Breach Model Validation Programme

Internal Erosion Initiated Breach: Model Performance Review & Validation

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Executive summary

This project was initiated by Electricité de France with the goal of reviewing, testing and validating the performance of existing, industry applicable breach prediction models, which could predict breach formation caused by internal erosion through dams and levees.

The definition of 'industry applicable' is a model which can be applied by practising engineers, is commercially available, ideally with some form of support, uses parameters which can be reasonably estimated or measured, and which takes seconds or minutes to run rather than hours or days.

The definition of an 'internal erosion initiated breach model' is a model that simulates growth from a defined internal pipe through to open breach and catastrophic failure of the dam or levee.

A total of 4 phases of modelling work were undertaken, with 5 different models being tested (AREBA, DLBreach, EMBREA, Rupro and WinDAM C). Throughout the programme of work (2019-2024) ~15 different modellers participated, drawn from industry, academia and NGOs both in Europe and the USA.

The different phases of modelling work related to:

- Phase 0 Initial setup of approach and model application to a hypothetical test case (1 case)
- Phase 1 Model application to field test data cases (4 cases)
- Phase 2 Model application to dam failure case studies (2 cases)
- Phase 3 Analysis of modelling and data uncertainty related to dam failure case studies (2 cases)

Modelling Phases 0-1-2 focused on comparing model performance against different sets of data, from hypothetical (Phase 0) through field test data (Phase 1) and on to real dam failure cases (Phase 2). Key outcomes from this work included:

- Despite defining many modelling parameters, some modellers varied these parameters or adapted other hidden parameters during the blind modelling, making a direct comparison of model performance difficult. This also highlights the challenges new modellers face when applying models; a thorough understanding of how the model performs is essential to obtain reasonable results
- Where modellers were required to make parameter value estimations, the estimated values varied significantly from modeller to modeller. In addition, model sensitivity to different parameters varies from model to model
- Modelling of the internal erosion pipe formation and roof collapse process appeared to have a relatively small effect on overall prediction of peak flood flow conditions – perhaps in comparison to the significant effect of uncertainties in other parameters such as soil erodibility
- Aware modelling demonstrated that the performance of most models could be improved by varying key parameters – in particular for soil erodibility

For Phase 3 of the work programme, a detailed analysis of test case data uncertainty was compared against Monte Carlo breach analyses using defined parameter uncertainty ranges and distribution. The goal here was to assess whether, given the right selection of parameter values, the models could predict conditions within the observed range of data. Key outcomes from this Phase 3 programme of work were:

- Given the right combination of modelling parameters, most models could predict observed conditions

- The range of uncertainty in the predictions – accumulating uncertainty from the modelling parameters – can be very large
- The range of uncertainty in predictions (max/min etc) varies between the different models – but also between different modellers. Modellers understanding of how a model works and should be applied is important
- The impact of correctly simulating pipe formation and roof collapse through to open breach depends upon a variety of factors and is not as significant as having a more reliable measurement or estimation of soil erodibility
- The use of PR functions to identify 'best runs' to achieve a certain performance function was a useful way of seeing how close models could get to 'observed' conditions

Overall project conclusions then included:

- Many of the models have the potential to predict the observed conditions given use of the right parameters and the correct model application
- Best estimates using the mean of Monte Carlo modelling results gave good estimations for many parameters and can be within ±15-20% of observed data
- Predicted result uncertainty bands arising from uncertainty in modelling parameters are very large. Whilst using the mean average predicted values seems to give good performance the max-min result ranges are very large (often factors of 2 or 3 above or below observed). There is a need to reduce uncertainty in parameter value measurement/estimation - in particular for soil erodibility, K_d and estimation of Manning's n/model flow roughness value
- Pipe formation and roof collapse routines within the models do affect breach peak outflow/time to peak outflow estimations, but the impacts appear to be smaller than might be initially assumed
- The accuracy of the modelling results depends significantly on the understanding and judgement of the modeller; significant differences in applications can be seen within this group of experts, relating to detailed knowledge of model setup
- Comparing predictive breach modelling results to simple peak discharge equations shows that the range of prediction from simplified equations (depending on high/med/low soil erodibility) is larger than from the physically based models
- An action most likely to improve the accuracy of breach modelling is to improve our ability to measure and predict and apply soil erodibility (K_d) more accurately for different dams and levees

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1 Introduction

Dams and levees typically breach through overtopping or erosion initiated by internal erosion, eventually leading to open breach and catastrophic failure. A range of different methods exist to predict potential breach conditions, with physically based predictive models offering the most flexible tools for engineers wishing to assess performance and flood risk. However, not all breach models allow for the simulation of internal erosion initiated conditions and different models adopt different methodologies. It is unclear which approaches offer the best solutions, hence the goal for this project was to review, test and validate the performance of existing, industry applicable breach prediction models, which could predict breach formation initially caused by internal erosion through dams and levees.

The project comprised the following stages of work, which are detailed in the following sections:

1. Undertaking an initial review of breach models
2. Establishing an international group for applying and reviewing model performance
3. Reviewing existing/available internal erosion initiated breach data sets
4. Implementing a programme of breach model testing and validation using different data sets and case studies
5. Drawing conclusions regarding model performance, validated through the international group

The project was commissioned by Electricité de France (EDF) and ran from 2019-2022.
HR Wallingford managed the overall programme of work, data analysis and group discussions leading to the overall conclusions.

2 Review of models and creation of modelling team

A review of existing, industry applicable, internal erosion initiated breach prediction models for inclusion within the test programme was undertaken. This review drew on:

- HR Wallingford (HRW) expertise (+20-year rolling programme of breach research and development work)
- Internet search
- Emailed invitations to participate via the ICOLD Internal Erosion Working Group, ICOLD European Working Group on Overflow and Overtopping Erosion and EDF and HRW known experts working in this field

The definition of 'industry applicable' for this project is a model which can be applied by practising engineers and:

- is commercially available, ideally with some form of support
- uses parameters which can be reasonably estimated or measured
- takes seconds or minutes to run rather than hours or days

The definition of an 'internal erosion initiated breach model' is a model that simulates growth of internal erosion through to complete open breach failure of the dam or levee. It is not sufficient to simply predict that a form of internal erosion might occur, or to predict initiation but not growth to breach (unless simply linking with another model to provide an overall solution). Hence this requires the model to:

- Either predict absolute initiation processes (eg ICOLD bulletin 164 (ICOLD, 2015)) or assume an initial 'pipe' through the structure, and
- Predict growth of that pipe with any associated erosion processes, and hence also allow for upstream and downstream hydraulic boundary conditions

The model may or may not include internal soil conditions/pore pressures etc as appropriate.

2.1 Internal erosion initiated breach models

The review highlighted that there appeared to be three categories of model available:

1. Internal erosion process models (limit state equations etc predicting whether different forms of internal erosion will occur – but not predicting beyond that)
2. Complex CFD modelling – perhaps simulating particle processes, but not 'industry applicable' by the definition above
3. Industry applicable breach models, but with internal erosion initiation assumed as a starting flow through a defined flow path (hole)

Category 3 models were considered appropriate for the evaluation programme. The models identified at this initial stage are summarised in Table 2.1 below.

Table 2.1: Industry applicable models identified for evaluation

Model	Contact	Organisation	Country
AREBA	Myron van Damme	TU Delft	Netherlands
DLBREACH	Weiming Wu	Clarkson University	USA
EMBREA	Mohamed Hassan	HR Wallingford	UK
RUPRO	André Paquier	INRAE (formerly IRSTEA)	France
WinDAM C	Sherry Hunt	USDA-ARS-HERU	USA

In addition to these models, the following models (Table 2.2) were also considered:

Table 2.2: Additional models considered for evaluation

Model	Action
DAMBRK – NWS, USA	Whilst innovative in the 1980s this was now considered redundant and not included within the programme.
Telemac – EDF, France	Unclear whether this complied with the 'industry applicable' definition – not included.
UCL, Belgium – CFD modelling	Does not comply with the 'industry applicable' definition – not included.
ARUP, UK – modified form of AREBA	ARUP was included, but the model changed from modified AREBA to a new code OvaBreach. However, as testing progressed it became clear that additional development work was needed, and the model was not used in the later testing stages of the project.

Descriptions of each model, covering their functionality and approach for breach prediction can be found in Appendix A.

2.2 The model evaluation team

During the initial search for models to participate in the testing programme, model developers, researchers and practitioners alike were also invited to participate in the testing programme. This open invitation was very successful and resulted in a team comprising representatives of each of these different sectors (i.e. developers, practitioners, researchers). In addition, whilst some participants chose to apply and test a single model, some chose to apply and test multiple models. This meant that we could compare results of model applications undertaken by different people, with different levels of model familiarity – more closely representing practice in real life as compared to how, say, the model developer would apply their model.

Ultimately, a project team was established with the members as summarised in Table 2.3 below:

Table 2.3: The model evaluation team

Name	Organisation	Country	Sector	Role
Jean-Robert Courivaud Julien Cintract	EDF-CIH	France	Industry	Project Director
Mark Morris	HR Wallingford (HRW)	France	Consultant/Applied Research	Project Manager
Tony Wahl	USBR, DSO	USA	Government	Participant/Modeller
Ghada Ellithy	ERAU	USA	University	Participant/Modeller
Sherry Hunt Ron Tejral Darrel Temple Abdelfatah Ali	USDA-ARS-HERU (ARS)	USA	Government	WinDAM C Development & Modelling
Mohamed Hassan	HR Wallingford (HRW)	UK	Industry	EMBREA Development & Modelling
Myron van Damme	TU Delft/Rijkswaterstaat	Netherlands	University/Government	Participant
Weiming Wu	Clarkson University (UniClrk)	USA	University	DLBREACH Development & Modelling

Name	Organisation	Country	Sector	Role
Stanislav Kotaška Jaromir Riha	Brno University of Technology	Czech Republic	University	AREBA Development & Modelling
Al Preston	Geosyntec	USA	Industry	Participant/Modeller
Mitch Neilsen Antony Atkinson	Kansas State University	USA	University	WinDAM C Development & Modeller
André Paquier Theophile Terraz Stéphane Bonelli	INRAE	France	Government	RUPRO Development & Modeller
Veronika Stoyanova	ARUP	UK	Industry	OVABREACH Development & Modeller
Rafael Moran	UPM	Spain	University	Participant/Modeller

ERDC / ERAU: Ghada Ellithy worked initially at ERDC and subsequently at ERAU. Either reference relates to modelling work undertaken by Ghada.

VUT / BUT: Both relate to the Brno University of Technology; VUT is the Czech abbreviation. Either reference relates to modelling work undertaken by Stanislav Kotaška.

3 The Model Testing and Validation Programme

The need to be able to distinguish model effects from modeller effects, and to be able compare 'like with like' were recognised as key issues from the outset. Several of the team members had participated in the earlier CEATI DSIG breach modelling project and hence had experienced these challenges before. The general approach adopted was therefore to:

- Define each test case as clearly as possible, keeping options for modellers to make to a minimum and asking modellers to follow those defined test conditions as closely as possible – as far as their models permitted
- Undertake both blind and aware model tests. Blind tests are where the test case is defined, but observed or measured data are not supplied; aware tests are where the observed or measured data are subsequently supplied, and modellers are invited to improve their results

The challenges that arose in following this process included:

- Some models used different parameters for their analyses, hence some uncertainty arises in calculating equivalent values
- Some models/modellers used additional factors to adjust conditions within the model. Without recognising these factors, like for like comparisons are not truly being undertaken
- Some modellers varied the assumptions or parameters for the test case, rather than rigidly following the defined parameters
- Some models were adapted, updated, corrected as the work progressed, hence their performance for later tests may differ slightly from earlier tests

Each of these aspects needed to be taken into consideration in drawing conclusions from the programme of testing.

3.1 The test programme

The test programme evolved as the testing progressed, resulting in the overall programme shown in Table 3.1 below. It should be noted that the original programme envisaged 3 phases of modelling, with in person team meetings after each phase to review and assess model performance. With the COVID 19 pandemic starting in the Spring of 2020, a few months after the kick off meeting in Stillwater, Oklahoma, the schedule changed to mainly online discussions. In addition, the programme was extended to include 4 phases of modelling instead of 3 to adapt to the modelling challenges found.

Table 3.1: Model evaluation test programme

Phase	Description	Period
Kick off workshop	Workshop at USDA-HERU, Stillwater.	15-16 October 2019
Phase 0	A single hypothetical test case intended to test templates, and the modelling and data analysis process. Test comprised: • Hypothetical failure • See Appendix B for the test case details	Launch April 2020 ⇳ Review July 2020

Phase	Description	Period
Phase 1	Four tests, focussing on large scale field test data. Tests comprised: • Modified hypothetical failure • IMPACT (EC IMPACT Project) • ARS P1 (ARS HERU (Stillwater) breach test programme) • ARS P4 (ARS HERU (Stillwater) breach test programme) • See Appendix C for the test case details	Launch July 2020 ⇳ Review Part 1: Jan 2021 Review Part 2: Mar 2021
Phase 2	Two tests, focussing on real dam failures. Tests comprised: • Big Bay Dam Failure (12 March 2004) • Lawn Lake Dam Failure (15 July 1982) • See Appendix D for the test case details	Launch Dec 2020 ⇳ Review July 2021
Phase 3	Revisiting the Phase 2 test cases, but with an in depth consideration of modelling and test case data uncertainty. Tests comprised: • Big Bay Dam Failure (12 March 2004) • Lawn Lake Dam Failure (15 July 1982)	Approach Development Meeting (UPM): Nov 2022 Launch: Apr 2023 ⇳ Review: Nov 2023

3.2 Modelling data analysis and conclusions

Sections 4-7 of this report detail the work undertaken in modelling the various cases under Phases 0 – 3 of the test programme. These sections present an assessment of the modelling results, along with any conclusions that may be drawn (at that stage). An overview of test setup and modelling results for each phase of the programme can be found in the associated Appendix.

More detailed information, along with all the modelling results, including the plots used for performance comparison, can be found in a series of Excel spreadsheets associated with each test. Details of the files needed to review this data are summarised at the start of each Appendix.

It is important to recognise that modelling results arise from a combination of both model and modeller capabilities. To get as objective an assessment as possible regarding model performance, it is necessary to compare 'like with like' in terms of modelling parameters and model application; challenges arise when different models use different parameters or techniques to address the same processes or modellers apply the models in different ways or use different embedded (hidden) modelling factors.

To minimise any confusion over modelling assumptions and approach, modellers were asked to provide a summary of key modelling parameters used for each test case. These are presented in the Appendices alongside the modelling results. The modelling results are plotted firstly showing all models and results compared on one set of plots (i.e. Flow, breach dimensions, water levels etc), secondly in greater detail (where needed) and thirdly comparing modellers using the same models (i.e. plots of results separately using AREBA, EMBREA, DLBreach etc).

Since the project team comprised a mix of model developers, researchers and practitioners, it was also interesting to see whether any trends arose in the way a different type of user applied the models. For example, whether modelling undertaken by model developers was generally more accurate than that undertaken by non-developers.

4 Phase 0: Hypothetical Test Case Evaluation

4.1 Modelling Objectives

The objective of the Phase 0 modelling was to test the modelling procedures setup such as test definition, data exchange, results analysis etc. To achieve this, a hypothetical test case was formulated. Summary data tables of the case and plots of modelling results (as referred to in the following sections) can be found in Appendix B.

4.2 Modeller Assumptions

Table B.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table immediately highlights that, despite trying to define a simple test case, the modellers have assumed a range of different values and conditions for their simulations. For example:

- **Modelling Approach**: Assumptions included assuming homogeneous using core material, or body material, or using an external layer. Some modellers chose to simulate breach with headcut, some breach through surface erosion
- **Dam Foundation**: Since the test case showed a dam constructed on a slope, different modellers chose different foundation base levels (since all models assume a flat base to the dam section)
- **Initiating Diameter**: The modeller was allowed to choose the initial seepage flow hole size for their model. Shapes included rectangular and circular. Sizes varied from 1-3 cm (side or diameter)
- **Location Along Dam**: Varied significantly – modeller choice
- **Initiating Timing**: Some modellers chose t=0 s, others aligned with the start of the flood hydrograph or peak of flood hydrograph
- **Soil Erodibility**: Estimated soil erodibility (K_d) varied from 0.1 to 170 cm^3/NS and has a significant impact on the breach predictions
- **Density**: Estimates varied from 2000-2140 Kg/m^3
- **Cohesion**: Varied from 0 to 25 Kpa
- **Friction Angle**: Varied from 26-30 degrees
- **Porosity**: Varied from 0.23 to 0.62
- **Critical Shear Stress**: Varied from 0 to 20 Pa
- **Manning's n**: Varied from 0.016 to 0.04 (and 5*)
- **Timestep**: Varied from 0.08s to 360s

The hypothetical test case was typical of the situation facing many modellers, whereby detailed data is often limited, and only descriptive data exists for the soils. The result is that many modellers come to very different conclusions when estimating key modelling parameters.

**Note that a value of 5 was listed by ARS_Ali using WinDAM C. This is likely an error and perhaps refers to an imperial units value of 0.05. This extremely high value is not listed in any later tests.*

4.3 Review of the Modelling Results

For this test case (Hypothetical), there are no observed results to compare modelling results against, hence the results are considered in relation to each other.

4.3.1 Comparison of all modelling data

Figure B.3, Figure B.4 and Figure B.5 show a comparison of all modelling data for the Hypothetical test case. Results that stand out include:

- Flow:
  - A majority of simulations predict breach flow within the first 1000 s or so, whilst OvaBreach and Rupro show failure much later at ~17000-20000 s
  - Cluster of modelling results in 0-1000 s range show differing flood hydrograph characteristics:
    - ARS_Ali WinDAM C shows immediate breach and drawdown – too fast
    - ERAU WinDAM C appears triangular – insufficient resolution?
    - VUT WinDAM C shows near vertical drop off in flood hydrograph – odd!
- **Breach Width**: Predictions vary from 7 m to above 40 m. This is a very large range
- **Breach Depth**: Tend to match the allowable depth defined in model setup
- **U/S Water Level**:
  - Reflects the breach formation timing (i.e. drops as breach occurs and the reservoir drains)
  - VUT DLBreach simulation shows instability in WL calculation
- **D/S Water Level**: Tends to confirm the d/s boundary condition established by modellers

Conclusions:

- The large variation in modeller assumptions makes it difficult to pick out any model performance trends
- Different model breach hydrographs show some unexpected characteristics (eg instant breach, instant drawdown, instability in flow prediction etc). To be watched on later tests
- The timing of OvaBreach and Rupro stood out as clearly different from the rest (correctly or incorrectly – without real data to compare, it cannot be determined)
- DLBreach (both VUT and HRW) showed instability in u/s reservoir level prediction

4.3.2 Assessment of AREBA modelling data

Figure B.6 shows a comparison of results from modellers (VUT and ERAU) both using AREBA.

Both predict quick breach formation, but ERAU predicts a faster process with double the peak outflow.

Modeller Assumptions can explain the relative results since:

- ERAU assumed 0.1 m initiating pipe dimension compared to VUT 0.03 m
- ERAU assumed K_d of 53 compared to VUT K_d of 6 cm^3/NS

Conclusions:

- No clear trends in model performance can be identified, but it highlights the significant role that modeller judgement plays in selecting modelling parameters.

4.3.3 Assessment of EMBREA modelling data

Figure B.7 shows a comparison of results from modellers (VUT, HRW, two members of staff from USDA-ARS-HERU (ARS_Tejral and ARS_Ali) using EMBREA.

A range of breach speeds and hence peak discharges are predicted.

Modeller Assumptions can explain the relative results since:

- Assumed K_d values cover 0.14, 1.85, 6 and 100 cm^3/NS
- Other values also vary – but K_d is likely to have the greatest impact

Conclusions:

- No clear trends in model performance can be identified, but again, the modeller assumed values have a significant impact

4.3.4 Assessment of DLBreach modelling data

Figure B.8 shows a comparison of results from modellers (UniClrk, HRW, ERAU, Geosyntec, VUT) using DLBreach.

A range of breach speeds and hence peak discharges are predicted.

Modeller Assumptions can explain the relative results since:

- Assumed K_d values cover 0.14, 1.85, 6, 8 and 53 cm^3/NS
- Other values also vary – but K_d is likely to have the greatest impact

Conclusions:

- No clear trends in model performance can be identified
- DLBreach showed instability in flow and upstream water level from 2 modellers (but this may relate to the test case definition)

4.3.5 Assessment of WinDAM C modelling data

Figure B.9 shows a comparison of results from modellers (ERAU, Geosyntec, two members of staff from USDA (ARS_Tejral, ARS_Ali), EDF and VUT) using WinDAM C.

A range of breach speeds and hence peak discharges are predicted, although ARS_Tejral and Geosyntec show very slow erosion processes.

Modeller Assumptions can explain the relative results since:

- Assumed K_d values cover 0.14, 17.7, 53, 100 and 173 cm^3/NS
- Other values also vary – but K_d is likely to have the greatest impact

Conclusions:

- No clear trends in model performance can be identified

4.3.6 Assessment of OvaBreach & Rupro modelling data

Figure B.10 shows a comparison of results from modellers using OvaBreach (ARUP) and Rupro (INRAE). Results from these two models differed notably from the others but were similar in comparison to each other.

Modeller Assumptions do not explain similarities since:

- Very little information was provided for Rupro
- One model (Rupro) ignores the core whilst the other (OvaBreach) treats the core and fill separately

Conclusions:

- No clear trends in model performance can be identified
- Look for differences between Rupro and OvaBreach and other models during later tests

4.4 Conclusions

The main conclusions drawn from the Phase 0 Hypothetical test case were:

1. There is a clear need to define the test case more precisely so as to reduce the number of decisions and assumptions made by the modellers. This should lead to model setups which are closer in terms of modelling parameters allowing for more direct comparisons of model performance
2. Even when working with a group of experienced breach modellers, the range of assumptions regarding model setup and modelling parameters – in particular soil erodibility – is very wide, leading to significantly differing modelling predictions
3. OvaBreach and Rupro appeared to show significantly different timing to the other models
4. Breach hydrograph characteristics varied significantly from model to model, with some showing clearly artificial constructs (eg instant failure, instant drawdown, triangular profiles etc)

4.4.1 Next steps

Based upon these conclusions, we proceeded to the Phase 1 modelling programme adopting a more detailed definition of modelling parameters. To see whether we could improve the hypothetical modelling performance we also included a 'Modified Hypothetical' test case, with an even more simplified setup and clearer parameter definitions.

5 Phase 1: Modified Hypothetical Test Case Evaluation

The overall objective of the Phase 1 modelling was to undertake model performance assessments against a range of large scale test data. Four tests were considered, comprising:

i Modified Hypothetical test case
ii IMPACT project test case
iii ARS P1 test case
iv ARS P4 test case

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendices C, D, E & F.

5.1 Modelling Objectives

The objective of the Phase 1 – Modified Hypothetical case was to assess whether modelling predictions were clustered more closely given a simplified and more detailed specification for the modelling work.

The modified hypothetical test case differed from the previous hypothetical test case by:

- Simplified, homogeneous structure with flat foundation level
- Simplified soils description
- Simplified reservoir bathymetry (at lower level)
- Simplified inflow hydrograph (steady inflow)
- Defined initial pipe flow dimensions
- Assumed no downstream water level effects on breach process (i.e. no drowning)

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix C.

5.2 Modeller Assumptions

Table C.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table immediately highlights that, despite trying to define a simple test case, the modellers have assumed a range of different values and conditions for their simulations. For example:

- **Modelling Approach**: Still some varying assumptions in modelling approach (eg inclusion of grass cover or not)
- **Dam Foundation**: Despite simplifying the structure, some modellers used 412.00 mAD and other 414.96 mAD
- **Initiating Diameter**: A majority of modellers followed the guidance and used 0.05 m dimension for initiation
- **Density**: Estimates varied from 1740-2770 Kg/m^3
- **Cohesion**: Varied from 7 to 20 Kpa
- **Friction Angle**: Varied from 32 – 45.6 degrees
- **Porosity**: Varied from 0.24 to 0.65
- **Critical Shear Stress**: Varied from 0 to 20 Pa
- **Manning's n**: Varied from 0.0188 to 0.03
- **Timestep**: Varied from 0.2 s to 10 s

The range of values used is generally smaller than for the Phase 0 hypothetical test but nevertheless reflects the different assumptions modellers make despite efforts to restrict the choices.

5.3 Review of the Modelling Results

5.3.1 Comparison of all modelling data

Figure C.2 and Figure C.3 show a comparison of all modelling data for the Phase 1 Modified Hypothetical test case.

- Flow:
  - A majority of simulations predict breach flow within the first 2000s or so, whilst OvaBreach and Rupro show failure later at approx. 3000-4000 s
  - Cluster of modelling results in 0-2000 s range show differing flood hydrograph characteristics. Some models show an instant step to peak flow and progressive drawdown; others the reverse; others a more symmetric profile. These differences may reflect pipe failure assumptions (eg roof collapse relationships)
  - DLBreach simulations appear to show an instantaneous jump from small flow to peak flow. [This may be due to the assumption that after roof collapse, these materials are assumed to be instantly removed]
  - Whilst results appear to be 'clustered' the peak flow variation (all results) is ~225-750 m^3/s and peak flow timing 500 – 1300 s
- **Breach Width**: Predictions generally vary from 7 m to 17 m. A more focused range than for Phase 0
- **Breach Depth**: Varies from ~5 to 17 m. A significant difference over what was observed in Phase 0
- **U/S Water Level**:
  - Reflects the breach formation timing (i.e. drops as breach occurs and the reservoir drains)
- **D/S Water Level**: Tends to confirm the d/s boundary condition established by modellers. (No downstream conditions were defined within the test case, assuming no downstream influence)

Conclusions:

- Modelling results clustered slightly more than Phase 0 results. This shows the importance of providing clear data sets to modellers. Something that was ensured in the coming phases of the project
- Different model breach hydrographs still show some unexpected characteristics (eg instant breach, instant drawdown, instability in flow prediction etc). To be watched on later tests
- The timing of OvaBreach and Rupro still stood out as clearly different from the rest, albeit the difference is significantly less than Phase 0

5.3.2 Assessment of Developers modelling data

Figure C.4 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- The range of results covers a similar range to that of the whole modelling group

Conclusions:

- There does not appear to be a notable difference in modelling performance between developers and the whole group

5.3.3 Assessment of AREBA modelling data

Figure C.5 shows a comparison of results from modellers (VUT) using AREBA.

With only one modeller using AREBA for this case, no comparisons can be made.

Observations:

- The hydrograph drawdown characteristic looks strange, in that it drops almost instantly and then predicts a steady 300 m^3/s flow. The flow may arise because of a fixed reservoir water level as an upstream condition

Conclusions:

- None – model setup questioned

5.3.4 Assessment of EMBREA modelling data

Figure C.6 shows a comparison of results from modellers (VUT, HRW and ARS_Ali) using EMBREA.

Observations:

- The three results are clustered well with regards to timing
- Two are very similar for peak flow (approx. 400 m^3/s; a third is approx. 600 m^3/s) – differences in assumed foundation level and soil density are likely causes

Conclusions:

- Three sets of modelling results seem broadly consistent (subject to modeller variations in assumptions)

5.3.5 Assessment of DLBreach modelling data

Figure C.7 shows a comparison of results from modellers (UniClrk, VUT, HRW, two members of staff from USDA (ARS_Ali and ARS_Tejral)) using DLBreach.

Three observations can be noted from this comparison:

- 4 of 5 of the simulations show the same characteristic – an instant jump to peak flow and then drawdown of the reservoir, reflected in the reducing flow hydrograph. The instantaneous jump seems odd, but may be due to the assumption that after roof collapse, materials are assumed to be instantly removed
- Unclear why there is a relatively wide range of results from the same model – variations in modeller assumptions regarding porosity (eg 0.35 to 0.65), timestep and Manning n (eg 0.0188 to 0.03) may contribute here. The variations are significant in peak flow 250 – 600 m^3/s for example

Conclusions:

- For the majority of modelling results, the model prediction characteristics show an instant jump to peak flow conditions
- Range of results suggests modeller assumptions are critical

5.3.6 Assessment of WinDAM C modelling data

Figure C.8 shows a comparison of results from modellers (VUT, EDF, Geosyntec, two members of staff from USDA (ARS_Tejral and ARS_Ali)) using WinDAM C.

Observations:

- Geosyntec modelling results appear spurious and will be ignored for this test
- Other results seem reasonably well clustered both in timing and peak outflow predictions.
- Differences may be attributed to modeller assumptions

Conclusions:

- No clear trends in model performance can be identified

5.3.7 Assessment of Rupro modelling data

Figure C.9 shows a comparison of results from modellers using Rupro (EDF) and Rupro (INRAE).

Observations:

- Hydrograph characteristics are very similar, but timing of the breach differs

Conclusions:

- Only difference between modeller assumptions appears to be timestep

5.4 Conclusions

The main conclusions drawn from the Phase 1 Modified Hypothetical test case were:

1. Despite efforts to define the modified test case more clearly than the original Phase 0 test case, there are still variations in model setup arising from (i) different modeller assumptions and (ii) incorrect model setup
2. Modeller assumptions – both in model setup and estimating parameters used by the models – have a significant impact on modelling accuracy
3. Since there are no 'observed' results for this test case against which to compare the modelling results, no observations can be made regarding overall model performance. Some trends in modelling outputs can be seen regarding the characteristic shape of predicted hydrographs and the timing of breach initiation such as:
    a. DLBreach modelling tends to show instant collapse and then drawdown of the reservoir (vertical leading face to hydrograph) with other models tending to show a more progressive rate of erosion (slower development of the hydrograph surge, which seems more realistic)
    b. The predicted rate of erosion affects the timing, magnitude of peak flow and duration of the flood hydrograph, with slower erosion prediction leading (logically) to lower peak discharge and a longer flood hydrograph
    c. Rupro and OvaBreach consistently predict a much slower breach initiation compared to the other models
    d. Breach width and depth predictions vary significantly between models

6 Phase 1 – IMPACT Test Case Evaluation

The objective of the Phase 1 – IMPACT modelling was to assess how models performed against a large scale test case. This test was performed in Norway as part of the European funded IMPACT project in October 2003. The levee was 4.3 m high and constructed from moraine material. The pipe flow was triggered by allowing flow to run through a perforated PVC pipe built into the base of the levee, which was surrounded by sand. The pipe flow removed the sand leading to larger pipe initiation and subsequently levee failure.

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix D.

6.1 Modeller Assumptions

Table D.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table also includes comments from modellers as to any assumptions or simplifications made in creating their models, as well as any changes made between 'blind' and 'aware' modelling. [Blind modelling is where modellers use the test case data, without access to observed results; Aware is where modellers adapt their models to improve the prediction based upon access to the test case results].

Key observations from the table include:

- Some model specific assumptions in model setup that create differences in approach
- Significant differences in estimated K_d values, ranging from 4 to 90 cm^3/NS
- Some variations in soil density, reflecting different forms of the parameter between models, but also some modeller inconsistencies
- Small variations in critical shear stress, Mannings' n and timestep assumptions
- Most modellers did not do aware as well as blind runs (volume of work limitations). Where done, variation to K_d (increasing it) was made. Also consistent with HRW analysis to increase K_d value for this test case

6.2 Review of the Modelling Results

6.2.1 Comparison of all modelling data

Figure D.2 shows a comparison of all modelling data for the Phase 1 IMPACT test case. This test case relied upon water control some kilometres upstream from the test site, hence water levels at the test site dropped and rose as efforts were made to maintain test conditions.

Modelling observations:

- Flow:
  - Most modelling results for flow clustered around the observed data, but this was typically driven by the timing of pipe flow initiation and timing of inflow from upstream
  - Since we have inflow for this test case, differences between observed and model prediction are magnified (compared, say, to a simple draining reservoir situation)
- **Breach Width**: Predictions were scattered either side of the observed, ranging from final widths of approx. 3 m to 23 m (observed final being approx. 14 m). Rates of breach width growth varied above and below observed
- **Breach Depth**: Many modellers/models incorrectly predict the time of roof collapse (hence max breach depth timing) – often too early
- **U/S Water Level**: Many models predict breach formation too early, as shown by surges in flow around 17300s instead of 18750s, predictions of breach depth developing at similar times and

lowering of the upstream water level earlier than observed. Whilst some predictions are related to an error in predicted initial water level, others are not

Conclusions:

- Performance related to flow hydrograph may be misleading due to the imposed boundary conditions
- Many models predict varying rates of breach width growth which are similar to observed, but often the timing is inaccurate
- Many models seem to predict too rapid roof collapse to open breach

6.2.2 Assessment of Developers modelling data

Figure D.3 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- The range of results covers a similar range to that of the whole modelling group

Conclusions:

- There does not appear to be a notable difference in modelling performance between developers and the whole group

6.2.3 Assessment of AREBA & OvaBreach modelling data

Figure D.4 shows a comparison of results from modellers using AREBA (VUT) and OvaBreach (ARUP).

With only one modeller using each for this case, comparisons are limited.

Observations:

- Both recreate the flood hydrograph but underestimate breach width and miss significant variations in u/s water level

Conclusions:

- Performance seems poor

6.2.4 Assessment of EMBREA modelling data

Figure D.5 shows a comparison of results from modellers (VUT, HRW and ARS_Ali) using EMBREA.

Observations:

- The predicted flow hydrographs are centred around the observed; HRW prediction is good, whereas the other two are less so
- For breach width, one over predicts, one under and one (HRW) is close (on average)
- Two of the results recreate the drop in upstream water level that was a feature of the test case

Conclusions:

- The three modellers used significantly different K_d values (90, 20, 4.5); K_d of ~20 gave the best results (HRW)

6.2.5 Assessment of DLBreach modelling data

Figure D.6 shows a comparison of results from modellers (UniClrk, VUT, HRW, two members of staff from USDA (ARS_Ali and ARS_Tejral), and ERAU) using DLBreach.

Observations:

- Most (but not all) of the results tended to be on the fast side compared to observed. As with other models/modellers, this may relate to choice of K_d value
- Timing of characteristics (rate of breach width; changing u/s water level) also perhaps reflects choice of K_d

Conclusions:

- Model predicts breach process characteristics, but timing of processes seems influenced by K_d choices which varied significantly across modellers

6.2.6 Assessment of WinDAM C modelling data

Figure D.7 shows a comparison of results from modellers (VUT, EDF, Geosyntec, two members of staff from USDA (ARS_Ali, ARS_Tejral)) using WinDAM C.

Observations:

- The scatter of modeller results perhaps seems a little wider than for the other models
- Average rate of breach width growth appears broadly correct, but rate of initiation and final widths vary significantly about the observed data
- Some modellers recreate the u/s water level variations, whilst others do not

Conclusions:

- Slightly wider scatter of results than with some of the other models (arising from modeller use and choice of parameters)

6.2.7 Assessment of Rupro modelling data

Figure D.8 shows a comparison of results from modellers using Rupro (INRAE and EDF).

Observations:

- INRAE compared three different versions of the Rupro model. Rupro 1 performed better than Rupro 2 and much better than Rupro 3
- EDF Rupro flow results were close to INRAE Rupro 1 whilst breach growth predictions were different

Conclusions:

- EDF Rupro and INRAE Rupro 1 & 2 modelling results seemed comparable to other breach modelling results. Some characteristics are over or under predicted though

6.3 Aware modelling

Aware modelling was undertaken by UniClrk using DLBreach and INRAE using Rupro#3; plots showing the influence of parameter variation on predicted outflow can be seen in Appendix D, Section D.4.

The variations made by UniClrk using DLBreach did not appear to make significant differences to the results, whilst the variations by INRAE using Rupro#3 significantly improved the Rupro#3 prediction.

6.4 Conclusions

The main conclusions drawn from the Phase 1 IMPACT test case were:

1. The nature of the test data (defined inflow) means that models should get a reasonable approximation to the flood hydrograph simply by a flow volume balance
2. Many of the models predicted breach too early
3. The variation in final breach width prediction was significant, although many models predicted broadly the correct rate of erosion (i.e. just for too long or too short a period)
4. The choice of K_d value has a significant influence on the accuracy of model prediction, also demonstrated by the 'Aware' modelling results

7 Phase 1 – ARS P1 Test Case Evaluation

The objective of the Phase 1 – ARS P1 modelling was to assess how models performed against a carefully controlled field test case.

This test was performed at the USDA ARS site in Stillwater, Oklahoma. The levee consisted of a homogeneous earth embankment 1.2 m high, 9.75 m long, with a crest width of 1.98 m and slopes of approximately 1 in 3. A pipe of 0.04 m diameter was created through the levee 0.28 m from the base, by removing a rigid pipe of that diameter, which had been constructed into the levee.

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix E.

7.1 Modeller Assumptions

Table E.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table also includes comments from modellers as to any assumptions or simplifications made in creating their models, as well as any changes made between 'blind' and 'aware' modelling.

Key observations from the table include:

- Some model specific assumptions in model setup that create differences in approach
- Significant differences in estimated K_d values, ranging from 50 to 210 cm^3/NS
- Some variations in soil density, reflecting different forms of the parameter between models, but also some modeller inconsistencies. (Ranging 1740-2770 kg/m^3)
- Significant variations in critical shear stress (0.14-6.89 Pa), Manning's n (0.009-0.03) and timestep assumptions (0.01-60 s)
- Where modellers did aware as well as blind runs, most varied K_d (reducing it) and critical shear stress

7.2 Review of the Modelling Results

7.2.1 Comparison of all modelling data

Figure E.2 shows a comparison of all modelling data for the ARS P1 test case.

