

**Author Comments (version 2) to review of “Network performance of leaky barriers NHESS 2019-394 Discussion Paper”.** B. Hankin, R. Lamb, I. Hewitt, 14-07-2020.

Dear Editor and Reviewers,

Thank you very much for the review comments on our NHESS discussion paper on a network performance model for leaky barriers. We have provided a response to each reviewer comment in the table below and think the proposed changes and corrections will lead to an improved and clearer manuscript.

This paper came out of an applied mathematics group challenge to understand the performance of a network of nature-based, leaky-barriers and to establish a fast model capable for testing many configurations at larger scales as a proof-of-concept. Our preliminary investigation uses a synthetic case with *extreme* flows, specifically chosen to stress-test the network and produce failures (and cascade failures) of the leaky barriers. These test cases also *incorporate substantial storage* (which may not be apparent from a first look at the equations), made possible using a convenient factor,  $\lambda$ , which may have been overlooked in the review. We are very much in agreement regarding the need for leaky-barrier type solutions to connect to as much storage as possible and the formulation provided with a solution in one-dimensional format permits such testing for a wide range of situations.

We first use the synthetic case to establish that the model can account for open channel flow with variable storage in 2d networks of leaky barriers whilst incorporating under-flow, over-flow, porous through-flow, barrier failure and cascade failure. We demonstrate its flexibility in application to a real network of existing barriers, and consider that the paper and model we have provided on *gitlab* will help others to answer pressing questions on configuring spatial strategies for large networks of leaky barriers.

In addition, in response to editor comments, we have undertaken the following:

- (a) Added a legend to Figures 8 and 10, so it is obvious what the size and colour mean without searching in the text.
- (b) Removed footnotes.
- (c) Corrected numbering of 'Code availability' and 'References'.

**We first make some general comments in response to Reviewer #1 and then tabulate our response to each reviewer point in the table further below**

**Reviewer #1**

We have assessed the comments made by Reviewer #1. We have looked to address the constructive suggestions for improvements in the reviewer’s comments; our proposed responses are given in a table that follows. We also have some general comments in response to this review, where we denote text taken from the reviewer’s commentary in **red**.

The reviewer structured their comments around a cross-check of “**Deliveries and scientific questions as defined by the author Barry Hankin.**”

We believe that in formulating their “check list” the reviewer has gone beyond the actual objectives we stated, as we explain below. Whilst we would concur that the objectives suggested by the reviewer are all of interest, they do not altogether represent our objectives in this paper.

**REVIEWER #1 Check of deliveries:**

- Delivery of network based model. Fulfilled. (215 lines matlab- code; 25 lines calculation, 190 lines I/O and plotting)

Agreed.

- Rapid assessment. Fulfilled.

Agreed.

- Design advice. Not fulfilled. (No quantification, the given design proposal is not derived from the network model)

We did not set out to offer design advice, believing that this would be premature. We are not aware of having set out any specific “given design proposal”. Rather, our hope is that the paper demonstrates a method that could be applied further to inform design advice with respect to the configuration of systems of leaky barriers (as opposed to construction advice about individual assets).

We think the wording of our original text in places does not reflect this position clearly enough and propose to amend it accordingly. The first sentence in the Conclusions sums up our position “We have formulated a network model for a catchment area that **allows for** simple exploration of the effectiveness of different dam placements and designs and is **sufficiently cheap to solve that it may be useful** in analysing risks that require a large ensemble of simulations.” (with emphasis added here to stress that we see this as enabling the application to inform design advice, rather than delivering design advice *per se*).

- Understanding of effective risk reduction strategies. Partially fulfilled. (advices are given, but on a predictable level)

This was not one of our stated objectives as written in general terms by the reviewer. Although it can be inferred that the analysis of NFM systems does, overall, seek to understand effective risk reduction strategies, our objectives stated on page 3 were in fact more specific, namely:

- 1) We want to understand NFM features as systems of assets, and to assess those systems within a risk-based analysis that considers the whole-system performance in terms of risk reduction
- 2) We wish to understand the impact of different spatial configurations of the leaky barriers, taking into consideration three possible performance issues:
  - a. under-utilisation / redundancy
  - b. undesired synchronisation
  - c. structure failures including cascades

### Check of scientific questions:

- Effective dynamic utilization of storage at network dam placements. Partially fulfilled. (There is no criteria given how effectiveness is defined in this context. The effectiveness is not exactly quantified. The effect for the given examples is low. The effectiveness is not linked to the given design flood.)

This is one of stated objectives (2a above). We agree it has been fulfilled in part and have made amendments to clarify how we define effectiveness (please see table and definitions in text based on percentage peak flow reduction).

- Identification of placements that reduce the risk of (cascade) failure. Partially fulfilled. (just qualitative analysis possible, because both dam leakage and fragility are not known.)

This is not one of our stated objectives. Our objective was to understand the impact of different spatial configurations of barrier networks, including the possibility of single and cascade failures (2c, above). This could of course help to identify placements that reduce the risk, as we have discussed; however, we did not set out to identify specific placements (in an optimisation sense) as one of our objectives, only broad strategies.

We disagree with the reviewer's assertion that the analysis is qualitative. The results are very clearly quantified in the graphs plotted in Figures 8 and 10. Dam leakage and fragility are explicitly parameterised (i.e. quantitatively) in our model. The selection of suitable values for the leakage and fragility functions is of course a different question; in the discussion and conclusions we comment on the desirability of calibration in future applications of the model.

- Do small-scale interventions using NFM (Natural Flood Management) combine to create a large scale benefit at large scale? Not fulfilled. (No definition what a large benefit in a large scale is. No model investigation carried out analysing this particular question. The models used are by far too small.)

This was not one of our stated objectives.

It is an important, broader question that we raised for context in the introduction to the paper. We also comment in our discussion and conclusion that our model could be helpful in exploring this question further. But we did not set out to tackle scaling in this paper. Rather, we see the paper as a precursor step to demonstrate a model that is capable of scaling, which could be applied further as we mention in the discussion and conclusions.

- Reliability and performance of NFM-measures under plausible hazards. Not fulfilled. (No plausible hazard given. The test case with  $Q_{max} = ca. 10m^3/s$  is completely synthetic. The case study area is not given, but the small N100 resulting in  $Q_{max} = 0,5m^3/s$  indicate a very small catchment area, which probably is not representative.)

We agree that the plausibility of the test cases require further discussion and clarification. We have given details of proposed amendments in our response to Reviewer #2, who raised some specific questions about this issue. Reviewer #2 identified some potential for misunderstanding about how additional channel/floodplain storage is represented in the equations, which we propose to clarify through amendments (see table).

We cannot agree with the comment that the small catchment area in Penny Gill is “probably not representative”. This is a real system with real NFM features that were placed there to address a real flood risk issue (The village downstream, Flimby, is categorized as a community at risk by the Environment Agency – flooding is caused by backing up upstream of culverts where any reduction in peak flow would be useful). How can that not be “representative” when it is a real case? Of course, it will not be representative of all other NFM systems, and we are not claiming that it is. We hope that other researchers will consider downloading the model code for applications to other systems elsewhere.

- Resilience of a network of NFM-measures. Not fulfilled. (Resilience is neither defined nor addressed in the results.)

Again, network resilience analysis *per se* is not one of stated objectives. Rather, it can be considered as a potential derived outcome of our objectives (1) and (2c). We have illustrated how system resilience can be addressed in the results (Figs 8 and 10 again) where we examine survival of barriers under an ensemble of loading conditions. We agree that resilience was not defined – and have tightened this up – see table below.

Tables of responses to REV 1 and REV 2

ID	REV 1 Comment	Response	Proposed change
	<b>Check of deliveries with issue:</b>		
1	Design advice. Not fulfilled. (No quantification, the given design proposal is not derived from the network model)	We are worried that the first reviewer thinks we were trying to present the method as a "for-real" real risk assessment, rather than as a proof of concept. We were demonstrating the approach as a proof of concept to encourage use of the approach in other configurations and to demonstrate the flexibility to experiment with configurations.	We have amended the abstract (line 24) and discussion in text on page 3 at lines 19-20 to make it clear that we are demonstrating a model that can be used to help inform the robust design of <b>networks</b> of NFM assets (rather than advice about the construction of an individual asset). We have changed the last sentence of the abstract to: <i>"The efficient scheme permits rapid assessment of the whole system performance of dams placed in different locations in real networks, demonstrated in application to a real system of leaky barriers built in Penny Gill, a stream in the West Cumbria region of Britain"</i>
2	- Understanding of effective risk reduction strategies. Partially fulfilled. (advices are given, but on a predictable level)	We made comments we think are supported by the analysis in terms of width of site, slope of site, location of barriers in network.  We have avoided making more generic statements without further research and justification – for instance, the fragility assumption really needs to be based on observed failure rates with estimated storm probability to take the conclusions further. We have recommended this as an important area for further research.	We have definitions of effectiveness in Section 1.0 (final paragraph) and added the following sentence below Figure 14 quantifying the system effectiveness: <i>"Whilst storage may be improved well above that for the real system (355m<sup>3</sup> as opposed to 235 m<sup>3</sup>), the dynamic utilisation of that storage in the network does not result in a better reduction in peak flow (97% as opposed to 96%), this being a key measure to assess the effectiveness of the whole system. This highlights the unpredictability of the network and whole system performance and demonstrates why such a model is important at larger scales."</i>

	<b>Check of scientific questions:</b>		
3	<p>Effective dynamic utilization of storage at network dam placements. Partially fulfilled. (There is no criteria given how effectiveness is defined in this context. The effectiveness is not exactly quantified. The effect for the given examples is low. The effectiveness is not linked to the given design flood.)</p>	<p>The effectiveness of the dynamic utilisation is expressed in terms of the overall reduction in peak flow whilst considering the number of barriers used. We demonstrate this in the second application to Penny Gill.</p> <p>The important effect here is that whilst we may be able to add lots of storage, it does not always fill and release fast enough that it can be used to take water away from the peak of a storm event.</p> <p>Conversely there is evidence of some leaky barriers not filling, which also make them less effective. We believe that the different trade-offs in terms of slope, positioning and storage lead to complexity and it is not always possible to pre-determine optimal arrangements, but it is possible to explore with this rapid network model.</p>	<p>Please see our response to last point and amendment.</p> <p>We have added a further comment in the discussion in the last para of Section 4:</p> <p><i>“In summary, the network analysis has demonstrated how the effectiveness of the system of leaky-barriers quantified overall using the integrating measure of percentage peak flow reduction at the bottom of the network. The approach accounts for additional storage volumes put in place, but also how it is utilised dynamically, something that will also vary across the system depending on the spatial and temporal pattern of runoff inputs, slot dimensions and leakiness.”</i></p>
4	<p>Identification of placements that reduce the risk of (cascade) failure. Partially fulfilled. (just qualitative analysis possible, because both dam leakage and fragility are not known.)</p>	<p>Figures 8-10 and discussion in Section 3.3 provide quantitative analysis of failure risk, and enable a comparison of two idealised network designs. Both leakage and fragility can be parameterised within the model, i.e. treated quantitatively.</p>	<p>We have demonstrated that the network model is capable of quantitative analysis of single and cascade failures graphically using Figures 8 / 10 and through the discussions of percentage change to peak flow.</p>
5	<p>- Do small-scale interventions using NFM (Natural Flood Management)</p>	<p>We demonstrate the model as a proof of concept at the small scale and think this objective is the next natural progression of</p>	<p>We have made the sentence starting on line 22 more emphatic in relation to the need for a network model.</p>

