09	1.18E-09	1.03E-09	1.41E-09	2.54E-10	3.25E-10	1.06E-10
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	1.16E-10	2.32E-10	6.64E-10	1.58E-09	5.97E-10	1.06E-09	5.34E-10
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	1.00E-10	1.00E-10	1.00E-10	5.91E-06	1.49E-05	3.58E-05	4.48E-05	7.64E-05
P(Failure)	9.00E-11	8.00E-12	2.95E-08	5.20E-08	1.01E-07	2.01E-08	2.42E-08	7.64E-09
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	2.84E-13	5.78E-09	3.36E-08	1.14E-07	4.74E-08	7.88E-08	3.84E-08

Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
	Tide Interval							
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	5.00E-02	7.43E-02	4.91E-01	7.91E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	5.00E-02	9.09E-02	5.54E-01	8.27E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	5.95E-02	1.23E-01	6.72E-01	8.86E-01	9.99E-01
	0.00E+00	0.00E+00	5.00E-02	7.07E-02	1.69E-01	7.61E-01	9.15E-01	9.99E-01
	0.00E+00	0.00E+00	5.00E-02	8.37E-02	2.09E-01	8.14E-01	9.32E-01	9.99E-01
	0.00E+00	0.00E+00	5.00E-02	9.98E-02	2.44E-01	8.43E-01	9.47E-01	9.99E-01
	0.00E+00	0.00E+00	5.63E-02	1.17E-01	2.91E-01	8.75E-01	9.62E-01	9.99E-01
	0.00E+00	0.00E+00	6.23E-02	1.32E-01	3.27E-01	8.99E-01	9.74E-01	9.99E-01
	0.00E+00	0.00E+00	6.74E-02	1.56E-01	3.58E-01	9.10E-01	9.84E-01	9.99E-01
	0.00E+00	0.00E+00	1.79E-05	1.83E-04	3.39E-04	1.92E-03	2.78E-03	3.32E-03
P(Failure)	0.00E+00	7.16E-07	1.00E-06	1.30E-06	4.51E-06	1.17E-06	1.22E-06	3.32E-07
PLL	0.000	0.036	0.263	0.714	1.187	2.421	3.319	5.093
Risk	0.00E+00	2.55E-08	2.64E-07	9.31E-07	5.35E-06	2.84E-06	4.05E-06	1.69E-06
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	2.50E-12	2.50E-12	2.50E-12	7.86E-12	1.60E-11	4.51E-11	6.90E-11	1.33E-09
P(Failure)	2.25E-12	2.00E-13	5.18E-14	5.97E-14	1.22E-13	2.85E-14	2.79E-13	1.33E-13
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	7.11E-15	1.01E-14	3.86E-14	1.37E-13	6.72E-14	9.07E-13	6.66E-13

Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
	Tide Interval							
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.21E-01	4.52E-01	9.99E-01
P(Failure)	0.00E+00	0.00E+00	0.00E+00	1.25E-04	5.42E-04	1.68E-04	2.90E-04	9.99E-05
PLL	0.000	0.036	0.196	0.647	1.187	2.421	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	8.08E-05	6.44E-04	4.07E-04	9.63E-04	5.09E-04