Modelling observations:

- Flow:
  - The main area of interest is the first ½ hr (1800 s) which is where the initiation of erosion affects the overall timing of the breach formation (and outflow). Not many models matched this timing except EMBREA runs by HRW and USDA. Most showed erosion initiation too soon; some (DLBreach and Rupro) too late
  - The convergence of all models to 2.5 m^3/s simply reflects the model simulating steady inflow/outflow after the breach has formed
- **Breach Width**: Predictions were scattered either side of the observed, ranging from final widths of >3.5 m to <10 m (observed final being approx. 6.5 m). Rates of breach width growth varied above and below observed
- **Breach Depth**: Many modellers/models incorrectly predict the time of roof collapse (hence max breach depth timing) – often too early
- **U/S Water Level**: Some models overpredicted the water levels due to underestimating erosion rates, whilst many models predict breach formation too early leading to

underprediction of the upstream water level and overprediction of breach depth and width rates

- **D/S Water Level**: Prediction of the d/s water level varies significantly across the models. Accurate representation is important since the observed conditions have the potential to drown conditions within the breach opening

Conclusions:

- The rates of erosion predicted by models varied above and below observed
- There was a tendency for modellers to use lower than measured K_d values to improve results
- Many models predict varying rates of breach width growth both above and below observed
- Many models seem to predict too rapid roof collapse to open breach

7.2.2 Assessment of Developers modelling data

Figure E.3 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- The range of results covers a similar range to that of the whole modelling group

Conclusions:

- There does not appear to be a notable difference in modelling performance between developers and the whole group

7.2.3 Assessment of AREBA and OvaBreach modelling data

Figure E.4 shows a comparison of results from modellers using AREBA (VUT) and OvaBreach (ARUP). With only one modeller using each for this case, comparisons are limited.

Observations:

- The AREBA simulation shows immediate roof failure but then open breach growth at a rate similar to observed (i.e. wrong roof collapse timing but correct widening rates)
- The OvaBreach simulation is slow to predict roof collapse and open breach formation; rate of widening is slower than observed

Conclusions:

- Performance seems poor for timing of roof collapse

7.2.4 Assessment of EMBREA modelling data

Figure E.5 shows a comparison of results from modellers (VUT, HRW and USDA (ARS_Ali)) using EMBREA.

Observations:

- Two of the predicted flow hydrographs are close to the observed (USDA and HRW); whereas the other (VUT) shows failure too early. The two close results are the best overall of all modelling
- For breach width, both VUT and USDA over predict the final breach width (approx. 10 m), whilst the HRW result is close (approx. 6 m versus approx. 6.5 m observed). All predicted rates of width erosion that are faster than observed
- All of the results predict a drop in upstream water level too early
- The VUT simulation becomes unstable after breach has occurred - hence oscillations in flow prediction. (Something to do with d/s boundary setup)

Conclusions:

- The three modellers used the same K_d values but differed in choice of density and Manning's n values

7.2.5 Assessment of DLBreach modelling data

Figure E.6 shows a comparison of results from modellers (UniClrk, VUT, HRW, two members of staff from USDA (ARS_Ali, ARS_Tejral) and Geosyntech) using DLBreach.

Observations:

- All modellers, except for UniClrk, predicted a very fast failure compared to observed; UniClrk predicted a slower failure than observed. This probably reflects the choice of K_d=10.3 cm^3/N.s by UniClrk compared to 120 cm^3/N.s by others. UniClrk later revised K_d to 60 for the aware run
- Manning's n also varied 0.016-0.03 across the modellers
- Rates of breach width growth vary above and below observed
- U/S and D/S water level by VUT appears wrong, suggesting wrong model setup

Conclusions:

- Model predicts breach process characteristics, but timing and rate of processes seem influenced by K_d and Manning's n choices which varied significantly across modellers
- The need to use a different K_d value to that measured for the test suggests that model inherently over/under predicts some processes

7.2.6 Assessment of WinDAM C modelling data

Figure E.7 shows a comparison of results from modellers (VUT, EDF, Geosyntec, and two members of staff from USDA (ARS_Ali, ARS_Tejral)) using WinDAM C.

Observations:

- The flow modelling results are all very similar – but all predict failure to occur before the observed event and underpredict the peak outflow
- All overpredict the breach width, estimating approx. 10 m final width instead of approx. 6.5 m which was observed
- ARS_Tejral from USDA predicts a breach widening rate similar to observed – unlike the others which over predict the rate – yet ARS_Tejral used K_d=210 cm^3/N.s whilst others use lower values (eg K_d=120 cm^3/N.s). However, there is a question for ARS_Tejral on the choice of timestep and bulk density that remains unresolved

Conclusions:

- Predicted flow characteristics look good, but timing is poor
- Significant variations in breach width despite apparently close simulations in breach flow

7.2.7 Assessment of Rupro modelling data

Figure E.8 shows a comparison of results from modellers using Rupro (INRAE and EDF).

Observations:

- INRAE compared three different versions of the Rupro model
- All simulations failed to create the observed flow characteristics
- Rupro 3 most closely matched the observed breach widening rate, with the other simulations underpredicting the rate
- Only parameter differences appeared to be choice of timesteps

Conclusions:

- Poor representation of roof collapse and flow surge (i.e. poor recreation of flow characteristics)

7.3 Aware modelling

Aware modelling was undertaken by USDA ARS using WinDAM C and DLBreach, BUT using TUD AREBA, EMBREA, WinDAM C and DLBreach and UniClrk using DLBreach. The influence of parameter variation on predicted outflow can be seen in Appendix E, Section E.4.

For USDA-ARS, changing the K_d value in the DLBreach model changed the shape of the hydrograph but without a significant improvement in overall prediction accuracy. Results for the WinDAM C model appeared unchanged.

BUT undertook runs using AREBA, EMBREA, WinDAM C and DLBreach using different erodibility values, based upon the respective model guidance. AREBA results were significantly improved; EMBREA slightly improved, WinDAM C slightly improved and DLBreach significantly improved. However, the significance of the different magnitudes of K_d value change with respect to each model has not been investigated.

UniClrk undertook aware modelling using DLBreach which improved upon the blind modelling. However the blind modelling already did not use the defined test parameters, so cannot be compared directly against the other modelling results. It was also noted that changes to other modelling parameters were made for the DLBreach simulations (eg pipe inlet losses).

7.4 Conclusions

The main conclusions drawn from the ARS P1 test case were:

1. The area of interest in the modelling results is the first 30 mins, where the models predict initiation, breach growth, roof failure and open breach. Many models over predicted the time to roof collapse; a few under predicted; only a couple of results (from EMBREA) came close to predicting both the flow characteristics and timing
2. Even where flow characteristics were reproduced, breach width growth rate and final value were often over or under predicted
3. Some model results suggested some errors in setup
4. Variation in choice of K_d, density, Manning's n and timestep could lead to significantly different modelling results – as shown by the 'Aware' modelling results
5. Rupro failed to recreate the outflow characteristics even though breach growth rate was close to observed for Rupro 2

8 Phase 1 – ARS P4 Test Case Evaluation

The objective of the Phase 1 – ARS P4 modelling was to assess how models performed against a carefully controlled field test case. Unlike the P1 test, the P4 test material was far less erodible and despite running for many hours, did not result in an open breach. Some backward erosion did occur, but not sufficiently to change the initial pipe dimensions through the upstream levee face.

Hence this test case offers a specific challenge to the models to predict a non-failure case, rather than erosion leading to an open breach.

This test was performed at the USDA ARS site in Stillwater, Oklahoma. The levee consisted of a homogeneous earth embankment 1.24 m high, 9.75 m long, with a crest width of 1.98 m and slopes of approximately 1 in 3. A pipe of 0.04 m diameter was created through the levee 0.23 m from the base, by removing a rigid pipe of that diameter, which had been constructed into the levee.

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix F.

8.1 Modeller Assumptions

Table F.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table also includes comments from modellers as to any assumptions or simplifications made in creating their models, as well as any changes made between 'blind' and 'aware' modelling.

Key observations from the table include:

- Use of K_d=0.1 cm^3/N.s was uniform across all modellers; use of critical shear stress of 35 Pa was also uniform across all modellers <u>except</u> for UniClrk (DLBreach) who used lower values
- There was some variation in use of density, Manning's n and timestep values
- All models initiated erosion with a 0.04 m square or diameter hole

8.2 Review of the Modelling Results

8.2.1 Comparison of all modelling data

Figure F.2 shows a comparison of all modelling data for the ARS P4 test case.

Modelling observations:

- Flow:
  - The observed data shows a very low flow with a gradual increase in discharge
  - Most models show no erosion (hence just a very small constant flow through the pipe)
  - OvaBreach and Rupro both show an instant increase in flow and then a near steady flow at ~0.6 m^3/s or breach progression (EDF Rupro). These predict an instant breach roof failure
  - EDF WinDAM C shows some (but too much) erosion and came closer than ARS WinDAM C, results – it appears by using a smaller timestep and higher Manning's n value (both erodibility K_d and critical shear stress Tc values being the same)
  - UniClrk DLBreach came closest to observed <u>but</u> did not follow use of defined parameters – instead using a lower value of critical shear stress (5 Pa instead of 35 Pa)
- **Breach Width/Pipe Dia**: Most models did not predict pipe erosion. Those which did, showed a greater rate of erosion than observed (as reflected by the flow plots)
- **U/S Water Level**: Most models predicted a steady upstream water level – except those predicting a breach where the level dropped

Conclusions:

- Most models using the defined test parameters did not show any erosion progression. Those that did show erosion progression were:
  - Rupro and OvaBreach – both predicted instant roof failure and breach
  - EDF WinDAM C – over predicted erosion but appeared to do so through varying timestep and Manning's n values
  - UniClrk DLBreach predicted progressive erosion, but only by using critical shear stress values different to those prescribed for the test
- Hence models struggled to predict the observed conditions using the measured parameters – but with adjustments to parameters may be able to recreate observed

8.2.2 Assessment of Developers modelling data

Figure F.3 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- The range of results covers a similar range to that of the whole modelling group.

Conclusions:

- There does not appear to be a notable difference in modelling performance between developers and the whole group

8.2.3 Assessment of AREBA and OvaBreach modelling data

Figure F.4 shows a comparison of results from modellers using AREBA (VUT) and OvaBreach (ARUP).

With only one modeller using each for this case, comparisons are limited.

Observations:

- The AREBA simulation shows no erosion
- The OvaBreach simulation shows instant roof failure

Conclusions:

- Neither model recreates the observed conditions with the defined parameters

8.2.4 Assessment of EMBREA modelling data

Figure F.5 shows a comparison of results from modellers (VUT, HRW and USDA (ARS_Ali)) using EMBREA.

Observations:

- None of the simulations showed erosion

Conclusions:

- The model did not recreate the observed conditions with the defined parameters (but can probably recreate observed conditions by varying input parameters away from defined values)

8.2.5 Assessment of DLBreach modelling data

Figure F.6 shows a comparison of results from modellers (UniClrk, VUT, HRW, two members of staff from USDA (ARS_Ali, ARS_Tejral), and Geosyntech) using DLBreach.

Observations:

- All modellers, except for UniClrk, predicted no erosion
- UniClrk predicted some erosion by using a critical shear stress value different from that defined for the test

Conclusions:

- The model did not recreate the observed conditions with the defined parameters (but can probably recreate observed conditions by varying parameters such as critical shear stress and/or internal model inlet loss parameters)

8.2.6 Assessment of WinDAM C modelling data

Figure F.7 shows a comparison of results from modellers (VUT, EDF, Geosyntec and two members of staff from USDA, (ARS_Ali, ARS_Tejral)) using WinDAM C.

Observations:

- Most modellers do not predict any erosion. However, EDF did produce erosion by varying Mannings n and timestep

Conclusions:

- The model did not recreate the observed conditions with the defined parameters (but can probably recreate observed conditions by varying parameters away from defined values)
- Changes in Mannings n and timestep appear to affect the results

8.2.7 Assessment of Rupro modelling data

Figure F.8 shows a comparison of results from modellers using Rupro (INRAE and EDF).

Observations:

- INRAE compared three different versions of the Rupro model
- All simulations failed to create the observed flow characteristics; the model predicted instant failure

Conclusions:

- Poor representation of roof collapse and flow surge (i.e. poor recreation of flow characteristics)

8.3 Aware modelling

Aware modelling was undertaken by UniClrk using DLBreach, BUT using TUD AREBA and DLBreach, HRW using DLBreach and EMBREA and INRAE using Rupro. The influence of parameter variation on predicted outflow can be seen in Appendix F, Section F.4.

For UniClrk, it should be first noted that some of the parameters used for the blind test did not follow the defined values, which undermines the comparison of results against the other models results.

For the aware tests, the parameter changes improved the flow prediction but made the breach width prediction worse. Some changes also included parameters specific to the model, hence not necessarily changeable for the other models.

BUT, using modified parameters in AREBA, managed to improve both flow and breach width prediction simultaneously. A similar trend was also observed using DLBreach.

By modifying the critical shear stress HRW managed in improve the DLBreach modelling results and, in a similar way, the EMBREA modelling results.

INRAE managed to improve the performance of Rupro, but results were still significantly away from the observed data.

8.4 Conclusions

The main conclusions drawn from the ARS P4 test case were:

1. Most models – using the defined parameters – fail to predict any erosion. Rupro goes in the opposite direction and predicts instant failure
2. Variation in Manning's n and timestep can affect the modelling results
3. Models are likely able to predict observed flow characteristics by varying parameters away from the measured data – for example, critical shear stress, inlet losses etc. This is highlighted by the aware modelling results

9 Phase 2 – Lawn Lake Test Case Evaluation

The overall objective of the Phase 2 modelling was to undertake model performance assessments against real dam failure case studies. Two case studies were considered, comprising:

- i Lawn Lake dam failure
- ii Big Bay dam failure

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendices G & H respectively.

9.1 Modelling Objectives

The objective of the Phase 2 – Lawn Lake modelling was to assess how models performed against a real dam failure case. Unlike the Phase 1 modelling data, information relating to the dam, dam failure process and the associated erosion and release of water are far less certain. Case study data has been sought from various sources, as referenced in the data files.

The Lawn Lake Dam failure occurred on 15 July 1982 during the early morning. The dam comprised a 7.9 m high earth structure, located in the Rocky Mountain National Park. The breach, initiated through internal erosion, released an estimated 0.83 Mm^3 of water with a peak flow approx. 500 m^3/s.

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix G.

9.2 Validity of observed data

Unlike the Phase 1 test data, which were measured in field or laboratory tests, the Lawn Lake Case is a real dam failure event for which data has been collected from the best available sources. As such, flood conditions are back calculated, soil properties are descriptive, and breach dimensions measured after the event. This means that the test case data is likely to have considerably greater bands of uncertainty around individual parameters than the field and laboratory test data.

Data details and references can be found through the test case spreadsheets (referenced in the Appendices).

The peak outflow (Q_p) was back estimated using DAMBRK analyses. This assumed trapezoidal overflow breach flow and field observations matching to DAMBRK flood routing. With severe debris flow, scour and deposition in the valley we should allow a considerable range of uncertainty about this value (perhaps 300-700 m^3/s).

Breach dimensions were observed after the event and are probably more reliable than the Q_p estimate. Consider that during the breach process, sides were vertical or undercut, meaning that the actual 'flow controlling' dimension is likely to be closer to the base than top width (i.e. probably between 17 m (base) and 23 m (average).

Some models can predict sub foundation erosion. The height of the dam was approx. 7.2 m. Costa & Jarrett (1986) suggests the outlet pipe was located on bedrock (3344.35 m) giving a max potential scour from the crest as approx. 8 m (not 7. 2 m or 9.3 m as used by many modellers).

Whilst these uncertainties make the modelling performance assessment harder, they do reflect the typical information available when undertaking analysis of real dams or failure cases.

9.3 Modeller Assumptions

Table G.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table also includes comments from modellers as to any assumptions or simplifications made in creating their models, as well as any changes made between 'blind' and 'aware' modelling.

Key observations from the table include:

- Estimation of K_d varied from 10 to 50 between modellers; estimation of critical shear stress was between 0.04 and 6 Pa
- There were some significant variations in the estimation of density, Manning's n and timestep values

Hence, differences in modelling results reflect a combination of model and modeller effects.

9.4 Review of the Modelling Results

9.4.1 Comparison of all modelling data

Figure G.2 shows a comparison of all modelling data for the Lawn Lake test case.

Modelling observations:

- Flow:
  - Results show a wide range of predictions – probably reflecting wide range in choice of K_d
- Breach Width:
  - Results also show a wide range of predictions both above and below the observed final widths
  - Impossible to assess rate of breach growth from case study data – just final breach width. However all models seem to predict very rapid growth of breach
- Breach depth:
  - Final depths vary between 7.2 and 9.3 depending on modeller assumptions
- U/S Water Level:
  - Geosyntec and USDA EMBREA model setup incorrect (fixed u/s level)

Conclusions:

- With modellers using judgment on the selection of modelling parameters, a wide range in many values has been used
- This results in wide variations in model predictions – whether for flow, breach width, water levels etc
- All modellers predict a very rapid erosion rate for the breach formation

9.4.2 Assessment of Developers modelling data

Figure G.3 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- The range of results seems more tightly clustered than that of the whole modelling group
- Tendency to predict Qp lower than 'observed'
- Tighter range of K_d chosen (15-50) but still varied

Conclusions:

- Perhaps reflects better judgement on use of models where data is highly uncertain

9.4.3 Assessment of AREBA modelling data

Figure G.4 shows a comparison of results from modellers using AREBA (VUT).

With only one modeller using AREBA for this case, comparisons are limited.

Observations:

- The predicted Q_p is within bounds of uncertainty, but breach width is over predicted

Conclusions:

- AREBA appears to be performing within the bounds of data uncertainty

9.4.4 Assessment of EMBREA modelling data

Figure G.5 shows a comparison of results from modellers (VUT, HRW and USDA (ARS_Ali)) using EMBREA.

Observations:

- All simulations underpredict Qp
- Breach width predictions spread across observed
- Breach depth predictions vary according to modeller setup

Conclusions:

- Results are varied – but closer check on modelling parameters (K_d, density and dam height) show that each modeller used a different combination of parameters (leading to differing results):

	HRW	USDA ARS	BUT
K_d (cm³/N.s)	15	50	30
Q_p (m³/s)	250	290	350
Breach Width (m)	23	14	43
Breach Depth (m)	7	9.3	6.85
Soil Density (Kg/m³)	1416 (dry)	2650	2050

9.4.5 Assessment of DLBreach modelling data

Figure G.6 shows a comparison of results from modellers (UniClrk, VUT, HRW, USDA (ARS_Ali) and Geosyntech) using DLBreach.

Observations:

- 3 of 4 flow predictions are close to observed
- 2 of 4 breach width predictions are in range
- Different maximum breach depths have been assumed

Conclusions:

- Initial flow results look good, but breach width and depth results vary
- Closer look at modeller data shows that (i) UniClrk and USDA ARS used exactly the same data; (ii) BUT achieved close Q_p with lower K_d, but by using increased critical shear and Manning's n values. (See modellers data table for more details)

	HRW	USDA ARS	BUT	UniClrk
K_d (cm³/N.s)	15	50	17.7	50

	HRW	USDA ARS	BUT	UniClrk
Q_p (m^3/s)	342	509	520	509
Breach Width (m)	33	30	58	30
Breach Depth (m)	7.2	9.3	?	9.3
Soil Density (Kg/m^3)*	1416	2650	2650	2650

**Care is needed when defining the soil density since the format of information used by the various models differs. DLBreach requires the user to specify gravity (eg 2.65) and porosity (eg 0.35, 0.4 etc) rather than soil density. It is listed here as a general parameter used by many models.*

9.4.6 Assessment of WinDAM C modelling data

Figure G.7 shows a comparison of results from modellers (VUT, EDF, Geosyntec and USDA (ARS_Ali)), using WinDAM C.

Observations:

- All modelling results for Q_p appear low; Geosyntec model shows no breach
- Again, mixed modeller choice of max breach depth. Modeller choice of K_d varies significantly
- Predicted breach widths are closer than many of the other models/modellers

Conclusions:

- Varying model predictions, which appear on the low side for Q_p but acceptable for breach width – but again with significantly varying modeller parameter assumptions:

	BUT	EDF	USDA ARS	Geosyn
K_d ($cm^3/N.s$)	17.7	10	50	?Low?
Q_p (m^3/s)	264	182	278	Minimal
Breach Width (m)	20	14	30	?
Breach Depth (m)	7.2	7.2	9.3	?
Soil Density (Kg/m^3)	2050	1692.6	2650	?

9.4.7 Assessment of Rupro modelling data

Figure G.8 shows a comparison of results from modellers using Rupro (INRAE and EDF).

Observations:

- Results similar in trend to WinDAM C – Q_p on the low side; Breach width in the right area

Conclusions:

- Limited modelling parameter details available to comment:

	Rupro#1	Rupro#2	EDF
K_d ($cm^3/N.s$)	Equiv?	Equiv?	Equiv?
Q_p (m^3/s)	307	307	233
Breach Width (m)	25	25	18
Breach Depth (m)	7.2	7.2	
Soil Density (Kg/m^3)			2650

9.5 Aware modelling

Aware modelling for the Lawn Lake Dam failure case was undertaken by HRW using EMBREA and BUT using WinDAM C, EMBREA and AREBA. The influence of parameter variation on predicted outflow can be seen in Appendix G, Section G.4.

For HRW with EMBREA, modification of the modelling parameters allowed for the model to match exactly the observed peak outflow. This was achieved through modifying the soil erodibility parameter, and also using a multiple of that parameter for the overflow erosion as compared to the pipe formation erosion. Since there is not yet a confirmed relationship between the use of K_d for internal erosion as compared to K_d for overflow erosion, this seemed an interesting approach to investigate, for which the results are quite positive (for this example).

For BUT, with all three models tested, parameter variation allowed for a significant improvement in case prediction with estimated peak outflows very close to the back calculated observed data. For these simulations BUT changed the soil erodibility differently with each model (but also based upon the original blind estimation of erodibility, which also varied per model).

It is clear from this modelling that variation key parameters such as soil erodibility allow the models to predict the observed conditions much more closely. The approach taken by HRW to consider different erodibility values for internal erosion development as compared to overflow erosion development gave positive results and should be investigated further.

9.6 Conclusions

The main conclusions drawn from the Lawn Lake case study were:

1. Significant variation in modeller parameter and breach assumptions led to wide range of modelling predictions
2. Developer predictions slightly more tightly clustered – perhaps suggesting better parameter estimation
3. Similar results for some metrics are achieved by varying different modelling parameters (K_d, critical shear stress, Manning's n, density etc)
4. There is perhaps a tendency of models to either predict Q_p and underpredict breach width, or to predict breach width and over predict Q_p
5. Uncertainty in the case study data makes it more difficult to determine the accuracy of modelling – particularly with greater variation introduced by the modeller parameter assumptions. However, the aware modelling results by HRW and BUT demonstrated that varying erodibility values within reasonable ranges could result in much better modelling results
6. The HRW aware modelling using different soil erodibility for internal erosion processes as compared to overflow erosion processes should be studied further

10 Phase 2 – Big Bay Test Case Evaluation

The objective of the Phase 2 – Big Bay modelling was to assess how models performed against a real dam failure case. Unlike the Phase 1 modelling data, information relating to the dam, dam failure process and the associated erosion and release of water are far less certain. Case study data has been sought from various sources, as referenced in the data files.

The Big Bay Dam failure occurred on Friday March 12th, 2004. An increased discharge from an existing seep was first noticed by a maintenance man on Thursday 11th March. The seepage gradually increased, with the flow carrying material by the next morning. At mid-morning on March 12th the seepage was inspected and was noted that it had about a 0.01 m head height. By 12:15 water "shot up out of the hole." Shortly after this the seepage was observed to be "spouting approximately 60-90 cm in height, with a diameter of about 45 cm." The area around the boil then collapsed and the embankment began to rapidly erode. This was the point where the breach was assumed to start, at about 12:20. The final breach dimensions occurred from about 13:10 when "breach widens to approx. 60 m" to 13:15 when the flood flow downstream of the embankment reached its maximum extent. Full breach formation was assumed to occur at 13:15. The breach formation time was estimated to be 55 min. (Summarised from T.R. Burge, 2004.)

Summary data tables and plots of modelling results (as referred to in the following sections) can be found in Appendix H.

10.1 Validity of observed data

Unlike the Phase 1 test data, which were measured in field or laboratory tests, the Big Bay Case is a real dam failure event for which data has been collected from the best available sources. As such, flood conditions are back calculated, soil properties are descriptive, and breach dimensions measured after the event. This means that the test case data is likely to have considerably greater bands of uncertainty around individual parameters than the field and laboratory test data.

Data details and references can be found through the test case spreadsheets (referenced in the Appendices).

The peak outflow (Q_p) was back estimated using HEC-RAS analyses (Yochum et al., 2008). A peak discharge of approx. 4200 m^3/s was estimated.

Breach dimensions were observed and noted during the event (Burge, 2004) and are likely to be more reliable than the Q_p estimate.

Whilst these uncertainties make the modelling performance assessment harder, they do reflect the typical information available when undertaking analysis of real dams or failure cases.

10.2 Modeller Assumptions

Table H.1 shows a summary of modeller assumptions and parameters used by the different modellers using the range of breach models. This table also includes comments from modellers as to any assumptions or simplifications made in creating their models, as well as any changes made between 'blind' and 'aware' modelling.

Key observations from the table include:

- Estimation of K_d varied from 10 to 84 cm^3/N.s between modellers; estimation of critical shear stress was between 0.15 and 3 Pa
- There were some significant variations in the estimation of density (1866-2650 kg/m^3), Manning's n (0.016-0.07) and timestep values (0.01-36 s)

Hence, differences in modelling results reflect a combination of model and modeller effects.

Note that for the test setup data, two curves for the reservoir stage volume relationship were provided due to conflicting information in published papers. Both curves start and finish with the same water levels and storage volume but deviate slightly in between. Where referenced, these relate to Option 1 and Option 2.

10.3 Review of the Modelling Results

10.3.1 Comparison of all modelling data

Figure H.2 shows a comparison of all modelling data for the Big Bay test case.

Modelling observations:

- Flow:
  - Predictions ranged from low values to 5250 m^3/s compared to an estimation of 4200 m^3/s using HEC-RAS.
- Breach Width:
  - Predictions from very high to very low; majority are lower than 96 m though
- Breach depth:
  - Clearly two depths simulated by modellers
  - Rupro and WinDAM C predict slower growth than the other models
- U/S Water Level:
  - Geosyntec model setup either incorrect or very slow erosion

Conclusions:

- With modellers using judgment on the selection of modelling parameters, a wide range in many values has been used. This results in wide variations in model predictions – whether for flow, breach width, water levels etc
- Models predict varying rates of erosion rate for the breach formation – some very fast; some very slow

10.3.2 Assessment of Developers modelling data

Figure H.3 shows a comparison of results from just the modellers who are developers. These persons are likely to have a more detailed understanding of their model and perhaps other models, which may lead to more accurate modelling results.

Observations:

- Results still seem widely ranged
- Most predicted Q_p are close to or less than estimated observed. DLBreach, EMBREA and AREBA are the closest
- Breach width is either close to or less than observed. DLBreach, EMBREA and AREBA are also the closest

Conclusions:

- DLBREACH, EMBREA and AREBA predictions seem the closest to observed or estimated values:

	K_d ($cm^3/N.s$)	Q_p (m^3/s)	Width (m)	Depth (m)	Density (Kg/m^3)
BUT – AREBA Em	27	4100	109	17.4	2020
BUT – AREBA An	27	1750	45	17.4	2020
UniClrk - DLBreach	25	5055	77	21.4	2650 (?1855)
INRAE – Rupro#1	?	2120	53	17.4	?

	K_d (cm³/N.s)	Q_p (m³/s)	Width (m)	Depth (m)	Density (Kg/m³)
INRAE – Rupro#2	?	1250	26	17.4	?
INRAE – Rupro#3	?	3000	71	17.4	?
USDA – WinDAM C	25	1780	13	21.4	2650
HRW – EMBREA	50	3850	100	17.4	1667

Still significant variations in modeller parameter assumptions, which makes model performance assessment more difficult.

Note that AREBA Em and AREBA An refer to two development versions – namely AREBA Empirical (earlier model) and AREBA Analytical (newer model version).

10.3.3 Assessment of AREBA modelling data

Figure H.4 shows a comparison of results from modellers using AREBA (VUT).

With only one modeller using AREBA for this case, comparisons are limited; however two model versions are applied (as introduced above).

Observations:

- AREBA_EM1 gives a better result than AREBA_An1

Conclusions:

- AREBA_EM1 appears to be performing within the bounds of data uncertainty

10.3.4 Assessment of EMBREA modelling data

Figure H.5 shows a comparison of results from modellers (VUT and HRW) using EMBREA.

Observations:

- HRW prediction is close to observed Q_p but high on breach width; VUT is low on Q_p but close on breach width
- The above is likely to be due to different K_d and density values used

Conclusions:

- Results are within uncertainty bands
- Modeller differences in parameter estimation are highlighted:

	K_d (cm³/N.s)	Q_p (m³/s)	Width (m)	Depth (m)	Density (Kg/m³)
BUT – EMBREA	27	2750	95	16.5	2020
HRW – EMBREA	50	3850	100	17.4	1667

10.3.5 Assessment of DLBreach modelling data

Figure H.6 shows a comparison of results from modellers (UniClrk, VUT, HRW, USDA, ERAU and Geosyntech) using DLBreach.

Observations:

- Most flow predictions are close to observed – except for Geosyntech
- Most breach width predictions are close (albeit low) except for HRW
- Maximum breach depths vary

Conclusions:

- Modeller parameter assumptions vary (see below); other parameters such as Manning's n and critical shear stress also vary:

	K_d (cm^3/N.s)	Q_p (m^3/s)	Width (m)	Depth (m)	Density (Kg/m3)
BUT – DLBreach	10	4500	83	?	2400
UniClrk – DLBreach	25	5055	77	21.4	2650 (?1855)
USDA - DLBreach	25	4750	75	21.4	2650 (?1855)
HRW – DLBreach	50	5300	187	17.4	1667
Geosync – DLBreach	?	1350	25	17.4	?
ERAU – DLBreach	14	4990	60	11	1866

10.3.6 Assessment of WinDAM C modelling data

Figure H.7 shows a comparison of results from modellers (BUT, EDF, Geosyntec and USDA) using WinDAM C.

Observations:

- BUT Q_p prediction is close, but the others are all low (<50%)
- BUT breach width is high(x2) whilst the others are all low (<25%)

Conclusions:

- Predictions seem to be over or under – wide ranging – reflecting significant variations in modeller choice of parameters:

	K_d (cm^3/N.s)	Q_p (m^3/s)	Width (m)	Depth (m)	Density (Kg/m^3)
BUT – WinDAM C	84	4335	160	?	2020
EDF – WinDAM C	5	1820	29	21.6	2000
USDA – WinDAM C	25	1780	13	21.4	2650
Geosync – WinDAM C	?	750+	11+	6+	?

10.3.7 Assessment of Rupro modelling data

Figure H.8 shows a comparison of results from modellers using Rupro (INRAE and EDF).

Observations:

- All results for Q_p and breach width seem to underestimate the observed or estimated values

Conclusions:

- Limited modelling parameter details available to comment

10.4 Aware modelling

USDA-ARS, ERAU, HRW, INRAE and BUT all undertook aware modelling for the Big Bay Dam failure case as follows:

- USDA-ARS investigated the performance of: EMBREA Pro and DLBreach
- ERAU investigated the performance of: DLBreach
- HRW investigated the performance of: EMBREA

- INRAE investigated the performance of: Rupro
- BUT investigated the performance of: AREBA, DLBreach, EMBREA and WinDAM C

The influence of parameter variation on predicted outflow can be seen in Appendix H, Section H.4.

The analyses of USDA-ARS showed that for both EMBREA and DLBreach, increasing the soil erodibility typically resulted in an increase in the peak discharge, along with more rapid dam failure.

ERAU modelling demonstrated a dependence of the DLBreach model predictions upon the initial pipe size assumptions.

HRW modelling showed that varying the soil erodibility and its distribution between internal erosion application and overflowing erosion, along with critical shear stress could allow better prediction of the observed results.

The INRAE modelling demonstrated model dependence upon assumed Manning's n values – and for this case an improvement in discharge could be obtained but through a worsening in timing prediction.

BUT modelling showed that by varying soil erodibility, the predictions from all models tested (AREBA, DLBreach, EMBREA and WinDAM C) could be improved.

10.5 Conclusions

The main conclusions drawn from the Big Bay case study were similar to the Lawn Lake analyses, namely:

1. Significant variation in modeller parameter and breach assumptions led to wide range of modelling predictions
2. Similar results for some metrics are achieved by varying different modelling parameters (K_d, critical shear stress, Manning's n, density etc)
3. Uncertainty in the case study data makes it more difficult to determine accuracy of modelling – particularly with greater variation introduced by modeller parameter assumptions. However, the aware modelling results demonstrated that varying erodibility values within reasonable ranges could result in much better modelling results
4. Modelling by ERAU and INRAE demonstrated model results dependence on Manning's n value and internal erosion initial pipe dimensions
5. The HRW aware modelling using different soil erodibility for internal erosion processes as compared to overflow erosion processes should be studied further

11 Interim Conclusions – Where from here?

A large amount of modelling work was undertaken via the Phase 0, 1 and 2 test programmes. Stepping back from the individual sets of modelling data, some broad observations may be made as follows:

1. Despite trying to define modelling parameters more rigorously through the Phase 0 and 1 tests, modellers were 'innovative' in finding additional parameters to adjust – or simply used differing values to those defined – in order to achieve better modelling results. Where common modelling parameters were changed (eg K_d, critical shear stress, Manning's n, density etc) these can be seen in the modeller summary tables. However, in some models (eg DLBREACH) additional parameters (such as inlet loss coefficients) were sometime tweaked. This has made the approach of comparing model performance by comparing modelling results very difficult. It has also emphasized the importance of the modeller having a detailed and thorough understanding of how a model has been developed and how it may be setup for any particular situation
2. Where data for parameters had not been provided, modeller estimations for these values tended to vary significantly. In addition, modellers may sometimes achieve similar results whilst using different parameter combinations
3. The focus for this programme of work was to assess the performance of breach models for predicting internal erosion initiated breach. This means that the models start from an assumed hole through the embankment and simulate erosion, subsequent hole growth leading to eventual roof collapse and open breach. Whilst some models use circular hole assumptions and others rectangular, along with different rules for roof stability and collapse, it appears that the differences between these approaches have a minimal effect on the overall prediction of Q_p, timing and breach width. This may be due to the fact that the breach characteristics arising from the internal erosion process are often relatively small in comparison to the conditions arising once open breach has developed. They may also be small in comparison to the variation in results arising from modeller parameter assumptions and uncertainties in observed data
4. The 'aware' modelling work highlighted that by varying certain parameters such as soil erodibility and critical shear stress the modelling results could be improved significantly. The question then remains as to whether the changes in parameter values required for better results are reasonable or not. The 'aware' modelling also highlighted model sensitivity to assumed Manning's n values and other parameters such as the initial pipe diameter assumptions
5. The 'aware' modelling undertaken by HRW also looked at using different erodibility values (albeit linked ratios) for internal erosion rates as compared to surface erosion rates. Modelling results from this initial work looked promising

Since we have not been able to definitively answer the question "Which model(s) perform best?" an alternative approach to performance analysis was considered for the final phase of modelling work. In this phase we consider the following question:

If you consider the uncertainties within test data and the uncertainties within the modelling data (and subsequent predictions), can we see whether – given the right choice of modelling parameter combinations – models have the potential to correctly predict the observed conditions?

This would allow us to broadly determine whether a model includes the core physical processes needed to predict breach formation, or whether some fundamental processes were likely missing.

12 Phase 3 Approach to Uncertainty Analyses

12.1 Concept

The broad approach for Phase 3 modelling is to compare observed conditions – including uncertainty bounds within measurements or back calculations – against modelled conditions – again including uncertainty bounds in modelling parameters and outputs. If the two sets of data 'overlap' then it can be considered that the model had the potential to simulate the observed conditions, if the correct combination of modelling parameter values are chosen.

Hence, for example, considering the prediction of Q_p and its timing, the observed values may be represented as shown in Figure 12.1 below, where the observed values have a range of uncertainty around them.

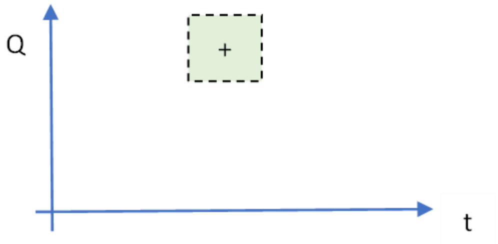


Figure 12.1: Theoretical uncertainty around observed Qp value and its timing

Subsequently, for the modelling analysis, uncertainty bounds are estimated for each of the parameters used by the models, allowing Monte Carlo analyses with the modelling to predict a range for values rather than a single value. The results will provide the most likely value, but also upper and lower ranges.

For Q_p and its timing, results may be considered as shown in Figure 12.2 below where the green, amber and red lines represent the upper and lower ranges of different models. If the two ranges (observed and modelled) overlap, the model has the potential to predict the observed conditions; if not, then the model is likely to be lacking representation of important processes.

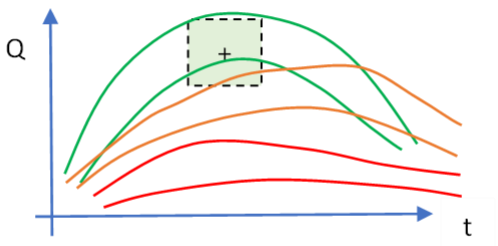


Figure 12.2: Theoretical uncertainty around observed Qp value and its timing vs modelling results

12.2 Interdependency of modelling parameters

In order to proceed with this approach, we need to consider and define potential uncertainty ranges for all of the modelling parameters used. When you start this process two questions arise:

1. Is there inter dependence between those parameters? If so, does it matter?
2. What probability distribution characteristic might we allocate to the parameter ranges?