	<p>combine to create a large scale benefit at large scale? Not fulfilled. (No definition what a large benefit in a large scale is. No model investigation carried out analyzing this particular question. The models used are by far too small.)</p>	<p>this work. We also identify on Page 3, line 2 how scaling up is a ‘significant outstanding research question’. We have not set out to tackle this problem here, but we think simplified network model will help with this.</p>	<p>Rather than stating “<i>Probably more importantly...</i>” To:  <i>“However, general advice on design can over-simplify, and the final example has demonstrated that this type of network model can be used effectively to rapidly test different arrangements of dams and to assess which are likely to work best to reduced risk given the unpredictability of the whole system response”.</i></p> <p>Please also see our discussion of scaling and future research that was already in the last paragraph of the paper.</p>
<p>6</p>	<p>- Reliability and performance of NFM-measures under plausible hazards. Not fulfilled. (No plausible hazard given. The test case with <math>Q_{max}</math>= ca. 10m<sup>3</sup>/s is completely synthetic. The case study area is not given, but the small N100 resulting in <math>Q_{max}</math> = 0,5m<sup>3</sup>/s indicate a very small catchment area, which probably is not representative.)</p>	<p>We explain the choice of flows and storage which we think may have been overlooked.</p> <p>The initial analysis was deliberately a synthetic case –as part of proof of concept. Please see this as a test of a method that uses dimensionless equations and can be scaled to other flows.</p> <p>The synthetic flows selected were deliberately extreme but were selected (combined with additional storage) to stress-test the barriers for failure. see answer to Rev 2).</p> <p>The flows for Penny Gill are realistic and due to backing up at infrastructure, flooding does occur.</p>	<p>We have included more context on Page 2, line 10 about the case study, Penny Gill. The text now reads:</p> <p><i>Penny Gill micro-catchment, drains to the small community at risk of Flimby on the west coast of Cumbria and is designated at risk because of the interaction of the stream with infrastructure downstream of the test-site. In this case the capability to attenuate the peak flows for this small sub-catchment (&lt;0.5km<sup>2</sup>) is important to avoid backing up and flooding from culverts.</i></p>
<p>7</p>	<p>- Resilience of a network of NFM-measures. Not fulfilled. (Resilience is neither defined nor addressed in the results.)</p>	<p>We demonstrate resilience in a number of ways. With regards to the synthetic case it can be interpreted as the survival of the leaky barriers in different configurations and the capacity of the system to deliver a reduction in peak flow within an ensemble of potential forcing</p>	<p>We reinforce the notion of ‘system resilience’ by discussing this at Page 12, line 15 and at the bottom of original page 13 in terms of whether the system can still deliver a reduction in flood peak downstream even if elements within it are allowed to fail. This has now been discussed more fully in relation to Figure 8/10, which</p>

		events and failure scenarios. We have use graphical representation of resilience in the form of Figure 8.	we have chosen to use in place of a single number.
8	- Definitions on effectiveness, resilience and “large benefit at large scales”	Ok –	We have ensured these terms are defined (for resilience see last answer) such that they can be interpreted using Figs 8 and 10. See page 2, line 2 and last paragraph of Section 1.0 where we have specified the measures of percentage peak flow reduction and frequency of barrier failure. See paragraph under figure 14.
9	- Hydrological aspects (f.i. definition of design hydrograph, impact of convective or orographic rainfall events on efficiency)	ok	See definition of ‘design event’ in first paragraph of section 4.0 and definitions of effectiveness discussed above.
10	- Ecological aspects	We didn’t set out to cover this but we could add to discussion – the design advice could include fish passage	We have referenced the potential issue of blocking fish passage if the under-flows are too narrow (this was prepared in a previous draft but omitted to save space). Text added P18: <i>“These different designs also need to consider the trade-offs in barrier design (slot height, leakiness) when considering ecological impacts such as fish passage, for example a narrow slot might improve flood attenuation but make passage more difficult”</i>
11	- Arguments for the development of the network model (there are well-proven 1d-tools as MIKE11 or HECRAS)	The packages mentioned do not allow for rapid assessment of collapse and cascade collapse of structures with leakiness factors in arbitrary network configurations.  It would be a lot of work to manually implement the automated solver used here that <i>continues</i> after barrier failure and multiple failures.	We have clarified why there are advantages to developing a new model and the ability of the solver to continue following failure or cascade failure by adding this sentence on Page 8, line 38: <i>“Whilst there are a number of hydraulic modelling packages solving similar equations with a diverse range of hydraulic units, these do not permit rapid assessment of collapse and cascade collapse of barriers having leakiness factors and a channel</i>



			<i>storage multiplier making it easier to test arbitrary networks of configurations.”</i>
12	- Validation of proposed network model for normal flow conditions and dam-break	The case studies are a form of sensitivity analysis – but we agree validation in future useful – it would be good to make comparisons where upstream and downstream measurements are available	We clarify on Page 14, line 9: <i>“We consider the application of the model in a ‘sensitivity to change investigation’ to a site on Penny Gill, West Cumbria (figure 1).”</i>

**Response to REV 2 overleaf**

**REVIEWER #2**

ID	REV #2 Comment	Response	Proposed change
1	I really enjoyed this paper. It was novel, innovative and thought provoking. I do have a number of fundamental detailed points that I will outline below.	Thank you	
2	Overall I am concerned that the RAF/Flood features being shown are not really good features to build or simulate. However, the theoretical analysis suggested that small networks of barriers can have a huge impact on flood reduction.	That is for the hypothetical case investigated in the workshop. We then apply to a more real-world situation that was already constructed.	See responses below with regards to an error in figure 5, but especially in relation to the $\lambda$ factor which permits investigation of more storage including floodplain storage.
3	This needs a number of clarification points. Firstly the feature shown are small and create small volumes of temporary storage. The features are trapped within the channel and hence seem to have little capacity to store flow. Hence the conclusion must be that it is the roughness that is slowing the flow???? This type of feature works better when the water is forced onto a floodplain and into extra storage areas. So I would be	<p>Please see further responses on the synthetic case, where the <math>\lambda</math> factor accounts for floodplain storage, and for Penny Gill where the barriers are robust and quite tall.</p> <p>It is the temporary (in-channel) storage behind the relatively large barriers which is attenuating the peaks, combined with the roughness.</p> <p>This system has been built and is not at the design stage.</p>	<p>We have highlighted the nature of the theoretical channel, with additional storage represented using the <math>\lambda</math> factor, and for the real channel at Penny Gill we have added more context under figure 11:</p> <p><i>Penny Gill stream is incised and there is little possibility of greater connection with the floodplain, so hence the barriers are relatively tall and rely on extended in-channel storage, except for the relatively wide segment in figure 11 which reflects an area of channel well</i></p>

	suggesting that RAF design is key, i.e. using the barriers in combination with other storage and velocity reducing zones, e.g. shallow flow across floodplains.		<i>connected with a relatively wide depression.</i>
4	A criticism of the Metcalf work is that the features did not allow flow onto to the farmland next to the channels.	This is not a comment on our paper	
5	However, let's go with the network of within channel features and discuss that.	ok	
6	One more 'picky' thing the author uses the term 'we' a lot. In the past this would not be allowed but I know we live in enlightened times and we can now use 'we'. I do think we have an overzealous use of the term 'we', especially when occurs 4 or 5 times in paragraph. 'We have formulated a network method...' could be 'A network method has been formulated...'	Ok	The editor has stated he does not have a language preference in this respect.
7	I think earlier work on network models could be included and referenced. E.g Nicholson et al., 2019, Quinn et al., 2015 which was included in the WWNP report. I think Bhoko et al.,	Ok. We have found it hard to find research on the performance of the type of leaky barriers under investigation (i.e. solid hose-jump type barriers).	The Nicholson et al paper is already cited on page 8 line 16 of original submission and is relevant. Quinn reference added in same place based on the WWNP citation (2013)

	<p>219/20? Could be added.</p> <p>The work of Nicholson included observations and this study has used a theoretical approach and this may need to be highlighted.</p>		
8	<p>Bhoko comments on beavers and the impact on flood flow. It is also important to stress that beaver dams are full of water all the time. So that is why they are useless to flooding!</p>	<p>Ok.</p>	<p>This has not been included since the main point we wished to make here in relation to beaver dams is to learn about performance failure; in this case we assume that leaky-barriers for NFM are designed not to be full of water before a flood (ie they are leaky with additional storage that is used at high flows)</p>
9	<p>I think your model is not simulating a typical leaky dam, even though figure 1 is a little leaky. Most leaky dams are usually large woody debris that are very porous/leaky. You are thus assuming that all leaky dams operate as a sluice gate. This may not be a good assumption. But, let's go with the impact of a network of sluice gates on flood flow.</p>	<p>We chose this type of barrier and have seen quite a few in operation (as in the case-study) where the barriers are solid and there is an underflow.</p> <p>Our model does also include a porosity term. Whilst we set porosity to zero for the cases investigated in the paper, in general this can be changed using the factor <math>k</math> in eq 7. For Penny Gill the leakage is in practice very small compared to the underflow.</p>	<p>Note on Page 18, line 18 (last sentence) we already do emphasise the diversity in leaky barriers:</p> <p><i>"..compounding factor is that there are many forms of leaky barrier or large woody debris dams (Addy et al., 2019) including placing large woody debris in channel, to the horse-jump type barriers in use in Penny Gill, combined with engineered log jams which will also reduce passage of debris should a structure fail."</i></p> <p>In this location we now also note that the leakiness factor in the equations, in addition to the slot dimensions can be experimented with to represent a broader range of features.</p>
10	<p>Fig 2 is crucial as is section 2.2. What is the size of the</p>	<p>The synthetic case was developed based on approximate reference scales (the equations are non-</p>	<p>There was a mistake in the left hand size of Figure 5 that has now been amended and</p>

	<p>catchment? Add a scale to fig 2. I know you want it to be a dimensionless model, but surely a catchments size would help. You need to state the density of barriers. I deduced later that the model had 100m lengths between nodes. Later you show the model resulting as being based on Fig 2b, so this means the length of channel is 500m, This means the catchment is about 1km<sup>2</sup>? SO, how do you get a flow of 15 m<sup>3</sup>/s (fig 7). This would be a flow rate for a 10-20km<sup>2</sup> catchment. That catchment area would require 100 RAFs. Can the authors sort this out or address my misunderstanding.</p>	<p>dimensionalised). We wanted to stress-test the system using extreme flows that would trigger failures with the assumed fragility function, so that we could demonstrate an approach that can test system resilience.</p> <p>Please see our response to the next point which should address your concerns here; we think there is a potential for misunderstanding that we can avoid with a minor revision.</p>	<p>reported with track changes – the peak flow was 10m<sup>3</sup>/s not 15m<sup>3</sup>/s that was from an earlier numerical experiment.</p> <p>Above equation (15) we have added the text:  <i>Here an extreme flow of 10 m<sup>3</sup>/s is used to fully stress-test the system, also permitting flows and depths with magnitudes capable of failing leaky-barriers.</i></p> <p>We have changed figure 2 caption to add the text:  <i>'The lengths of the network edges can be varied depending on the scale of interest; in the examples shown in this paper they are typically on the order of 100m long.'</i></p> <p>This should be considered in combination with the fact a large <math>\lambda</math> factor has been used so there is adequate storage capable of providing attenuation.</p>
11	<p>The model result then suggest a huge reduction in flow caused by 5 barriers?? 15m<sup>3</sup>/s down to 6m<sup>3</sup>/s. But the storage behind these barriers must be tiny. Surely they would be overwhelmed by such high flows. SO I may have missed something. SO what is causing the reduction? Is it roughness? Could the authors show some</p>	<p>There was a mistake in the legend for figure 5 – see above.</p> <p>There are also two factors at work here – 1) we wanted to use extreme flows to test performance, as noted above, and 2) we introduced a channel storage factor, <math>\lambda</math>, which can represent extra channel or floodplain storage per segment, which as you correctly point out is essential for gaining significant attenuation when using leaky barriers.</p> <p>The parameter <math>\lambda</math> is introduced in equation (14), which increases the lateral storage or, in effect, represents</p>	<p>Figure 5 has been corrected.</p> <p>Please see new edit below regarding the use of a large <math>\lambda</math> factor, which is a convenient way of representing extra channel or floodplain storage. This is highlighted under equation 14 in relation to this discussion:</p> <p><i>“where <math>\lambda \geq 1</math> is this enhancement factor that accounts for a larger volume being stored behind the dam. In practice, this would have to be estimated for each dam</i></p>