Section 4

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	0.00E+00	2.63E-06	1.15E-05	1.53E-05	1.91E-05	3.33E-05	4.47E-05	8.46E-05
	0.00E+00	2.63E-06	1.15E-05	1.53E-05	1.91E-05	3.33E-05	4.47E-05	8.46E-05
	0.00E+00	2.63E-06	1.15E-05	1.53E-05	1.91E-05	3.33E-05	4.47E-05	8.46E-05
	0.00E+00	2.49E-05	1.05E-04	1.39E-04	1.73E-04	2.27E-04	2.29E-04	2.37E-04
	0.00E+00	7.03E-05	1.87E-04	2.36E-04	2.85E-04	3.60E-04	3.60E-04	3.60E-04
	0.00E+00	1.41E-04	2.82E-04	3.42E-04	4.01E-04	4.90E-04	4.90E-04	4.90E-04
	0.00E+00	3.56E-04	4.71E-04	5.20E-04	5.69E-04	6.40E-04	6.40E-04	6.40E-04
	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04	8.10E-04
	0.00E+00	1.01E-07	2.45E-07	3.06E-07	3.67E-07	5.24E-07	6.10E-07	9.12E-07
P(Failure)	4.55E-08	1.38E-08	2.76E-09	1.68E-09	1.78E-09	2.84E-10	3.04E-10	9.12E-11
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	4.92E-10	5.40E-10	1.09E-09	2.00E-09	6.68E-10	9.90E-10	4.58E-10
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	3.33E-05	6.24E-05	8.38E-05	9.29E-05	1.02E-04	1.23E-04	1.32E-04	8.09E-04
P(Failure)	4.31E-05	5.85E-06	8.84E-07	4.87E-07	4.49E-07	6.36E-08	1.88E-07	8.09E-08
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	2.08E-07	1.73E-07	3.15E-07	5.03E-07	1.50E-07	6.12E-07	4.07E-07

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.27E-01	4.61E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.33E-01	4.72E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.38E-01	4.80E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.42E-01	4.87E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.47E-01	4.94E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.50E-01	5.00E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.54E-01	5.05E-01	9.99E-01
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	4.16E-05	1.92E-04	3.90E-04	8.32E-04
P(Failure)	0.00E+00	0.00E+00	0.00E+00	1.04E-07	4.68E-07	1.46E-07	2.44E-07	8.32E-08
PLL	0.000	0.036	0.196	0.714	1.187	2.421	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	7.43E-08	5.56E-07	3.52E-07	8.11E-07	4.24E-07
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	3.84E-11	2.89E-10	1.87E-09	2.56E-09	3.26E-09	4.91E-09	5.62E-09	8.11E-09
P(Failure)	1.47E-10	8.63E-11	2.21E-11	1.46E-11	1.63E-11	2.63E-12	2.74E-12	8.11E-13
PLL	0.000	0.036	0.196	0.647	1.120	2.354	3.252	5.026
Risk	0.00E+00	3.07E-12	4.33E-12	9.41E-12	1.83E-11	6.19E-12	8.92E-12	4.07E-12

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	5.00E-02	2.21E-01	4.52E-01	9.99E-01
P(Failure)	0.00E+00	0.00E+00	0.00E+00	1.25E-04	5.42E-04	1.68E-04	2.90E-04	9.99E-05
PLL	0.000	0.036	0.196	0.647	1.187	2.421	3.319	5.093
Risk	0.00E+00	0.00E+00	0.00E+00	8.08E-05	6.44E-04	4.07E-04	9.63E-04	5.09E-04

Section 17

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tide Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	5.38E-07	2.03E-05	3.48E-05	4.10E-05	4.71E-05	8.41E-05	1.03E-04	1.68E-04
	3.59E-06	2.34E-05	3.79E-05	4.40E-05	5.05E-05	9.34E-05	1.12E-04	1.77E-04
	5.96E-06	2.57E-05	4.02E-05	4.64E-05	5.77E-05	1.01E-04	1.19E-04	1.84E-04
	9.41E-05	2.72E-04	4.02E-04	4.50E-04	4.54E-04	4.63E-04	4.66E-04	4.79E-04
	2.46E-04	5.05E-04	6.94E-04	7.20E-04	7.20E-04	7.20E-04	7.20E-04	7.20E-04
	4.46E-04	7.59E-04	9.80E-04	9.80E-04	9.80E-04	9.80E-04	9.80E-04	9.80E-04
	8.72E-04	1.13E-03	1.28E-03	1.28E-03	1.28E-03	1.28E-03	1.28E-03	1.28E-03
	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03
	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03
	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03	1.62E-03
	2.40E-07	5.60E-07	7.92E-07	8.66E-07	9.21E-07	1.22E-06	1.36E-06	1.85E-06
P(Failure)	3.60E-07	5.41E-08	8.29E-09	4.47E-09	4.28E-09	6.43E-10	6.41E-10	1.85E-10
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	5.10E-08	1.46E-08	4.09E-09	3.16E-09	3.72E-09	8.05E-10	9.99E-10	3.99E-10
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	1.67E-04	2.25E-04	2.68E-04	2.86E-04	4.86E-04	2.53E-03	3.41E-03	6.51E-03
P(Failure)	1.76E-04	1.97E-05	2.77E-06	1.93E-06	6.03E-06	1.49E-06	1.98E-06	6.51E-07
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	2.49E-05	5.31E-06	1.37E-06	1.36E-06	5.25E-06	1.86E-06	3.09E-06	1.40E-06