In an attempt to help to answer these questions, a number of runs were undertaken using the EMBREA model using the Big Bay case data. In these runs soil parameters were varied, looking in particular at the relation between the erodibility coefficient (K_d) and the modelling outputs such as the peak breach outflow (Q_p), time to peak (T_p) and final breach width (B_w).

The model runs comprised:

1. Monte Carlo (MC) analysis assuming dependence between parameters, with a triangular profile probability distribution (For results see the blue circles in Figure 12.3, Figure 12.4 and Figure 12.5 below). The number of simulations per MC run was 300
2. Two deterministic runs using upper and lower bound parameter values (For results see the red circles in Figure 12.3, Figure 12.4 and Figure 12.5 below) - assuming dependence between parameters (where logical)

For the deterministic runs, the inter-dependencies were defined according to the matrix shown in Figure 12.6. Where parameter links were assumed, a direct or inverse correlation between the position in the parameter range of values was taken based upon the initial selected position in the range of values for K_d.

Analysis of these run results shows that the upper and lower deterministic results sit at either end of the MC distribution. In fact they sit just beyond the bounds of the probability distribution, reflecting that undertaking 300 runs gives a range of results that is close to, but not completely representing, the full extremes, particularly for the lower bound.

A comparison of the MC and deterministic modelling results shows what might be expected in selecting upper and lower extreme values. However, it does not tell us about dependencies other than that it does not show any unexpected behaviour in generating the range of values (i.e. that using some combination of mid-range parameter values does not appear to give rise to more extreme upper or lower range results than are obtained by using upper or lower bound parameter values).

Plots of K_d versus Q_p, T_p and B_w, for the Big Bay case, show a nonlinear relationship. Whilst there may be a linear relationship between K_d and erosion rate, there is not a linear relationship between the rate of change in the breach invert level and hence change in discharge, hence shear stress, hence rate of change in erosion.

Figure 12.3, Figure 12.4 and Figure 12.5 show that Q_p and B_w increase more rapidly as K_d increases; T_p slows more rapidly as K_d decreases. Based on this, one can broadly say that despite assuming no dependence between the parameters in the MC runs, their results look plausible and are sat within the upper and lower bounds of the deterministic runs in which dependencies were taken into account.

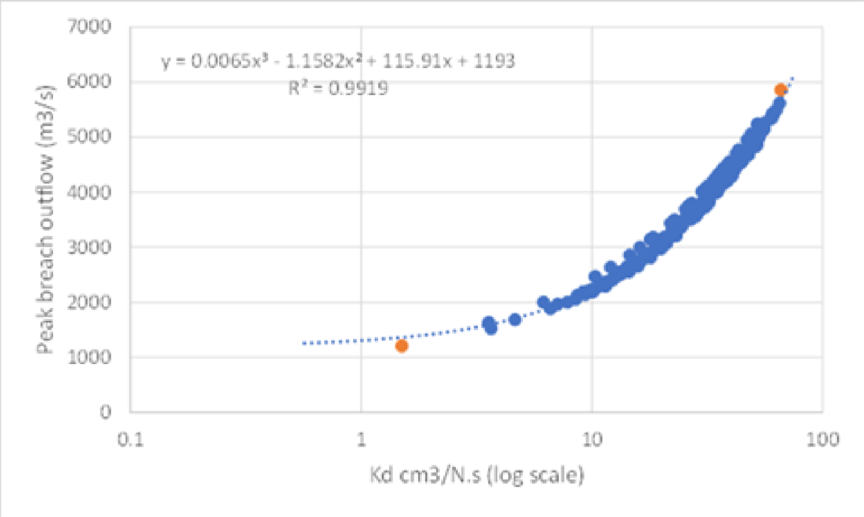


Figure 12.3: Big Bay: Variation in Q_p against K_d

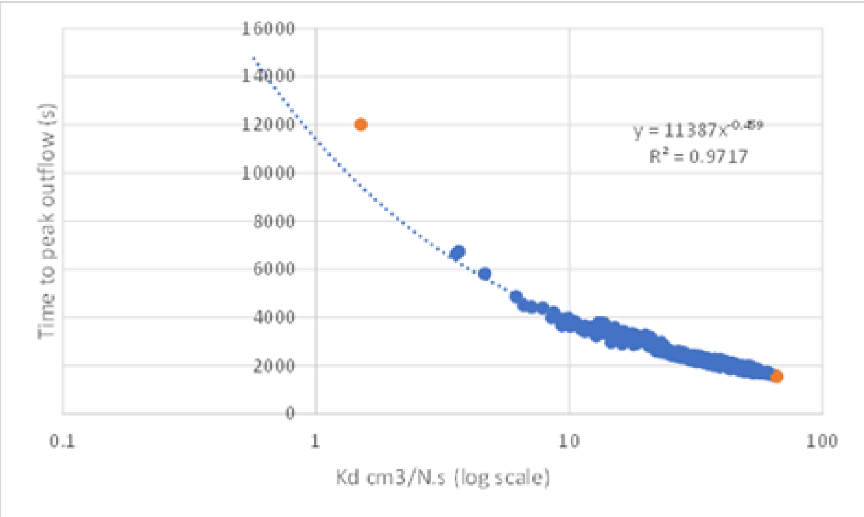


Figure 12.4: Big Bay: Variation in T_p against K_d

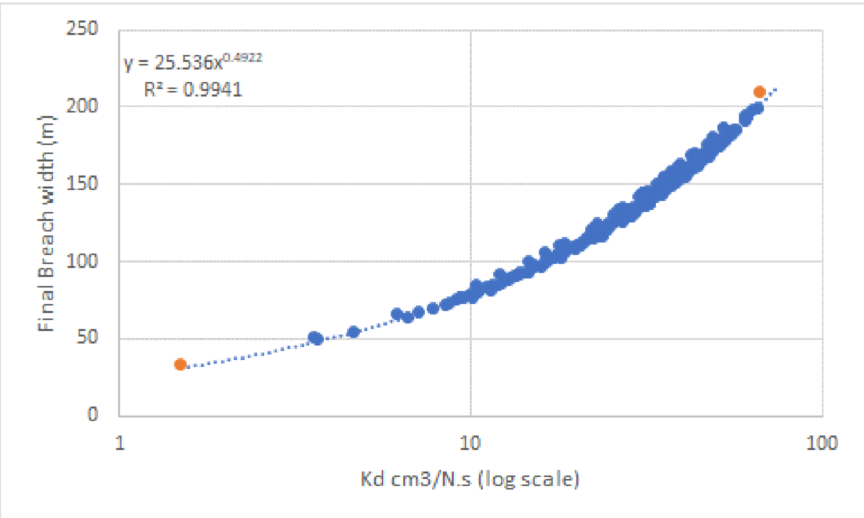


Figure 12.5: Big Bay: Variation in B_w against K_d

Assumed parameter interdependencies

Interdependence legend (top right sector):

None – Some – Strong – Not Decided – Same parameter

Interdependence type legend (bottom left sector):

Inverse – Direct – No - Not decided – Same parameter

	D_{50}	PI	Gd	Cu	SDW	Wc	P or e	k	Ø	C	Kd	IHET	Tc
D_{50}													
PI													
Gd													
Cu													
SDW													
Wc													
P or e													
k													
Ø													
C													
Kd													
IHET													
Tc													

Soil parameters listed:

a) Grading size distribution (or simply D_{50} if data not available)
b) Plasticity index (PI)
c) Grain density (Gd)
d) Unconfined compressive strength (Cu)
e) Soil densities and/or unit weights (e.g. dry, unsaturated and saturated) (SDW)
f) Water content (Wc)
g) Porosity (p) and/or void ratio (e)
h) Permeability (k)
i) Friction Angle (Ø)
j) Cohesion (C)
k) Jet Test Erodibility Coefficient (Kd)
   a. HET Erodibility Index (if available) (IHET)
   b. Critical shear stress (Tc)

Figure 12.6: Assumed parameter interdependencies (Big Bay Analyses)

Based upon the observations above, we could adopt one of the following two approaches:

i Define dependencies and adapt any MC analyses to reflect these, or

ii Ignore dependencies (i.e. assume they have already been taken into account when selecting the range for the various parameters so impossible combinations are, to a great extent, eliminated)

The following points were considered by the team:

i If we adopt the first approach and putting aside the challenge of defining different parameter dependencies, and implementing that functionality within any MC analysis code needed to apply with the models, consider what the end effect of such an approach would be?

ii If we define dependencies, we then take, for example, K_d, randomly select a value within the chosen range and then apply that random value to the other parameter ranges according to their type of dependency (eg direct, inverse etc). This will ignore uncertainties within the defined dependency and indeed the real value of each parameter relative to each other. Should we then apply some form of tiered MC analysis, where the dependent variable also has a further range of uncertainty – if so, how?

iii The modelling above shows that using the extreme range parameter values results in corresponding extreme range results. Hence it may be concluded that adding dependencies to the modelling would tend to flatten the results probability distribution (for any parameter) by distributing results more widely across the potential results spectrum (for example, see Figure 12.7). If so, does this matter for us? For breach modelling, we typically look at the upper and lower bounds of results, plus the most likely value rather than the weight of any distribution

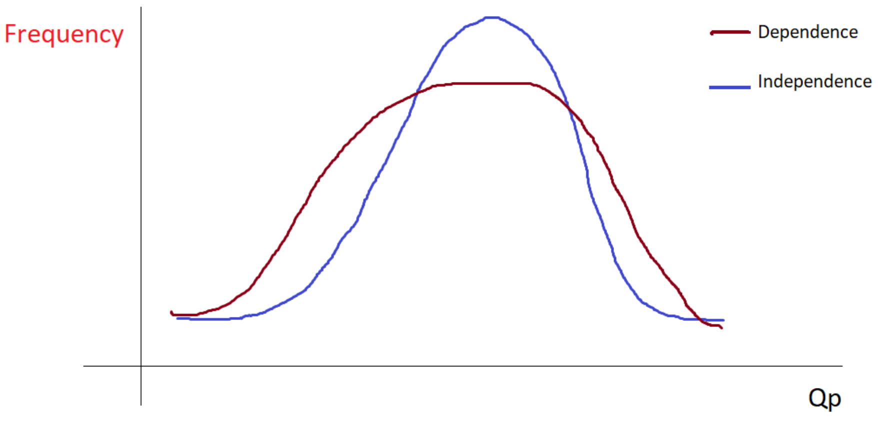


Figure 12.7: Possible effect of dependence of the results probability distribution

iv Probability Distribution: We have defined a range for different parameters – upper and lower based around the assumed best estimate. Hence, we should give more weight to our best estimate than the upper and lower bound

12.2.1 Conclusions

After considering the issues outlined above the team decided to proceed with the Phase 3 analyses on the assumptions that:

- i Parameter values can be considered independent, in the sense that estimated values and uncertainty ranges will inherently relate to values and ranges assumed for other values. No 'manual' adjustments are required
- ii A simple triangular probability distribution centred on best estimate, maximum and minimum values was considered sufficient for these analyses

12.3 Specification for Phase 3 modelling

This specification was developed based upon the discussions and conclusions drawn at the hybrid team meeting of 28th November 2022 plus subsequent feedback.

12.3.1 Overview

Having looked at a variety of ways in which modelling data can be presented to facilitate a clearer understanding of modelling uncertainty, it was concluded that presenting data in a variety of ways (rather than a single approach) was the best approach. This requires that we:

1. Undertake MC breach analyses, using parameters ranges for a number of key modelling parameters
2. Collate data in a format that allows us to:
   a. go back and extract any single run data as a deterministic data set (or extract the parameters needed to recreate the deterministic data set)
   b. generate probability distribution plots for key metrics
   c. calculate model performance indicators – linked with the modelling parameters used for each MC simulation

12.3.2 Modelling specification

Modellers were provided with:

- i The test case conditions, including parameter value ranges for all modelling parameters
- ii The observed best case conditions

Modellers were then asked to:

1. Collate data in a format that allows us to generate probability distribution plots for key metrics including:
   a. Peak discharge (Q_p)
   b. Time to peak discharge (T_p)
   c. Final breach width (B_w)
   d. Final breach depth (B_d)
   e. Time to pipe flow roof collapse (Tpc)
   f. Breach width and depth at roof collapse (both before and after collapse) (Bwbc; Bwac; Bdbc; Bdac)

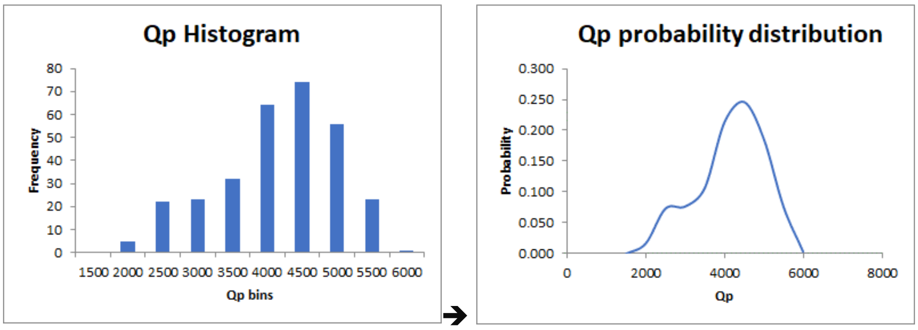
   For clarity, Bw and Bd refer to the dimensions of the breach opening itself, not the elevation or position of the opening within the dam. Further subscripts bc and ac refer to before collapse and after collapse.

2. Include calculation of model performance functions for each MC run, comparing the model calculation against an observed best case. Three 'Pr' functions were calculated representing (1) performance in predicting peak discharge conditions; (2) performance in predicting IE roof collapse and (3) performance in predicting both IE roof collapse and peak discharge conditions. These equations would comprise:
   - Pr1 = [[Ln(Qp/Qpm)]^2 + [Ln(Tp/Tpm)]^2 + [Ln(Bw/Bwm)]^2]^0.5
   - Pr2 = [[Ln(Tpc/Tpcm)]^2 + [Ln(Bwc/Bwcm)]^2 + [Ln(Bdc/Bdcm)]^2]^0.5
   - Pr3 = [[Ln(Qp/Qpm)]^2 + [Ln(Tp/Tpm)]^2 + [Ln(Bw/Bwm)]^2 + [Ln(Tpc/Tpcm)]^2 + [Ln(Bwc/Bwcm)]^2 + [Ln(Bdc/Bdcm)]^2]^0.5.

   Outputs provided included the individual ratio components (i.e. Ln(X/Xm)) as well as the summed results for Pr1, Pr2 and Pr3.

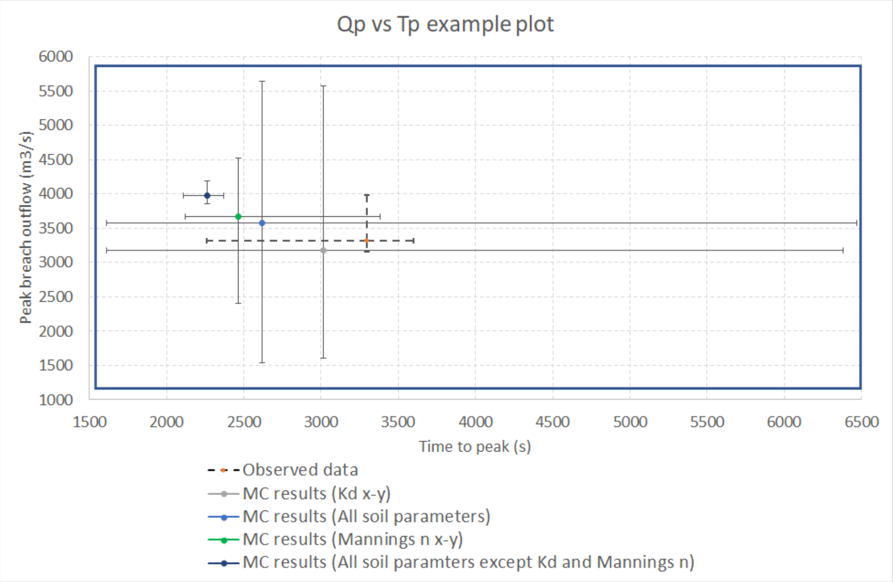
   Outputs were linked with the model parameter values selected for each MC simulation.

   Outputs should provide the ability to extract deterministic run data (Qp, Bw, Bd, etc) for any specific Monte Carlo simulation case (or extract the input parameters needed to recreate the deterministic data set)
3. Provide plots as well as providing the base data, results should be plotted using the (excel graph formats provided) showing:
   a. Probability distribution plots for Qp, Tp, Bw, Bd, Tpc, Bwc, Bdc. These can usually be produced once you have the Monte Carlo runs data using, for example, the 'Histogram' analysis tool in MS excel then using the histogram frequencies it can be converted into a probability distribution. An example of such plots is shown below for Qp probability distribution



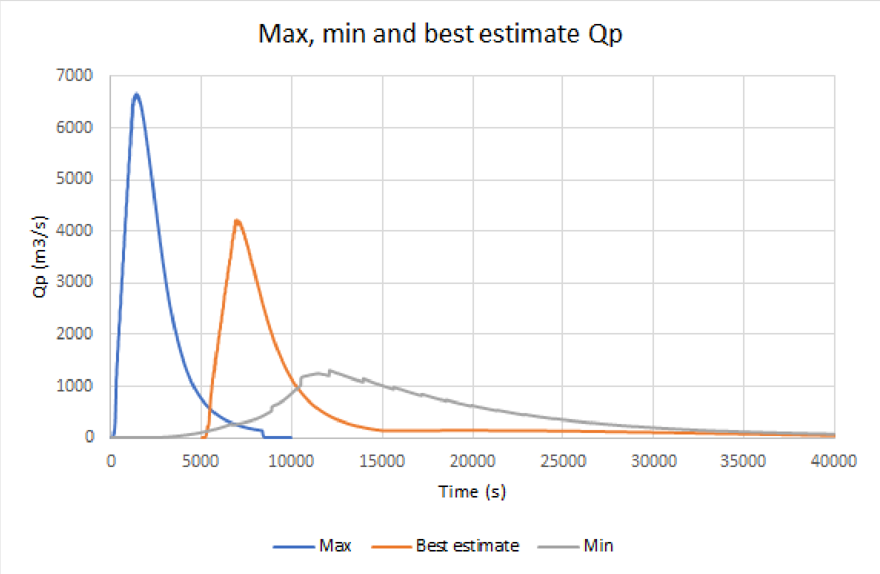
   b. Maximum – Minimum – Best Estimate range plots showing:
      i Qp versus Tp
      ii Qp versus Bw
      iii Bw versus Bd

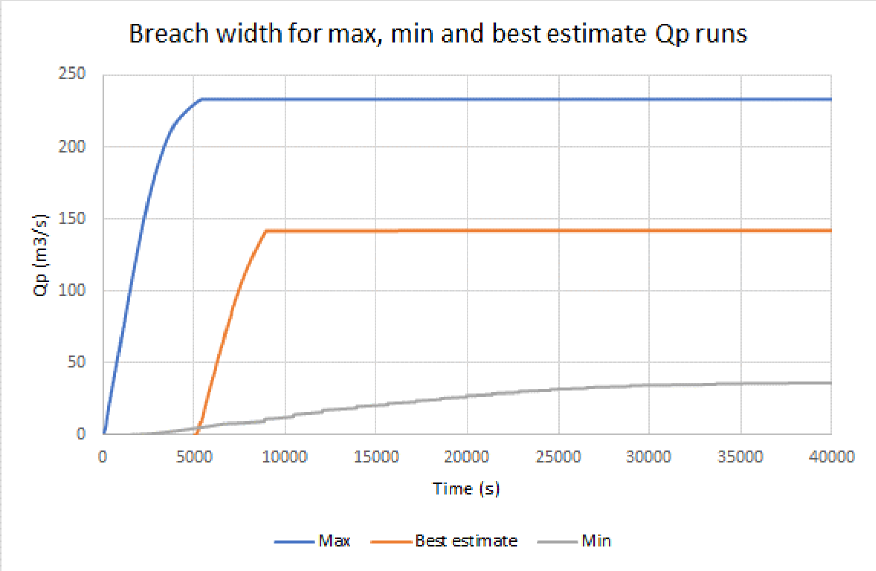
   An example of a Qp versus Tp plot is shown below:



c. Deterministic plots for:
   i Flow hydrographs for best estimate, max and min values of Qp
   ii Breach width vs time for best estimate, max and min values of Bw

Examples of the above plots are shown below:





d. Table showing:
   i Number of MC runs undertaken
   ii Top 20 simulations according to each of the three PR ratings (ie minimal PR value) including sub PR components and modelling output and input values

12.3.3 Analysis of results

The estimated uncertainty in the test case data was shared after the participants have undertaken their modelling work, as part of the review of model performance.

Each set of modelling results was reviewed and compared according to the plots and table specified above to see whether there are clear trends, differences between models, modelling assumptions etc.

To help compare models, data was manually extracted and combined to allow comparison of:

1. Probability distribution plots
2. Parameters max-min-best estimate range plots
3. Performance factor values

13 Phase 3 Modelling Results

Four team members participated in this final phase of the modelling analyses. These were:

1. HR Wallingford Applying EMBREA
2. BUT Applying AREBA and DLBREACH
3. USDA/KSU Applying WinDAM C and DLBREACH
4. Geosyntech Applying WinDAM C

Two test cases were used for analysis. These were the ARS P1 test and the Big Bay Dam failure case.

For each test case, an analysis of uncertainty in the observed data was undertaken and specifications for the modellers produced. This helps to ensure that each modellers using the same parameter values, with a triangular distribution reflecting best estimate, max and min values.

Details for each test case can be found in Appendices E and H, whilst details of parameter uncertainty ranges can be found in Appendix I.

The amount of modelling and format of results varied between the participants, hence each set of modelling results is presented separately rather than trying to integrate and directly compare the data.

13.1 HRW Modelling

13.1.1 ARS P1 Observations

The modelling results and plots can be found in the file: ARSP1_EMBREA_MC_Outputs_v1p6_0.xlsx.

Copies of key tables and plots can be found in Appendix J.

A total of 300 simulations were undertaken for each MC run, using a triangular probability distribution for each input parameter.

The range of modelling parameters used is summarised in the table below:

Table 13.1: Phase 3 – HRW: ARS P1 modelling parameter ranges (triangular distribution)

Run	Parameter	Lower Value	Most Likely	Upper Value
Geometric	Height (m)	1.2	1.3	1.4
	Crest Width (m)	1.78	1.98	2.18
Soil	Manning's n	0.016	0.025	0.033
	K_d (cm^3/N.s)	23	120	270
	Porosity	0.33	0.34	0.40
	Friction Angle (Degrees)	30	32	34
	Dry Density (KN/m^3)	15.7	16.7	17.8
	Cohesion (KN/m^2)	4	7	9
	Critical Shear Stress (Pa)	0	0.144	0.16
Other	Piping Level (m)	30.68	30.78	30.88

Key observations can be summarised as:

Considering Q_p:

- All model prediction results uncertainty ranges envelop the observed Q_p value
- The best estimate comes from varying just K_d and Manning's n

Considering B_w:

- All model prediction results uncertainty ranges envelop the observed B_w value
- The best estimate comes from varying just K_d and Manning's n

Considering T_{pc}:

- All model prediction results uncertainty ranges envelop the observed T_{pc} value
- Smallest band uncertainty and closest fit comes from varying just K_d and Manning's n
- PR1 offers the closest fit when considering results from PR1, PR2 and PR3

Comparing aware parameter values to PR1 best fit values, both K_d and T_c are significantly reduced.

Analysing the distribution of PR runs values from all of the MC analyses (Figure J.8) it can be seen that:

- PR2 runs offer a generally poorer fit to the observed than PR1 or PR3. (Compared to PR1 and PR3 points the PR2 points are significantly away from the observed data)
- Since PR3 is a combination of PR1 and PR2, this means that PR1 should be the best performing ratio to use – which is reflected in Figure J.8

This shows that if we optimise the model run to predict conditions based upon pipe roof collapse, we get a worse prediction of Q_p and T_p than if we optimise conditions based upon Q_p and T_p. So if the model more accurately reflects the pipe formation/roof collapse process it less accurately predicts the extreme 'open breach' conditions. This perhaps suggests that, for this example, the model does not correctly represent the pipe/roof collapse process by adopting the same modelling approach as for open breach growth. However, the influence of this on overall peak discharge conditions seems relatively small.

13.1.2 Big Bay Observations

The modelling results and plots can be found in the file: BBay_EMBREA_MC_Outputs_v2p1_0.xlsx.

Copies of key tables and plots can be found in Appendix J.

The range of modelling parameters used is summarised in the table below:

Table 13.2: Phase 3 – HRW: Big Bay modelling parameter ranges (triangular distribution)

Run	Parameter	Lower Value	Most Likely	Upper Value
Geometric	Height (m)	15.51	15.56	15.61
	Crest Width (m)	11.59	12.2	12.81
Soil	Manning's n	0.016	0.025	0.035
	K_d (cm3/N.s)	1.5	33	66
	Porosity	0.23	0.3	0.35
	Friction Angle (Degrees)	30	32	34
	Dry Density (KN/m3)	18	19.5	21
	Cohesion (KN/m2)	5	10	15
	Critical Shear Stress (Pa)	1	3	5
Other	Piping Level (m)	71.4	71.5	73.7

Key observations can be summarised as:

Considering Q_p:

- Only MC runs soil, K_d & Manning's n and All parameters encompass the observed Q_p range
- Best estimate comes from varying just K_d

Considering T_p:

- All runs overlap uncertainty ranges

Considering B_w:

- All model prediction results overlap with the observed uncertainty range

Considering T_{pc}:

- Only MC runs soil, K_d & Manning's n and All parameters encompass the observed T_{pc}

There does not seem to be a clear winning run; all of the best estimates from MC analyses tend to cluster in an area predicting a quicker T_p but lower Q_p.

Analysing the distribution of PR runs values from all of the MC analyses (Figure B.8) it can be seen that:

- PR2 runs offer a slightly poorer fit to the observed than PR1 or PR3. (Compared to PR1 and PR3 points the PR2 points are further away from the observed data)
- There is no clear performance difference between the various MC analysis results

13.2 BUT Modelling

BUT undertook modelling using AREBA and DLBreach for both the P1 and Big Bay test cases.

Copies of key tables and plots can be found in Appendix K.

A total of 10000 simulations were undertaken for each, using a triangular probability distribution as requested.

13.2.1 P1 test case

Note that parameter ranges used differed for some parameters from the specification ranges, as shown in slide S4, Appendix K.

Key observations:

AREBA – P1 Modelling Results:

- Plots show predicted max, med and min – however, all results show failure that is too quick compared to observed
- There is a big variation in the use of K_d value

AREBA - P1 parameter correlations:

- T_p ~40% and T_c ~66% correlation with Q_p
- T_p to B_w and T_c ~50%

The deterministic best run:

- Flow & water level simulation is good; but B_w too large
- NOTE that BUT parameters used are outside of the recommended range K_d 4.3 (instead 23-270); Tc 5.7 (0-0.16); n 0.08 (0.016 – 0.033)

DLBreach – P1 Modelling Results:

Plots for max/med/min show:

- A very wide range of results
- B_w is odd since it exceeds the defined dam width
- PR1/2/3 – odd PR1 & 3 significantly out (→ PR1 basis is worse than PR2)?

Perhaps there is an error in the model setup? As with AREBA results there is a big variation in use of K_d value.

- The uncertainty bands predicted from the DLBreach model are very large – they do encompass the observed data, but their range is massive

Prob distributions:

- There is the same issue over B_w exceeding the size of the dam

Deterministic runs:

- There appears to be no surge in the outflow
- Some parameters are outside of the specified modelling range

Correlations:

- Between many parameters are high
- T_c has a high impact on Q_p and T_p for small dams

13.2.2 Big Bay test case

A summary of parameter ranges used is shown in slide S16, Appendix K.

Areba – Big Bay Modelling Results:

- The range calculated encompasses all results – but this is a big band
- Correlations:
  - T_c to Q_p, T_p, B_w are high

DLBreach – Big Bay Modelling Results:

- There appears to be a modelling issue over +10000 data
- The range calculated encompasses observed – but it is even bigger than the AREBA range

13.2.3 Some conclusions

BUT identified the following points from their Phase 3 modelling results:

1. **Roof Collapse Calculations:**

They tried many equations within AREBA but found that for the P1 test the impacts were significant whilst for the Big Bay test they were less significant.

However, it should be recognised that these are affected by the inflow hydrograph timing hence if there is a large variation, timing is important.

2. **Downstream drowning:**

It is important to include drowning effects in the breach modelling simulation.

3. **Mannings n dependency:**

Modelling results show a signifcant dependency upon the choice of Manning's n value.

13.3 USDA/KSU Modelling

The approach taken by USDA/KSU was to wrap models using the 'Dakota' programme in order to permit Monte Carlo simulation without the need for modifying the model codes. This approach was applied to the DLBREACH and WinDAM C models. Further, rather than simulating with each modelling parameter defined independently, parameters were grouped.

Results from this approach are reported in Appendix L.

Some modelling observations:

1. Probability density results are not normally distributed – these are further affected by grouped parameters
2. DLBreach results have a multi peaked Q_p distribution – why is unclear

3. DLBreach tends to use a Manning's n value of 0.016 only
4. Predicted results have big ranges but their median values are close to observed
5. There is something odd with the DLBreach model set up for P1 – the simulation breaches quickly and then drains but through a predicted flow of 0
6. Approach used a model approximation model to avoid having to run models
7. The performance accuracy depends on what you optimise on
8. For greater clarity, we should do full parameter modelling

13.4 Geosyntec Modelling

Geosyntec undertook modelling of the Big Bay test case using WinDAM C.

Details can be found in Appendix M.

The main observations were:

1. Noted that above Tc = 3.25 then WinDAM C did not predict failure – raising also the importance of the initial pipe hole diameter size assumptions:
   - For the optimised runs (PR1) Q_p tended to be just below 100% of observed and B_w just over 100% of observed
   - Lowest simulation is significantly different to observed (both for Q_p and B_w). Range of uncertainty used is perhaps too large
   - Analysis of parameter correlations emphasises the importance of K_d (and T_c but to a lesser extent)

13.5 Comparing Phase 3 modelling results

The following tables provide a simple overview of modelling results for predicting Q_p, T_p and B_w, including observed versus minimum, average and maximum values (including % deviation) and % deviation of the PR1 best fit.

(KSU here refers to Kansas State University, working on behalf of USDA ARS).

Table 13.3: Comparing Phase 3 modelling results for ARS P1

ARS P1	Q_p (m³/s) Min-Av-Max (m³/s) Min-Av-Max (% Obs) PR1 Best Fit	T_p (s) Min-Av-Max (s) Min-Av-Max (% Obs) PR1 Best Fit	B_w (m) Min-Av-Max (m) Min-Av-Max (% Obs) PR1 Best Fit
Observed	**2.98**	**1560**	**6.5**
HRW – EMBREA	2.25 – **2.74** – 3.65 75% – 92% – 122% 2.98 (101%)	400 – **1809** – 6809 26% – 116% – 426% 1574 (101%)	3.3 – **6.9** – 9.5 51% – 106% – 146% 6.47 (100%)
BUT – AREBA	2.52 – **3.47** – 5.81 85% – 116% – 195% 2.53 (85%)	104 – **177** – 3999 7% – 11% – 256% 3577 (229%)	9.75 – **9.75** – 9.75 150% – 150% – 150% 4.6 (71%)
BUT – DLBreach	0 – **3.49** – 6.48 0% – 117% – 217% 3.92 (132%)	10 – **940** – 2390 1% – 60% – 153% 7219 (463%)	0 – **9.3** – 18.9 0% – 143% – 291% 4.58 (70%)
KSU – WinDAM C	2.6 – 3.65	0 – 7500	8.9 – 24
KSU – DLBreach	0.25 – 3.6	150 - 1005	2.5 - 9

Table 13.4: Comparing Phase 3 modelling results for Big Bay

Big Bay	Q_p (m³/s) Min-Av-Max (m³/s) Min-Av-Max (% Obs) PR1 Best Fit	T_p (s) Min-Av-Max (s) Min-Av-Max (% Obs) PR1 Best Fit	B_w (m) Min-Av-Max (m) Min-Av-Max (% Obs) PR1 Best Fit
Observed	**3313**	**3300**	**96.2**
HRW - EMBREA	1288 – **2910** – 5051 40% - 88% - 152% 2910 (88%)	1730 – **2975** – 8650 52% - 90% - 262% 3150 (95%)	36 – **110** – 189 37% - 115% - 197% 107 (111%)
BUT - AREBA	861 – **3511** – 7102 26% - 106% - 214% 3329 (100%)	1169 – **2348** – 9188 35% - 71% - 278% 2507 (76%)	9 – **85** – 185 9% - 88% - 192% 79 (82%)
BUT – DLBreach	848 – **7691** – 9999 26% - 232% - 302% 2120 (64%)	1569 – **2171** – 8950 48% - 66% - 271% 9756 (296%)	14 – **133** – 226 15% - 138% - 235% 31 (32%)
KSU – WinDAM C	0 – **2345** – 3736 0% – 71% – 113% 2789 (84%)	0 – **2880** – 24840 0% – 87% – 753% 3240 (98%)	0 – **105** – 132 0% – 113% – 137% 104 (108%)
KSU – DLBreach	1266 – **5967** – 8673 38% – 180% – 262% 2975 (90%)	1185 – **1890** – 18760 36% – 57% – 568% 4080 (124%)	52 – **229** – 349 54% – 238% – 363% 122 (127%)
Geosyntec – WinDAM C	1036 – x – 3829 31% – x – 116% 3238 (98%)	2520 – x – 15480 76% – x – 469% 3240 (98%)	23 – x – 126 24% – x – 131% 98 (102%)

13.6 Comparison to Simple Qp Prediction Equations

An approach often used by engineers seeking a quick prediction of potential breach flow is to use simplified equations, typically based upon use of parameters reflecting reservoir volume, dam height and perhaps type of soil erodibility. These equations have been developed by matching historic dam failure data against observed/predicted flow conditions, hence at best will reflect an average estimate of conditions for an 'average' dam structure and failure condition.

Three equations were considered here for performance comparison:

1. Froehlich 1995
2. Xu & Zhang 2009
3. CLF 2020

These were applied to both the P1 and Big Bay cases. Since the equations only predict Q_p, the time offset in these plots is simply to show a comparison; only the Y axis (flow) position is relevant.