	<p>sensitivity analysis of Mannings 'n' on the model output?</p>	<p>a well-connected floodplain for values &gt; 1.</p> <p>This enabled us to represent significant storage by setting the value of <math>\lambda</math> to 50 (see Figure 5) – which assumes <b>a lot</b> of extra storage (conceptually a wide floodplain) – and hence perhaps why you were uncomfortable with the magnitude of the reduction.</p> <p>Thus, in these equations we are able to represent more than simply the wedge-storage in the channel behind a barrier. This means we can represent the type of extended floodplain storage that the reviewer has mentioned in previous comment</p>	<p><i>location, but provides a useful mechanism for exploring different NFM designs, for instance it can be used to conceptualise reaches where leaky barriers are being used to enhance floodplain reconnection, thus accessing additional storage with greater potential for peak flow attenuation."</i></p> <p>When we use the real case of Penny Gill then the <math>\lambda</math> factor is 1, which ties in with peak flows and storage volumes that are realistic for this system.</p>
12	<p>I like the analysis introducing branches, but again I do worry about scale and flows. The point being made about branches have less danger is good, as this reflects that a scale appropriate positioning of RAFs is needed, as they work better and have less problems in smaller channels.</p>	<p>We have addressed this in our response to the previous comment.</p> <p>The equations are non-dimensional so different assumptions can be made about the appropriate scale, and we have provided the model.</p>	<p>See discussion in last paragraph of paper about scaling up.</p>
13	<p>Overall I was less impressed with the analyses and cascade failures. I am not addressing the analysis which is good, but a NFM team would either design in the failure so that lower dams trap debris or if there is a threat than a debris traps would be built.</p>	<p>In terms of design taking in these considerations there is no current definitive guidance in the UK, (although CIRIA are in the process) so 'NFM teams' do need some guidance, and internationally, The WWF "Green Atlas" (<a href="https://www.worldwildlife.org/publications/natural-and-nature-based-flood-management-a-green-guide">https://www.worldwildlife.org/publications/natural-and-nature-based-flood-management-a-green-guide</a>) provides case studies to advise on 'good practice'</p>	<p>We do already state that the Penny Gill implementation has mitigated risk of failure this to some extent through the use of logs within the channel that would trap debris (see penultimate sentence of section 4).</p>

14	<p>SO, now we look at Penny Gill, the flow rate is 0.4m<sup>3</sup>/s. This analysis section is interesting. However, it reinforces my point about the flow rates used above. You really have a chalk and cheese comparison. The features in Penny Gill should have been better designed, i.e. to use zones with active floodplains (if appropriate) or maybe the wood should have been distributed over larger areas to create more roughness, thus could be more tree planting in the channel, e.g. willow.</p>	<p>We were using Penny Gill to illustrate a simple application – not purpose of paper to debate whether it was the most appropriate choice of NFM system design. It has already been built and we were studying it.</p> <p>Agree the scales are different – we were partially addressing the problem that you had with the synthetic workshop example, and wanted to apply the proof of concept derived in a workshop to a ‘real world case’</p> <p>The incision in the Gill makes what you state difficult – there is one area where the floodplain is within reach of realistic flood flows and that was the one wide area identified in Figure 11, 325m.</p> <p>The West Cumbria Rivers Trust have sought as much wedge-storage as possible in the gill, but we do advise like you to reconnect floodplain where possible. However as stated the channel is highly incised.</p>	<p>See responses above (point 6 for first reviewer and point 3 for the second reviewer).</p> <p>It is useful to show the flexibility across different situations.</p>
15	<p>I hope the authors do not think I am being over critical. I really did enjoy the paper. I may not fully understand the model, the scale, the flows, the volumes and the RAF density assumptions. If you sort this out then this paper can be published. If it is a simple clarification of the model and the assumptions then this would be just a minor edit. If there is a fundamental issue</p>	<p>No - these are good points, thank you, and help improve the paper.</p> <p>In the synthetic example from the workshop, we chose to scale the non-dimensional equations with values that yielded interesting cases – such as failure and attenuation.</p> <p>We could have chosen small values of <math>\lambda</math> to reflect many installations of leaky barriers we have seen, but there would have been little response in the hydrograph and this would reflect your comments on the need to reconnect floodplain storage.</p>	<p>Hopefully the various clarifications above, corrections to some of the legends and our introductory remarks address these concerns.</p>

	with the flow and the impact then major corrections are needed.		
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# A risk-based, network analysis of distributed in-stream leaky barriers for flood risk management

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**Abstract.** We develop a network-based model of a catchment basin that incorporates the possibility of small-scale, in-channel, leaky barriers as flood attenuation features, on each of the edges of the network. The model can be used to understand effective risk reduction strategies considering the whole-system performance; here we focus on identifying network dam placements promoting effective dynamic utilisation of storage, and placements that also reduce risk of breach or cascade failure of dams during high flows. We first demonstrate the model using idealised networks and explore risk of cascade failure using probabilistic barrier-fragility assumptions. The investigation highlights the need for robust design of nature-based measures, to avoid inadvertent exposure of communities to a flood risk, and we conclude that the principle of building the leaky-barriers on the upstream tributaries is generally less risky than building on the main trunk, although this may depend on the network structure specific to the catchment under study. The efficient scheme permits rapid assessment of ~~performance-of-the whole system performance of~~ dams placed in different locations in real networks, demonstrated in application to a real system of leaky barriers built in Penny Gill, a stream in the West Cumbria region of Britain ~~and which leads to further design advice.~~

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## 1 Introduction

The concept of “green infrastructure” is embedded within environmental policy in Europe (European Commission, 2007, 2013a,b, EEA, 2015) and the UK (Defra, 2019) as a strategic approach involving the design and management of networks of natural and semi-natural environmental features to deliver a wide range of ecosystem services. Echoing this approach, projects around the world have been blending natural and engineering approaches to deliver multiple social and environmental benefits (WWF, 2016, Bridges et al., 2018). In Flood and Coastal Risk Management (FCRM) there has been a growing interest in so-called ‘nature-based’ measures, including small-scale, distributed storage features, tree planting and soil structure improvement to prevent fast overland flow. These measures have collectively become known as Natural Flood Management (NFM) in the UK (see Dadson et al., 2017 and Lane, 2017), or Working With Natural Processes (WWNP) after the Pitt Review of the UK 2007 summer floods (Pitt, 2008), a term adopted in the recent UK Evidence Directory (Burgess-Gamble et al., 2018). Internationally they have also been termed “nature-based approaches” or “engineering with nature” (Bridges et al., 2018).

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One such nature-based measure is to encourage in-channel flood attenuation (e.g. see Metcalfe et al., 2017), using small dams or barriers, usually made from wood (Figure 1). These barriers, which are often deliberately built to be permeable (and sometimes called “leaky barriers”), allow low flows to pass under or through, but hold back high flows, providing temporary water storage analogous to beaver dams. It is hoped that a large collection of such features deployed in a catchment may hold

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back enough flood water (in-channel or on the floodplain) to mitigate flood risk downstream (Figure 2(a)). In the UK, use of leaky barriers has been incentivised under the current environmental stewardship grants across England and Wales [UK Government, 2017]<sup>1</sup>. However, whilst the effectiveness of systems of runoff attenuation features and leaky barriers in terms of peak flow reduction has been investigated recently (e.g. Metcalfe et al., 2017 and Addy and Wilkinson, 2019), these studies do not consider performance failure, and there remains much trial-and-error installation of different designs which could be improved upon for more efficient risk-reduction strategies at the large scale.

There have been many attempts at representing the effects of leaky-barriers on flow, with methods ranging from increasing roughness in 1d models to full 3d representation, but relatively few have been able to test the accuracy of the physical representation (see Addy and Wilkinson, 2019). The NERC project, Q-NFM (Lancaster Environment Centre, 2017)<sup>2</sup> has developed a set of small, accurately monitored 'micro-catchments' in Cumbria to attempt to quantify the effect of different nature-based interventions. ~~One of the micro-catchments is~~ The Penny Gill ~~micro-catchment, drains~~ drains to a small community at risk of Flimby on the west coast of Cumbria and is designated at risk because of the interaction of the stream with infrastructure downstream of the test-site. ~~In this case the capability to attenuate the peak flows for this small sub-catchment (<0.5km<sup>2</sup>) is important to avoid backing up and flooding from culverts.~~ This stream has had ten robustly constructed leaky barriers (Figure 1 shows two examples from Penny Gill) that are in sequence on the main stem of the stream. The larger, lower eight of these structures were surveyed and have been modelled to try and understand the attenuation and storage during times of flooding, and the system of leaky barriers is used as real-life network to see if the network model presented here can help with design-guidance and effective deployment strategies.

<sup>1</sup><https://www.gov.uk/countryside-stewardship-grants/rp33-large-leaky-woody-dams>

<sup>2</sup><https://www.lancaster.ac.uk/lec/sites/qnfm/>



**Figure 1: Leaky barriers in sequence in Penny Gill, Cumbria (B. Hankin)**

Significant research questions remain about whether “many small interventions (each creating local benefits) [will] combine to create large benefits at large scale” (Dadson et al., 2017) and whether “the lack of demonstrable effect at large scale is because noticeable flood mitigation could not be achieved in a large catchment, or because a sufficiently large-scale set of interventions have not yet been implemented”. Meanwhile, in the UK at least, the government’s approach is to see working with natural processes as complementary to conventional, engineered flood risk management measures. In England, this is reflected in the latest long-term investment planning scenarios published by the Environment Agency (2019), whilst noting uncertainty about the effectiveness of NFM to manage large floods and large catchments.

If this complementarity is to be realised in practice, we believe there is a pressing need to integrate NFM more tightly within the cost, benefit and risk assessment frameworks that apply to “conventional” flood management. This means that we want to understand NFM features as systems of assets, and to assess those systems within a risk-based analysis that considers the whole-system performance in terms of risk reduction. A risk-based analysis of NFM asset systems should take account of both the reliability of the assets and their performance as a whole system under different plausible hazard or loading scenarios. One vital lesson from conventional flood management is that even when flood mitigation measures are in place, the residual risk cannot be ignored.

Some initial work to test the effectiveness of catchment-wide NFM under a range of spatially distributed extreme rainfalls has been reported by Hankin et al. [2017a], but without consideration of the reliability of the underlying NFM assets. Here,

we focus instead on the resilience of a network of NFM features as an asset system. To do this, we develop a simple network-based model of a river catchment that incorporates the possibility of leaky barriers being installed on each edge of the network, similar to the approach taken by Metcalfe et al. [2017]. We wish to understand the impact of different spatial configurations of the leaky barriers, taking into consideration three possible performance issues. These are:

- 1) under-utilisation of dynamic storage (see Metcalfe et al., 2018), i.e. redundancy in the network of leaky barriers that could be regarded as an inefficient use of resources;
- 2) undesired synchronisation of flood peaks (see Pattison et al., 2014), where measures intended to slow the flow could result in flood peaks being increased under some scenarios;
- 3) structural failure and cascade failure of barriers.

In Section 2 we develop a mathematical drainage network model and show how leaky barriers can be incorporated in a form that is simple enough to enable solution of the resulting system of equations, but sufficiently realistic to describe key hydraulic modes of behaviour. We then apply the equations in Section 3 to study the performance of idealised one- and two-dimensional stream networks subjected to single-peaked and multi-peaked flood events, including the potential for failure of individual or multiple assets ([quantified in terms of the frequency of barrier failure and percentage change to peak flow](#)).

Multi-peaked flood events are a more effective test to the resilience of the system aimed at providing dynamic storage, that can be re-used on consecutive events, and it is this kind of event that often resulted in more severe impacts. We discuss the findings in terms of the risk reduction ([quantified as percentage peak flow reduction](#)) and the residual risk achieved by the systems of NFM features under different configurations, and how the idealised cases may help inform analysis of real NFM systems. In Section 4, the model is applied to the real system of leaky barriers in Penny Gill, West Cumbria and conclusions are drawn about more effective designs and placement.