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tide Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	9.89E-02	5.46E-01	8.42E-01	1.00E+00
	0.00E+00	0.00E+00	0.00E+00	9.75E-02	1.26E-01	6.62E-01	9.21E-01	1.00E+00
	0.00E+00	0.00E+00	0.00E+00	1.07E-01	2.15E-01	8.68E-01	9.83E-01	1.00E+00
	0.00E+00	0.00E+00	9.75E-02	1.48E-01	3.50E-01	9.61E-01	9.94E-01	1.00E+00
	0.00E+00	0.00E+00	1.04E-01	2.09E-01	4.71E-01	9.81E-01	9.98E-01	1.00E+00
	0.00E+00	0.00E+00	1.29E-01	2.82E-01	5.82E-01	9.92E-01	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	1.71E-01	3.98E-01	7.05E-01	9.96E-01	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	2.15E-01	4.89E-01	7.94E-01	9.98E-01	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	2.62E-01	5.66E-01	8.57E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	3.99E-05	2.61E-04	5.47E-04	2.28E-03	3.03E-03	3.33E-03
P(Failure)	0.00E+00	1.59E-06	1.50E-06	2.02E-06	5.66E-06	1.33E-06	1.27E-06	3.33E-07
PLL	0.142	0.337	0.494	0.774	0.937	1.318	1.626	2.224
Risk	0.00E+00	5.37E-07	7.43E-07	1.56E-06	5.31E-06	1.75E-06	2.07E-06	7.40E-07
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	3.66E-09	8.18E-09	1.15E-08	1.30E-08	1.44E-08	1.77E-08	1.91E-08	2.41E-08
P(Failure)	5.33E-09	7.89E-10	1.23E-10	6.84E-11	6.42E-11	9.20E-12	8.64E-12	2.41E-12
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	7.55E-10	2.13E-10	6.05E-11	4.84E-11	5.58E-11	1.15E-11	1.35E-11	5.19E-12

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tide Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	9.75E-02	3.93E-01	7.00E-01	1.00E+00
	0.00E+00	0.00E+00	0.00E+00	2.44E-04	9.82E-04	2.73E-04	3.40E-04	1.00E-04
	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	0.00E+00	0.00E+00	0.00E+00	1.72E-04	8.55E-04	3.42E-04	5.30E-04	2.16E-04