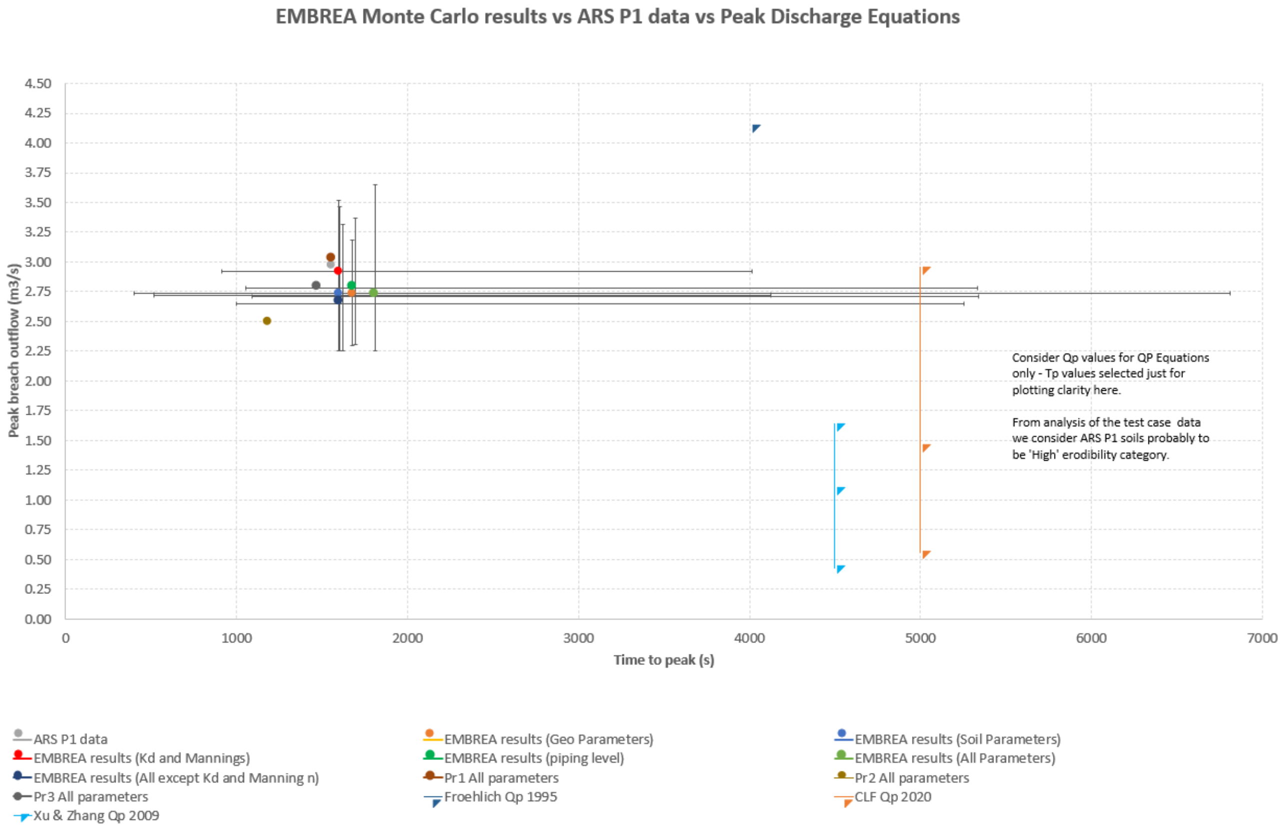


Figure 13.1: Comparing Peak Discharge Equation Performance for ARS P1

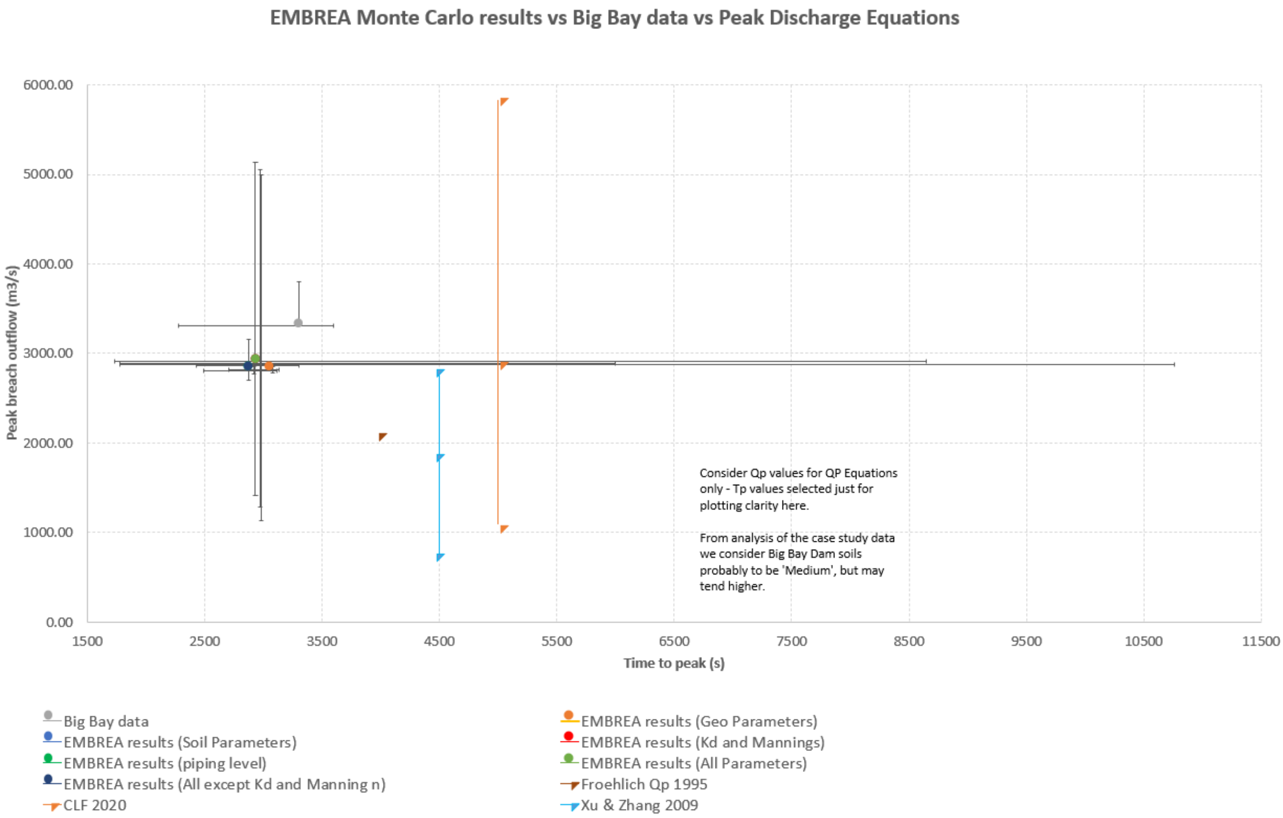


Figure 13.2: Comparing Peak Discharge Equation Performance for Big Bay

Hence observations for these two test case comparisons:

- The P1 test condition is probably outside of the typical data range from real dam failures used to create the equations – i.e. dam height relatively small (closer to a levee), and storage volume small compared to a typical reservoir (however, this also reflects the dangers of using such equations for breach through flood levees, where conditions are also likely to differ from historic dam failure records)
- Froehlich 1995 overpredicts for P1 and underpredicts for Big Bay. For P1 it is outside of the EMBREA modelled range, whilst for Big Bay it falls within
- Xu & Zhang underpredicts (for all erodibility values) for P1, but some states fall within the EMBREA predicted range for Big Bay. Nevertheless, all of the results are low of 'observed'
- CLF overlaps with 'observed' data in both P1 and Big Bay, but the range of uncertainty presented is very large – larger than EMBREA max/min for both cases

These results are consistent with expectations when keeping in mind that the equations are based upon averaging a limited dataset and use a limited number of parameters to reflect a complex breaching process. Hence, as an indicator of potential breach peak flow, the equations offer a very simple solution, but a more refined estimate may be achieved using the physically based models.

13.7 Conclusions from Phase 3 Modelling Programme

The Phase 3 modelling programme allowed us to assess model performance from a different perspective. The key observations and conclusions from this were:

1. Given the right combination of modelling parameters, the models can predict observed conditions:
   a. Hence, suggesting that there are no obvious parameters or processes missing from the models
2. The range of uncertainty in the predictions – accumulating uncertainty from the modelling parameters – can be very large:
   a. But using the mean average predicted values seems to give good performance
   b. Mean average values can be in the range ±15-20% (see Table 13.3 and Table 13.4)
   c. From the HRW analysis, use of uncertainty analysis on just K_d & Manning's n gave close or better than analysis of all parameters
   d. Hence, reducing uncertainty in choice of K_d and Manning's n would have the most beneficial impact on modelling accuracy
3. Performance of models: The range of uncertainty in predictions (max/min etc) varies between the different models – but also between modellers:
   a. Even at this level of modelling expertise, modellers setup models differently and have difficulties in correctly applying models that they are not so familiar with, leading to significant differences between model users, even within the same organisation.

      Table 13.3 and Table 13.4 show how the results can vary
4. The impact of correctly simulating pipe formation and roof collapse through to open breach depends upon a variety of factors:
   a. Logically, the timing of pipe & roof collapse affects the open breach growth and eventual Q_p, T_p, B_w etc but if the reservoir and inflow conditions are stable, the apparent impacts are minimal; how the collapse timing then interacts with the load hydrograph becomes significant
   b. If you're looking at Q_p & B_w only, impacts may be minimal; if T_p is important, then closer attention is needed (and assumptions regarding pipe initiation diameter affect this)
   c. Differences between existing model approaches seemed to have minimal impact in the context of wider modelling uncertainties

5. Use of PR functions to identify 'best runs' to achieve a certain performance function was a useful way of seeing how close models could get to 'observed' conditions:
   a. Note the limitations in how many MC runs we could reasonably do
   b. PR1 seemed to offer the best results – but maybe that's because PR1 assesses fit to Q_p, T_p, B_w and these are the parameters that we tend to use for flood risk assessment of breach (as compared to pipe formation & roof collapse)
6. We tried repeatedly to separate model and modeller to allow an objective comparison of model performance… and failed:
   a. Modellers apply models in the way they think best
   b. Where modellers are not so familiar with a code, applications can vary!
   c. Where modellers are very familiar with a code, tweaks to 'internal modelling parameters' can be made which are difficult to identify:
      i Consider these points in relation to someone with limited experience applying a breach model
7. Comparing simple 'peak discharge equations' to physically based model performance:
   a. The comparison showed that the most commonly used simple equations were not as 'accurate' as the physically based models through either missing a reasonable prediction of the observed data, or predicting a wider range of uncertainty, or both
   b. This finding is consistent with what might reasonably be expected when comparing the performance of a simplified approach against a more complex, physically based approach

14 Summary of Main Project Conclusions

Key project conclusions are:

1. Several of the models consistently predict results such that the modelled uncertainty range overlaps with the observed data:
    a. The models have the potential to predict the observed conditions given the right parameters
    b. The use of the PR1 function showed this very well
2. **Best estimates using the mean of Monte Carlo modelling results gave good estimations for many parameters:**

    Can be within ±15-20% of observed (see Table 13.3 and Table 13.4)
3. Predicted result uncertainty bands – arising from uncertainty in modelling parameters are very large:
    a. Whilst using the mean average predicted values seems to give good performance the max-min result ranges are very large (often factors of 2 or 3 above or below observed)
    b. There is a need to reduce uncertainty in parameter value measurement/estimation – in particular for K_d and estimation of Manning's n/model flow roughness value
4. Pipe formation & roof collapse routines within the models do affect Q_p/T_p conditions, but the impacts appear to be smaller than might be initially assumed:
    a. It may be that the focus for studies are often on peak flood conditions rather than pipe roof collapse
    b. The time of roof collapse and time of peak flood is poorly predicted by the models used in this project
5. The accuracy of the modelling results depend significantly on the understanding and judgement of the modeller:
    a. Significant differences in applications can be seen within this group of experts, relating to detailed knowledge of model setup
6. Comparing predictive models to peak discharge equations shows that:
    a. The range of prediction (depending on high/med/low K_d) can be large – larger than the physically based models
    b. The reliability of the predictions reduces as the application deviates from the 'average' dam and/or reservoir
7. **An action most likely to improve the accuracy of breach modelling is to improve our ability to measure and predict and apply K_d for different dams and levees**

15 Acknowledgements

Whilst this project was funded by EDF (represented by Jean-Robert Courivaud), and managed by HR Wallingford (represented by Mark Morris), the modelling work was undertaken by a variety of organisations and their representatives as summarised below. We would like to acknowledge their contributions to this work, which could not have been undertaken without their involvement.

Table 15.1: Organisations and representatives

Name	Organisation	Model and/or Role
Abdelfatah Ali	USDA HERU, Stillwater, USA	WinDAM C & DLBREACH modelling
Anthony Atkinson	Kansas State Uni, USA	WinDAM C & DLBREACH modelling
Ghada Ellithy	ERAU (Embry-Riddle Aeronautical University, USA; Formerly ERDC, Vicksburg).	Modelling and advisory
Mohamed Hassan	HR Wallingford, UK	EMBREA and DLBREACH modelling; breach perspectives
Sherry Hunt	USDA HERU, Stillwater, USA	Advisory
Stanislav Kotaska	Brno Uni Tech, Czech Republic	AREBA and DLBREACH modelling
Mitch Nielson	Kansas State Uni, USA	WinDAM C & DLBREACH modelling
André Paquier	INRAE, France	RUPRO modelling
Al Preston	Geosyntec, USA	WinDAM C modelling; breach perspectives
Pierre Squillari	GeophyConsult, France	WinDAM C & Rupro modelling
Veronika Stoyanova	ARUP, UK	OvaBreach modelling
Ron Tejral	USDA HERU, Stillwater, USA	WinDAM C & DLBREACH modelling
Myron van Damme	TU Delft/Rijkswaterstaat, NL	AREBA modelling; breach perspectives
Tony Wahl	USBR, Denver, USA	Advisory
Weiming Wu	Clarkson Uni, USA	DLBREACH modelling

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Appendices

A Model Descriptions

Table 2.1 (copied below) provides a summary of the models used in performance evaluation programme. The following sections in this appendix provide a brief summary of each of those models.

Table A.1: Industry applicable models identified for evaluation

Model	Contact	Organisation	Country
AREBA	Myron van Damme	TU Delft	Netherlands
DLBREACH	Weiming Wu	Clarkson University	USA
EMBREA	Mohamed Hassan	HR Wallingford	UK
RUPRO	André Paquier	INRAE (formerly IRSTEA)	France
WinDAM C	Sherry Hunt	USDA-ARS-HERU	USA

A.1 The AREBA model

A.1.1 An introduction to the AREBA model

Q1: What does the model predict?

AREBA is able to predict the breach hydrograph within the bounds of uncertainty that originate from the uncertainty in model input parameters.

Q2: Why was the model developed – any specific end user or application in mind?

It was developed for fast predict of the breach hydrograph within the bounds of uncertainity.

A.1.2 Modelling approach

Q3: What broad approach does the model take to simulate breach development?

(eg Section by section, predefined failure process, etc?)

The dam breach model written in Matlab (Octave), for fast predict of dam breaching with Monte Carlo simulations. It has also include analytical process method based on dilation influencing erosion. Simplified approach of dam breach with average erosion on crest and average erosion on airside of a dam.

Q4: What are the advantages and disadvantages of this approach?

Advantages - The speed of prediction, Monte Carlo simulation, modellers can modify the model and see the code. Disadvantages - Some approaches are simplified or are not considered.

Q5: How does the model predict the internal erosion growth process? What initial assumptions – if any – must be made? Is the internal erosion growth process (i.e. shape and mechanism) predefined or free format?

User defines initial circular pipe diameter and position. Model then calculates flow shear within pipe combined with erosion relationship to predict pipe growth.

Q6: Does the model predict roof instability above the internal erosion, followed by collapse and subsequent open breach formation? How?

Analyses roof stability by weight of roof which must be equal to stabilizing force. If the roof fails the sediment is immediately eroded away.

Q7: How are the open breach formation and widening stages simulated? Are the processes and breach shape predefined or free format?

Option of headcut or surface erosion with calculation of erosion by empirical solution by Hanson 2005 or by analytical based process solution by Van Damme 2020.

The breach shape is defined as a trapezoidal. There is no geotechnical stability.

Q8: What erosion relationship(s) does the model use? Are these predefined or can the modeler choose? Do these apply throughout all stages of breach development (from initiation growth through to open breach formation and widening)?

User defines erosion relationship to use - can choose from empirical formulae taken over HR Breach or can apply analytical based process method for soils with cohesion under 4500 Pa and flow rate over 1,5 m/s.

Q9: How does the model calculate flow through the breach (from initiation to open breach)?

Pipe flow rate is calculated using Bernouilli's energy equation; Simple weir flow calculation, Flow surface calculated from critical flow.

Q10: How does the model simulate the upstream boundary conditions (reservoir, river etc)?

Upstream reservoir routing: volume - height conditions; inflow: flow-time conditions, functional objects: flow-time conditions; It is necessary to set maximal surface and volume of reservoir, altitudes of functional objects.

Q11: Does the model simulate downstream conditions, and if so, does it take drowning of the breach into account?

Yes, by set the condition of water level at downstream area and by surface area of downstream valley. Analyses the effect of drowning on breach growth.

Q12: Does the model allow the user to investigate uncertainty in parameters/prediction?

Yes by Monte Carlo simulations of parameters.

Q13: Does the model allow for forms of Monte Carlo analysis or inclusion of other means of parameter uncertainty?

Yes it allow for forms of Monte Carlo analysis.

A.1.3 Modeller assumptions and model performance

Q14: What key parameters is the modeller required to define when setting up the breach model? (include any computational as well as material and structure definition parameters.)

Initial level of the pipe [asl], Initial pipe diameter [m]; value for the soil cohesion in [kN/m^2]; mannings coefficient in [s/m^3];weir coefficient, Density of the soil in [kg/m^3]; D50 in [mm]; critical shear stress in [N/m^2]; Hydraulic conductivity in m/s; Initial porosity [-]; Critical porosity [-]; Density particles [kg/m^3]; Erodibility [cm^3/Ns]; Internal friction angle [deg]; Top load [kN/m^2].

Q15: Is guidance provided on selecting these parameters? If so, how and on what is that guidance based?

No. (Its writtening now.)

Q16: How sensitive is performance of the model to key parameter selection? Which are the key parameters?

The empirical model is for soils with cohesion over 4500 Pa and flow rate under the 1,5 m/s with analytical based erosion process method. The key parameters are hydraulic conductivity, porosity, erodibility, critical shear stress and cohesion.

Q17: How was the model performance validated during development? Where appropriate, what data sets have been used - and how - to validate performance?

AREBA empirical solution have been validated against the IMPACT data (www.impact-project.net), and benchmarked against HR BREACH version 4.1. The based process erosion method for horizontal and vertical side was validated, but for circular pipe is not validated yet.

Q18: What is an indicative duration for a model simulation? (eg <1s; <30 s; a few minutes; 5-10 minutes; 10-30 minutes; > 30 minutes; Hours.)

< 30 s ~ about 5 s for one simulation. In case of MonteCarlo, the time is multiplied by number of simulations.

Q19: What are the model strengths and weaknesses?

The strangest are that the modeller gets the whole code of model which can be modified, and errors can be traced. It can be used Monte Carlo simulation for parameters which the modeler chooses. The weaknesses are simplified of mechanism breaching, and need of additional software (MATLAB, Octave).

A.1.4 The AREBA model development history and availability

Q20: When was the model first developed? By whom?

The model was developed by Van Damme et al. as part of the FRMRC2 programme.

Q21: What language is the model developed in?

Matlab – English.

Q22: What platforms can the model run on?

MATLAB, Octave.

Q23: How can a user access and run the model to undertake breach analyses?

Contact the breach developers to get the model.(Myron van Damme, Stanislav Kotaška).

Q24: Are there any costs to use the model?

No.

Q25: Is technical support available? How?

Yes, contact the breach developers to get support. (Myron van Damme, Stanislav Kotaška) (The guideline is under process now).

A.2 The DLBreach model

A.2.1 An introduction to the DLBreach model

Q1: What does the model predict?

DLBreach simulates the breaching processes of non-cohesive and cohesive, homogeneous and composite embankments due to overtopping and piping in rivers, estuaries and coastal zones. It predicts the breach hydrograph, width and depth, as well as the water level in the reservoir or bay.

Q2: Why was the model developed – any specific end user or application in mind?

DLBreach can be applied to simulate the breaching processes of dams and levees in inland rivers, as well as dikes and barriers in coastal zones.

A.2.2 Modelling approach

Q3: What broad approach does the model take to simulate breach development?

(eg Section by section, predefined failure process, etc?)

DLBreach calculates the non-equilibrium transport of noncohesive sediments from the reservoir, bay, or ocean to a downstream channel or storage. It simulates the cohesive embankment breach erosion processes in the form of headcut migration or surface erosion, and the breaching of composite embankment with clay core and cover. The model handles dam, levee and barrier breaching by implementing different algorithms to determine the head and tail water levels. It allows embankment base erosion. DLBreach can handle both one- and two-direction breaches under river flows, tidal flows and waves.

Q4: What are the advantages and disadvantages of this approach?

Advantages: DLBreach is computationally efficient and can provide results in seconds to minutes. It can handle the embankment breaching in riverine, estuarial and coastal waters. It is public free.

Disadvantages: DLBreach is a simplified physic-based breach model, so it adopts assumptions, simplifications and approximations.

Q5: How does the model predict the internal erosion growth process? What initial assumptions – if any – must be made? Is the internal erosion growth process (i.e. shape and mechanism) predefined or free format?

The piping breach is approximated as a flat pipe with rectangular cross-section until the pipe roof collapses, and then overtopping takes place. The flow in the pipe is calculated with the orifice flow equation. The erosion at the pipe perimeter is determined using different transport models for non-cohesive and cohesive sediments. The erosion thickness is assumed to uniformly distribute on the pipe surface and along the length. The pipe is enlarged at each time step until the collapse of the roof part of the embankment.

Q6: Does the model predict roof instability above the internal erosion, followed by collapse and subsequent open breach formation? How?

The failure of the roof part is determined by comparing the driving and resistance forces in both vertical and horizontal directions. The failure is assumed along the vertical planes extended from two side walls of the pipe. Once the driving force is larger than the resistance force in either vertical or horizontal direction, the roof part above the pipe will collapse. Then, the overtopping flow module is then used to simulate the breach process.

Q7: How are the open breach formation and widening stages simulated? Are the processes and breach shape predefined or free format?

The failed pipe roof is assumed to move out of the breach immediately, but stored in a virtual tank. In the next time steps, the model calculates the flow and sediment transport without considering the failed pipe roof material, but does not change the breach geometry until the mass stored in the virtual tank is completely eroded away. It allows the gradual release of the failure block to the downstream and avoids the possible instability caused by sudden, discrete mass failure events during the breaching process.

Q8: What erosion relationship(s) does the model use? Are these predefined or can the modeler choose? Do these apply throughout all stages of breach development (from initiation growth through to open breach formation and widening)?

In DLBreach, noncohesive soil erosion is calculated by using the non-equilibrium sediment transport model, and cohesive soil erosion is calculated by using the linear erosion law and headcut migration model.

DLBreach divides the breach process into two phases: Intensive breaching phase and general breach (inlet) evolution phase, particularly for coastal dike/barrier breaching. In the intensive breaching phase, the breach flow is modelled with the broad-crest weir flow equation or orifice flow equation in the cases of overtopping and piping breach, respectively. In the general evolution phase, the flow through the breach or inlet is modelled using the Keulegan equation. This treatment can also be used in the case of inland levee breach with a long evolution period. The breach flow in the intensive breaching phase are typically supercritical and upstream control, whereas the breach flow in the general evolution phase is subcritical or mixed sub-/ supercritical and experiences significant downstream tailwater effect.

Q9: How does the model calculate flow through the breach (from initiation to open breach)?

The flow through the pipe is calculated with the orifice flow equation.

Q10: How does the model simulate the upstream boundary conditions (reservoir, river etc)?

The water level in a reservoir or bay is calculated by using the water balance equation considering river inflow and breach flow. The water level in a river and the ocean is determined by measurements or simulations using a third-part model, such as HEC-RAS.

Q11: Does the model simulate downstream conditions, and if so, does it take drowning of the breach into account?

Because DLBreach handles one- and two-direction breaches, the upstream and downstream boundaries are treated using the same methods and can be switched. The drowning or submergence on the outflow side is considered in the weir flow equation, the orifice flow equation or the Keulegan equation.

Q12: Does the model allow the user to investigate uncertainty in parameters/prediction?

DLBreach does not have a function to investigate parameter uncertainties.

Q13: Does the model allow for forms of Monte Carlo analysis or inclusion of other means of parameter uncertainty?

DLBreach can be incorporated with a third-part model to conduct the uncertainty analysis.

A.2.3 Modeller assumptions and model performance

Q14: What key parameters is the modeller required to define when setting up the breach model? (include any computational as well as material and structure definition parameters)

Time step (seconds), simulation period (seconds), embankment height (m), crest width (m) upstream and downstream slopes (vertical/horizontal), length (m), breach mode, overtopping mode, initial overtopping breach depth and width (m), breach location, hard bottom elevation (m), Manning n ($s/m^{1/3}$), noncohesive or cohesive sediment, sediment diameter (m), specific gravity (unitless), porosity (unitless), clay content (in fraction), cohesion (Pa), internal friction coefficient (unitless), noncohesive sediment adaptation length parameter, cohesive soil erodibility k_d

(cm^3/Ns), critical shear stress (Pa), initial upstream and downstream water levels (m), clay core geometric parameters, reservoir or bay parameters or water level time series, downstream channel parameters or water level time series, reservoir or bay inflow, waves, wind, tides, etc.

Q15: Is guidance provided on selecting these parameters? If so, how and on what is that guidance based?

Yes. The model has been tested in many cases. Users can select parameters using the examples similar to their cases.

Q16: How sensitive is performance of the model to key parameter selection? Which are the key parameters?

For cohesive sediments, the erodibility coefficient k_d is the key parameter. It needs to be measured or calibrated. For noncohesive sediments, the particle diameter and the adaptation length coefficient lamda are important. The Manning coefficient n is important for the flow and bed shear stress calculations.

Q17: How was the model performance validated during development? Where appropriate, what data sets have been used – and how – to validate performance?

DLBreach was first tested using 50 sets of laboratory experiment and field case study data on dam breaching. Then it was tested in several cases of riverine levee and coastal dike and barrier breaching. The model performance is highly dependent on the erodibility coefficient k_d for cohesive soils.

Q18: What is an indicative duration for a model simulation? (eg <1 s; <30 s; a few minutes; 5-10 minutes; 10-30 minutes; > 30 minutes; Hours).

Each simulation using DLBreach takes seconds to minutes.

Q19: What are the model strengths and weaknesses?

DLBreach is able to handle dam, riverine levee, coastal dike/barrier breaches.

A.2.4 The DLBreach model development history and availability

Q20: When was the model first developed? By whom?

The first version of DLBreach was based on the journal article of Wu (2013). The present version was released in 2016. The model was developed by Prof. Weiming Wu, Clarkson University, USA.

Q21: What language is the model developed in?

DLBreach is written in Fortran.

Q22: What platforms can the model run on?

The DLBreach executable runs on PC.

Q23: How can a user access and run the model to undertake breach analyses?

The DLBreach executable code, technical report, and user guidance can be downloaded from https://webspace.clarkson.edu/~wwu/DLBreach.html.

DLBreach has been implemented in the HEC-RAS model and released to the public. Note that HEC-RAS uses its own flow module and adopts only the sediment transport and morphology modules of DLBreach.

Q24: Are there any costs to use the model?

DLBreach is free.

Q25: Is technical support available? How?

Short questions have been answered by Weiming Wu without charge.

A.2.5 Anything else?

Add any other information you wish to provide.

N/A

A.3 The EMBREA model

A.3.1 An introduction to the EMBREA model

Q1: What does the model predict?

The model predicts the breach outflow hydrograph and breach growth with time (i.e. breach depth and width vs time). The model also predicts the following processes:

- Initial erosion of embankment surface protection (grass or rock cover)
- Breach growth through overtopping flow of homogeneous and layered embankments (cohesive or non cohesive materials - including consideration of head cut and the analysis of breach side slope instability)
- Breach growth through overtopping flow of simple composite embankment structures (i.e. simple zoned structures)
- Breach growth through internal erosion and subsequent collapse of homogeneous and layered embankments

Q2: Why was the model developed – any specific end user or application in mind?

The model was developed to meet industry needs such as, accurate modelling of breach processes, proven model performance and a user-friendly software.

A.3.2 Modelling approach

Q3: What broad approach does the model take to simulate breach development?

The model integrates hydraulics, soil mechanics and structural failure processes to a broadly consistent degree of complexity. The model undertakes analysis on a section-by-section basis through the model (Figure A.1) and, unlike other models, does not predefine the breaching process in terms of stages and geometry.

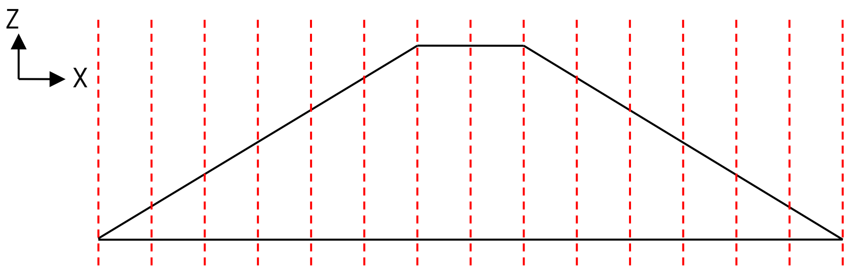


Figure A.1: Modelling embankment breach by division of embankment into sections

Figure A.2 provides a flow chart showing the order in which the hydraulics, soil and structural processes are analysed.

Q4: What are the advantages and disadvantages of this approach?

Advantages are the ability to accurately model the various breach processes within practically acceptable run times. One disadvantage is probably data requirements as the model requires more data than other simple and empirical approaches.

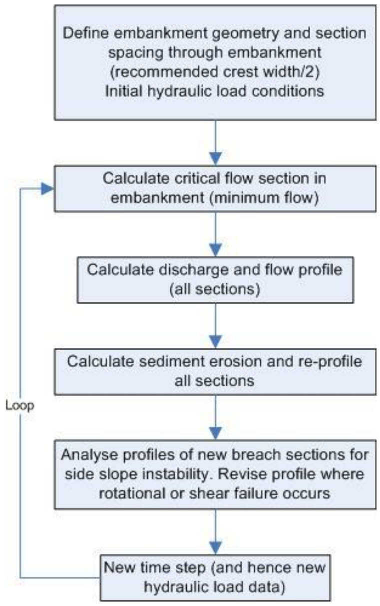


Figure A.2: EMBREA model processes

Q5: How does the model predict the internal erosion growth process? What initial assumptions – if any – must be made? Is the internal erosion growth process (i.e. shape and mechanism) predefined or free format?

To simulate breach growth through internal erosion, an initial assumption is made that a finite size pipe has already been established along the embankment. The model then simulates growth of this pipe, through to embankment failure, including the following processes:

- Erosion of material in the pipe (i.e. growth of the pipe diameter and hence flow through the embankment)
- Slumping of the downstream embankment face material above the pipe (simulating the cut back of the pipe exit in the downstream embankment face)
- Collapse of the embankment body above the pipe, either under its own weight or by the water pressure forces
- Following collapse of a pipe, erosion of the embankment body as an open breach

Q6: Does the model predict roof instability above the internal erosion, followed by collapse and subsequent open breach formation? How?

Yes, the model predicts roof instability above internal erosion followed by collapse of and subsequent open breach formation. A description of how this is done in the model is given below.

Slumping of the Downstream Face Material above the Pipe

After the formation of the pipe, the embankment material of the downstream face starts to fall into the pipe when it becomes unstable. Then the flowing water carries it away. This mechanism has been observed during the piping failure of the Teton dam in 1976. The vertical failure planes observed during Teton dam failure suggest that it is likely to be a shear failure. As shown in Figure A.3, the hatched wedge will fall into the water when the shear stress due to its own weight exceeds the shear strength of the embankment material above the pipe on the downstream face. The shear strength of the material consists of two components the cohesion and the friction. The friction forces are considered small and are ignored hence only cohesion forces are considered.

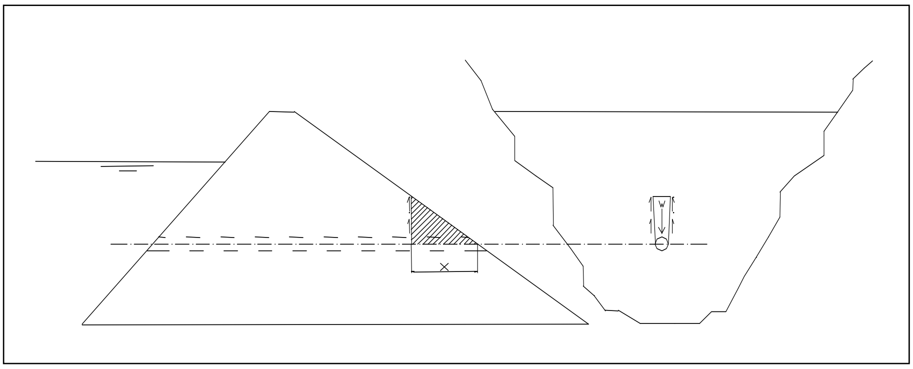


Figure A.3: Slumping of the downstream face material

Based on this, the factor of stability (FOS) of the wedge against shear failure can be expressed as follows:

$$\boldsymbol{FOS} = \frac{\boldsymbol{Stabilising\ forces}}{\boldsymbol{Destabilising\ forces}} = \frac{\boldsymbol{CAc}}{\boldsymbol{W}} \qquad (1)$$

where: C : Soil cohesion

A_c : Areas on the sides and the back of the wedge corresponding to the cohesion

W : Weight of the wedge taking into consideration the arching effect

<u>Collapse of the Top Part of the Embankment</u>

As the material slumps into the pipe (as explained above), the top of the dam gets thinner. If the water pressure forces are high enough to exceed the shear strength of the embankment material, then the top of the dam will collapse (See Figure A.4). Also, it can collapse under its own weight. This mechanism was also observed during the failure of Teton dam after the slump of the downstream material started.

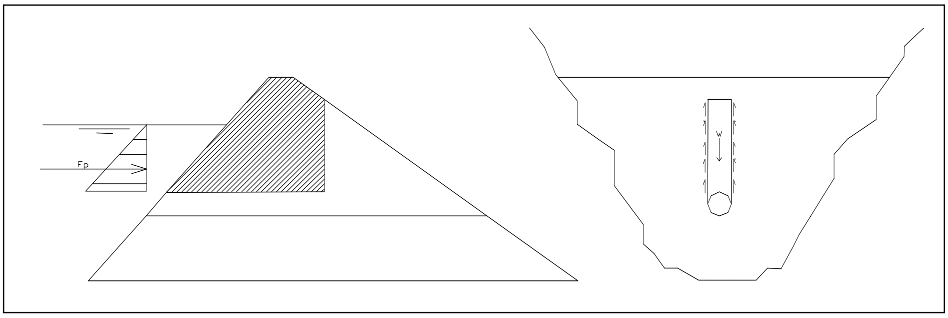


Figure A.4: Collapse of the top of the dam

If it fails because of hydrostatic pressure forces, then, the factor of stability can be expressed as follows:

$$\boldsymbol{FOS} = \frac{\boldsymbol{CAc}}{\boldsymbol{Fp}} \qquad (2)$$

where: A_c : Areas on the sides corresponding to the cohesion

F_p : Hydrostatic forces

If the top of the dam collapses under its own weight the factor of stability can be expressed as follows:

$$\boldsymbol{FOS} = \frac{\boldsymbol{CAc}}{\boldsymbol{W}} \qquad (3)$$

where: A_c : Areas on the sides corresponding to the cohesion.

W : Weight of the wedge taking into consideration the arching effect.

Q7: How are the open breach formation and widening stages simulated? Are the processes and breach shape predefined or free format?

The flow is established at the open breach formation stage as described in Q9. This flow is used in the following steps:

- Computation of water Depth and velocities using the non-uniform flow equation
- Computation of eroded material and update of the breach longitudinal and lateral profiles using a sediment transport or an erosion equation (See Q8)
- Assessment of the stability of the sections along the breach profiles to potential rotational and shear failure modes

Q8: What erosion relationship(s) does the model use? Are these predefined or can the modeler choose? Do these apply throughout all stages of breach development (from initiation growth through to open breach formation and widening)?

The model includes several equations to calculate erosion, these are:

Hanson (erosion eqn.)	Chen and Anderson (erosion eqn.)	Meyer-Peter-Müller (Sediment transport eqn.)
Yang (Sediment transport eqn.)	• 5- Visser (Bagnold-Visser) (Sediment transport eqn.)	

The model allows the user to define the erodibility coefficient and critical shear stress Hanson equation.

Q9: How does the model calculate flow through the breach (from initiation to open breach)?

The flow over the dam crest (Q_w) and through the breach (Q_b) is computed using the broad crested weir formula. The equations used in the model to compute these two components are as follows:

$$Q_w = C_d (L - B_b) H_w^{3/2}$$
$$Q_b = C_d B_b H_b^{3/2} \qquad (4)$$

where: C_d : Discharge coefficient

L : Crest length

B_b : Breach width

H_w : Total head over the crest

H_b : Total head over the breach

The values of the flow are corrected if the flow is submerged. Usually, this condition occurs after the reservoir water level has receded and there is no flow over the crest. It is therefore likely that only the value of the flow through the breach is going to be affected.

Q10: How does the model simulate the upstream boundary conditions (reservoir, river etc)?

The model can simulate the following upstream conditions:

- inflow hydrograph (i.e. flow vs time) which suits the routing of reservoirs
- Water level hydrograph (i.e. water level vs time) which can simulate the water level in a river but can also be used for reservoirs

Q11: Does the model simulate downstream conditions, and if so, does it take drowning of the breach into account?

Yes, the model simulates downstream conditions, and it takes drowning of the breach into account.

Q12: Does the model allow the user to investigate uncertainty in parameters/prediction?

Yes, the model allows the user to investigate uncertainty in parameters/prediction through the use of Monte Carlo simulations.

Q13: Does the model allow for forms of Monte Carlo analysis or inclusion of other means of parameter uncertainty?

Yes, the model allows user to run Monte Carlo simulations.

A.3.3 Modeller assumptions and model performance

Q14: What key parameters is the modeller required to define when setting up the breach model? (include any computational as well as material and structure definition parameters)

Key parameters that are the soil parameters such as the critical shear stress, erodibility coefficient, density, cohesion and friction. Other important parameters include time and space steps as they affect model performance.

Q15: Is guidance provided on selecting these parameters? If so, how and on what is that guidance based?

Yes, guidance is provided in the model user manual that it is based on typical ranges for soil parameters from the available literature and based upon the courant number for the selection of the time and space steps.

Q16: How sensitive is performance of the model to key parameter selection? Which are the key parameters?

Model is sensitive to the soil parameters, in particular, the critical shear stress and erodibility coefficient.

Q17: How was the model performance validated during development? Where appropriate, what data sets have been used – and how – to validate performance?

The model was not calibrated nor validated against a particular data set. Its performance was validated against various data sets at different scales, examples are:

- The EC IMPACT Project lab and field experiments (small and medium scales)
- The US Department of Agriculture (USDA) experiments (medium scale)
- Real dam failures such as Teton and Banqiao dams (large scale)

Q18: What is an indicative duration for a model simulation? (eg <1s; <30s; a few minutes; 5-10 minutes; 10-30 minutes; > 30 minutes; Hours).

The model typically takes a few minutes to run rather than seconds.

Q19: What are the model strengths and weaknesses?

EMBREA is probably the only model that can simulate the various erosion processes (i.e. surface, headcut and internal erosion). It allows the user to run single and Monte Carlo simulations. It can also simulate failure in several types of embankments (i.e. homogeneous, composite and layered). One weakness is the use of pool reservoir routing rather than dynamic reservoir routing.

A.3.4 The EMBREA model development history and availability

Q20: When was the model first developed? By whom?

Model was first developed in 2002 by Mohamed Hassan from HR Wallingford.

Q21: What language is the model developed in?

It was developed using the C++ language.

Q22: What platforms can the model run on?

Windows platform or via a web frontend.

Q23: How can a user access and run the model to undertake breach analyses?

The model has a free version that is available at www.dambreach.org. A Pro version is also available at the same website but at a fee for an annual license (currently £1500+vat). There is also a standalone version that runs on windows platform which is also available for one fee (currently £5000+vat), and it can be installed only on one computer.

Q24: Are there any costs to use the model?

Please see answer to the above question.

Q25: Is technical support available? How?

Yes for the Pro and standalone versions by email to mohamed.hassan@hrwallingford.com or via the www.dambreach.org website.