### 1.1 Performance of existing nature-based dams

There are a number of studies documenting the benefits of beaver dams in terms of habitat improvement, peak flow attenuation and water quality improvements (Puttock et al, 2017, 2018), so it is natural to try and emulate these types of benefits artificially. However, we should also study what happens in nature when things go wrong. Structural failure of natural beaver dams has been reported as occurring frequently by Butler and Malanson [2005], citing numerous cases of dam failure, that resulted in outburst floods. These floods have reportedly been “*responsible for 13 deaths and numerous injuries, including significant impacts on railway lines*”. Engineered NFM measures are likely to be more robust than beaver dams (contingent on maintenance in the longer term), but the relative risks of different configurations, positioning in relation to geometry, slope and proximity to each other, and build-design need a mechanism for appraisal. The intention is to help design safer and lower risk configurations of NFM, which is seen as a potentially low-cost complement to conventional flood risk management strategies.

Failure of beaver dam structures in the US has been reasonably well documented (Hillman, 1998; Butler and Malanson, 2005), and there have been two records (Tom Nisbet, *Pers. Comm.*) of leaky barrier performance failure at Pickering, UK to the authors’ knowledge, after two large flood events. The first flood event in November 2012 resulted in the washout of one of the larger dams on the main Pickering Beck and a shift to the edge/bank of a second dam below this. These features were relatively tall structures, and located within a straightened section of channel alongside a railway line, with limited floodplain storage. The logs from the failed dam were caught within the downstream reach between that and a third dam downstream. The failed and shifted dam plus one other were found to be deflecting flows into the river bank, causing some local scouring, placing a local railway at risk of undercutting so they were removed (2014) and replaced with five new dams on a better reach downstream. The second failure event occurred during the UK Boxing Day floods, 2015, where a total of 11 dams were damaged, all involving a shift/deflection in the dam by edge scour or loss/breakage of top logs, rather than a complete washout. These losses all involved the original, more natural design of cross logs used to construct dams in

2010/11, with no wiring used to secure logs in place. All of these have since been replaced using the now favoured semi-engineered design of horizontal stacked logs secured by wiring (a design also used in Penny Gill – see Figure 1). Additionally, Addy and Wilkinson [2016] report on complete failure of one structure during a 10% Annual Exceedence Probability (AEP) event for ‘engineered log jams’ that are albeit designed to trap sediment.

5 Siting, construction and improvements in engineering design are therefore important, and recent research (Dixon and Sear, 2014) shows logs 2.5 times channel width provide ‘near functional immobility’ - unlikely to be transported in an extreme event. ~~It is~~Such design-construction ‘rules of thumb’ can be very useful, but cannot always describe the complexity of the whole system response, which can be very place-specific, driving the need for a network model that can be rapidly set up to test different situations of the kind described here.

10 In this paper we explore network issues impacting the three performance issues categorised above, particularly with respect to spatial configuration of leaky barriers in a network that have a probability of failure defined by a fragility curve, an approach commonly used in the systems approach to quantifying flood risk [Hall et al., 2004]. The probability of failure is very difficult to define for the range of constructions that are being implemented – and how this varies with age, decay and sedimentation is not known. So here we attempt to understand what aspects of geometry, slope, proximity are the best trade-  
15 offs for a given, reasonable assumption about fragility. We later translate this back the real world example on Penny Gill, Cumbria, and the implications for spacing and siting.

## 2.0 Method

### 2.1 Introduction

We begin by setting up a network model for an arbitrary stream network, breaking the stream up into segments that may  
20 each potentially contain leaky barrier designed to attenuate high flows (often referred to generically as runoff attenuation features). Our aim here is to set out a mathematical formulation for the network of features that will enable us to describe and experiment numerically with different configurations of NFM features within a probabilistic analysis. The model is based on a consideration of essential hydraulic principles, with enough simplification to enable solutions to be obtained quickly for idealised cases. Rules for the storage and discharge (flux) in each segment are prescribed based on the slope,  
25 stream cross-section and roughness. Modifications of these rules to account for the effect of a leaky dam are developed. The model amounts to a series of coupled ordinary differential equations (ODEs) that are solved numerically given prescribed runoff inflow. We then explore solutions for some simple networks forced by idealised flood hydrographs, focussing on the response of the discharge at the downstream end of the network. We then examine the response to failure of the dams including cascade failure.

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### 2.2 Network set-up

We construct a network model in which segments of a channel (‘reaches’) are described in a lumped fashion (Figure 2). The primary variables are the average cross-sectional area  $A_i$  and discharge  $Q_i$ , which flows into the next channel segment downstream. The channel segments correspond to nodes of a graph, and the edges that transfer water downstream can be  
35 thought of as potential dams (i.e. the positions at which dams might be added). The connections between the channel segments are described using an adjacency matrix ( $a_{ij}$ ). The  $i^{\text{th}}$  row of this matrix is all zeros except for in the  $j^{\text{th}}$  column, where  $j$  indexes the node immediately downstream of the  $i^{\text{th}}$  node. Idealised network structures, with uniform widths and slopes are used, although positions and connections between the channel segments, as well as their lengths, and slopes, might be determined from studying a real drainage network, for example based on a two-dimensional digital elevation model  
40 (DEM) such as that used by Metcalfe et al. [2017], or the Penny Gill example discussed further below.

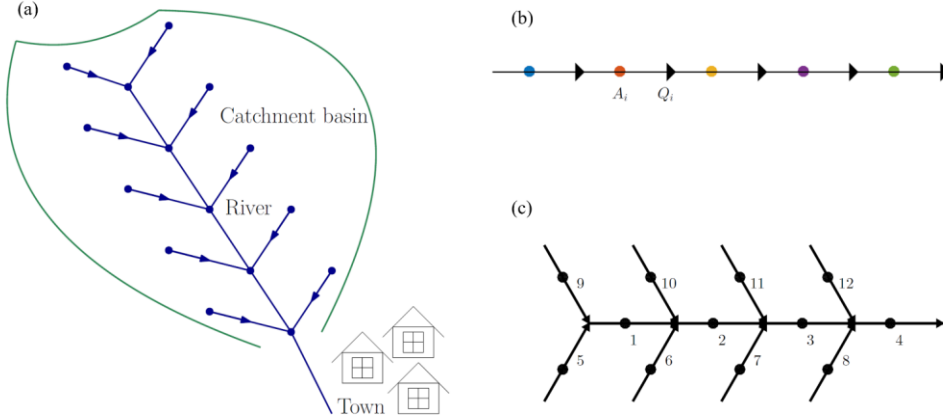


Figure 2: Schematic (a) and the 1d network (b) and 2d network (c) leaky barrier configurations investigated. Triangles indicate the location of dams which control the discharge,  $Q_i$  associated with the upstream reach where the average cross section is  $A_i$ . The lengths of the network edges can be varied depending on the scale of interest; in the examples shown in this paper they are typically on the order of 100m long.

Taking  $l_i$  to represent the length of the channel segments, volume conservation requires:

$$l_i \frac{dA_i}{dt} = \sum_{j=1}^N a_{ji} Q_j - Q_i + q_i \quad (1)$$

for each node ( $i = 1 \dots N$ ). The sum represents the fluxes from the immediately upstream nodes and  $q_i$  represents the inflow to each segment from rain/runoff from the surrounding land. It may be more convenient to think of (1) in terms of the water volumes  $V_i = A_i l_i$  stored in each channel segment. We assume that the lateral inflows,  $q_i(t)$ , are prescribed, although in a more complete treatment they might be taken from a two-dimensional model (using the shallow water equations for example), or they might be derived from rainfall data using a filter to represent the time-delay due to subsurface and/or overland flow.

Given the known slope of each channel segment  $S_i$  (which may be related to the bed angle  $\theta$  by  $S_i = \tan(\theta)$ ), we could relate the discharge and cross-sectional area. However, it turns out to be more convenient to express the discharge in terms of the water depth  $h_i$  behind the potential dam in each reach. In the case that there is no dam, or when the depth is below the bottom of the dam, this is simply the average water depth and we can relate this to the cross-sectional area and flow.

The relationship depends on the assumed shape of the channel, and on a parameterisation of turbulent flow. If we assume for simplicity that the channel has a rectangular cross-section with fixed width  $w_i$ , and use Manning's law, we have

$$A_i = w_i h_i, \quad Q_i = \frac{w_i^{5/3} h_i^{5/3} S_i^{1/2}}{(w_i + 2h_i)^{2/3} n} \quad (2)$$

where  $n$  is the Manning roughness coefficient and  $S_i$  is the slope. Since we can then relate  $Q_i$  directly to  $A_i$  (by eliminating  $h_i$ ), we can interpret (1) as a set of coupled ordinary differential equations for the  $A_i$ , forced by the inputs  $q_i$ . These can be solved numerically using a variety of methods. More generally, when we include dams, we write:

$$A_i = \tilde{A}(h_i; \cdot), \quad Q_i = \tilde{Q}(h_i; \cdot) \quad (3)$$

where  $\tilde{A}(h_i; \cdot)$  and  $\tilde{Q}(h_i; \cdot)$  are known functions, and the extra parameters ( $\cdot$ ) will describe the dam as well as the cross-section and slope (see below). We also define  $\tilde{h}(A; \cdot)$  to be the inverse of  $\tilde{A}$  (which is a monotonically increasing function of  $h$  and therefore has a well-defined inverse). Thus, we will still have a direct relationship between  $Q_i$  and  $A_i$ .



We take  $h$  to represent the height of the water behind the dam. The dam has its bottom at height  $b$  above the stream bed, and its top at height  $H$ . The description of the flow past the dam can then be divided into three modes represented in figure 3,  $h < b$  (corresponding to the water level being below the bottom and the dam doing nothing),  $b \leq h < H$  (when the dam is operating normally),  $h \geq H$  (when the dam is over-spilling).

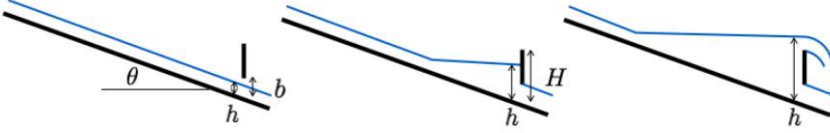


Figure 3: Flow modes for a leaky barrier.

For the first mode we use the same Manning relationship as given above to relate  $Q$  to  $h$ . For the second two modes we adopt relationships from hydraulic theory for the flow beneath sluice gates and over weirs (e.g. Munson et al., 2013). When the water depth is part of the way up the face of the dam the flow underneath is given by Bernoulli's equation, to be:

$$wbh \sqrt{\frac{2g}{b+h}} \quad (4)$$

An empirical correction factor to account for losses is often included in this formula, but we neglect it for simplicity. The flow through the (leaky) dam is assumed to similarly vary with the water depth (due to the hydrostatic pressure), and we write this as:

$$kw(h-b)\sqrt{2gh} \quad (5)$$

where  $k$  should be interpreted as the dam permeability. When the water depth is above the level of the dam, the over-flow is described as for flow over a weir, giving

$$\frac{2\sqrt{2g}}{3}w(h-H)^{3/2} \quad (6)$$

The leaky flow through the dam is then

$$kw(H-b)\sqrt{2gh} \quad (7)$$

In summary, therefore,

$$\bar{Q}(h; w, l, b, H, k, S) = \begin{cases} \frac{w^{5/3}h^{5/3}S^{1/2}}{(w+2h)^{2/3n}} & 0 \leq h < b \\ w\sqrt{2g} \left[ \frac{bh}{(h+b)^{1/2}} + k(h-b)h^{1/2} \right] & b \leq h < H \\ w\sqrt{2g} \left[ \frac{bh}{(h+b)^{1/2}} + k(H-b)h^{1/2} + \frac{2}{3}(h-H)^{3/2} \right] & H \leq h \end{cases} \quad (8)$$

When there is a dam,  $A_1$  no longer represents the uniform cross-section of the stream, but rather its average over the length. It is most straightforward to calculate the volume  $Al$  in terms of the water depth  $h$  for each of the three cases mentioned above. This again depends upon the precise geometry; for the rectangular channel we have

$$\bar{A}(h; w, l, b, S) = \begin{cases} wh & 0 \leq h < b \\ wb + \frac{w(h-b)^2}{2Sl} & b \leq h \end{cases} \quad (9)$$

The terms in the second expression here represent the volume of water in the stream up to the depth of the bottom of the dam, plus the volume of water stored in the triangular wedge that forms behind the dam. These relationships are shown in figure 4.