Section 18

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping	Eq Piping
	Embankment Piping Probability Earthquake followed by a Tide							
	0.00E+00	2.40E-05	5.74E-05	7.16E-05	8.57E-05	1.55E-04	1.98E-04	3.48E-04
	0.00E+00	3.24E-05	6.58E-05	8.00E-05	9.41E-05	1.81E-04	2.24E-04	3.73E-04
	1.21E-07	3.69E-05	7.03E-05	8.45E-05	9.86E-05	1.95E-04	2.37E-04	3.87E-04
	8.85E-06	4.08E-04	7.08E-04	8.36E-04	9.04E-04	9.24E-04	9.32E-04	9.62E-04
	2.11E-04	8.07E-04	1.24E-03	1.43E-03	1.44E-03	1.44E-03	1.44E-03	1.44E-03
	5.48E-04	1.27E-03	1.80E-03	1.96E-03	1.96E-03	1.96E-03	1.96E-03	1.96E-03
	1.46E-03	2.05E-03	2.48E-03	2.56E-03	2.56E-03	2.56E-03	2.56E-03	2.56E-03
	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03
	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03
	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03	3.24E-03
	2.70E-07	8.68E-07	1.41E-06	1.63E-06	1.77E-06	2.35E-06	2.67E-06	3.81E-06
P(Failure)	5.12E-07	9.10E-08	1.52E-08	8.48E-09	8.24E-09	1.26E-09	1.30E-09	3.81E-10
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	7.25E-08	2.45E-08	7.49E-09	6.00E-09	7.17E-09	1.57E-09	2.02E-09	8.21E-10
	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe	Fnd Pipe
	Foundation Piping Given a tide							
	3.82E-04	6.15E-04	7.86E-04	8.58E-04	9.30E-04	1.10E-03	1.17E-03	7.32E-02
P(Failure)	4.48E-04	5.60E-05	8.22E-06	4.47E-06	4.06E-06	5.67E-07	1.49E-05	7.32E-06
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	6.35E-05	1.51E-05	4.06E-06	3.16E-06	3.53E-06	7.09E-07	2.32E-05	1.58E-05

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop	Eq Otop
	Overtopping Earthquake followed by a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	0.00E+00	0.00E+00	0.00E+00	1.85E-01	2.31E-01	8.75E-01	9.92E-01	1.00E+00
	0.00E+00	0.00E+00	0.00E+00	1.85E-01	2.72E-01	9.38E-01	9.98E-01	1.00E+00
	0.00E+00	0.00E+00	0.00E+00	2.21E-01	4.16E-01	9.90E-01	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	1.85E-01	2.85E-01	5.95E-01	9.99E-01	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	1.93E-01	3.68E-01	7.10E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	2.30E-01	4.43E-01	8.01E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	2.71E-01	5.79E-01	8.77E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	3.33E-01	6.64E-01	9.28E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	3.82E-01	7.35E-01	9.56E-01	1.00E+00	1.00E+00	1.00E+00
	0.00E+00	0.00E+00	7.22E-05	6.96E-04	1.07E-03	3.10E-03	3.32E-03	3.33E-03
P(Failure)	0.00E+00	2.89E-06	3.84E-06	4.42E-06	8.34E-06	1.60E-06	1.33E-06	3.33E-07
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	0.00E+00	7.78E-07	1.90E-06	3.13E-06	7.26E-06	2.01E-06	2.07E-06	7.18E-07
	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe	Emb Pipe
	Embankment Piping Probability given a Tide							
	3.61E-10	6.65E-09	1.47E-08	1.82E-08	2.18E-08	3.00E-08	3.35E-08	4.60E-08
P(Failure)	3.15E-09	8.52E-10	1.64E-10	1.00E-10	1.04E-10	1.59E-11	1.59E-11	4.60E-12
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	4.47E-10	2.30E-10	8.12E-11	7.07E-11	9.01E-11	1.99E-11	2.48E-11	9.91E-12

	Tide Interval							
Description	AEP 1 to 10	AEP 10 to 50	AEP 50 to 100	AEP 100 to 200	AEP 200 to 1000	AEP 1000 to 2000	AEP 2000 to 10000	AEP >10000
Tidal Interval	9.00E-01	8.00E-02	1.00E-02	5.00E-03	4.00E-03	5.00E-04	4.00E-04	1.00E-04
	Tidal AEP [years]							
	1	10	50	100	200	1000	2000	10000
	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT	Emb OT
	Embankment Overtopping Probability given a Tide							
	0.00E+00	0.00E+00	0.00E+00	0.00E+00	1.85E-01	6.32E-01	9.10E-01	1.00E+00
P(Failure)	0.00E+00	0.00E+00	0.00E+00	4.64E-04	1.64E-03	3.85E-04	3.82E-04	1.00E-04
PLL	0.142	0.270	0.494	0.707	0.870	1.251	1.559	2.157
Risk	0.00E+00	0.00E+00	0.00E+00	3.28E-04	1.42E-03	4.82E-04	5.95E-04	2.16E-04

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