A.3.5 Anything else?

Key references for the EMBREA model:

- M.A.A. Mohamed, Embankment Breach Formation and Modelling Methods. PhD Thesis, The Open University, England, 2002
- M.W. Morris, Breaching of Earth Embankments and Dams. PhD Thesis, The Open University, England, 2011
- M.W. Morris, M. Hassan and C. Goff, EMBREA-Web: a tool for the simulation of breach through dams and embankments. In: INCOLD 2021 Symposium, New Delhi, India and Online, 2021

A.4 The Rupro model

A.4.1 An introduction to the Rupro model

The model aims at providing the breach hydrograph for dam break wave or levee breaching flooding calculation.

A.4.2 Modelling approach

The breach is defined by a control section circular or rectangular that evolves with time according to an erosion rate. The diameter of the circle increases while the rectangular breach deepens and widens.

In Rubar 20 (2 D shallow water equations solver), the breach model is considered as a structure and then is introduced between two edges respectively corresponding to the upstream toe and the downstream toe of the dike. In CastorDigue, upstream and downstream water elevations can be calculated using various simplified ways.

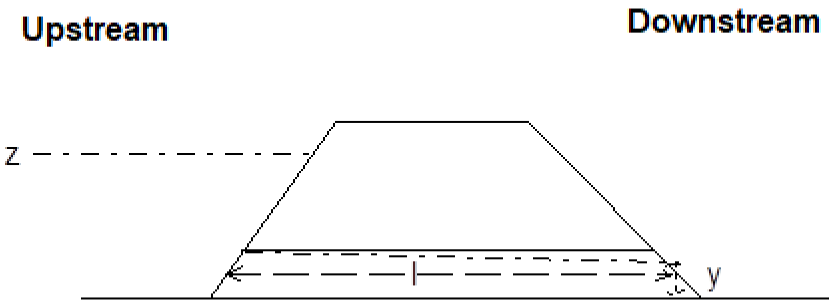
The software couples a hydraulic calculation determining the average hydraulic variables on the dike to a sediment transport calculation, which assumes the uniform erosion throughout the dike.

Hydraulic computation is carried out at free surface by solving the BERNOULLI equation with as downstream condition the elevation at the downstream edge. The pressure losses are either linear (MANNING-STRICKLER formula) or singular located on the upstream face of the dike.

The equation in *y* is:

$$z = y + \frac{S}{2l}\left(1 + \eta\frac{S^2}{S_e^2} + \frac{2gl}{K^2 R^{4/3}}\frac{S^2}{\overline{S}^2}\right)$$

where z is the elevation of the upstream edge, y the elevation at the downstream edge, S the section corresponding to y, S_e the upstream section, $\overline{S}$ the mean section equal to $(S + S_e)/2$, R the radius hydraulic, corresponding to $\overline{S}$, l the length of the erosion channel calculated at the center of the wetted section, therefore depending on y, η the head loss coefficient at the inlet of the erosion channel.



The software performs a simplified calculation of progressive erosion for a non-cohesive and supposedly homogeneous material. The sediment flow is determined from the MEYER-PETER-MULLER formula:

$$Q_s = \frac{8\sqrt{g}}{(\rho_s - \rho)\sqrt{\rho}}\left(\rho JR - 0.047 D_{50}(\rho_s - \rho)\right)^{3/2}$$

where Q_S is the sediment flow per unit of width (multiplied in this case by the mean wetted perimeter), ρ_S is the density of the solid material, ρ the density of the water, D_{50} the median diameter of the grains of the material, the frictional pressure drop calculated from a mean friction coefficient K. A variant calculates directly the erosion rate as a linear function of the difference between shear stress and critical shear stress; the equation can be then written as:

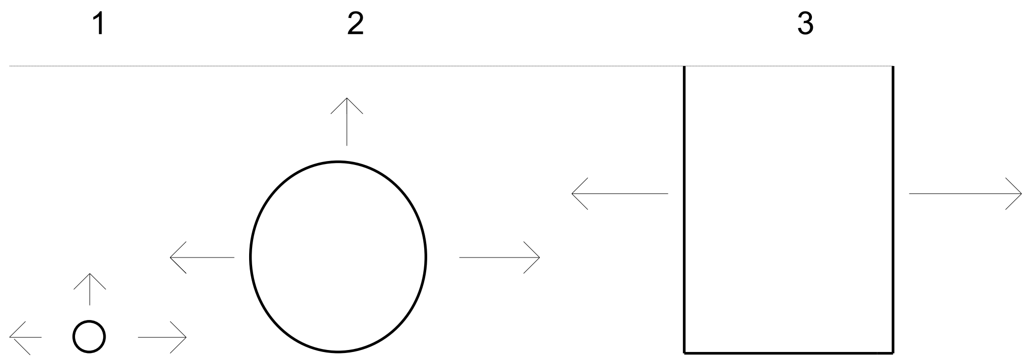
$$Q_s = \alpha\left(\rho gJR - 0.047\, gD_{50}(\rho_s - \rho)\right)$$

where α is the erosion rate coefficient and the critical shear stress is calculated from a not dimension coefficient value of 0.047.

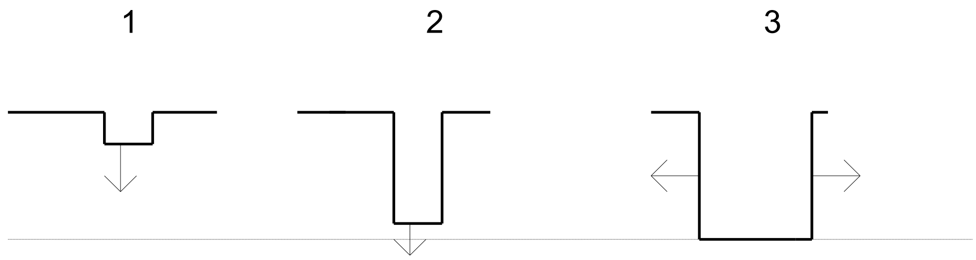
The dike is described by a trapezoidal type cross section defined by a crest width, a toe width, a crest and a toe elevation.

The software can simulate only 2 types of breaching:

- Erosion by piping; piping is schematized by a circular pipe which widens progressively (the low point of the circle remaining fixed) until its diameter reaches 2/3 of the height of the dike; it then collapses and the breach becomes rectangular then widens (without deepening)



- Submersion erosion; the breach is supposed to be rectangular; it deepens without widening until it reaches the substratum then widens to reach the maximum breach width unless the upstream will be emptied before



In both cases, the breach width is limited in CastorDigue by a value provided by the user and in Rubar 20 by the minimum between the length of the upstream edge and the length of the downstream edge.

The data set by the program are acceleration of gravity (9.81) and critical stress (0.047).

The user can give an erosion start time different from the start time of the general calculation (default value). In Rubar 20, one can also chain several successive breaches to represent more complex levee.

Because CastorDigue uses simplified assumptions, an uncertainty estimate is included in the software.

A.4.3 Modeller assumptions and model performance

The main parameters are grain diameter, porosity, density, initial breach dimensions, levee dimensions, friction coefficient, which are parameters easy to estimate.

The most sensitive parameters are, in the first place, the friction coefficient), the grain diameter and, for the overflow, the initial width of the breach.

CastorDigue is generally running in a few seconds while Rubar 20 runs a few minutes on a simple case of only one breach.

Rupro model is easy to handle but cannot represent complex processes. It can be trusted as first approach because validated many times in benchmarks against laboratory and field measurements.

A.4.4 The Rupro model development history and availability

Developed by Cemagref (now INRAE) in the 1980’s and continuously improved since. Encapsulated in software CastorDigue (simplified propagation), RubarBE (1D) and Rubar 20 (2D).

Written in Java or Fortran depending on software, it can be used on either platform.

Software and operating manuals are available on request at INRAE with eventual technical support paid to INRAE.

A.5 The WinDAM C model

1.1	**Model description:** *A simple overview…*	
Q1	*What does the model predict?*	The four essential functions of the software are: 1. Perform level surface routing of a hydrograph through a reservoir with or without flow over the top of dam 2. Predict performance of an overtopped homogeneous earth embankment with or without vegetation (grass) or riprap protection on the downstream face. Includes an estimate of extent and rate of erosion and, if a breach is predicted, the estimated breach outflow hydrograph 3. Predict performance of a homogeneous earth embankment having an existing horizontal flow path through the embankment. Includes an estimate of the erosion rate and, if a breach is predicted, the estimated breach outflow hydrograph. (Subsequent discussion will focus on this function of the software) 4. Predict the potential for breach of up to three earth or vegetated earth spillways for conditions where embankment breach is not being evaluated. The spillway evaluation does not provide prediction of breach outflow.
Q2	*Why was the model developed?*	During the last half of the 20th century, the USDA assisted in the design and construction of approximately 12,000 flood control dams across the United States. Most of these were earthen embankment dams and many have reached, or will soon reach, the end of their planned service life. Sedimentation, rodent activity, woody vegetation on the embankments, development in the downstream floodplain, and other issues associated with aging have increased the concerns related to the performance of these embankments during extreme events resulting in overtopping or flow through the embankment (internal erosion). These concerns resulted in the development of a research program focused on improving the understanding of the processes governing embankment performance and applying that understanding to develop tools to better predict that performance. This program has included physical models of embankments subjected to overtopping or internal erosion and the attempt to quantify the observed performance through application of fundamental principles in simplified numerical models. The initial attempt to quantify the processes observed in the relatively large-scale physical models resulted in the SIMBA (SIMplified Breach Analysis) model. This model was a research tool used to evaluate the potential for reproducing the observed performance using simplified dominant process models. WinDAM was developed to allow more general application of the resulting computational models outside of the research environment. However, it should be recognized that these computational models are simplified and represent a "first cut" at quantifying a subset of potential breach conditions. References describing the research program and development of SIMBA and WinDAM may be obtained through the USDA ARS Hydraulic Engineering Research Unit. https://www.ars.usda.gov/plains-area/stillwater-ok/hydraulic-engineering-research/.
1.2	**Modelling approach:**	

Q3	*What broad approach does the model take for simulating breach development?*	The approach to modeling the erosion/breach was to attempt to represent (quantify) the dominant physical processes in as simple a fashion as possible. Focus is on fundamental processes and their quantification in terms of measurable parameters. In implementing this approach, a number of simplifying (limiting) assumptions are made for computational purposes. These limitations are consistent with the physical model tests that were conducted to increase understanding of the overall process. Assumptions include: 1) A homogeneous earthen embankment in a rectangular valley with inerodible boundaries; 2) An existing horizontal flow path through the embankment that is of sufficient size to generate turbulent flow; and 3) Stepwise steady state conditions. These and other key simplifying assumptions are discussed further below.
Q4	*What are the advantages & disadvantages of this approach?*	Advantages of this approach include: 1. Outputs are relatively easy to interpret 2. Minimal computational time and resources are required 3. Embankment may be represented by measurable material parameters and other inputs are relatively straightforward. Disadvantages of this approach include: 1. The overall physics tend to be oversimplified 2. Scope of application in "real world" is limited by the simplifying assumptions 3. Interaction of the embankment with the foundation or abutment is not considered.
Q5	*How does the model predict the IE growth process? Is the process/shape predefined or free format?*	The model assumes that the dominant processes are expansion of the initial flow path (conduit) due to hydraulic shear and the potential for a headcut to develop at the outlet of the conduit onto the downstream slope of the embankment. The initial flow conduit is assumed to be rectangular, horizontal, and of sufficient size to generate turbulent flow (stress computed with Manning equation). Initial conduit dimensions and location are user inputs. The conduit is assumed to remain rectangular and expand equally in all directions unless/until an inerodible boundary is encountered in one of the directions of expansion. It is assumed to remain horizontal during expansion. The downstream slope of the embankment is assumed to have negligible surface protection in the area of the flow conduit exit allowing a headcut to form at the point of exit, deepen, and progress upstream effectively shortening the conduit.
Q6	*Does the model predict roof instability above the internal erosion, followed by collapse and subsequent open breach formation? How?*	The roof of the conduit is assumed to remain in place so long as any portion of the conduit is flowing full. The roof may be considered failed such that redevelopment of conduit flow cannot occur once the flow becomes partially full throughout the conduit and the conduit width becomes greater than twice the remaining distance between the top of the conduit and top of dam. Therefore, conduit roof collapse is not a factor in computations for most scenarios. Since the conduit is assumed rectangular throughout, widening and downward erosion of the breach area due to free surface flow through the breach area continue to be computed based on average hydraulic shear stress on the boundary in the same fashion as for conduit flow. The assumption implicit in this is that material from mass failure of the sides above the level of flow will be immediately washed away. In the case of the headcut progressing upstream into the reservoir, additional widening is that associated with headcut advance through the upstream slope of the embankment.

Q7	*How are the open breach formation and widening stages simulated? Are the processes and breach shape predefined or free format? What geotechnical stability analyses are performed?*	The breach shape is predefined as rectangular with vertical sides. As indicated above, widening is assumed to be governed by average hydraulic stress. No additional geotechnical analyses are performed relative to bank stability.
Q8	*What erosion relationship(s) does the model use? Are these predefined or can the modeler choose? Do these apply throughout all stages of breach development (from initiation growth through to open breach formation and widening)?*	The primary relation considered to govern the erosion process is the excess stress detachment rate relation. That is, detachment rate in volume per unit area per unit time is equal to the product of the detachment rate coefficient (material property) and the difference between the applied erosionally effective hydraulic stress and the critical stress (material property). In applying this relation, the erosionally effective stress is the spatially averaged stress over the wetted perimeter of conduit or breach area. This is recognized as a significant simplification and is the result of the approximating assumption of a horizontal conduit of constant cross section. This approach to computations is used whether the conduit is flowing full, partially full, or is free surface over its entire length. Two options are available for predicting headcut advance. These are an energy based model designated as the Temple/Hanson model and a stress based mass failure model designated as the Hanson/Robinson model. These models are discussed with appropriate referencing by Hanson et al. (2011). Hanson, G. J., D. M. Temple, S. L. Hunt, and R. D. Tejral. 2011. Development and characterization of soil material parameters for embankment breach. *Applied Eng. in Agric.*, Vol. 27(4):587-595.
Q9	*How does the model calculate flow through the breach (from initiation to open breach)?*	Flow through the breach area is computed using the previously stated assumptions of a rectangular conduit or flow channel and step wise steady state conditions. The hydraulic control for discharge computations is assumed to be at the current location of the headcut (initially the outlet of the conduit) and backwater computations are performed to determine whether the flow is free surface, partially full conduit flow, or full conduit flow. Energy losses associated with the conduit entrance are considered negligible. Critical flow conditions are assumed at the hydraulic control unless external tailwater conditions indicate a greater flow depth. Hydrostatic pressure conditions are assumed throughout. A Manning's n value of 0.02 is assumed. If the headcut is computed to have progressed into the upstream embankment face, energy losses are from the reservoir to the hydraulic control are considered negligible. The hydraulic control for purposes of discharge and stress computations remains at the most upstream position computed for the headcut even if the headcut is washed out (base of channel or conduit is base of dam).
Q10	*How does the model simulate the upstream boundary conditions (reservoir, river etc)?*	The upstream boundary is considered to be a reservoir with a defined stage storage relation. A level surface routing procedure is used to determine reservoir water surface elevation considering inflow and outflow through uncontrolled spillways as well as flow through the breach area. As previously noted, step wise steady state conditions are assumed, and erosion rates are considered constant throughout the time step. Discharge is computed as previously noted using the eroded geometry at the end of the time step.
Q11	*Does the model simulate downstream conditions, and if so,*	Downstream conditions may be represented by a relation between total discharge (including spillways) and tailwater elevation. This

	does it take drowning of the breach into account?	tailwater may result in computed submergence of the conduit or subcritical flow at the specified hydraulic control section.
Q12	*Does the model allow the user to investigate uncertainty in parameters/predictio n?*	No special provision is made for investigation of parameter sensitivity. Data sets may be easily modified, and impact of the modifications compared through plots or tables.
Q13	*Does the model allow for forms of Monte Carlo analysis or inclusion of other means of parameter uncertainty?*	No provision is made for direct application of Monte Carlo or other statistical analysis.
1.3	**Modeller assumptions and model performance**	
Q14	*What key parameters is the modeller required to define when setting up the breach model? (include any computational as well as material and structure definition parameters)*	In addition to the description of the inflow, the reservoir geometry, tailwater, embankment geometry, and initial dimensions and location of the flow conduit, the key parameters are those describing the embankment. Specifically, these are: 1. The detachment rate/erodibility coefficient expressed in volume per unit area per unit time per unit of stress (ft/h)/(lb/ft^2) 2. Critical shear stress (lb/ft^2). If the Temple/Hanson headcut advance model is used, a headcut advance rate coefficient is required (ft/h)/(ft/s$^{1/3}$). Alternately, if the Hanson/Robinson model is selected, the additional parameters required are: 1. The undrained shear strength of the material (lb/ft^2) 2. The total unit weight of the material (lb/ft^3).
Q15	*Is guidance provided on selecting these parameters? If so, how and on what is that guidance based?*	A substantial body of literature is available related to the excess stress relation. Hanson et al. (2011) contains a discussion of material parameters with appropriate referencing (Question 8). This includes guidance in determining or measuring values of the material parameters. Other pertinent references include: Hanson, G. J., and K. R. Cook. 2004. Apparatus, test procedures, and analytical methods to measure soil erodibility in-situ. *Applied Eng. in Agric.* 20(4):455-462. Briaud, J.-L., I. Shafii, H.-C. Chen, and Z. Medina-Cetina. 2019. *Relationship Between Erodibility and Properties of Soils*. Transportation Research Board.
Q16	*How sensitive is performance of the model to key parameter selection? Which are the key parameters?*	Sensitivity is scenario-dependent, and modelers are advised to investigate uncertainty for the scenario in question. Some examples, but by no means rules: • An order of magnitude increase (×10) in the detachment rate coefficient may bring about an approximate two-fold increase in maximum breach discharge • In simulations where the maximum applied stress and critical shear are of similar magnitude, small changes in critical shear stress determine whether erosion is predicted, often breach versus no breach. Therefore, over a small range, sensitivity to critical shear stress approaches infinity.

Q17	*How was the model performance validated during development? Where appropriate, what data sets have been used – and how – to validate performance?*	To date, verification of the internal erosion portion of the WinDAM model has been very limited. Additional verification is needed. The internal erosion tests underlying model development are described by Hanson et al. (2010) and Ali et al. (in press). Hanson, G. J., R. D. Tejral, S. L. Hunt, and D. M. Temple. 2010. Internal erosion and impact of erosion resistance. Proc. 30th U.S. Society on Dams Conf. Sacramento, CA. pp 773-784. CD-ROM. Ali, A. K., S. L. Hunt, and R. D. Tejral. (in press). Embankment breach research: observed internal erosion processes. *Trans. ASABE*.
Q18	*What is an indicative duration for a model simulation? (eg <1s; <30s; a few minutes; 5-10 minutes; 10-30 minutes; > 30 minutes; Hours).*	ca. 1 to 10 s.
Q19	*What are the model strengths and weaknesses?*	Key model strength is simplicity and reliance on fundamental principles. Key model weakness is oversimplification of processes and limited validation.
1.4	**Model development history and availability**	
Q20	*When was the model first developed? By whom?*	The first software named WinDAM came to be in the mid-2000s; however, many of its algorithms have their roots in previous USDA models, eg SITES. WinDAM C is developed cooperatively by United States Department of Agriculture-Agricultural Research Service (ARS), USDA-Natural Resources Conservation Service (NRCS), and Kansas State University. WinDAM is under phased development with expectations for additional modules to be added.
Q21	*What language is the model developed in?*	English and in U.S. Customary units.
Q22	*What platforms can the model run on?*	Microsoft Windows
Q23	*How can a user access and run the model to undertake breach analyses?*	Administrative privileges are required to install the software. For guidance operating WinDAM see https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=NRCSEPRD997406. Model may also be found at https://www.ars.usda.gov/plains-area/stillwater-ok/hydraulic-engineering-research/docs/technology-transfer/.
Q24	*Are there any costs to use the model?*	No.
Q25	*Is technical support available? How?*	Manuals available here: https://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/water/manage/hydrology/?cid=nrcseprd997406#downloadManuals. For the purposes of the internal erosion models evaluation project, support is available from USDA ARS Hydraulic Engineering Research Unit. For general support users are referred to the USDA Natural Resources Conservation Service. See https://www.nrcs.usda.gov/wps/portal/nrcs/detail/national/water/manage/hydrology/?cid=nrcseprd997406#contacts. Training has been provided cooperatively by USDA-ARS, USDA-NRCS, Kansas State University, and collaborators over the years through

		workshops, webinars, and other technology transfer means. Those interested in such training are advised to reach out to the USDA-ARS Research Leader, Sherry Hunt (Sherry.Hunt@usda.gov), and/or USDA-NRCS Hydraulic Engineer, Karl Visser(Karl.Visser@usda.gov). Additional publications related to the development of WinDAM may be found at: https://www.ars.usda.gov/plains-area/stillwater-ok/hydraulic-engineering-research/.

B Phase 0 Modelling Test Case

B.1 Phase 0 Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	T0_Hypothetical_Blind.xlxs
Analysis & comparison of modelling results	Phase0_ModellingComparison_20_09_01.xlxs

B.2 Test Case Description

Full details of this test case description can be found in the T0_Hypothetical_Blind.xlxs spreadsheet. Separate worksheets provide details of the site in general, reservoir storage, inflow hydrograph and initiating conditions. The test case was based upon a real dam and reservoir, but with some details simplified.

Figure B.1: Aerial view of reservoir used as the basis for the Phase 0 test case

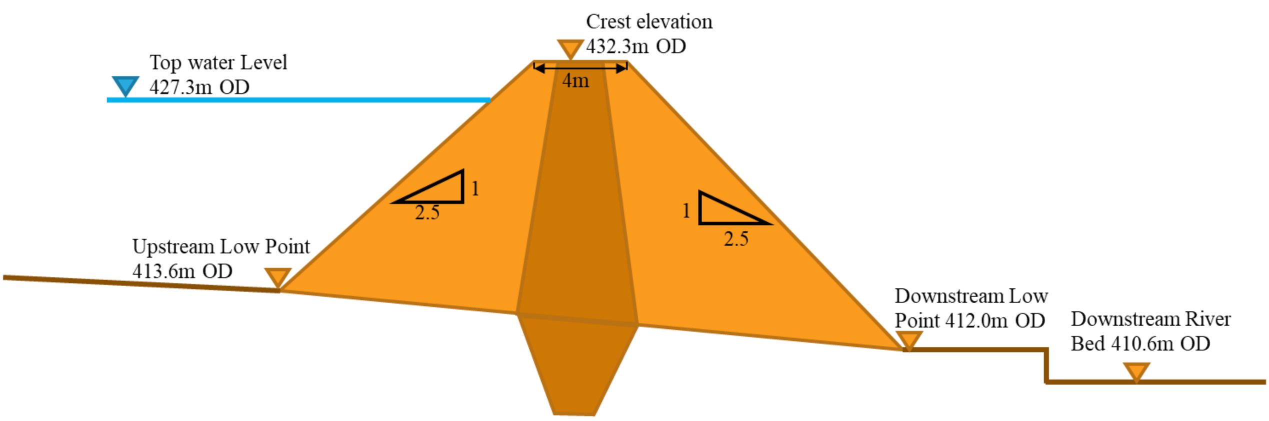


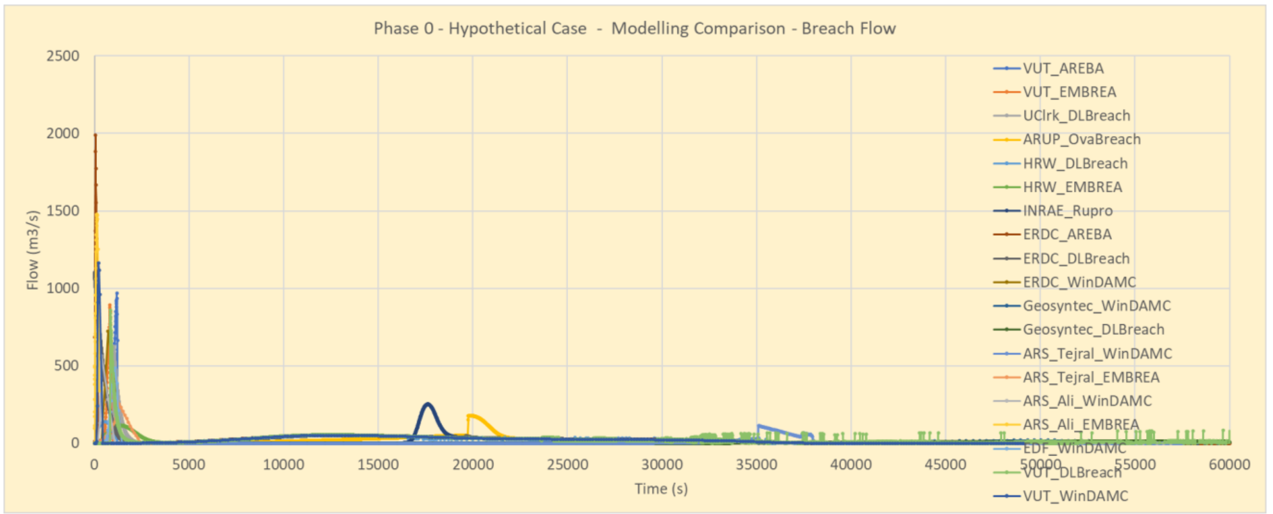
Figure B.2: Simplified schematic of test case dam

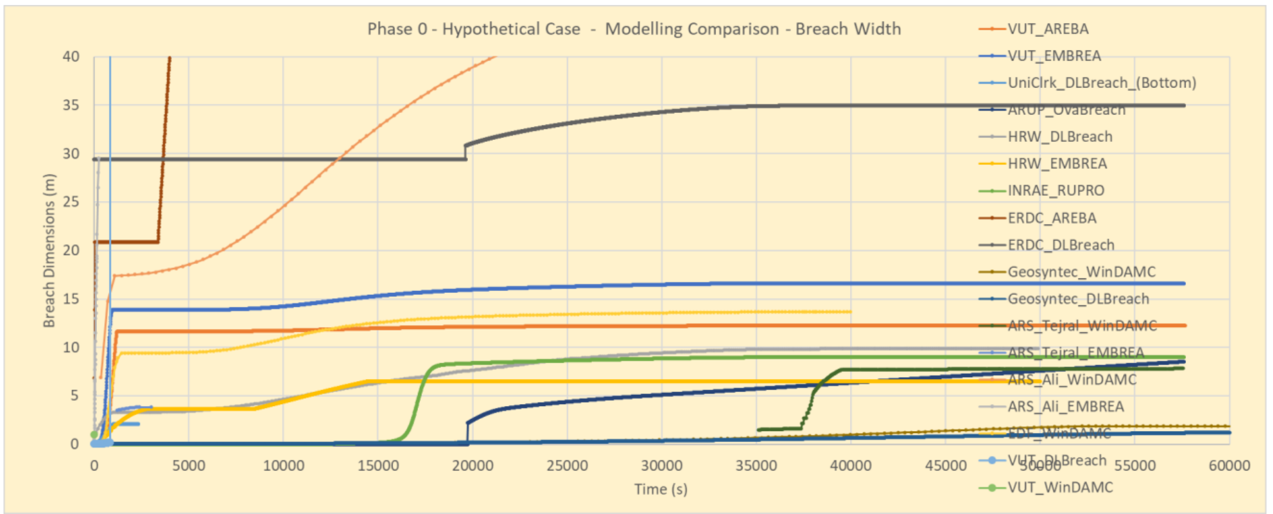
B.3 Phase 0 – Modeller Assumptions

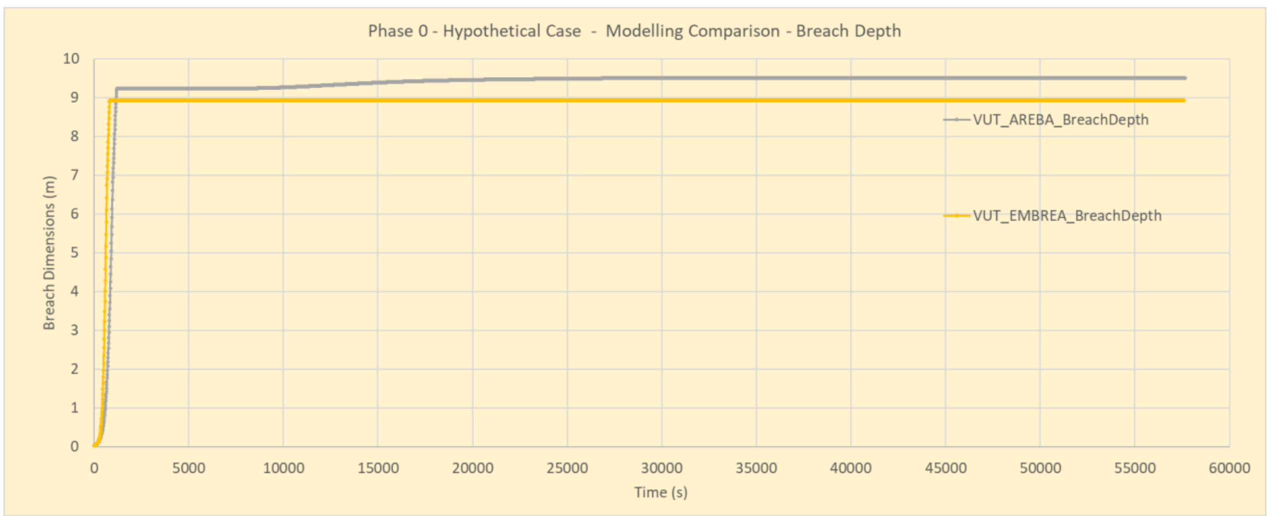
Table B.1: Phase 0: Modeller Assumptions

Models & Modellers:		Structure		Initiation			Soil Parameters								Flow	Computational					Reported Problems or observations
	Variables	Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	**Ron Teiral & Ali Abdelfatah**																				
	Tejral_WinDAMC	WinDAM C does not model zoned structures. However, Temple/Hanson headcut model allows user to input erodibility and advance rate. These inputs are intended to describe a single material. I assumed core would control rate of breach conduit width and height, while headcut migration rate would be governed moreso by fill material. Therefore, selected kd is based on core material, while Advance Rate *C* is based on fill. Value of *kd* (and *C* by correlation) followed from Hanson et al. (2011) Development and characterization of soil material parameters for embankment breach. Full geometry of dam, i.e. not core, was used.	412.00	0.03×0.03 rect	200	at time 0 of provided inflow hydrograph	0.14	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	3	0.02	0.01 hr	#N/A	#N/A	#N/A	0.004 s^-2/3	Stage-area table had to be modified slightly as WinDAM requires area to increase with elevation. Stage-area sampled to reduce to fewer than 50 points. Provided inflow hydrograph exceeded input time-step limitation. It was sampled at 0.1-hr increment.
	Tejral_WinDAMC Imperial units		1351.70 ft	0.1×0.1ft rect	656.00 ft		0.08 (ft/h)/(lb/ft^2)							0.06 psf						14 (ft/h)/(ft/s^1/3)	
	Tejral_EMBREA Lite	I assumed core to dominate breach process. Because EmbreaLite does not model zoned embankments, I used core properties to inform inputs.	412.80	0.1	#N/A	Time 0.	0.14	1630	20	26	0.62	#N/A	#N/A	3	0.02	2 s	2	Minimum flow	#N/A	#N/A	Because EmbreaLite does not model spillway flow, I elected to set the reservoir water surface elevation at the maximum associated with routing the inflow hydrograph through the spillway. Core material is assumed to control the breach process. Geometry and material properties of core were used. The simulation reported is a prediction of breach with maximum discharge of 300 m3/s at 1100 s. An excess of 30 simulations were run. The model was found to be highly sensitive to minimum flow and length tolerance.
	Ali_WinDAMC	The embankment is homogenous. I used Headcut advance model: Hanson/Robinson Stress Model. cu = 0 Assuming fill	412.50	0.1x0.1ft rect	61	0	100	N/A	N/A	N/A	N/A	N/A	N/A	0	5	0.1 h	N/A	N/A	100	N/A	
	Ali_WinDAMC, Imperial units		1353.41	0.1 X0.1 ft	200	0	56.2							0	5	0.1 h					
	Ali_EMBREA	Uses headcut Assumes fill controls the process. The embankment is homogenous. I selected a sediment erosion equation: Original Hanson (Cohesive)	412.00	0.03m Square	N/A	0	100	21	2	30	0.38	0.43		0.2	0.04	2 sec.	2		N/A	0.03	
ARUP	**Veronika Stoyanova**																				
	ARUP OvaBreach	Considers core and fill separately: Clay core >							20		0.5			5				???			
		Fill >							0					20					1.00E-05	0.00225	
HRW	Mohamed Hassan																				
	DL BREACH	Assumed homogeneous, using shell material properties	???	0.03		Multiple runs - peak is timing dependent	1.85	2140	0	30	0.39			0.1	???	???		???			Issue with storage volumes below 414.5mAD. Assumed dead storage below.
	EMBREA	Assumed homogeneous, using shell material properties	???	0.03		Multiple runs - peak is timing dependent	1.85	2140	0	30	0.39			0.1	???	???		Moves with critical flow section location			Issue with storage volumes below 414.5mAD. Assumed dead storage below.
ERDC	**Ghada Elithy**																				
	TUD AREBA	Assumed homogeneous, using shell material properties		0.1			53						2.00E-05								
	Rupro																				
	WinDAMC	Homogeneous - Non cohesive fill D50 30mm		0.1	1/3rd from left abutment		53							1.4		n/a	n/a				Hanson/Robinson stress model used
	DL BREACH	Core + fill ??		0.1 width			53		50 (core)												
Brno Uni	**Stanislav Kotaska**																				
	TUD AREBA	Homogeneous;	412.00	0.03		at start of Flood	6	2100	2	30	0.37	0.42	1.86 E-5	0.2	0.04	10s	n/a		n/a	n/a	
	EMBREA	Homogeneous;	412.00	0.03		at start of Flood	6	2100	2	30	0.37			0.2	0.04	10s	1		n/a	n/a	Long run time and instability with smaller time step (2s)
	WinDAM	Homogeneous; Headcut in toe of dam	412.00	0.03	80 m from left abutment	at start of Flood	173	2100	2	30	0.37			0.0012	0.04	0.5s	n/a			400 psf	Hanson/Robinson stress model used
	DL BREACH	Homogeneous;	412.00	0.03		at start of Flood	6	2100	2	30	0.37			0.2	0.04	0.08	n/a	one side	n/a	n/a	
Geosyntec	**Al Preston**																				
	WinDAM	Assumes core materials for whole embankment		0.03			0.14							1.9							
	DL BREACH	Assumes core materials for whole embankment		0.03			0.14							1.9	0.02						
INRAE	**André Paquier**																				
	Rupro	Core is ignored. Circular pipe - invert kept stationary		D90 (		At Peak of Flood															Smaller initial diameter delays process and may cancel erosion; Wider (x10) makes breach almost instant. But both have only a small effect on the peak flow. Alternative calc using just clay core gives similar results.
UniClrk	**Weiming Wu**																				
	DL BREACH	Permitted erosion to -0.8m but not needed	413.56				8		25	28	0.23			0.15	0.016						
EDF	**Pierre Squillari (Geophy)**																				
	TUD AREBA																				
	ARUP OvaBreach																				
	Rupro																				
	WinDAM	Simulated whole dam using core properties?		0.01			17.68	20						0	0.03						
	DL BREACH																				
	EMBREA																				