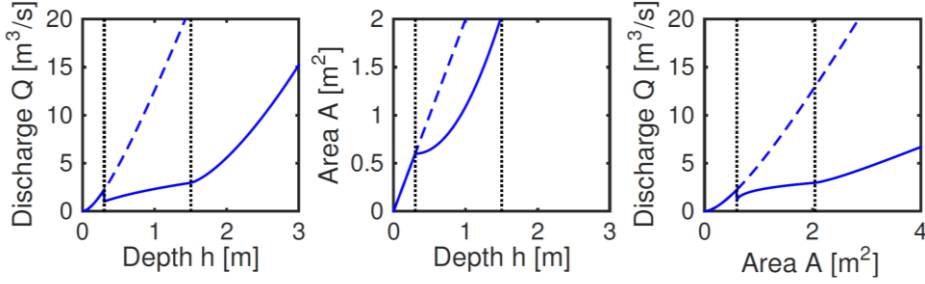


Figure 4: Examples of the relationships between discharge  $Q$ , cross-sectional area,  $A$ , and water depth  $h$ , for rectangular channel of width  $w=2\text{m}$ , slope  $S=0.01$ , reach length  $l=100\text{m}$ , and Mannings  $n$  of  $0.01$ . Dashed lines show the case of no dam. Solid lines show cases of  $b=0.3\text{m}$ ,  $H=1.5\text{m}$ , shown by the vertical dotted lines.

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### 2.3 Non-dimensional model

We expect that the flow through the dam will be small compared to that under and over it (the fact that it is allowed to be leaky makes it easier to construct, but the leakiness between logs is not fundamental to its operation in that there is leaking from underneath the barriers). Thus, for the results presented here, we assume this can be ignored and set the dam permeability coefficient  $k$  to zero for simplicity.

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We choose scales, denoted by square brackets, such that

$$[Q] = \frac{[w][h]^{5/3}[S]^{1/2}}{n}, \quad [A] = [w][h], \quad [t] = \frac{[l][w][h]}{[Q]} \quad (10)$$

For given typical values of  $[Q]$ ,  $[w]$ ,  $[S]$ , and  $[l]$ , these determine the scales  $[h]$ ,  $[A]$ , and  $[t]$ . Using typical values for small headwater drainage channels in UK or other humid temperate environments,  $[w] = 2\text{m}$ ,  $[Q] = 1\text{m}^3\text{s}^{-1}$ ,  $[S] = 0.01$ , and  $[n] = 0.01\text{s}^{-1}\text{m}^{-1/3}$ ,  $[l] = 100\text{m}$ , we find  $[h] = 0.17\text{m}$ ,  $[A] = 0.33\text{m}^2$ ,  $[t] = 33\text{m}^2$ .

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In non-dimensional form, and assuming negligible dam permeability, these are:

$$\bar{Q}(h; w, l, b, H, k, S) = \begin{cases} \frac{w^{5/3}h^{5/3}S^{1/2}}{(w+2\alpha h)^{2/3}} & 0 \leq h < b \\ w\gamma \left[ \frac{bh}{(h+b)^{1/2}} \right] & b \leq h < H \\ w\gamma \left[ \frac{bh}{(h+b)^{1/2}} + \frac{2}{3}(h-H)^{3/2} \right] & H \leq h \end{cases} \quad (11)$$

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$$\bar{A}(h; w, l, b, S) = \begin{cases} wh & 0 \leq h < b \\ wb + \beta \frac{w(h-b)^2}{2Sl} & b \leq h \end{cases} \quad (12)$$

where the dimensionless parameters are

$$\alpha = \frac{[h]}{[w]} \quad \beta = \frac{[h]}{[S][l]} \quad \gamma = \frac{\sqrt{2g}[w][h]^{3/2}}{[Q]} \quad (13)$$

These represent the ratio of depth to width (this is of little importance), the ratio of depth to elevation change across the segments, and the strength of gravity compared to friction –Mannings coefficient is implicit from the relationship for  $[Q]$  in (10). For the values given above we find  $\alpha \sim 0.08$ ,  $\beta \sim 0.17$ ,  $\gamma \sim 0.6$ .

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The parameter  $\beta$  helps link this mathematical analysis back to the real-world, in that it is related to ‘rule of thumb’ estimates of backwater length used by hydraulic engineers to understand influence upstream as  $0.7^*h/S$  (for example see Environment Agency, 2010). This estimate, like  $\beta$ , tells us that for a significant backwater (and therefore storage) we need to have a small slope and larger depth. In other words, small  $\beta$  indicates that the capacity to hold back a significant volume of water behind

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the dams is very limited. However,  $\beta$  can also be large due to large  $h$  – which can lead to increased probability of failure, if  $h > H$ , so in Section 4 the model is applied to understand different configurations.

A small value of  $\beta$  is also the first indication of why a large number of dams may be required to have even a noticeable effect on the discharge downstream. The small value here is an artefact of the assumed uniform width of the channel, but it is also consistent with the work of Metcalfe et al. [2017], where 57 leaky barriers were required in the 29km<sup>2</sup> Brompton catchment, along 4.7km area of the main stem and Ing Beck, before significant attenuation was achieved. It is likely that the locations for the dams may in reality be chosen to dam “reservoirs” that are wider than the average stream width, where the stream bed is particularly flat, or where there is capacity for significant overflow onto the floodplain, or additional off-line storage (e.g. [Quinn et al., 2013](#), Nicholson et al, 2019). We suggest that the contribution to the cross-sectional area due to the volume in the reservoir in (9) is therefore underestimated, and should be increased. Thus, we modify the cross-sectional area to:

$$\tilde{A}(h) = wb + \lambda \frac{w(h-b)^2}{2st}, \quad H \leq h \quad (14)$$

where  $\lambda \geq 1$  is this enhancement factor that accounts for a larger volume being stored behind the dam. In practice, this would have to be estimated for each dam location, [but provides a useful mechanism for exploring different NFM designs, for instance it can be used to conceptualise reaches where leaky barriers are being used to enhance floodplain reconnection, thus accessing additional storage with greater potential for peak flow attenuation.](#)

One potential concern with the above formulation is the discontinuity in the discharge-depth relation when the water depth reaches the bottom of the dam (Figure 4). This occurs in the model because the physics used to relate the depth to discharge is different in the two cases of free-stream flow (when we use Manning’s law to describe turbulent drag) and flow under the dam (when we use an essentially inviscid formula for flow beneath a sluice gate). Mathematically, provided the discontinuity in flux involves a reduction as  $h$  increases, there should be no problem. As the water depth reaches the bottom of the dam, the flow past it suddenly reduces and the water quickly fills up behind the dam until the depth has increased to allow sufficient flow to balance the inflow from upstream. There can be problems, however, if the discharge suddenly *increases* when  $h$  increases past  $b$  which, from (8), occurs if the slope is sufficiently small, or  $n$  sufficiently large. If that occurs, we continue to use the frictional formula in the first case of (8) until the second formula gives a lower value for the flux.

## 2.6 Solution method

The system of equations (1), coupled with the expressions for  $\bar{Q}(\mathbf{h})$  and  $\tilde{A}(\mathbf{h})$  in (8) and (9), is a system of non-linear equations for the temporal evolution of the water depths  $\mathbf{h}_t$ . The equations are forced by the source terms  $\mathbf{q}_t$ , and each equation is coupled the equations corresponding to the upstream edges of the network. The whole system is solved numerically using an ordinary differential equation solver in Matlab, using code that we have published in a repository as cited at the end of this paper. [Whilst there are a number of hydraulic modelling packages solving similar equations with a diverse range of hydraulic units, these do not permit rapid assessment of collapse and cascade collapse of barriers having leakiness factors and a channel storage multiplier making it easier to test arbitrary networks of configurations.](#)

## 3.0 Results

### 3.1 One-dimensional network

In this section we consider a simple example of the model, using the one-dimensional network shown in figure 2b. We suppose that each of the channel segments is the same (i.e. equal widths, lengths, and slopes), and the discharge in the final

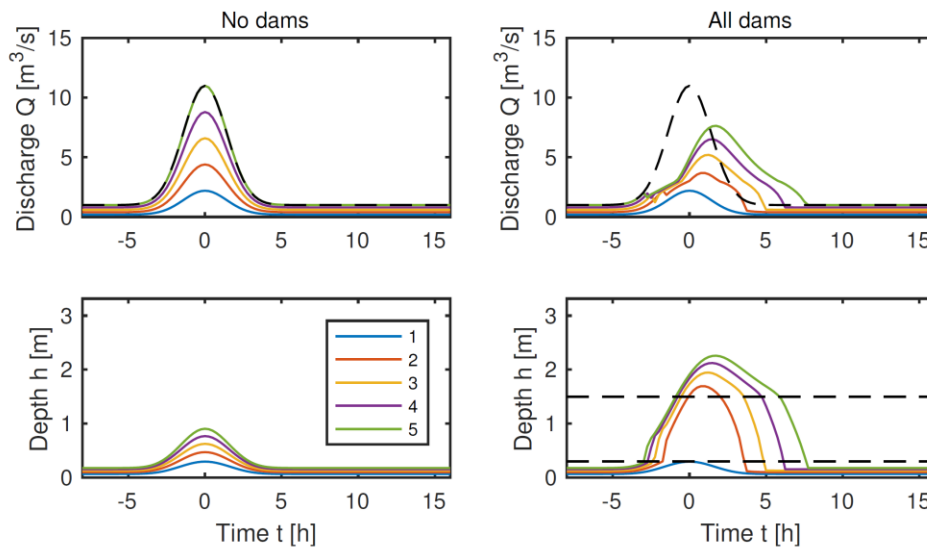
segment is of most interest for the community requiring protection. For these calculations (and all others shown in this report) we use the original flux and area formulas (8) and (9), with the enhancement factor described in (12).

The model is forced with a 'storm' input in the form of a hydrograph based on a simplified Gaussian functional form, as an approximation to a typical design storm estimated using unit hydrograph approach (used in the application to the real case in

5 Section 4). Here an extreme flow of 10 m<sup>3</sup>/s is used to fully stress-test the system, also permitting flows and depths with magnitudes capable of failing leaky-barriers.

$$q(t) = q_0 + q_{max} e^{-t^2/\sigma^2} \quad (15)$$

10 Where  $q_0$  is a baseline inflow (groundwater flow into the channel, say),  $q_{max}$  is the peak flood inflow at time  $t = 0$ , and the flood is spread over a time period  $\sigma$ . In figure 5 we compare the resulting modelled discharge in each channel segment with a large  $\lambda$  factor, between the case of no dams, and the case of having a dam on each segment (we use only five segments for ease of illustration; using more segments allows for greater potential of reducing the peak discharge).



15 Figure 5: Solutions for a one-dimensional 5-node network as in figure 2b, forced by uniform inflow to each node  $q(t) = q_0 + q_{max} \exp(-t^2/\sigma^2)$ , with  $q_0 = 0.22 \text{ m}^3\text{s}^{-1}$ ,  $q_{max} = 3.2 \text{ m}^3\text{s}^{-1}$ , and  $\sigma = 2 \text{ h}$ . Parameter values are as given in section 2.3, together with  $\lambda = 50$ ,  $b = 0.3 \text{ m}$  and  $H = 1.5 \text{ m}$ . Left-hand panels show the response with no dams, when the peak discharge is almost identical to the peak cumulative inflow, shown by the dashed line in the upper panels. Right-hand panels show the response if a dam is included on each of the 5 reaches. The dashed lines in the lower panel shows the heights of the bottom and top of the dams.

20 Figure 6 shows an example when the input has a double peak. In this case, as might be expected, the dams are less effective at reducing the height of the second peak, because they are already holding back a lot of water and have less capacity to store and delay water for the second storm. This indicates that testing of the performance of NFM, or any risk reduction measures, should potentially consider testing resilience against real storm series or double peaks and not simply single peaked storm

25 events, as are commonly assumed in practice when considering flood storage design analysis.

Commented [BH1]: Image corrected – previously no-dams flows were from different experiment with higher flow: now this is consistent

Commented [BH2]: These were typos and have been corrected

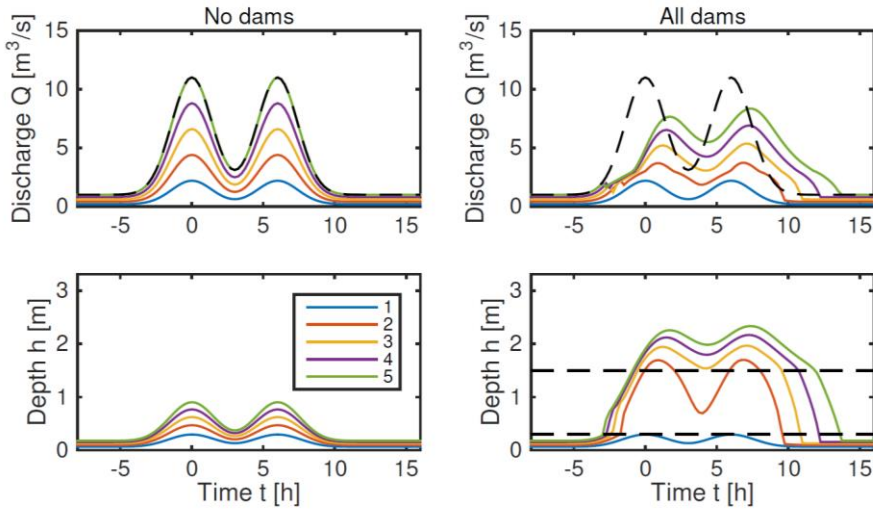


Figure 6: Solutions for a one-dimensional network as in figure 2b, forced by a double peaked input to each node. Parameter values are as in figure 5.

### 3.2 Two-dimensional network

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Here we consider a simple two-dimensional network as shown in figure 2c, which reflects a more likely pattern given the dendritic nature of channel formation in headwater catchments. There are more interesting questions to consider about the positioning of dams in this case. For example, if one has funding to build a certain number of dams, which of the channel segments are the best ones on which to put them? Putting them on the central trunk is likely to ensure that they are used (performance issue 1), but also means that they may more easily overspill and lose their effectiveness. They may also be more susceptible to cascade failure (performance issue 3 - discussed in the next section).

10

In figure 7 we show two examples of the response to a flood input of the form given by equation (15). In the first case, 4 dams are placed on the main trunk (nodes 1-4), whereas in the second case 4 dams are placed on the upper branches (nodes 5,6,9,10). The discharge from the final segment (node 4) is plotted, along with its maximum value. Both dam placements have the effect of slightly delaying and reducing the peak discharge, with the second design being marginally more effective. This is because the dams near the bottom of the central trunk are over spilling and losing their effectiveness, whereas the dams on the side branches are all having a significant effect.

15

However, for different sized floods or realistic spatial patterns of extreme rainfall (see Hankin et al., 2017a), the optimal arrangement can vary. Unfortunately, there does not appear to be a clear rule for the most effective dam placement, even in this simple example, where the resilience of distributed NFM in terms of temporary storage and tree-planting were tested against different storm extremes having spatially realistic patterns (Lamb et al., 2010). In this network study, the 'on average' performance of one particular system of NFM was tested allowing for utilisation and the risk-reducing or risk-increasing impacts of changes to tributary synchronisation (performance issue 2), using average annual losses as the integrated measure of risk reduction. However, the high-resolution model, with 180 million cells, took over 26 hours to run so only 30 extreme events were simulated with and without NFM measures, and *alternative* spatial strategies were *not* tested to understand which were more advantageous. Simplified network analyses such as those presented here could be used to rapidly explore such spatial strategies, without resorting to highly complex, and relatively slow models.

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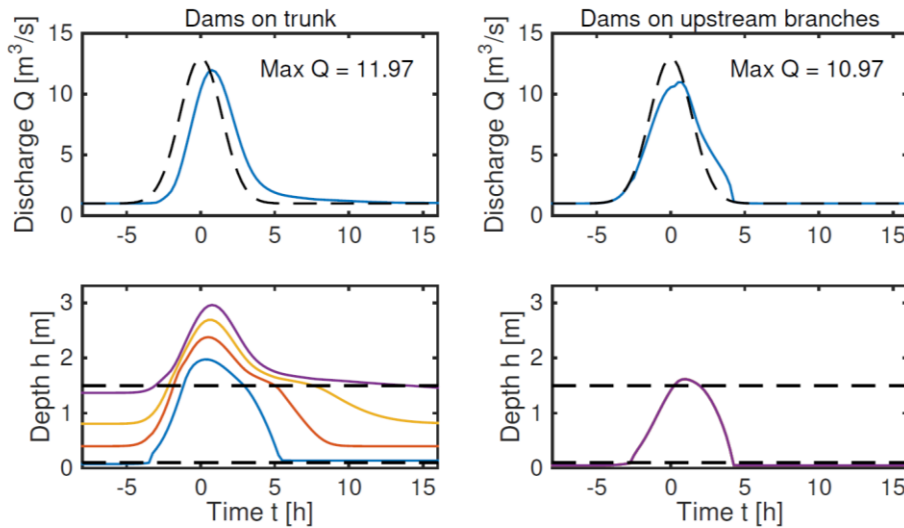


Figure 7: Solutions for a one-dimensional 5-node network as in figure 2cb, forced by uniform inflow to each of the 8 branch nodes  $q(t) = q_0 + q_{\max} \exp(-t^2/\sigma^2)$ , with  $q_0 = 0.125 \text{ m}^3/\text{s}^1$ ,  $q_{\max} = 1.53 \text{ m}^3/\text{s}^1$ , and  $\sigma = 2 \text{ h}$ . The upper panels show the discharge downstream (node 4), and the lower panels show the water depth at the 4 dams (nodes 1-4) for the case on the left, and the identically-behaving nodes 5,6,9,10 for the case on the right). Parameter values are as given in section 2.3, together with  $\lambda = 20$ ,  $b = 0.1 \text{ m}$  and  $H = 1.5 \text{ m}$ .

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### 3.3 Failure mechanisms

One of the potential risks of installing many dams in a catchment is the possibility that they all collapse in sequence, creating a flood surge that is larger than would have occurred if no dams had been installed at all. Provided each dam stores only a small reservoir of water, the collapse of one dam on its own should not be catastrophic. But if the collapse of one dam causes others further downstream to collapse too there is the obvious danger of the surge escalating. This risk may be an important factor in deciding the best placement of dams (perhaps outweighing the efficiency of peak-flow reduction under 'normal' operating conditions).

The main method suggested for analysing this risk is to run an ensemble of simulations of flood events, assigning a failure depth to each dam using the probability distribution suggested by a fragility curve. Ideally, this ensemble should include a range of storm conditions too. Such analysis could in principle use dynamical weather models or rainfall records to

construct statistical models and then sample from the modelled joint (spatial) distributions of rainfall forcing. Both approaches have been considered in the context of reviewing flood resilience in the UK (HM Government, 2016) and for flood risk analysis over large and complex infrastructure networks (Lamb et al., 2019). This type of spatially-structured risk analysis can be expensive, and so it may be desirable to establish some rules of thumb about which dam placements are more, or less, at risk from cascade failure.

Knowledge gained from this type of analysis might be used to plan for the size and strength of dams that should be built at different locations. For example, it could be that certain locations are particularly prone to collapse (downstream of merging tributaries for example), and building one stronger 'buffering' dam could significantly reduce the risk of a cascade.

As an example of cascade failure in the network model, we return to the one-dimensional example shown in figure 2b. We impose a regular storm inflow to the upstream node of the form given in (15) and examine an ensemble of 50 possible system states (describing different combinations of survival or failure of the individual dams). Each of the dams is assigned a critical water depth  $h_{ci}$  such that when  $h_i > h_{ci}$  the dam collapses; the critical depth is sampled from a normal distribution with mean 3.5m and standard deviation 0.5m (the top of the dam is at 1.5m so dam collapse usually occurs when the dam is already submerged). The results are shown in figures 8 and 9. Figure 8 shows the peak discharge  $Q_{max}$  at the downstream node for each simulation.

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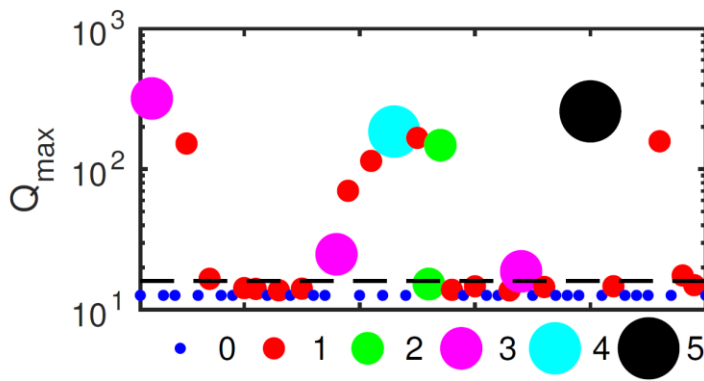


Figure 8 Maximum discharge at the downstream node for an ensemble of runs (indexed along abscissa) on the one-dimensional network in figure 2c, forced by the same upstream inflow  $q(t) = q_0 + q_{max} \exp(-t^2/\sigma^2)$ , with  $q_0 = 1 \text{ m}^3\text{s}^{-1}$ ,  $q_{max} = 15 \text{ m}^3\text{s}^{-1}$ , and  $\sigma = 2 \text{ h}$ . The size and colour of the dots indicates the number of dams that failed during each realisation, and the dashed line shows the peak discharge in the case that no dams are installed. Parameter values are as given in section 2.3 except with  $l = 1000 \text{ m}$ ,  $S = 0.005$ , and  $\lambda = 20$ , together with  $b = 0.3 \text{ m}$  and  $H = 1.5 \text{ m}$ .

Figure 8 helps illustrate whole-system resilience across a wide range of events, and is coloured by the number of dams that are predicted to collapse within each ensemble member; small blue dots indicate that no dams failed, and since the forcing is identical in each case, the peak discharge in this case is always the same. It is lower than what the peak would have been in the absence of any dams, so the dams are proving effective in these cases. Larger dots correspond to more dams having failed. In most of the ensemble members only one dam collapses, and the peak discharge recorded downstream is strongly dependent on which one fails (the red dots in figure 8). A larger peak occurs if the collapsed dam is further downstream, since if an upstream dam fails (and importantly does *not* precipitate a cascade of downstream failure) the sudden release of water from that dam is buffered by the dams further downstream.

If, on the other hand, a single dam failure leads to further collapse of two or more dams, the peak discharge can be much larger. In one example, all five dams collapse in quick succession, and the time series of this example (the black dot in figure 8) is shown in figure 9, where it is compared to an example with no failure. We have found that the pattern of failure in this one-dimensional model, including the likelihood for cascading failure, depends heavily on the assumed dam sizes, critical water depth distribution, and the magnitude of rainfall events.

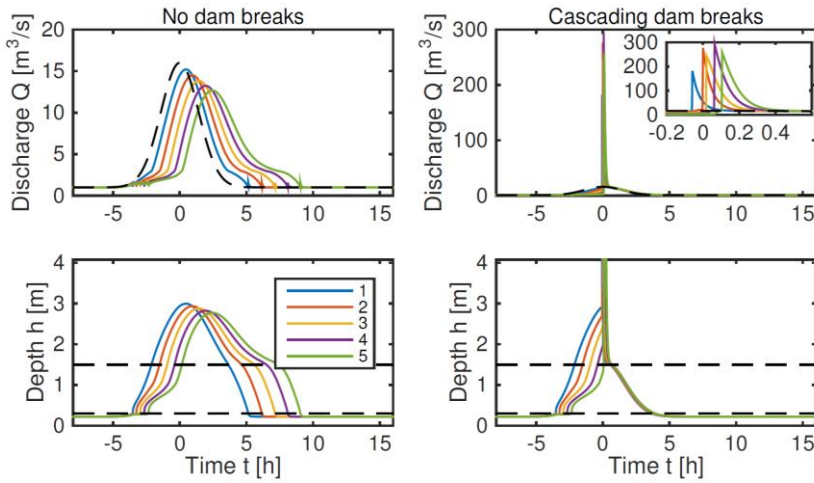


Figure 9: Example from an ensemble of runs on the one-dimensional network in Figure 2b, forced by the same upstream inflow  $q(t) = q_0 + q_{\max} \exp(-t^2/\sigma^2)$ , with  $q_0 = 1 \text{ m}^3\text{s}^{-1}$ ,  $q_{\max} = 15 \text{ m}^3\text{s}^{-1}$ , and  $\sigma = 2 \text{ h}$ . The dams have critical failure water depths  $h_{ci}$  drawn from a normal distribution with mean 3.5 m and standard deviation 0.5 m, and shown by the coloured dashed lines in the lower panels. In the case on the left, no dams fail, whereas in the case on the right (when the uppermost dam is particularly weak), they all fail in a cascade. Parameter values are as in figure 87.