B.4 Phase 0 Modelling Results







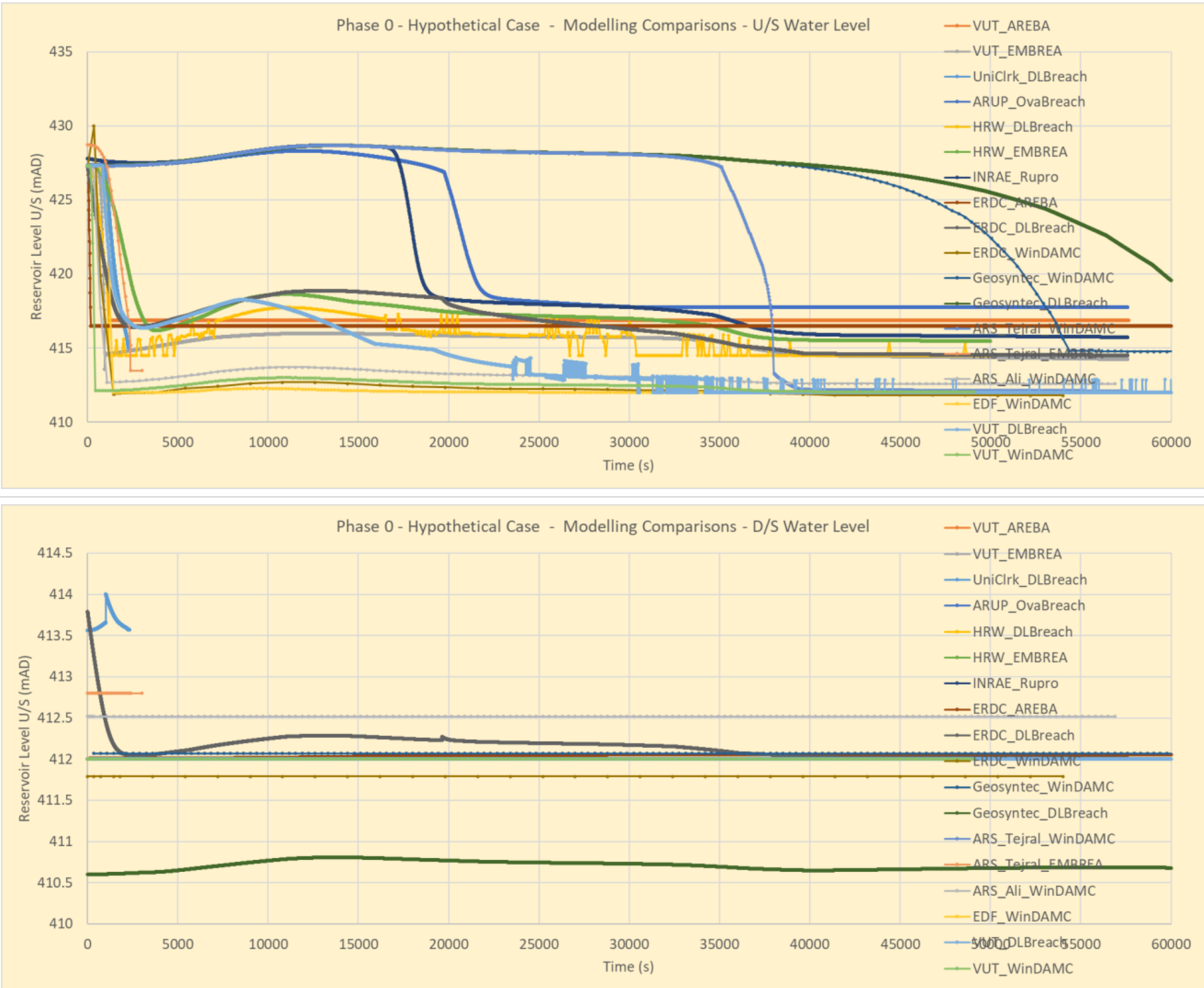


Figure B.3: Phase 0: All modelling results

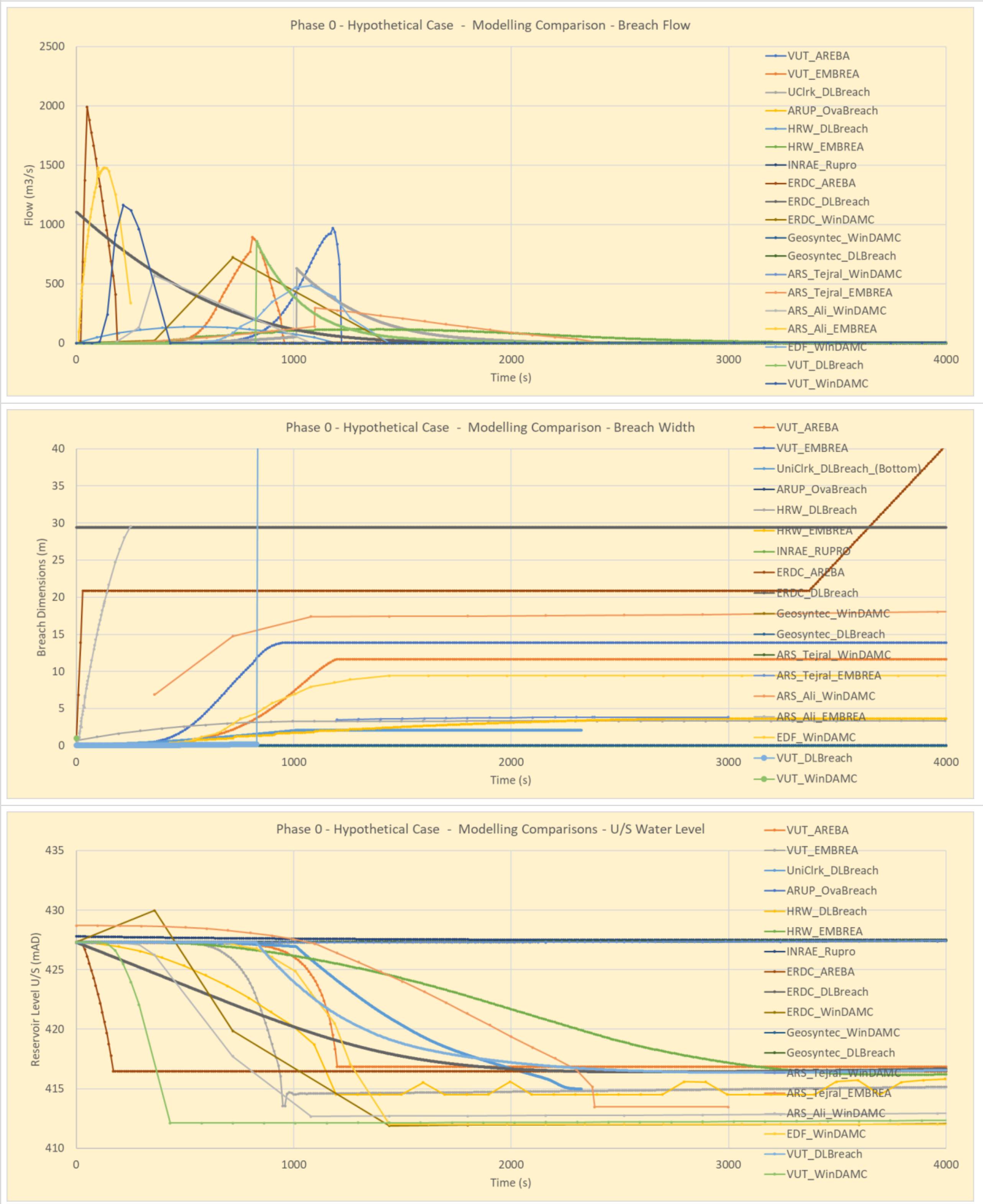


Figure B.4: Phase 0: All modelling results – focus on t=0-4000 s

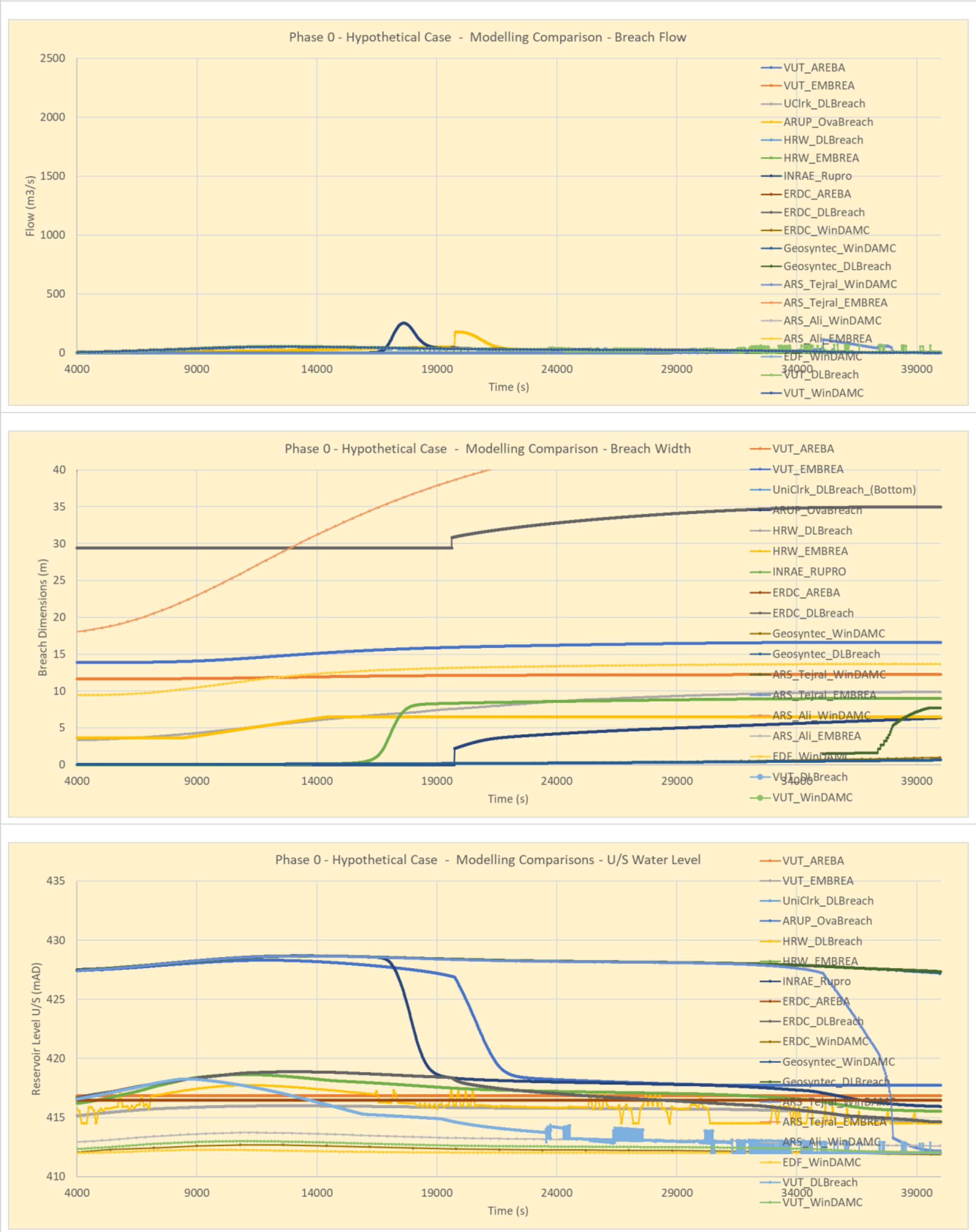


Figure B.5: Phase 0: All modelling results – focus on t=4000-39000 s

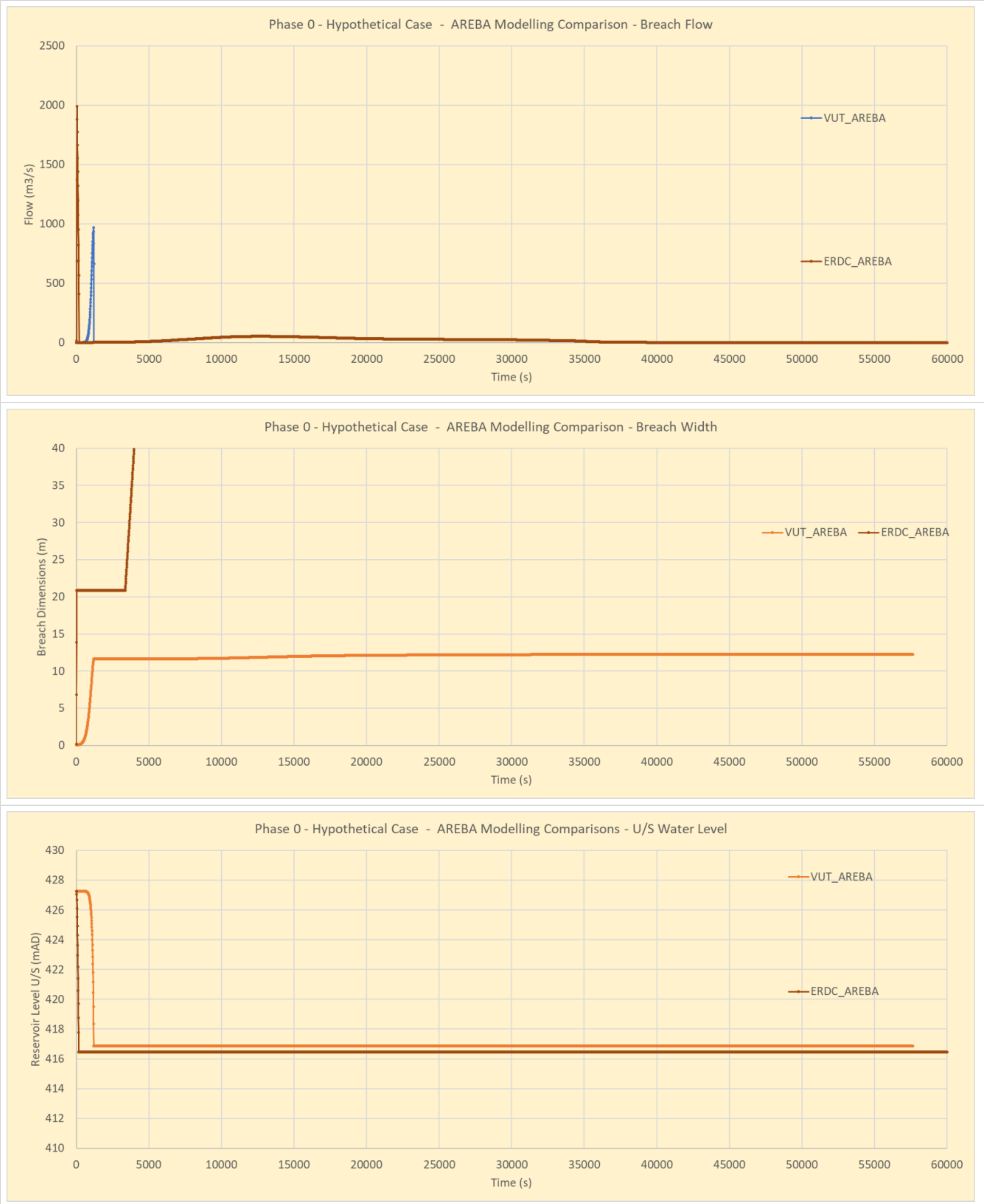
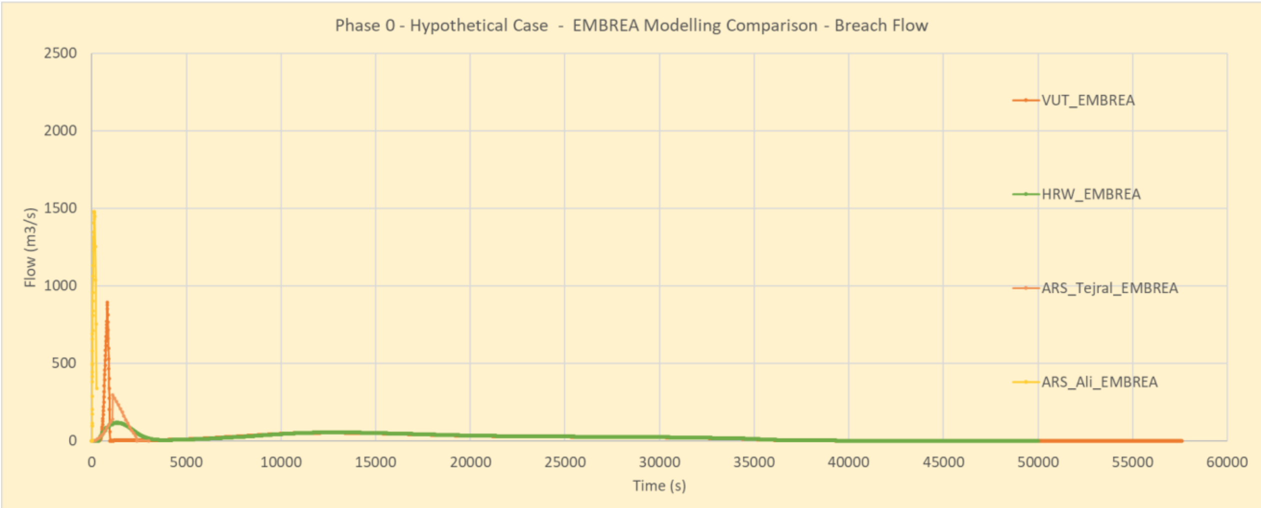
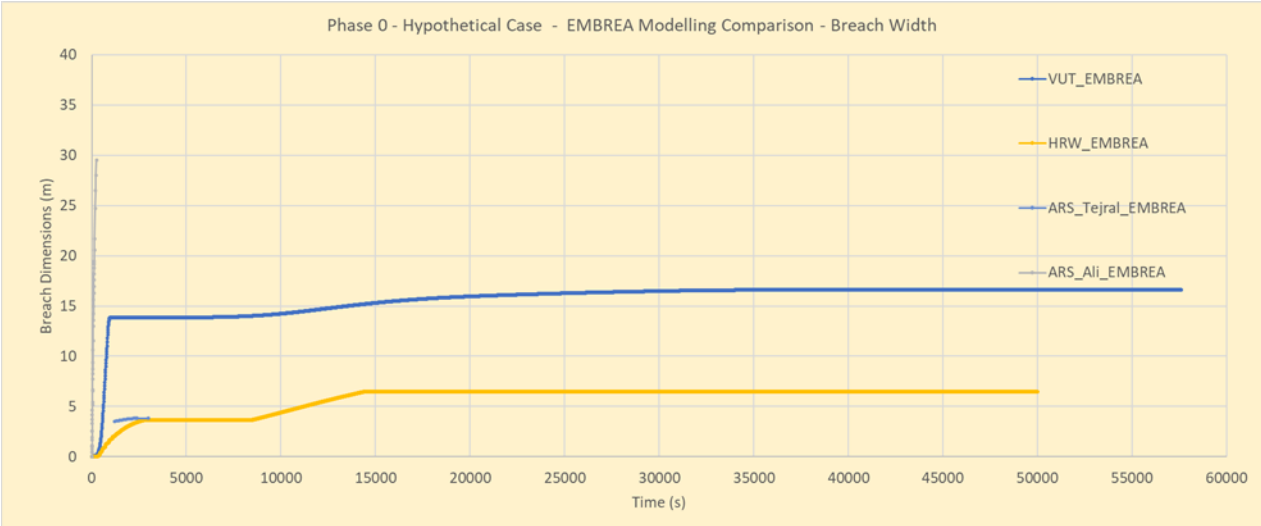


Figure B.6: Phase 0: Modelling results using AREBA





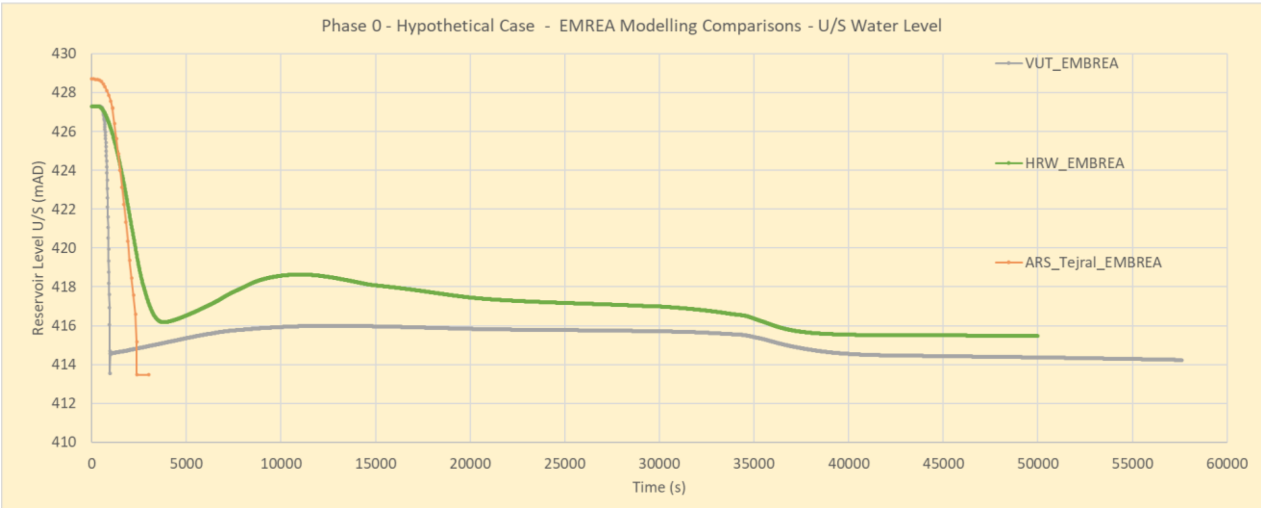


Figure B.7: Phase 0: Modelling results using EMBREA

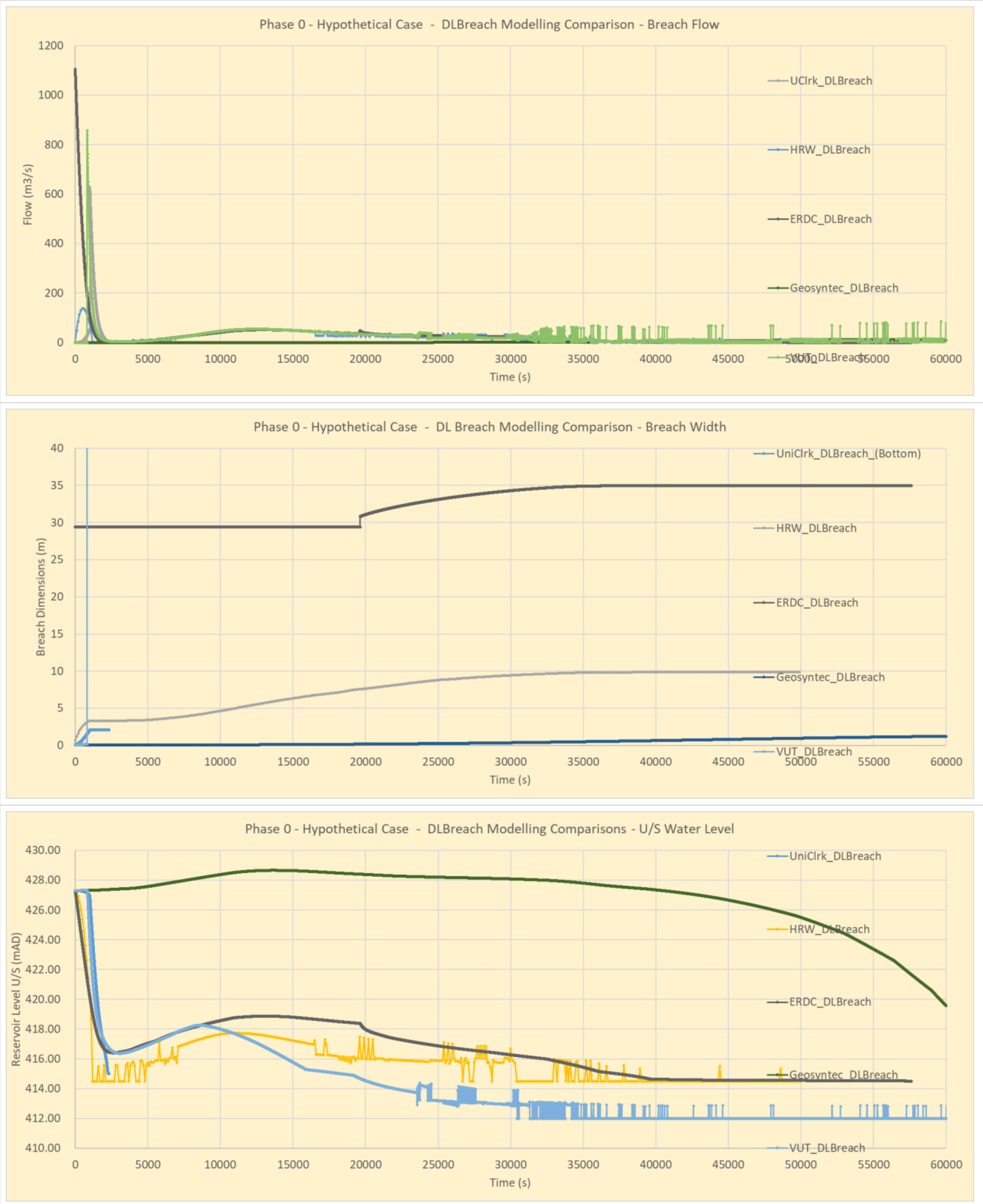


Figure B.8: Phase 0: Modelling results using DLBreach

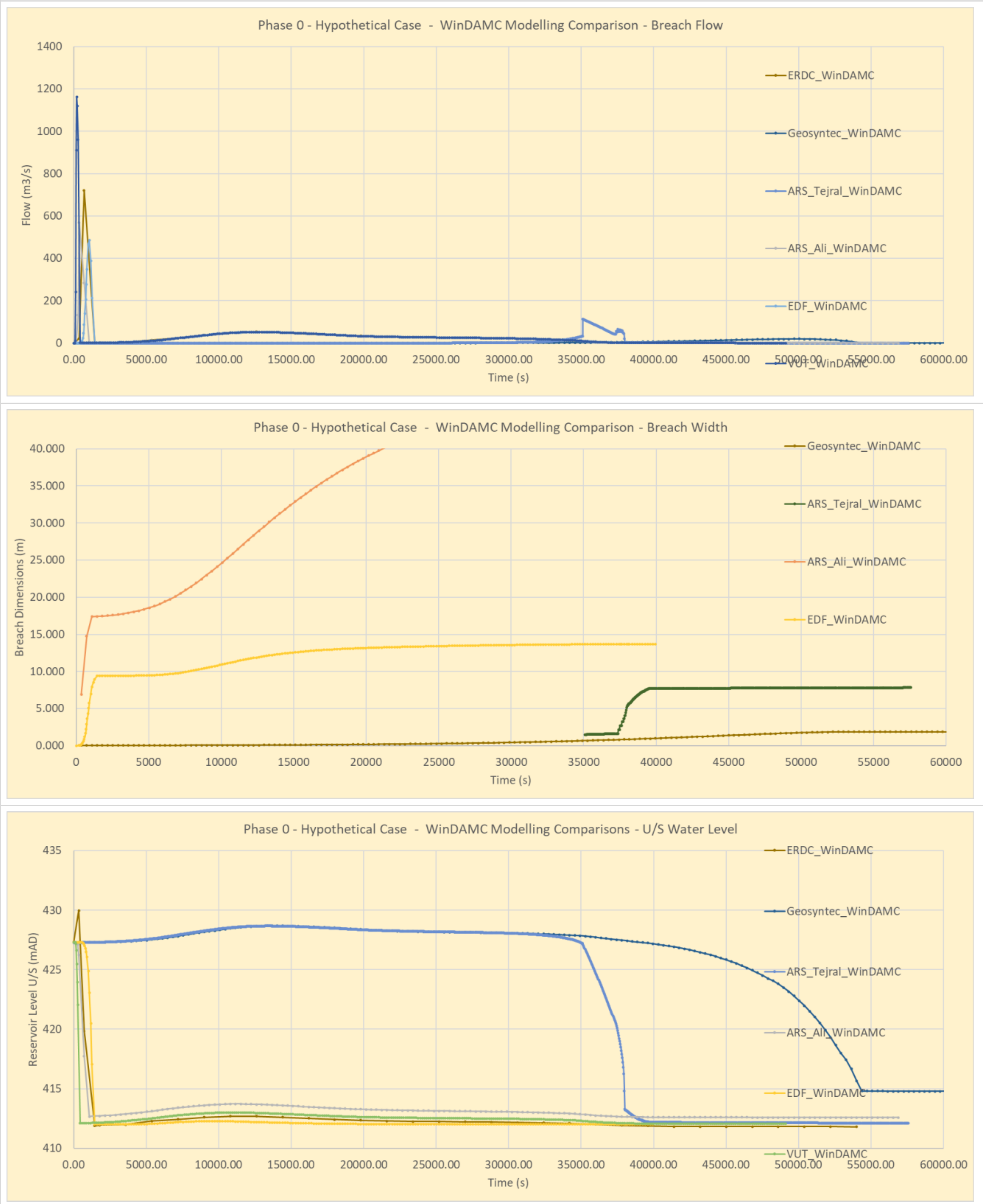


Figure B.9: Phase 0: Modelling results using WinDAM C

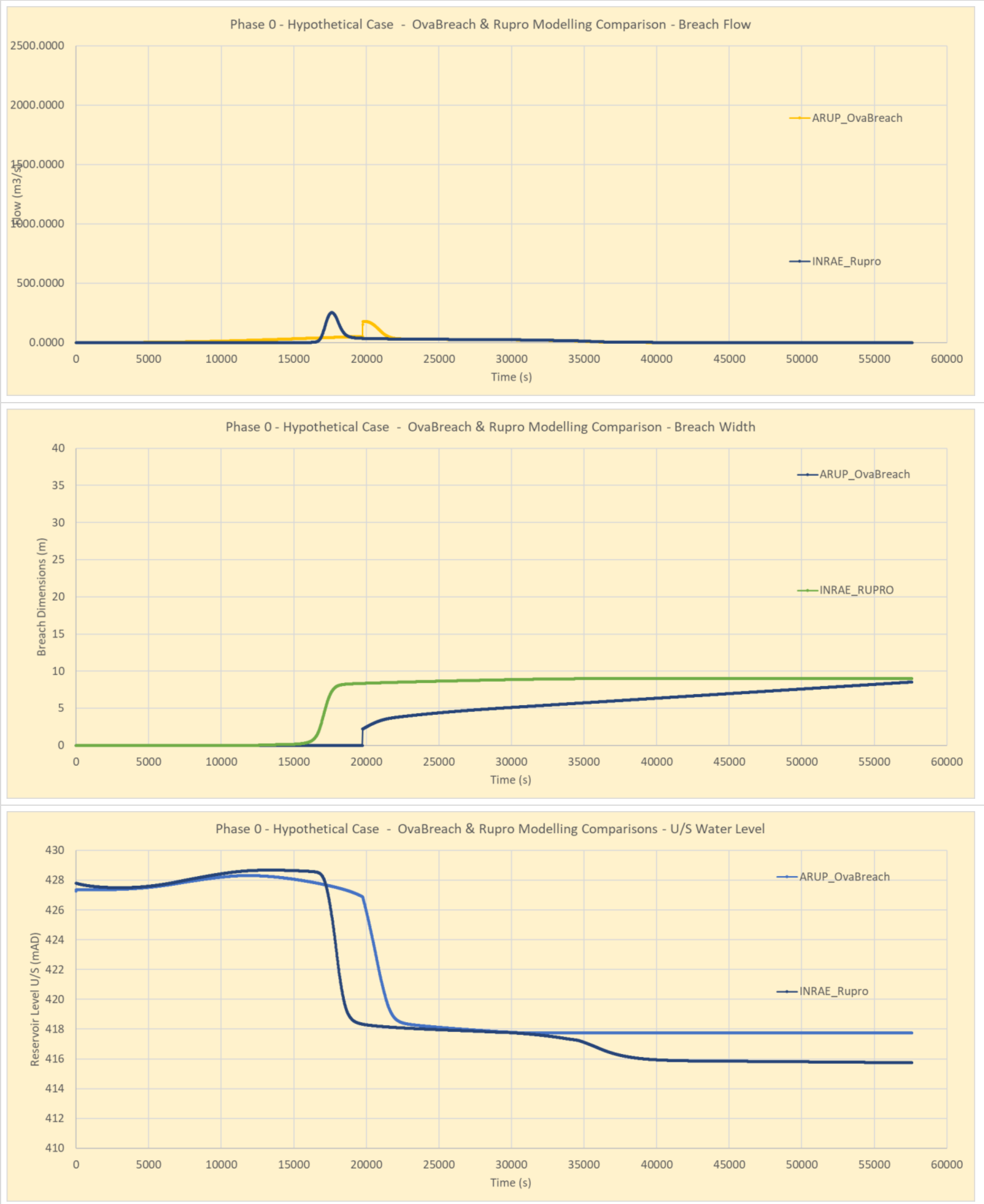


Figure B.10: Phase 0: Modelling results using OvaBreach & Rupro

C Phase 1 – Modified Hypothetical Test Case

C.1 Phase 1 – Modified Hypothetical Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	T1_Modified_Hypothetical_Blind_v3_mwm.xlsx
Analysis & comparison of modelling results	Phase1_ModellingComparison_ModHypothetical_21_01_07.xlsx

C.2 Test Case Description

This 'Modified Hypothetical' test case was based upon a real dam and reservoir, but with some details simplified. The test case differed from the previous 'hypothetical' test case by further simplification of parameters including:

- Simplified, homogeneous structure with flat foundation level
- Simplified soils description
- Simplified reservoir bathymetry (at lower level)
- Simplified inflow hydrograph (steady inflow)
- Defined initial pipe flow dimensions
- Assumed no downstream water level effects on breach process (i.e. no drowning).

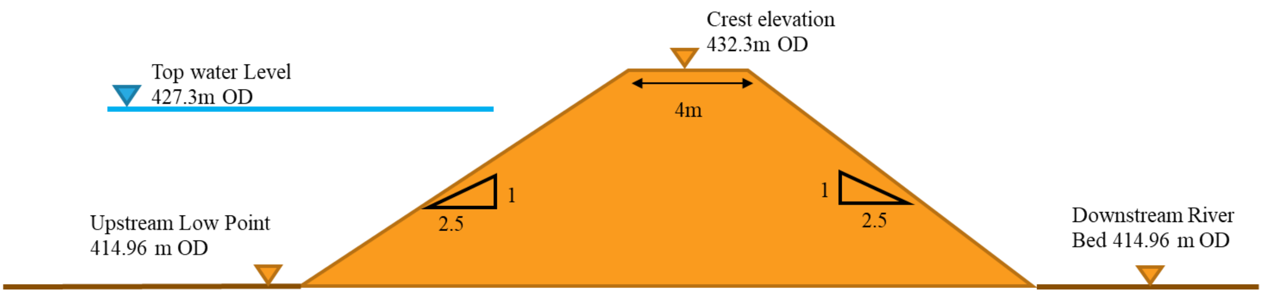
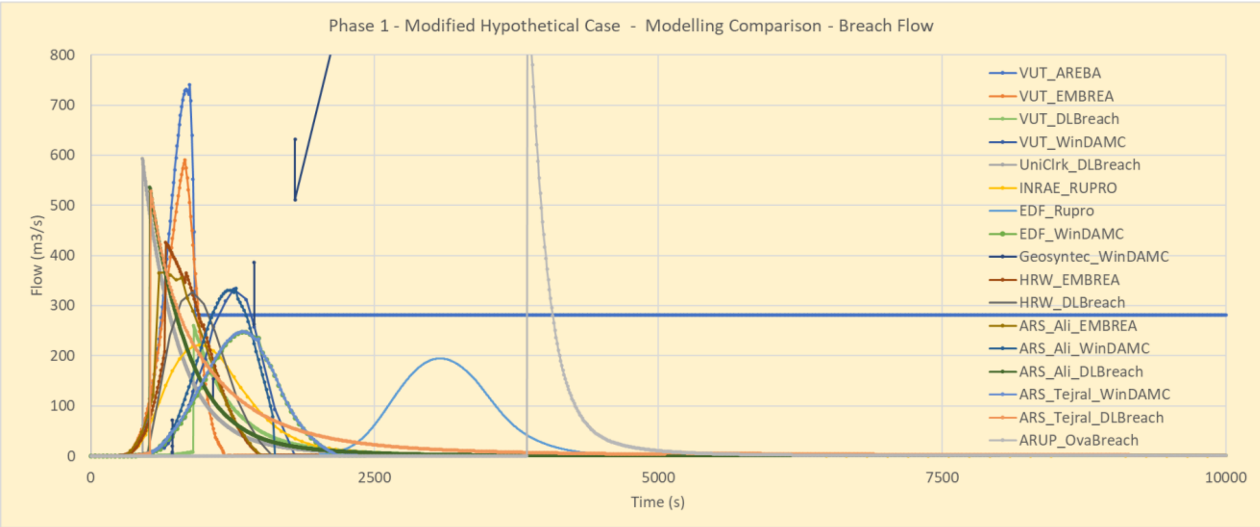


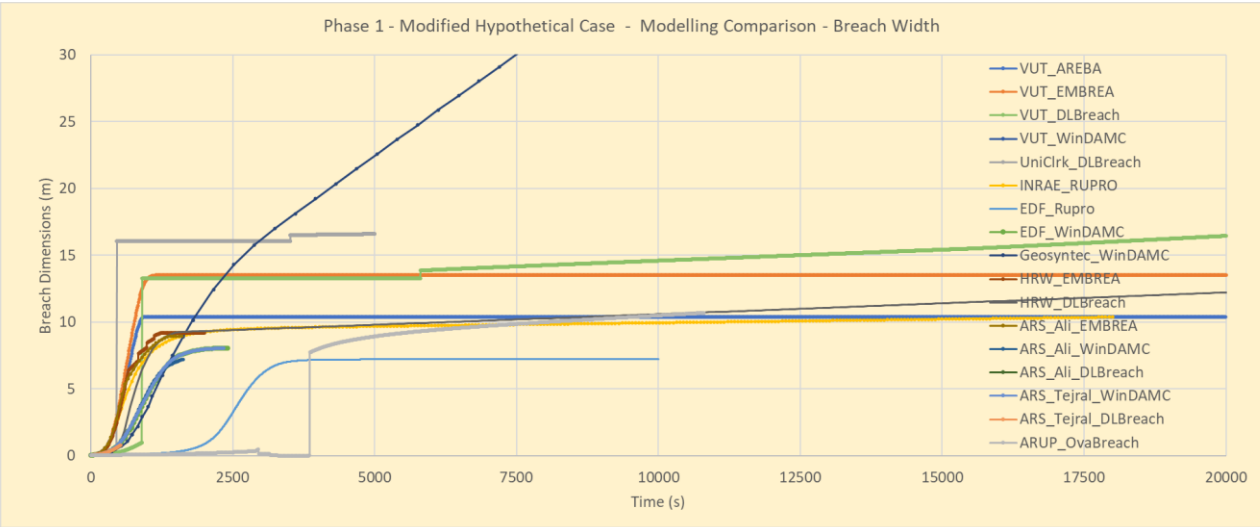
Figure C.1: Modified Hypothetical simplified schematic of test case dam

Table C.1: Phase 1 – Modified Hypothetical: Modeller Assumptions

Models & Modellers:		Structure		Initiation			Soil Parameters								Flow	Computational					Reported Problems or observations
		Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	**Ron Tejral & Ali Abdelfatah**																				
	Ali Abdelfatah																				
✓	DL Breach	The cross section of the dam is trapezodal and the hight is 17.32 m and the crest is 4 m, and the slop for upstream and downstrean are 1V:2.5H.	412.00	0.05*0.05 m Rec.	200 m, at the middle and 16.28 m below the dam crest	0 Sec	10	N/A	7	32	0.65	N/A	N/A	0.1	0.03	0.2	N/A	at the dam crest, it is a pipe of open-channel breach invert at u/s to outlet.	N/A	N/A	
✓	EMBREA		412.00	0.05*0.05 m Rec.	200	0 Sec	10	N/A	N/A	N/A	N/A	N/A	N/A	0.0021	0.02	0.01	N/A		N/A	N/A	
✓	WinDAMC		412.00	0.05*0.05 m Rec.	200	0 Sec	10	N/A	N/A	N/A	N/A	N/A	N/A	0.0021	0.02	0.01	N/A		N/A	N/A	
	Ron Tejral																				
✓	DL Breach	Modeled as cohesive, but assumed d50 was more representative of roughness than clay floc diameter.	0.00	0.05 × 0.05 m square	Center	0	10	N/A	7	32	0.65	N/A	N/A	0.1	0.03	1	N/A	From pipe or open-channel breach invert at u/s to outlet or headcut at d/s.	N/A	N/A	Peak discharge of 530 m3/s predicted at 0.147 hrs. The breach completes formation in a single timestep with breach top width expanding from 0.8 to 28 m.
✓	WinDAMC	cu = 12.6 kPa from given C and ϕ	412.00	0.05 × 0.05 m rect	200	0 sec	10	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	0	0.02	9	#N/A	Conceptually the breach dimensions apply from invert at u/s to headcut at d/s.	#N/A	#N/A	WinDAM has option of dividing user-entered timestep to limit %change in peak discharge and maximum water surface elevation. I had entered 36 s (0.01 hrs), but all time steps were subdivided at least once. 9 ≤ timestep ≤ 18 s.
HRW	Mohamed Hassan																				
✓	DL BREACH	Homogeneous structure - Instantaneous removal of collapsed pipe material - Ignore downstream slumps	414.96	0.05	416	0	10	1740	7	32	0.65	NA	NA	0.1	0.03	10	15	Critical section which moves with time and is not fixed	NA	NA	
✓	EMBREA	Homogeneous structure - Instantaneous removal of collapsed pipe material - Ignore downstream slumps	414.96	0.05	416	0	10	1740	7	32	0.65	NA	NA	0.1	0.03	10	15	Critical section which moves with time and is not fixed	NA	NA	
ERAU	Ghada Elithy																				
✓	DL BREACH	homogenous dam	364.81	0.2	mid	0	N/A	2770	20	45.6	0.244	N/A	N/A	N/A	0.035	0.2	N/A	?	N/A	N/A	I don't think the pipe dimensions are reported in the output
VUT	Stanislav Kotaska																				
✓	TUD AREBA	Homogenous dam with grass protection	412.00	0.05	1/3 right side	-	10	1900	7	32	0.65	-	-	0.1	0.03	10	-	-	-	-	
✓	EMBREA	Homogenous dam with grass protection	412.00	0.05	1/3 right side	-	10	1900	7	32	0.65	-	-	0.1	0.03	10	-	-	-	-	
✓	DL BREACH	Homogenous dam with grass protection	412.00	0.05	1/3 right side	-	10	1900	7	32	0.65	-	-	0.1	0.03	10	-	-	-	-	
✓	WinDAMC	Homogenous dam with grass protection	412.00	0.05	1/3 right side	-	10	1900	7	32	0.65	-	-	0.1	0.03	-	-	-	-	-	
Geosyntec	Al Preston																				
✓	WinDAM		414.96	0.05	180	0	10	1900	7	32	0.65			0.1	0.03	360					
	André Paquier																				
✓	Rupro #1			0.05		0		2650			0.35				0.03	10s					calculation 1 using castorDigue
✓	Rupro #2			0.05		0		2650			0.35				0.03	1.1 s					calculation 2 using Rubar 20 same assumptions as CastorDigue
✓	Rupro #3			0.05		0	10	2650			0.35			0.1	0.03	1.1 s					Calculation 3 using Rubar 20 and provided erodibility value
UniClrk	Weiming Wu																				
✓	DL BREACH	trapezodal cross-section: dam is 17.32 m high, dam crest is 4 m wide, upstream slope 1V:2.5H and downstream slope 1V:2.5H. The dam has an erodible foundation with 2.96 m thickness.	414.96	0.05	middle, 16.28 m below dam crest	at t=0 s	10	2650	7	32	0.343			0.1	0.0188	0.2		dam crest			
EDF	Pierre Squillari (Geophy)																				
✓	Rupro		414.96	0.05	centered : 125 m	no delay	-	2650 (grain)	-	-	-	-	-	-	0.03	1 s (no choice)	?	-	-	-	particle diameter is d50 = 0.13 mm
✓	WinDAM		414.96	0.05	centered : 125 m	no delay	10	1740	-	-	-	-	-	0.1	0.03	0.005 h	?	-	-	-	undrained shear strength is 150 kPa or 3000 psf

C.3 Phase 1 – Modified Hypothetical Modelling Results





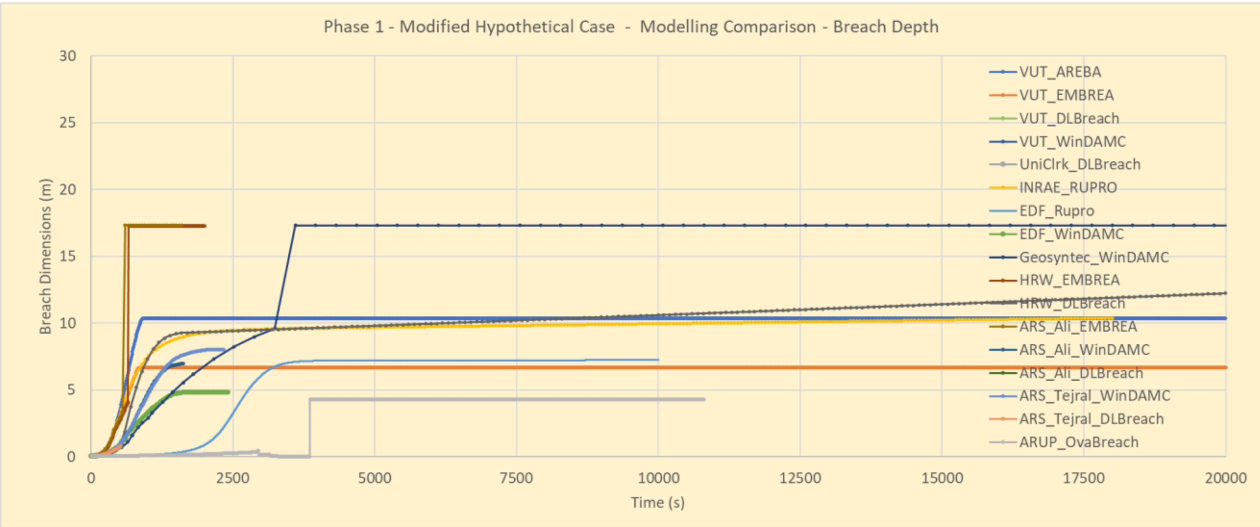


Figure C.2: Phase 1 – Modified Hypothetical: All modelling results

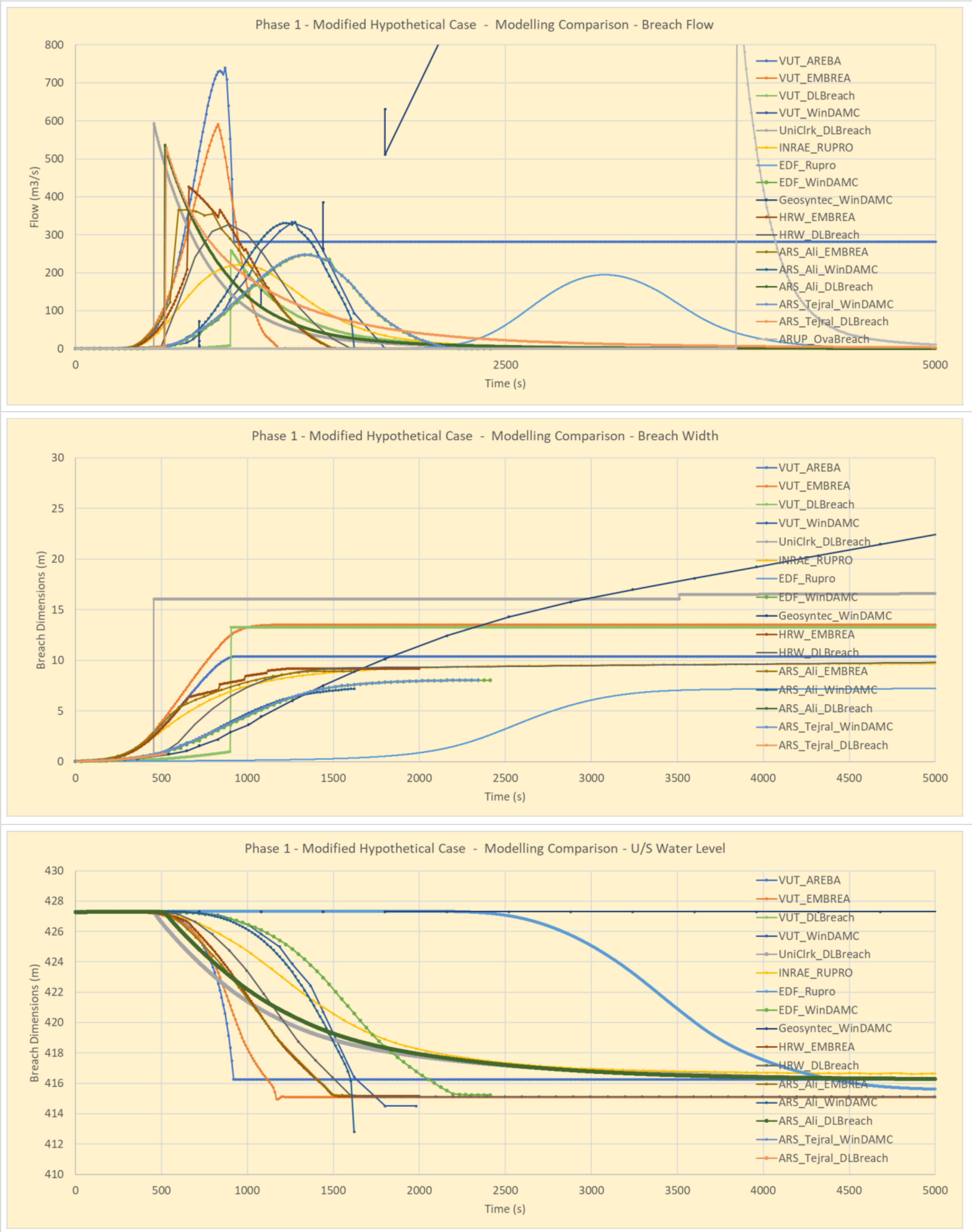


Figure C.3: Phase 1 – Modified Hypothetical: All modelling results – focus on t=0-5000 s

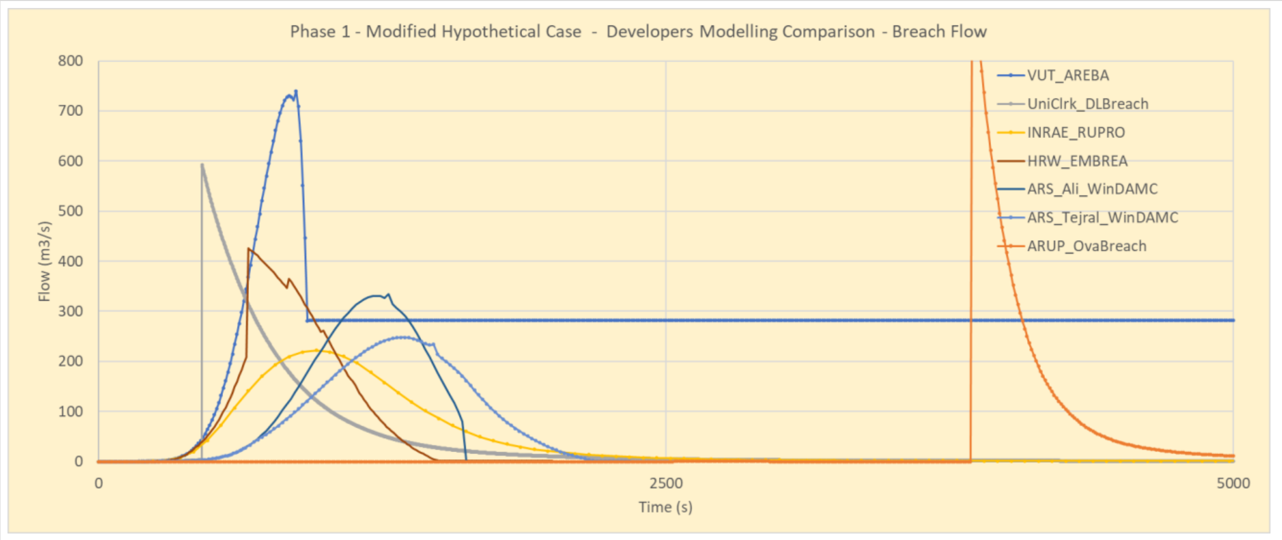


Figure C.4: Phase 1 – Modified Hypothetical: Developers modelling results

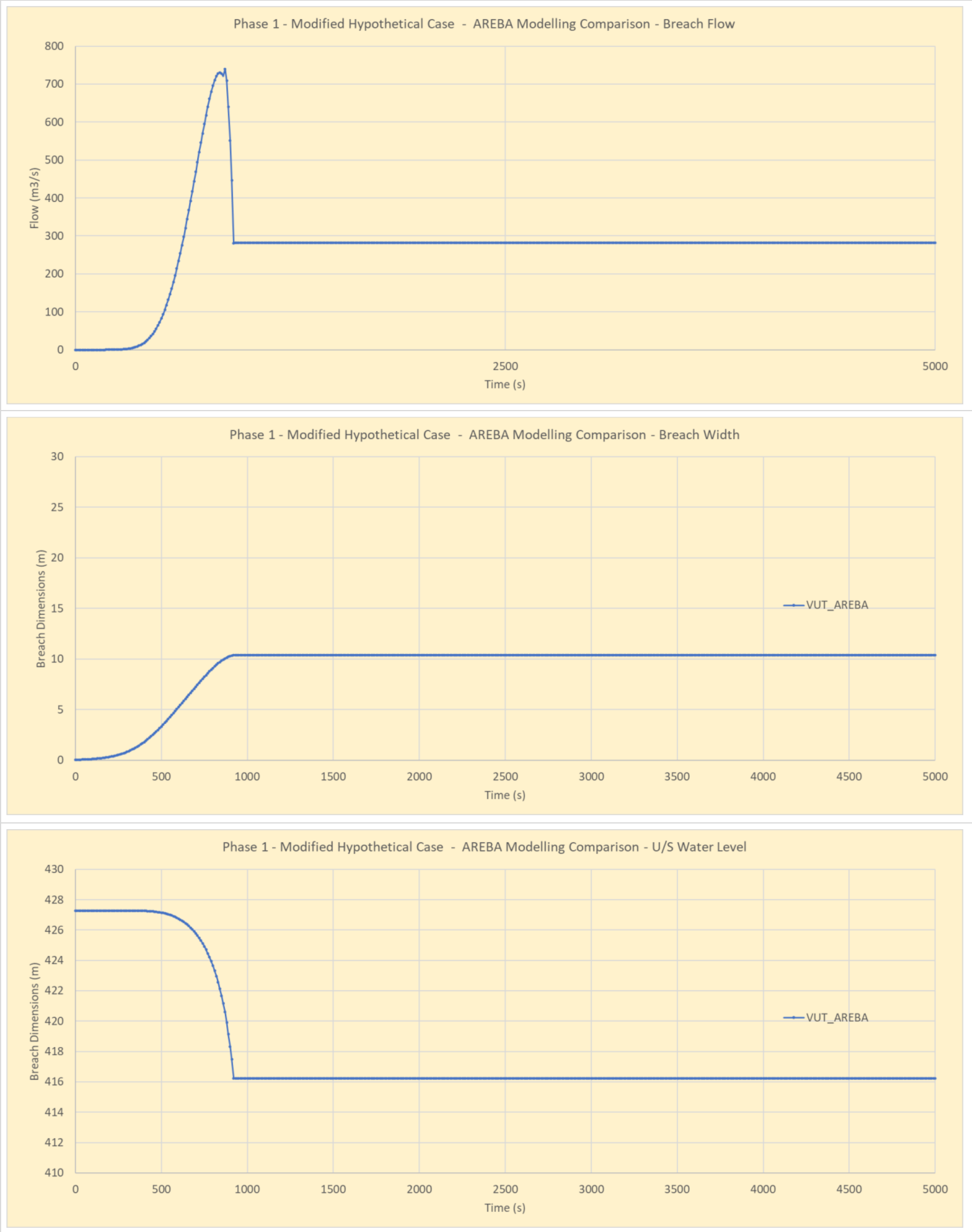


Figure C.5: Phase 1 – Modified Hypothetical: Modelling results using AREBA

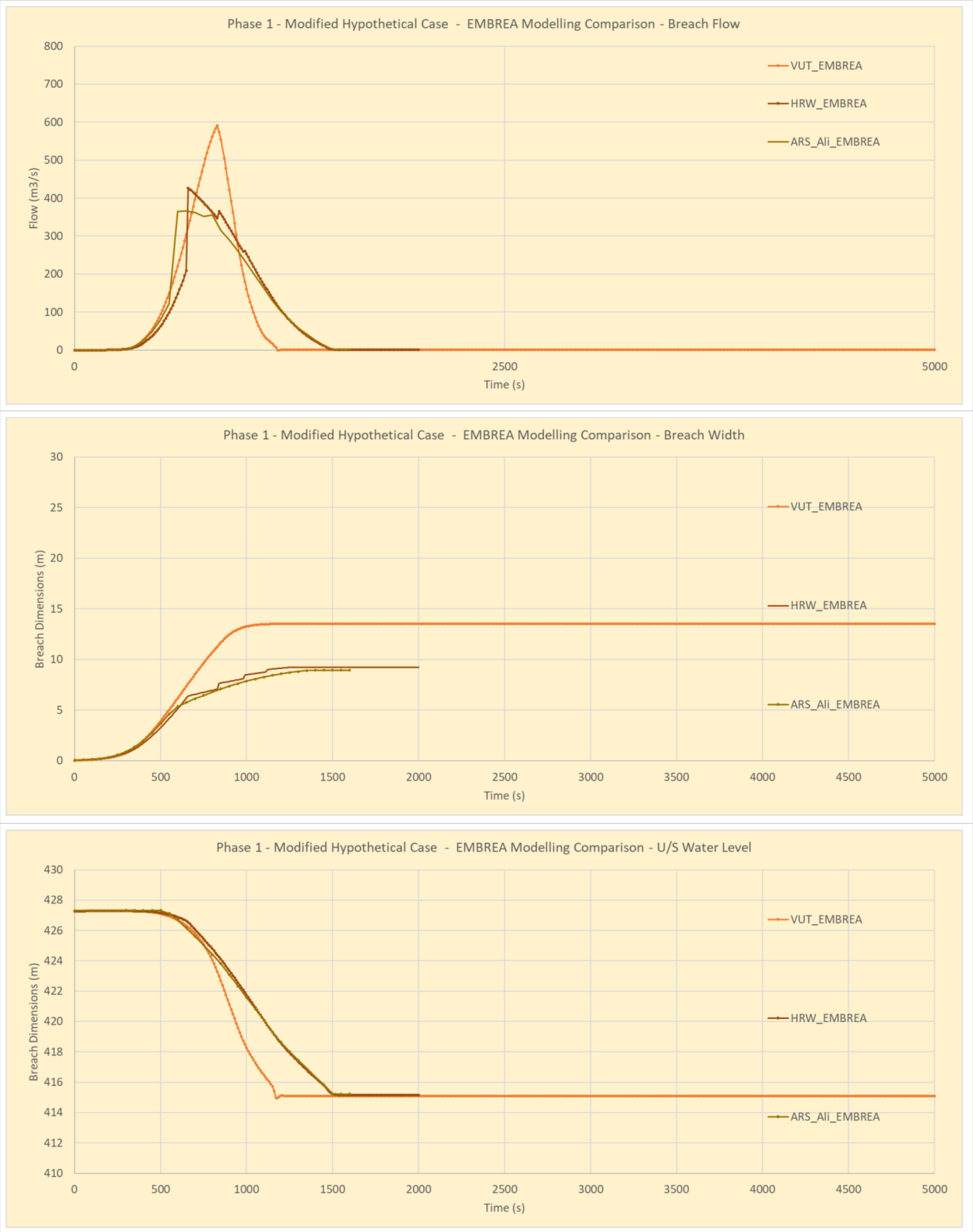


Figure C.6: Phase 1 – Modified Hypothetical: Modelling results using EMBREA

Figure C.7: Phase 1 – Modified Hypothetical: Modelling results using DLBreach

Figure C.8: Phase 1 – Modified Hypothetical: Modelling results using WinDAM C

Figure C.9: Phase 1 – Modified Hypothetical: Modelling results using Rupro

D Phase 1 – IMPACT Test Case

D.1 Phase 1 – IMPACT Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	T1_IMPACT_Blind_v3.xlsx ModellingPhase_11-TestCasesIMPACTT1_IMPACT_Aware_v2.xlsx
Analysis & comparison of modelling results	Phase1_ModellingComparison_IMPACT_21_01_07.xlsx

D.2 Test Case Description

This test case was undertaken in October 2003 as part of the European funded IMPACT Project test programme, where a series of large (4-5 m high) levee sections were constructed and then failed through overtopping or internal erosion flow. This particular test case was 4.3 m high and constructed from moraine. Internal erosion failure was induced by building a perforated flow pipe into the base of the levee and surrounding it by sand which rapidly eroded once flow was allowed through the pipe.

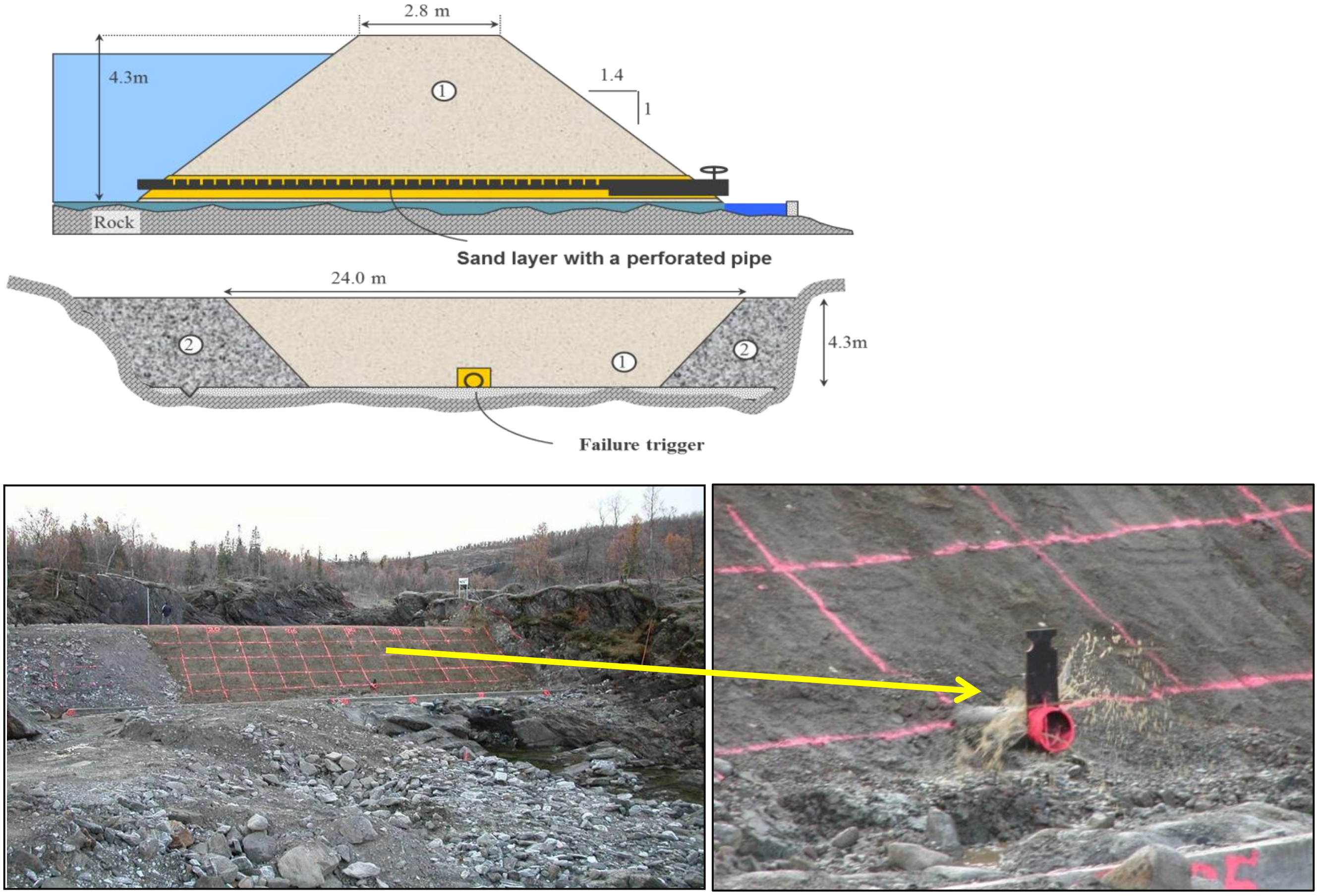
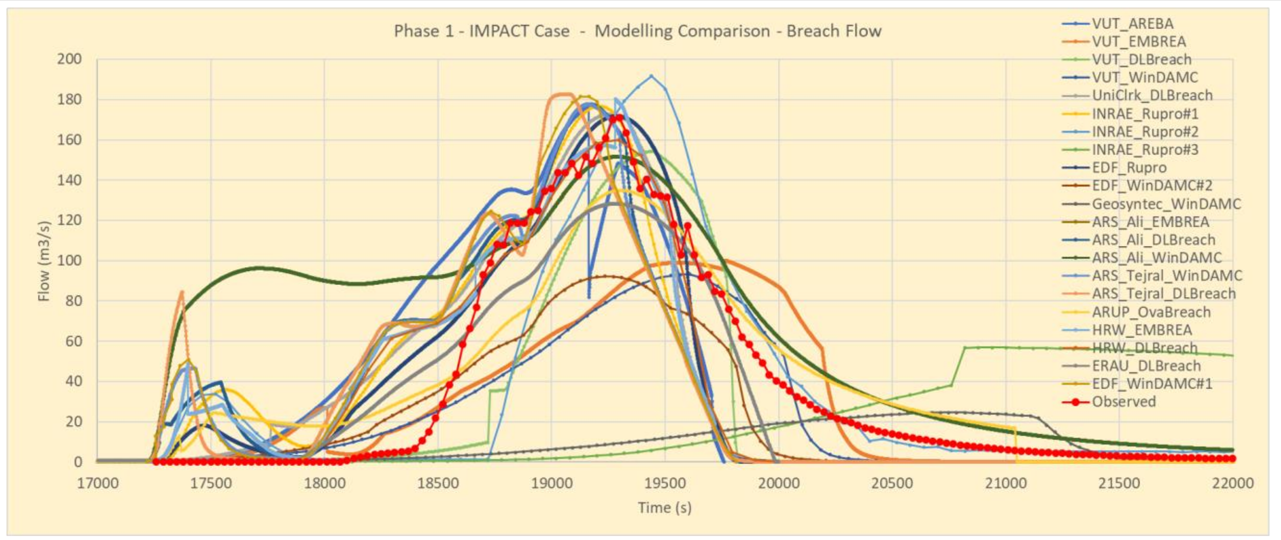


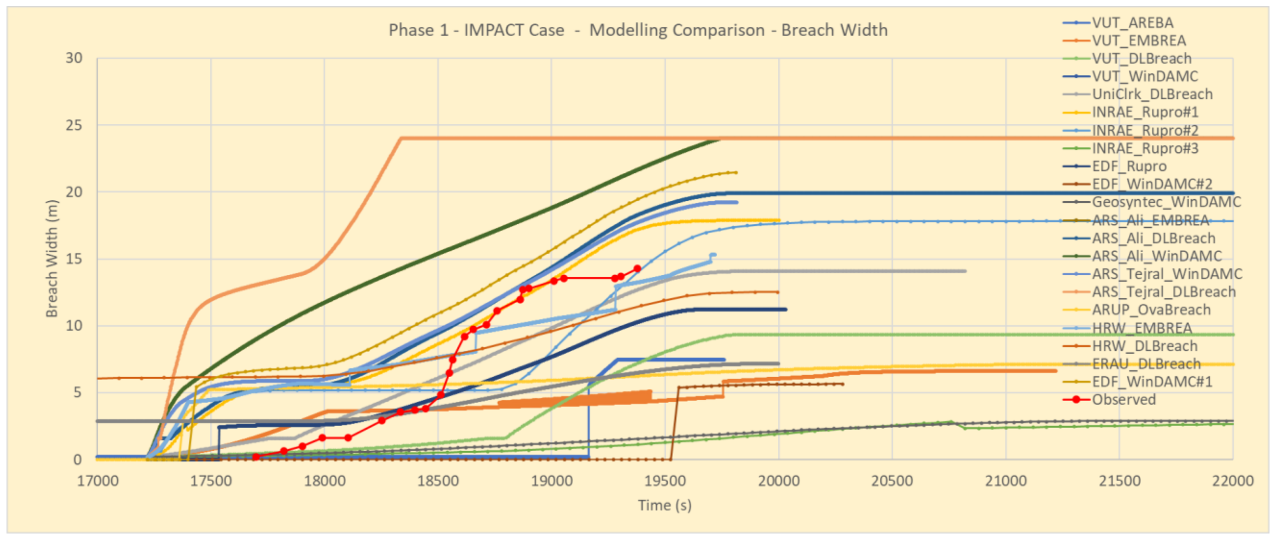
Figure D.1: IMPACT Project internal erosion test case

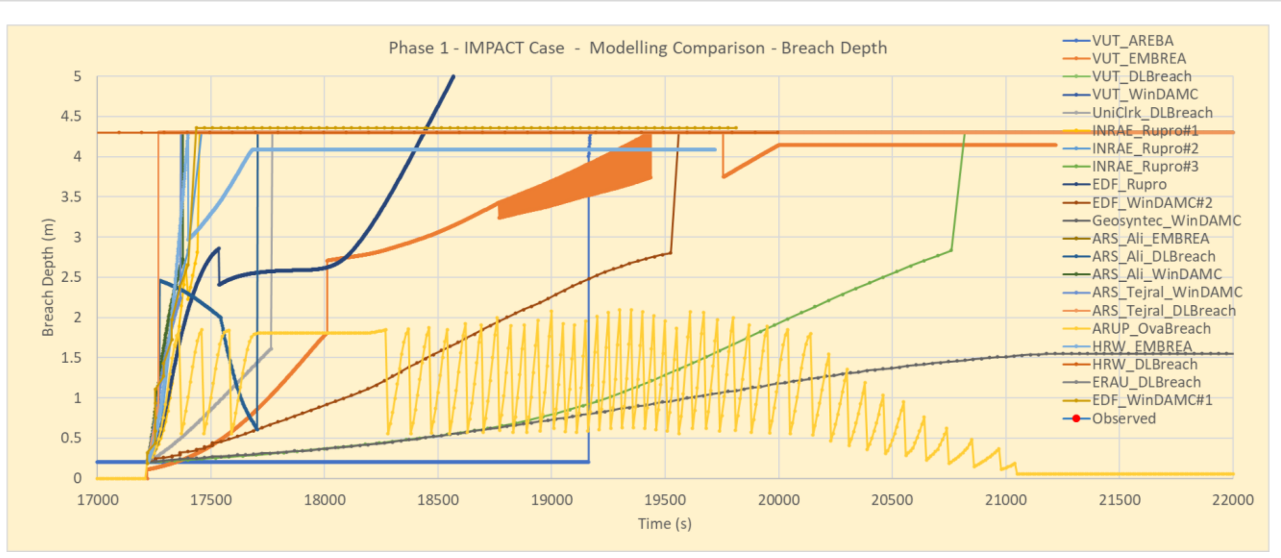
Table D.1: Phase 1 – IMPACT: Modeller Assumptions

Models & Modellers:		Structure	Initiation				Soil Parameters								Flow	Computational					Reported Problems or observations
		Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	Ron Tejral & Ali Abdelfatah																				
	Ron Tejral																				
Blind	WinDAMC	No special treatment of sandy region around pipe. c_u = 49 kPa from given C and ϕ	#REF!	0.20 × 0.20 m rect	#REF!	time zero elapsed coincides with opening of valve	#REF!	#N/A	20	46	#N/A	#N/A	#N/A	6.5	0.02	#REF!	#N/A	Conceptually the breach dimensions apply from invert at u/s to headcut at d/s.	#N/A	#N/A	User-entered timestep = 0.001 hrs = 3.6 s. WinDAM subdivided further for compuational timestep ranging from 0.9 to 1.8 s.
Blind	DL Breach	No special treatment of sandy region around pipe. Modeled as cohesive, but assumed d50 was more representative of roughness than clay floc diameter.	0.00	0.20 × 0.20 m rect	Center	0	90	N/A	20	89	0.24	N/A	N/A	0	0.016	0.2	N/A	From pipe or open-channel breach invert at u/s to outlet or headcut at d/s.	N/A	N/A	
	Ali Abdelfatah																				
Blind	WinDAMC	Undrained Shear Strength (kPa), Cu=50, Cu =(1/2)qu, qu (Unconfined Compressive Strength)	364.81	0.2X0.2 m rectangl	12	0	88.4	N/A	20	46	N/A	N/A	N/A	6	0.02	1	N/A	The breach width apply	N/A	N/A	timestep =0.005 hrs =18 sec.
Blind	DL Breach	Undrained Shear Strength (kPa), Cu=50, Cu =(1/2)qu, qu (Unconfined Compressive Strength)	364.81	0.2X0.2 m rectangl	12	0	88.4	N/A	20	45.6	0.244	N/A	N/A	6	0.02	1	N/A	The breach width apply	N/A	N/A	timestep =0.005 hrs =18 sec.
Blind	EMBREA	Undrained Shear Strength (kPa), Cu=50, Cu =(1/2)qu, qu (Unconfined Compressive Strength)	364.81	0.2X0.2 m rectangl	12	0	88.4	N/A	20	45.6	0.244	N/A	N/A	6	0.02	1	N/A	The breach width apply from invert at up/sto headcut at d/s	N/A	N/A	timestep =0.005 hrs =18 sec.
Aware	None																				
ARUP	Veronika Stoyanova																				
	ARUP OvaBreach	Homogenious, tail water depth ignored	364.81	0.2	as per diagram	17220.00	14.27	2215.4	20	45.6	0.244	n/a	n/a	2	0.0204555	10	n/a (1D model)		4.51386E-05	0.001	Assumed that the baseflow at the start is lost through seepage and the reservoir level was static at the point of the breach. The Manning's n and ks values are functions of d50.
Aware	None																				
HRW	Mohamed Hassan																				
	DL BREACH	All parameter values have been used as described in the test case spreadsheet. Kd was estimated using dry unit weight and % of clay and critical shear stress was estimated based on Kd value but then increased to stop overflowing from taking place (see problems/observations column).	364.81	0.2	364.91	0	20	2130	20	45.6	0.244	NA	NA	5	0.03	1	1	Critical section which moves with time and is not fixed	NA	NA	Erodibility coefficient (Kd) was initially estimated using the dry unit weight and % of clay and found to be 1.4 cm3 /N·s. Critical shear stress was also estimated to be 5 Pa. Based upon this, model shows little erosion and massive overflowing was observed which was not the case during the test case. Therefore Kd was increased until no significant overflowing takes place in the model. This was achieved at a value of Kd equals to 20 cm3 /N·s.
	EMBREA	All parameter values have been used as described in the test case spreadsheet. Kd was estimated using dry unit weight and % of clay and critical shear stress was estimated based on Kd value but then increased to stop overflowing from taking place (see problems/observations column).	364.81	0.2	364.91	0	20	2130	20	45.6	0.244	NA	NA	5	0.03	1	1	Critical section which moves with time and is not fixed	NA	NA	Erodibility coefficient (Kd) was initially estimated using the dry unit weight and % of clay and found to be 1.4 cm3 /N·s. Critical shear stress was also estimated to be 5 Pa. Based upon this, model shows little erosion and massive overflowing was observed which was not the case during the test case. Therefore Kd was increased until no significant overflowing takes place in the model. This was achieved at a value of Kd equals to 20 cm3 /N·s.
Aware	None																				
ERAU	Ghada Elithy																				
	DL BREACH	homogenous dam	364.81	0.2	mid	0	N/A	2770	20	45.6	0.244	N/A	N/A	N/A	0.035	0.2	N/A	?	N/A	N/A	I don't think the pipe dimensions are reported in the output
Aware	None																				
VUT	Stanislav Kotaska																				
Blind	TUD AREBA	Homogenous dam without protection	364.81	0.2	middle	17220	4.5	2090	20	45.6	0.24	-	-	0.001	0.03	1	-	-	-	-	
Blind	EMBREA	Homogenous dam without protection	364.81	0.2	middle	17220	4.5	2090	20	45.6	0.24	-	-	0.001	0.03	1	-	-	-	-	
Blind	WinDAM	Homogenous dam without protection	364.81	0.2	middle	17220	4.5	2090	20	45.6	0.24	-	-	0.001	0.03	-	-	-	-	-	
Blind	DL BREACH	Homogenous dam without protection	364.81	0.2	middle	17220	4.5	2090	20	45.6	0.24	-	-	0.001	0.03	1	-	-	-	-	The piping grow to 9.3 m without collapse of roof conduit and regardless to height of dam
Aware	DL BREACH						17.68														The kd is changed to 17.68 cm^3/Ns, which was used for the moraine sediment in DLBreach manual.
																					The pipe entrance head loss coefficient is ajusted from 0.05 to 1.5 by using the card Pipe_Entrance_Head_Loss_Coef 1.5
																					The dam length is changed to 16.2 m, which is bottom length.
																					The measured downstream water level is too far from the dam and cannot be used for the DLBreach, which does not calculate the downstrean channel flow routing. **The unit of the time is minute in the measured breach width data?**
Geosyntec	Al Preston																				
Blind	WinDAM		364.81	0.20	12.19	17220	1.4	64	20	45.6	0.244			6.22	0.03	36					
Aware	None																				
	André Paquier																				
Blind	Rupro #1			0.2		17220 s		2770			0.244				0.05	10s					calculation 1 using castorDigue
Blind	Rupro #2			0.2		17220 s		2770			0.244				0.05	0.4 s					calculation 2 using Rubar 20 same assumptions as CastorDigue
Blind	Rupro #3			0.2		17220 s	0.1	2770			0.244			30	0.05	0.4 s					Calculation 3 using Rubar 20 and low erodibility value: breach hydrograph certainly underestimated
Aware	Rupro #1	*The erosion rate was multiplied by 10 compared to blind modelling results 3.*																			
Aware	Rupro #2	*Moreover no upstream head loss was considered and the shear stress was not reduced along vertical walls*																			
Aware	Rupro #3																				
UniClrk	Weiming Wu																				
Blind	DL BREACH	trapezodal cross-section: dam is 4.3 m high, dam crest is 2.8 m wide, upstream and downstream slopes 1V:1.4H. The dam foundation is assumed nonerodible.	364.81	0.2	middle, at base	valve opening, in Oct 8, 13:34	8.5	2770	20	45.6	0.245			5.5	0.036	0.5		dam crest			
Aware	DL BREACH						17.68														The kd is changed to 17.68 cm^3/Ns, which was used for the moraine sediment in DLBreach manual.
																					The pipe entrance head loss coefficient is ajusted from 0.05 to 1.5 by using the card Pipe_Entrance_Head_Loss_Coef 1.5
																					The dam length is changed to 16.2 m, which is bottom length.
																					The measured downstream water level is too far from the dam and cannot be used for the DLBreach, which does not calculate the downstrean channel flow routing. **The unit of the time is minute in the measured breach width data?**
EDF	Pierre Squillari (Geophy)																				
Blind	Rupro		364.10	0.2	middle of the dam	no delay	-	2650 (grain density)	-	-	0.244	-	-	-	0.03	1 s		rectangular shape	-	-	
Blind	WinDAM	homogeneous trapezoidal embankment	364.8	0.6 ft x 0.6 ft 0.2 m x 0.2 m	middle of the dam and at elevation 1196.9	initiation at 13:34	Hanson gives 50 (ft/h)/(psf) (clay fraction below 10%, standard compaction effort, wet side of optimum)	2090 ≈ 130.5 pcf and not 133 pcf as it is in the Dam Levee Description sheet	0	-	-	-	-	-	0.03	0.01 h (at most)					Bare soil for downstream slope protection
Blind							Fell et al. 2008 give 2.705(ft/h)/psf (SM soil with fine content = 30%)														Kd and TauC determined from Hanson's graphical tools: wet side of the optimum, standard compaction effort, clay fraction 1-10 % --> Kd = 50 (ft/hr)/(psf) TauC = 0 kPa = 0 psf
Aware	None						NEH equation gives 2.706 (ft/hr)/psf (with clay fraction = 0% and dry density = 130.5 pcf														Undrained shear strength determined from