As a second more instructive example of cascade failure, we revisit the herring-bone network in figure 2c. We consider the two possible placements of 4 dams that were discussed earlier; either along the main trunk (nodes 1-4) or on the upstream side branches (nodes 5,6,9,10). In figure 7 we found that there was relatively little difference in the peak discharge measured at the downstream node with these different placements. However, in figure 10 we see that the first case is much more at risk from cascade failure and the whole system of dams is not resilient in this spatial configuration. This figure shows the peak downstream discharge in an ensemble of simulations, with the failure depths for each dam being different each time. The failure depths are sampled from the same distribution in each case.

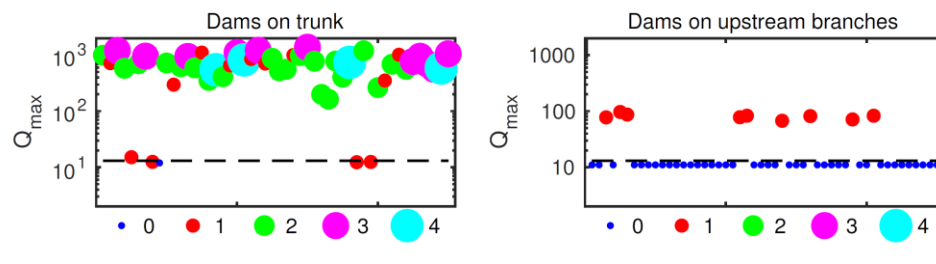


Figure 10: Maximum discharge at downstream node for an ensemble of runs (indexed on abscissa) on the two-dimensional network in figure 9, forced by uniform inflow to each of the 8 branch nodes  $q(t) = q_0 + q_{\max} \exp(-t^2/\sigma^2)$ , with  $q_0 = 0.125 \text{ m}^3\text{s}^{-1}$ ,  $q_{\max} = 1.5 \text{ m}^3\text{s}^{-1}$ , and  $\sigma = 2 \text{ h}$ . Each dam is assigned a failure depth  $h_{ci}$  drawn from a normal distribution with mean 3.5 m and standard deviation 0.5 m. In the first case, dams are placed on the four trunk segments (nodes 1-4), while in the second case they are placed on the upstream side branches (nodes 5,6,9,10). The size and colour of the dots indicates the number of dams that failed in each realisation, and the dashed line shows the peak discharge in the case that no dams are installed. Parameter values are as in figure 7.

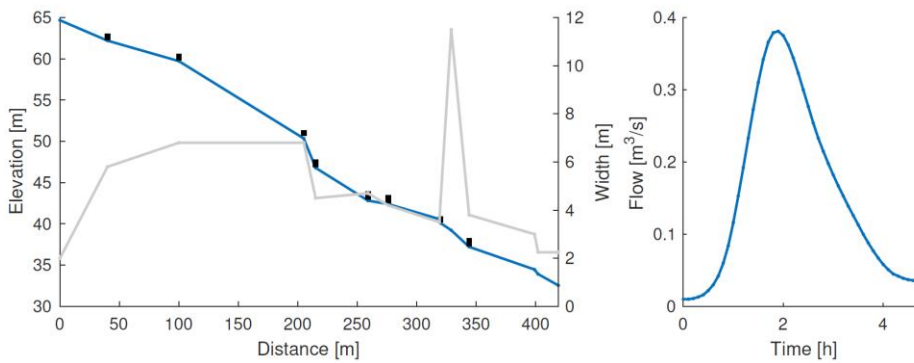
In almost every run with dams on the trunk, we see cascade failure occurring so that 3 or 4 of the dams collapse; this leads to extremely high (though short-lived) peak discharge. In contrast, when the dams are placed on the side branches, there is no possibility of cascade failure (the individual branches do not communicate with each other) and it is unlikely that more than

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one dam collapses. Thus, [having used the model to consider the resilience of the whole system of barriers for two cases, it can be seen that](#) although each of these dam placements is similarly effective at reducing the peak discharge, there may be strong reason for preferring the second design that places them on the upstream tributaries because this configuration is a more resilient system (even though the resilience of the individual dams is the same in both designs). The large surges predicted in the simple network model when multiple dams fail have some support in the literature. For example, Hillman [1998] describes a June 1994 outburst flood in central Alberta, Canada, releasing 7,500m<sup>3</sup> of water and a flood wave 3.5 times the maximum discharge recorded for that creek over 23 years. Although not reported as a cascade failure, large trees and debris from older beaver dams were carried further downstream, and five hydrometric stations downstream were destroyed.

#### 10 4.0 Application of model to Penny Gill, West Cumbria

[We consider the application of the model in a 'sensitivity to change investigation' to a site on Penny Gill, West Cumbria](#) (figure 1). The geometry of the network of leaky dams installed by the West Cumbria Rivers Trust is shown on the left of figure 11, with the inflow hydrograph on the right. The inflow hydrograph has this time been based on a 100 year return period design hydrograph ([synthetic hydrograph with estimated annual exceedance probability](#)) using Revitalised Flood Hydrograph or ReFH version 1 (Kjeldson, 2007), which is based on a unit hydrograph approach assuming empirical relationships with local catchment descriptors such as slope, annual average rainfall from the Flood Estimation Handbook (IH, 1999). ReFH also includes a losses model that accounts for hydrology of soil types and gives the hydrograph in this instance a slight tail due to slower baseflow contribution. It should be noted that the peak flow for the 100 design year event is relatively small but is likely to be under-estimated owing to contributions from old coal measures that are known to generate additional flows during times of prolonged rainfall.



**Figure 11: Left: Geometry of the Penny Gill network with given position of dams and widths labelled. Right inflow hydrograph based on ReFH approach described in the text.**

25 The model is as described in section 2, with mass conservation for each network given by equation (1) as before, although in this example water is all fed into the upper most segment of the network according to the input hydrograph  $q_{in}(t)$  shown on the right of figure 11. Discharge and cross-sectional area are related to water depth at the dams  $h_i$  by the functions  $Q_i = \bar{Q}(h_i)$  and  $A_i = \bar{A}(h_i)$  given in by dimensionless equations (11-13) in section 2. The only changes are that  $\lambda=1$ , and variable lengths, widths and slopes are being used for different segments, allowance for which was already made. [Penny Gill](#)

stream is incised and there is little possibility of greater connection with the floodplain, so hence the barriers are relatively tall and rely on extended in-channel storage, except for the very-relatively wide segment in figure 11 which reflects an area of channel well connected with a relatively wide depression.

A model calculation using the measured values for the dam parameters (bottom height  $b$  and top height  $H$ ) showed very little delay or reduction in the peak of the downstream discharge hydrograph relative to the input hydrograph. This is because the dams were hardly coming into operation at all. However, at a recent site-visit there was siltation and noticeably small slot heights for many of the leaky barriers, over much of the width of each construction. Therefore, an example was simulated with the bottom heights lowered to 0.02m, as shown in figure 12, which ensures more of the storage comes into use. It should be noted that for this 1-d model, the slot heights represent an *average* across the width of the barriers, which represents a further approximation.

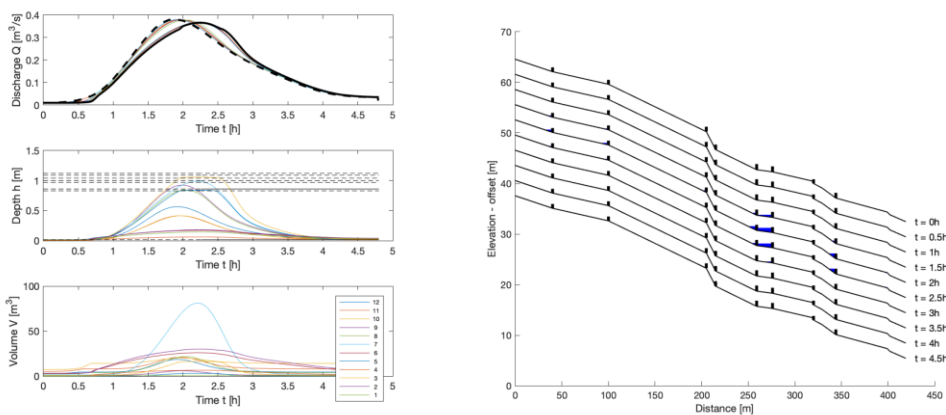
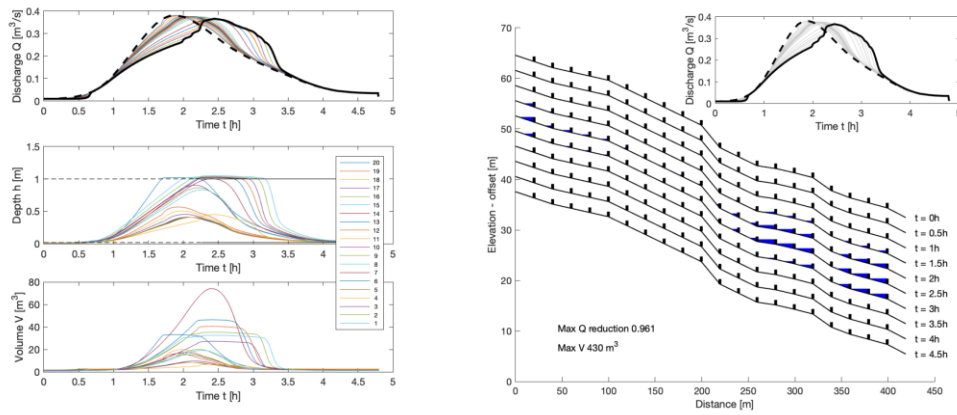


Figure 12: Model run using the given measured dam locations and bottom height of 0.02m. Note that each segment has a different width, which is not shown on the diagram, but which can lead to different rates of filling of the region behind different dams. We take a Manning roughness of  $0.1 \text{ s m}^{-1/3}$ . The reduction peak discharge is 96% of the inflow, and the maximum volume stored is  $245 \text{ m}^3$ .

We can use the network model to quickly explore alternative arrangements of the dams, and examine whether there are general rules about how to site the dams that could lead to more efficiency. To do this we break the stream into 20 segments, and allow for the possible siting of a dam on each one. All such dams are assumed identical, with bottom height  $b = 0.02 \text{ m}$  and top height  $H = 1 \text{ m}$ . An example calculation with dams on every section is shown in figure 13. The issue of under-utilisation is clear from this example. Note also that although more water is stored than in figure 12, the reduction in peak discharge is actually slightly less. This is because the dams are already full by the time that the peak now happens, and illustrates the subtleties involved in deciding how and where to place the barriers.





**Figure 13: Model run with more leaky dams. The reduction peak discharge is 96% of the inflow, and the maximum volume stored is 430 m<sup>3</sup>.**

- 5 We also consider randomly siting 8 dams on the 20 segments, to analyse which positions work well. Examples of some of these are shown in Figure 14. Some are considerably better at reducing the peak flow than others (though none are particularly good - simply because there is not enough storage overall to reduce the peak discharge substantially).

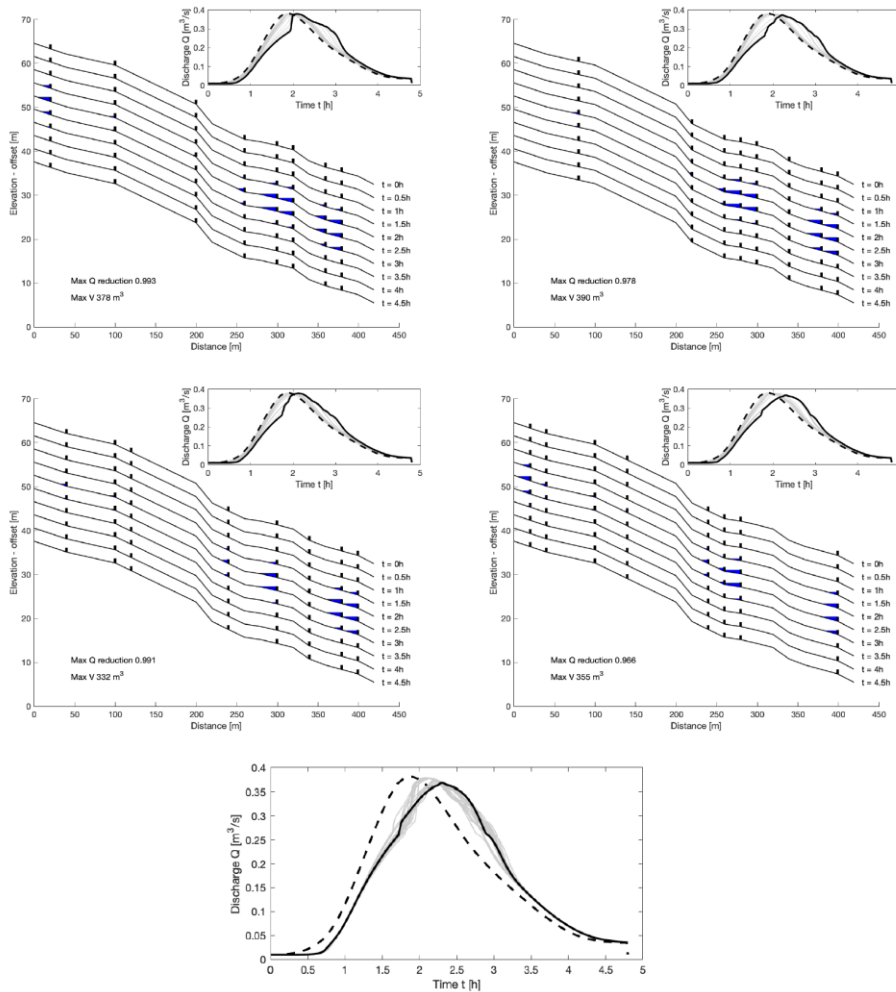
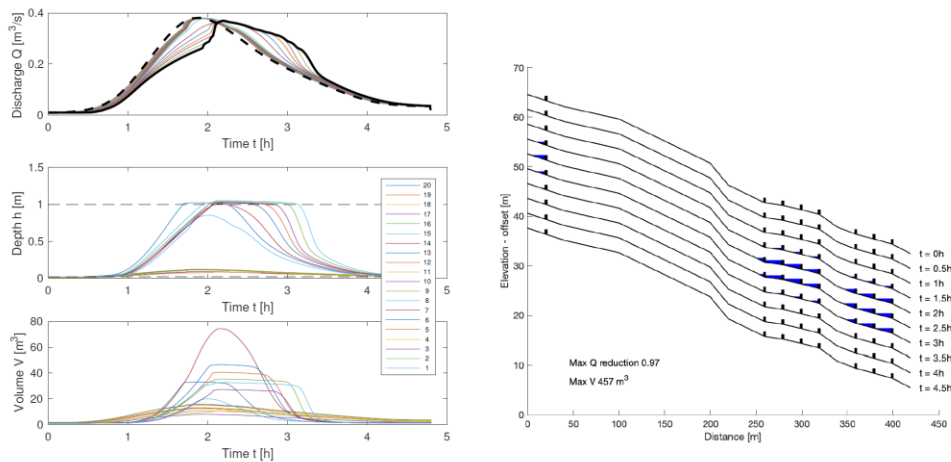


Figure 14: Example model runs with random arrangements of 8 dams. The lower panel shows the downstream discharge for each of 20 different arrangements, compared with the input hydrograph (dashed), and with the one that gives the greatest peak reduction (97%) highlighted in black, which is the lower right of the examples above and results in a maximum volume stored of 355m<sup>3</sup>.

Whilst storage may be improved well above that for the real system (355m<sup>3</sup> as opposed to 235 m<sup>3</sup>), the dynamic utilisation of that storage in the network does not result in a better reduction in percentage peak flow reduction (97% as opposed to 96%), this being a key measure to assess the effectiveness of the whole system. This highlights the unpredictability of the network and whole system performance and demonstrates why such a model is important at larger scales. By choosing to site dams on shallower segments of the network, storage is enhanced and under-utilisation is avoided; an example is shown in Figure 15. This has almost as good performance as building dams on all 20 segments (compare with Figure 13). Note however that this is partly due to assuming identical dams. Since the water depth would typically be lower on steeper stretches, it would be natural to have a smaller slot under the dam there so that the dam becomes active. Nevertheless, the inability to store significant water behind a dam on steeper slopes means that such locations should generally be avoided.

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**Figure 15: Model run with 8 dams, chosen to be sited on segments with lower slopes. The reduction peak discharge is 97% of the inflow, and the maximum volume stored is 457 m<sup>3</sup>.**

- 5 What insights can we learn from this? A good general principle is to site the barriers in low-lying areas or regions where the upstream area widens, so as to provide more storage per barrier. The lower height of the barrier should be made sufficiently high that it does not start to dam water too early; if the dam has filled before peak inflow conditions, it serves no further useful purpose in reducing the flow downstream. Dams in locations with a large backwater region (which is characterised by the local value of  $\beta$ , although it is more easily interpreted in terms of being shallow-sloping and wide) are worth building higher, and making correspondingly stronger so as to avoid the risk of failure.
- 10

Such advice could be used at a much larger scale to supplement the relatively scarce advice for where to locate leaky barriers, which also tends not to include details of geometries, leakiness or slot heights. For example, in England and Wales, countryside stewardship grants can be applied for to support construction of leaky barriers and the website [2017](https://www.gov.uk/countryside-stewardship-grants/rp33-large-leaky-woody-dams)<sup>3</sup> advises that leaky barriers should be sited on channels between 3 and 5 m, yet says nothing about slope, height and slot dimensions, which could be more of a determining factor in the effectiveness of potential storage. The dimensions of the slot height are also not clear, and vary in grey literature between 0.1- 0.3m, but in practice, are less (on average across a cross section) than this in locations like Penny Gill. A compounding factor is that there are many forms of leaky barrier or large woody debris dams (Addy et al., 2019) including placing large woody debris in channel, to the horse-jump type barriers in use in Penny Gill, combined with engineered log jams which will also reduce passage of debris should a structure fail and enhance floodplain reconnection. – These different designs all need to consider the trade-offs in barrier design (slot height, leakiness – both of which can be adjusted in the model reported here) when considering ecological impacts such as fish passage, for example a narrow slot might improve flood attenuation but make passage more difficult.

- 25 However, general advice on design can over-simplify, and the final example has demonstrated that this type of network model can be used effectively to rapidly test different arrangements of dams and to assess which are likely to work best to reduced risk given the unpredictability of the whole system response. Probably more importantly, the final example demonstrates that this type of model can be effectively used to quickly try out different arrangements of dams and to assess

<sup>3</sup> <https://www.gov.uk/countryside-stewardship-grants/rp33-large-leaky-woody-dams>

which are likely to work best. Only one particular input hydrograph has been used here, and a more thorough analysis ought to consider different amplitudes and shapes and multiple peaks, since there are likely to influence the effectiveness of the whole scheme. It would also be useful to test further failure scenarios, although the simplified 1d representation does not include log-jams that were also placed between some of the dams, which would help mitigate the risk of cascade failure explored in Section 3. In summary, the network analysis has demonstrated how the effectiveness of the system of leaky-barriers quantified overall using the integrating measure of percentage peak flow reduction at the bottom of the network. The approach accounts for additional storage volumes put in place, but also how it is utilised dynamically, something that will also vary across the system depending on the spatial and temporal pattern of runoff inputs, slot dimensions and leakiness.

## 5.0 Conclusions

We have formulated a network model for a catchment area that allows for simple exploration of the effectiveness of different dam placements and designs and is sufficiently cheap to solve that it may be useful in analysing risks that require a large ensemble of simulations. We have applied the model for relatively small idealised and real systems of around 10-20 dams, but its computational simplicity means that it would readily scale to consider much larger systems at little additional cost. Based on the analyses presented, we can make four practical conclusions focussing on the three performance issues highlighted, those of utilisation, synchronisation and failure or cascade failure:

- A large number of dams are needed to have any significant effect on the peak discharge downstream based on scale analysis alone, especially in reaches with steep gradient. When estimating storage requirements, it is not sufficient to simply estimate the total storage capacity in relation to the volume under the hydrograph for a set of NFM measures that are distributed around the network. Network analysis is required that also permits the assessment of the integrated impact of: **dynamic utilisation of storage; drain down between events** (tested by simulating on multi-peaked events); and **changes to flood-peak synchronicity** on overall risk reduction. This has been measured in terms of reduction to peak flow hydrograph at the bottom of the network between the pre- and post-NFM situation, providing an integrated measure of the effectiveness of the system of NFM features.
- The dams should be located in places with the potential to store a reasonable volume of water (in wide reaches of the channel), although with consideration that the loading on each structure is not excessive. With reference to a real-world example at Penny Gill, we have used the network model to highlight how locating dams in areas with wider channel width and low slope is more effective and are worth building higher, and making correspondingly stronger so as to avoid the risk of failure.
- These conclusions on placement help, in part, to understand whether there are any benefits from making an effort to place dams strategically, seeking an optimal network configuration, or whether it may be justifiable to install them opportunistically, or even randomly. The analysis indicates that, for the relatively simple system at Penny Gill, when considering potential dam sites at up to 20 locations, approximately 50% of effort could be saved in construction, costs and later maintenance, if fewer dams are placed more selectively. It remains to be seen whether there are any broader advantages at large scales (>100km<sup>2</sup>, say) of macro-scale strategies, such as targeting one whole side of a valley or another. The approach demonstrated here enables such analysis to be carried out.
- Cascade failure is a risk when dams are placed along a main artery and the risk may be lessened by spreading dams around tributaries. There are very large uncertainties in the fragility assumptions leading to failure, although here water depths of an order of *twice* the barrier height were used, which would present a considerable loading. Despite the uncertainties in the probability component of risk, the potential consequences, which appear to be evident in historical events in natural systems, should highlight the need for robust barrier design supported by good engineering design of leaky-barriers. For the case-study of Penny Gill investigated here, the placing of large woody material within the channel between dams is another good risk reduction strategy.

We envisage future risk assessments using this network approach at larger scales, taking into account additional factors including: uncertainties in geometry, roughness parameterisation, spacing, fragility assumptions, a wide range of spatial configurations of NFM measures, and a wider range of feasible storm types, durations and probabilities. These are all required not just for NFM, but also for improved integrated flood risk management, if we are to answer the types of simple questions that communities need to answer, such as: “With a limited budget, what’s the best approach for integrated flood risk management?”, or, “does spatial configuration even matter at a larger scale?”. We hope our conclusions here start to address such questions, but future analyses would also be better constrained with a more detailed understanding of the fragility of different types of barriers. More formal fragility curves can be directly generated (Lamb et al., 2019) based on analysis of observations of survival and failure of dams, if such data is recorded for the growing number of leaky barriers.

## 6 Author Contribution

Barry Hankin wrote the paper with major contributions from Ian Hewitt and Rob Lamb. All authors helped in developing the model at the Maths Foresees Study Group.

## 7 Acknowledgements

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## 8.15 Code Availability

The Matlab scripts developed are available on the JBA Trust GitLab repository:

<https://gitlab.com/jba-trust/leaky-barrier-network-analysis>

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