D.3 Phase 1 – IMPACT Modelling Results







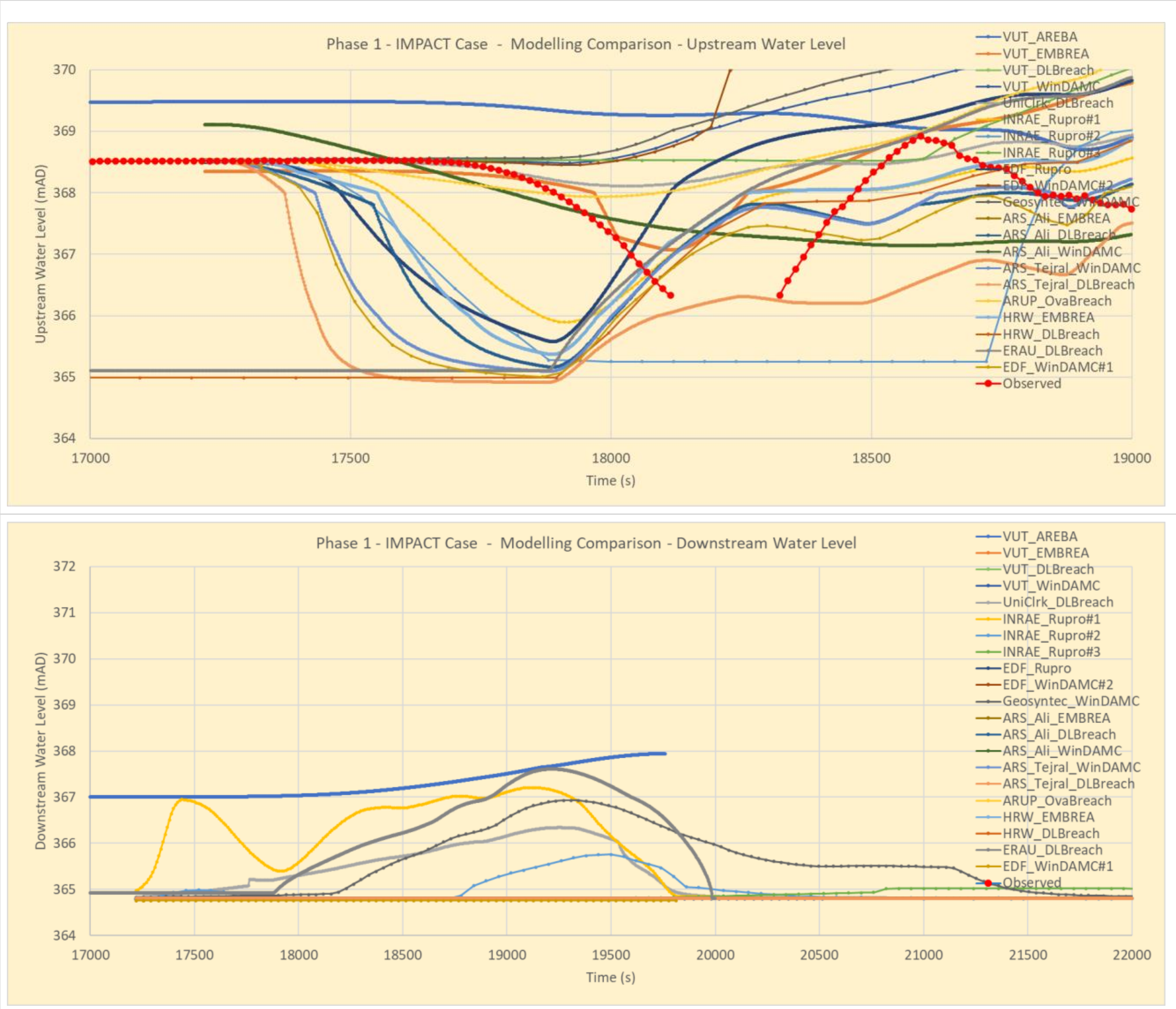


Figure D.2: Phase 1 – IMPACT: All modelling results

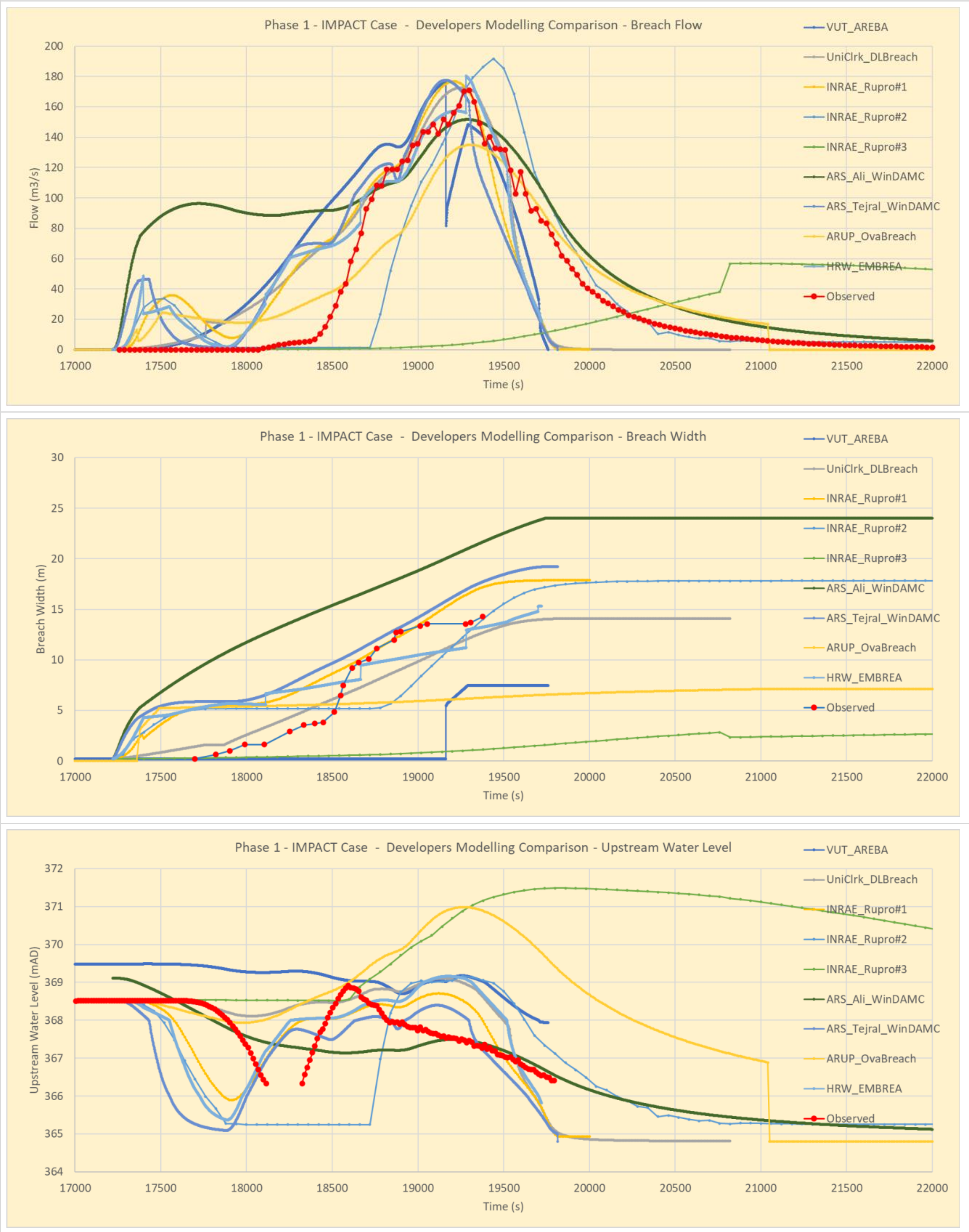


Figure D.3: Phase 1 – IMPACT: Developers modelling results

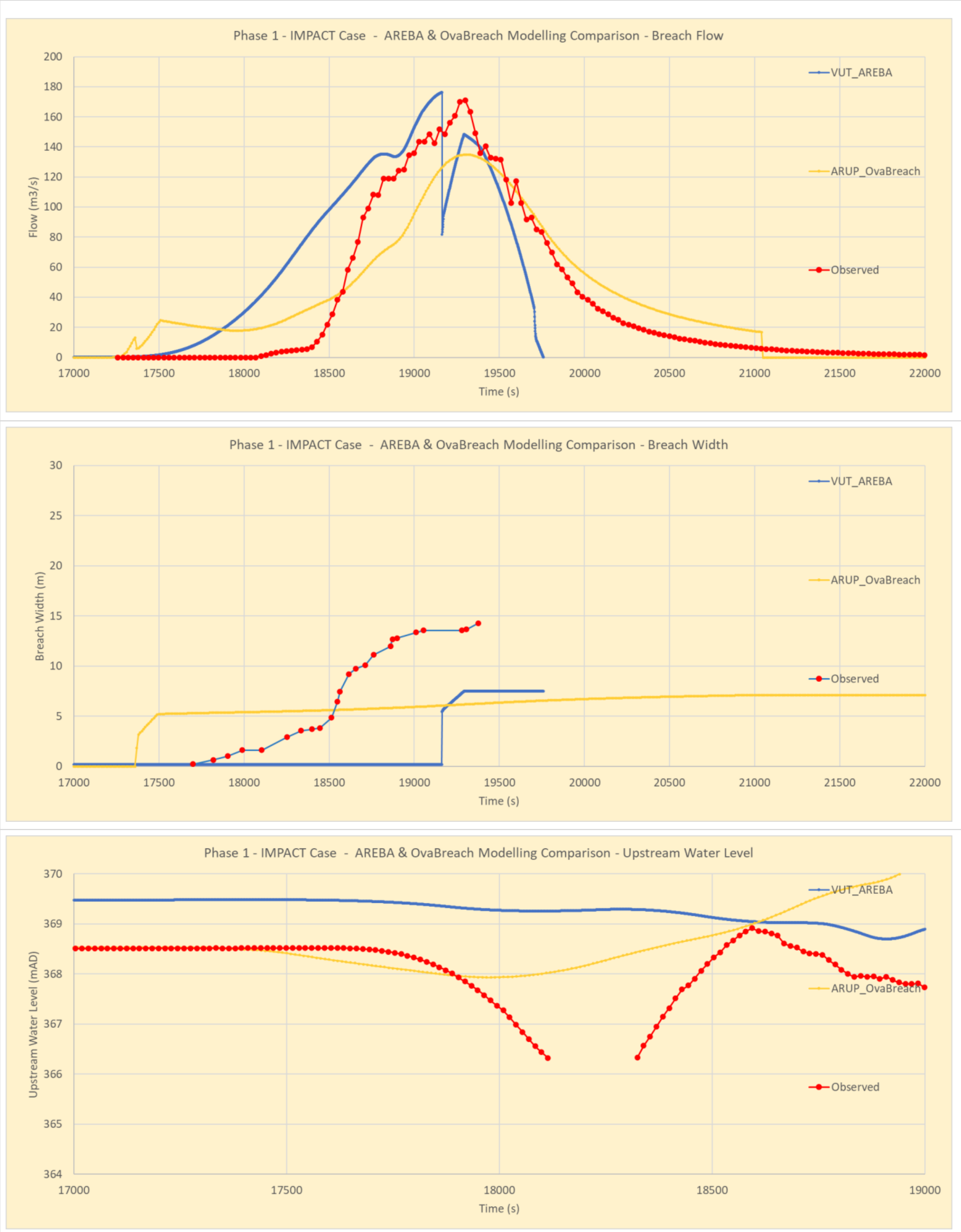


Figure D.4: Phase 1 – IMPACT: Modelling results using AREBA and OvABreach

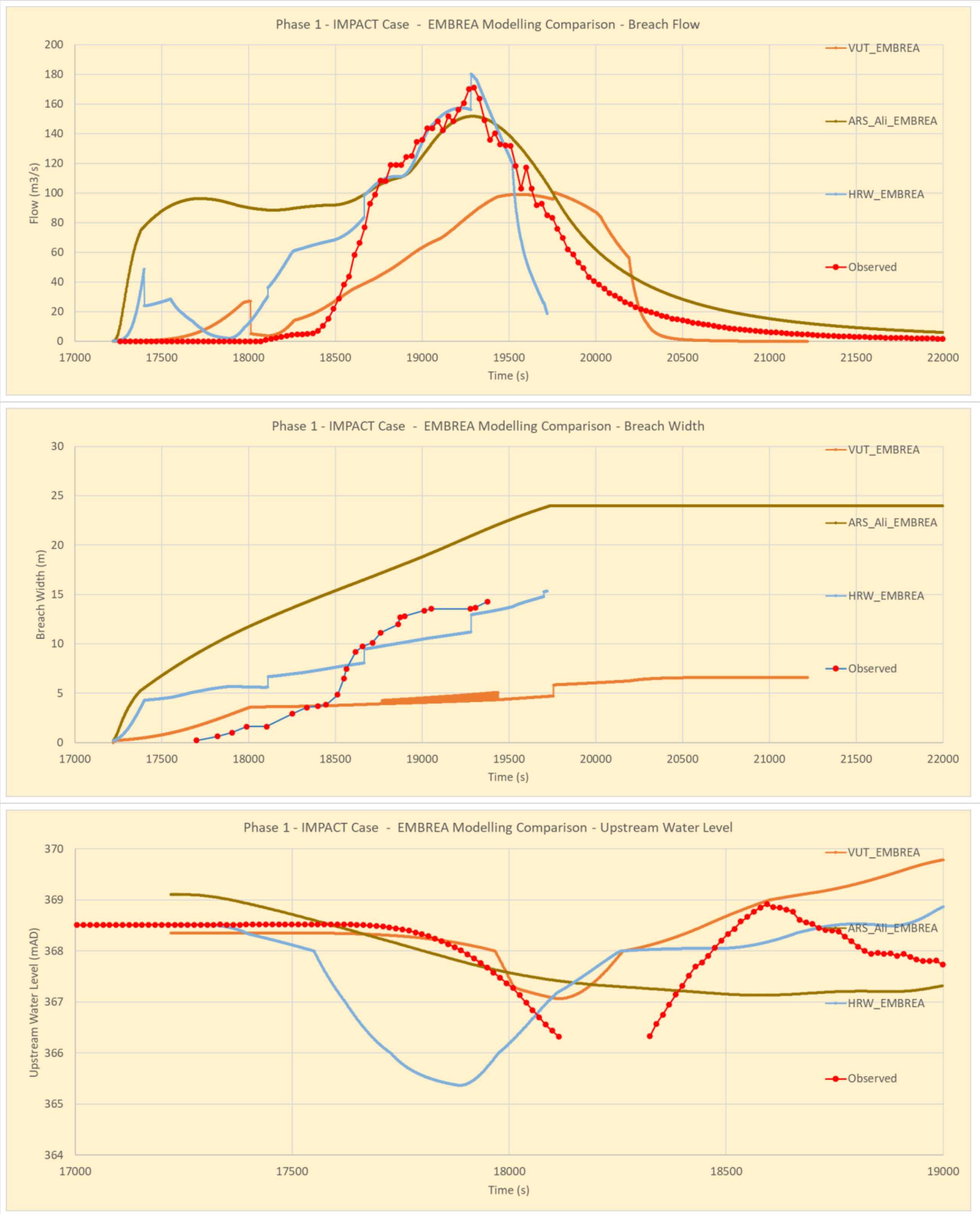


Figure D.5: Phase 1 – IMPACT: Modelling results using EMBREA

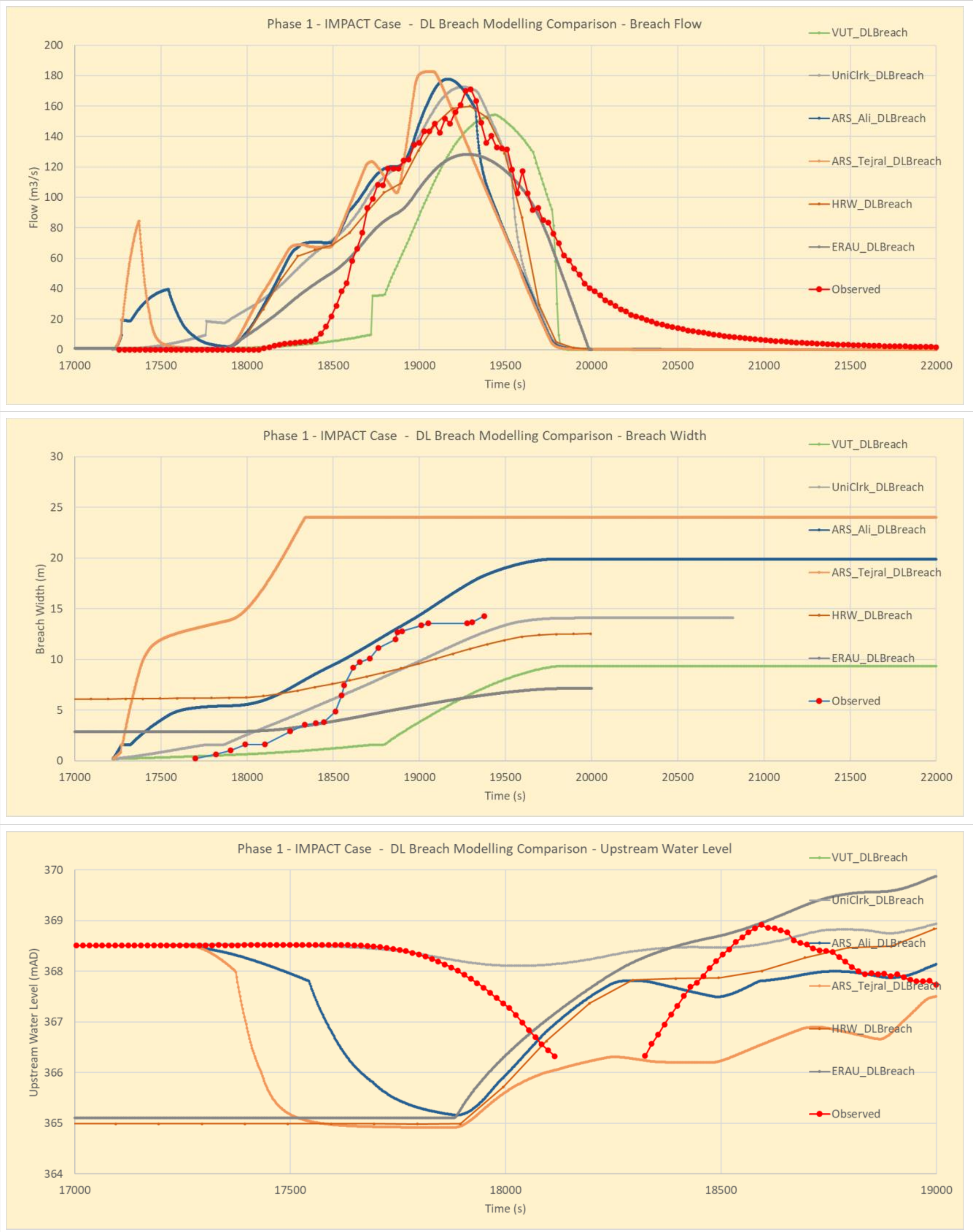


Figure D.6: Phase 1 – IMPACT: Modelling results using DLBreach

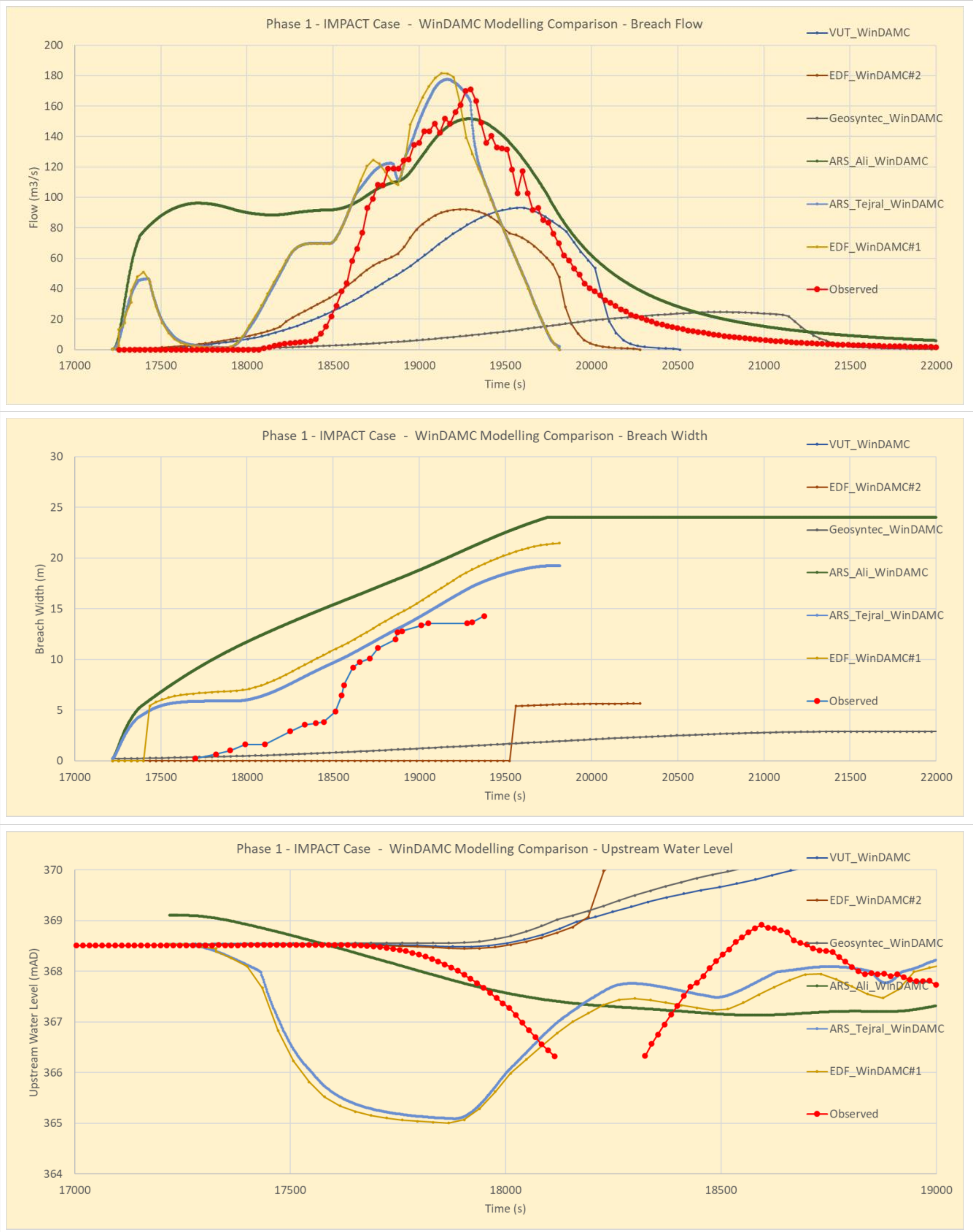


Figure D.7: Phase 1 – IMPACT: Modelling results using WinDAM C

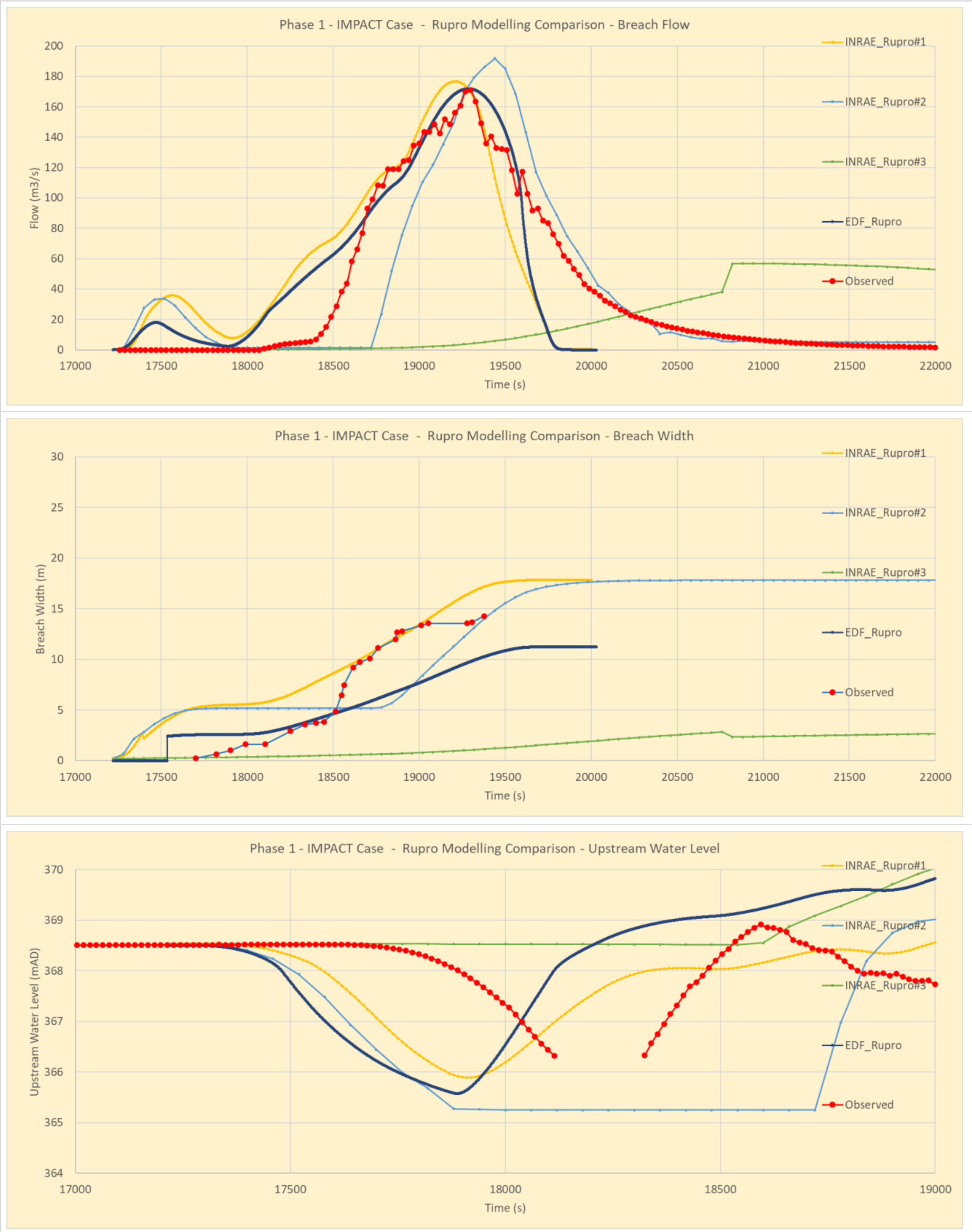


Figure D.8: Phase 1 – IMPACT: Modelling results using Rupro

D.4 Phase 1 – IMPACT Aware Modelling Results

Aware modelling results were submitted by UniClrk and INRAE for this test case:

UniClrk – DLBreach

In an effort to improve the modelling results, the following parameters were changed:

- The K_d was changed to 17.68 cm³/Ns, which was used for the moraine sediment
- The pipe entrance head loss coefficient was adjusted from 0.05 to 1.5 by using the card Pipe_Entrance_Head_Loss_Coef 1.5
- The dam length was changed to 16.2 m, which is bottom length
- The measured downstream water level is too far from the dam and cannot be used for the DLBreach, which does not calculate the downstream channel flow routing.

A comparison of observed blind and aware results for flow are shown in the Figure below. There does not appear to be a significant improvement in model performance.

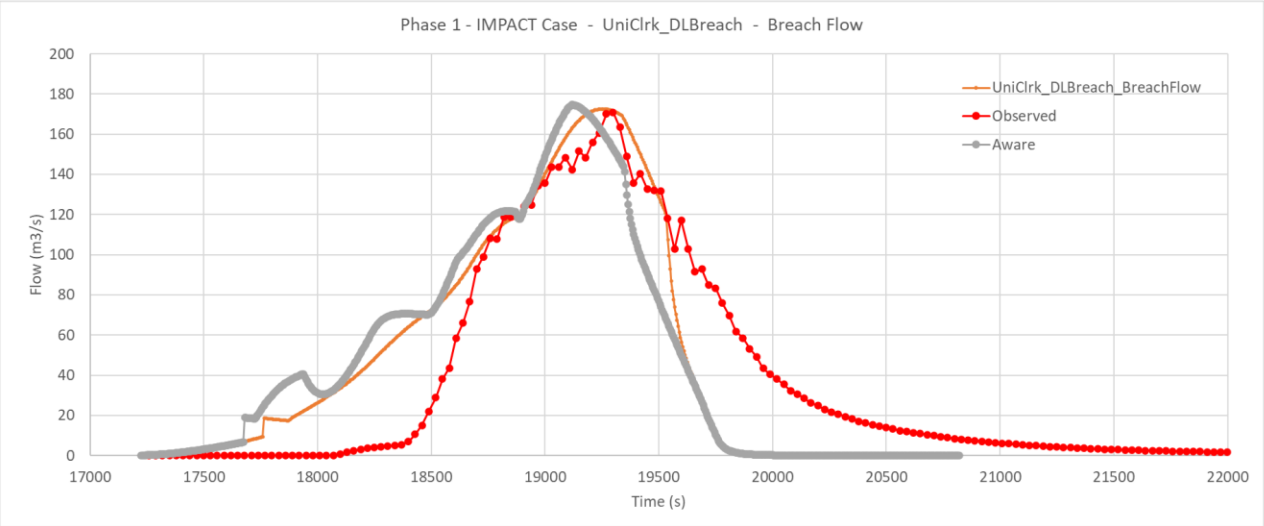


Figure D.9: Phase 1 – IMPACT: Aware modelling by UniClrk using DLBreach

INRAE – Rupro#3

Adjustments to the model allow for a significant improvement in results prediction for Rupro#3, as shown in the Figure below. However, the results are still within a similar band of error as shown by Rupro#1 and Rupro#2.

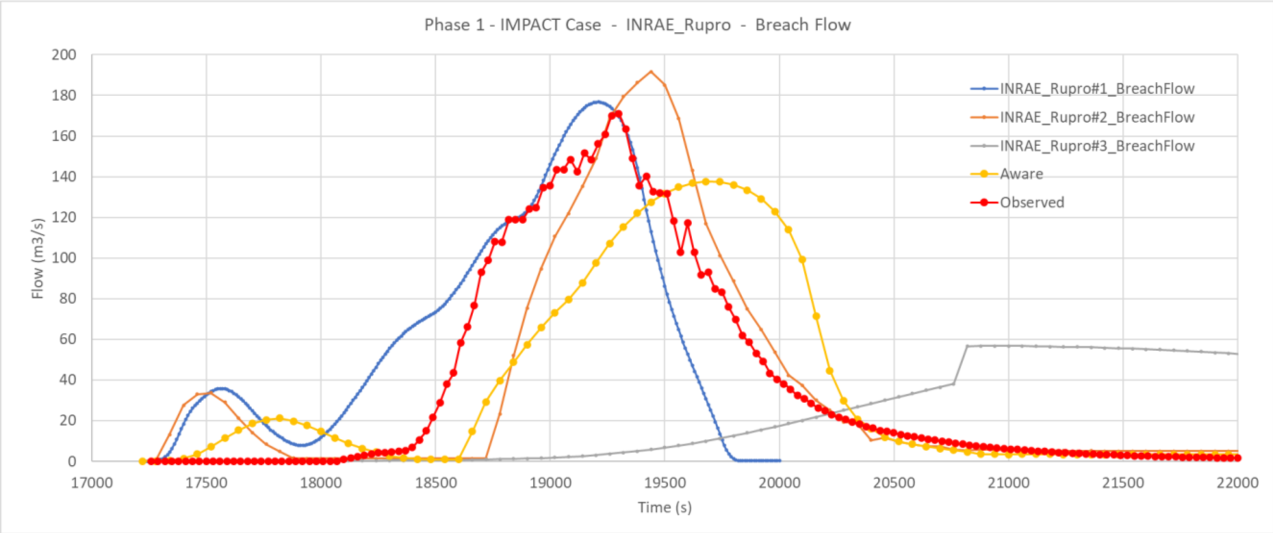


Figure D.10: Phase 1 – IMPACT: Aware modelling by INRAE using Rupro#3

E Phase 1 – ARS P1 Test Case

E.1 Phase 1 – ARS P1 Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	USDA-ARS-P1_Blind_v7.xlsx USDA-ARS-P1_Aware_v2.xlsx
Analysis & comparison of modelling results	Phase1_ModellingComparison_P1_21_03_01.xlsx

E.2 Test Case Description

This test case was performed at the USDA ARS site in Stillwater, Oklahoma and consisted of a homogeneous earth embankment 1.2 m high, 9.75 m long, with a crest width of 1.98 m and slopes of approximately 1 in 3. A pipe of 0.04 m diameter was created through the levee 0.28 m from the base, by removing a rigid pipe of that diameter, which had been constructed into the levee.

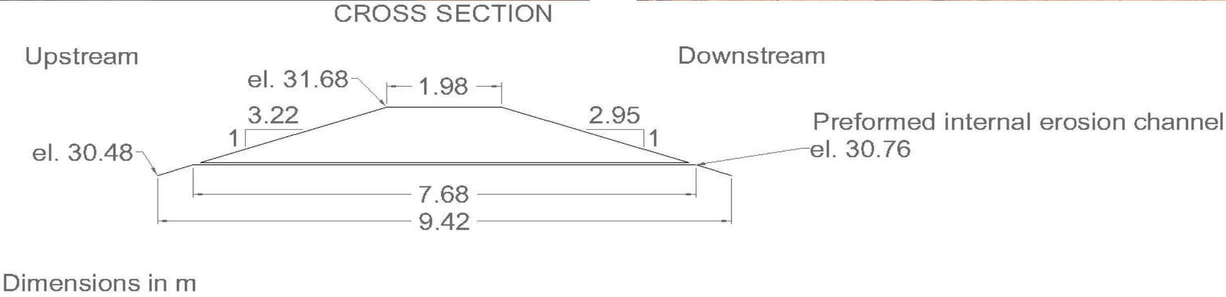
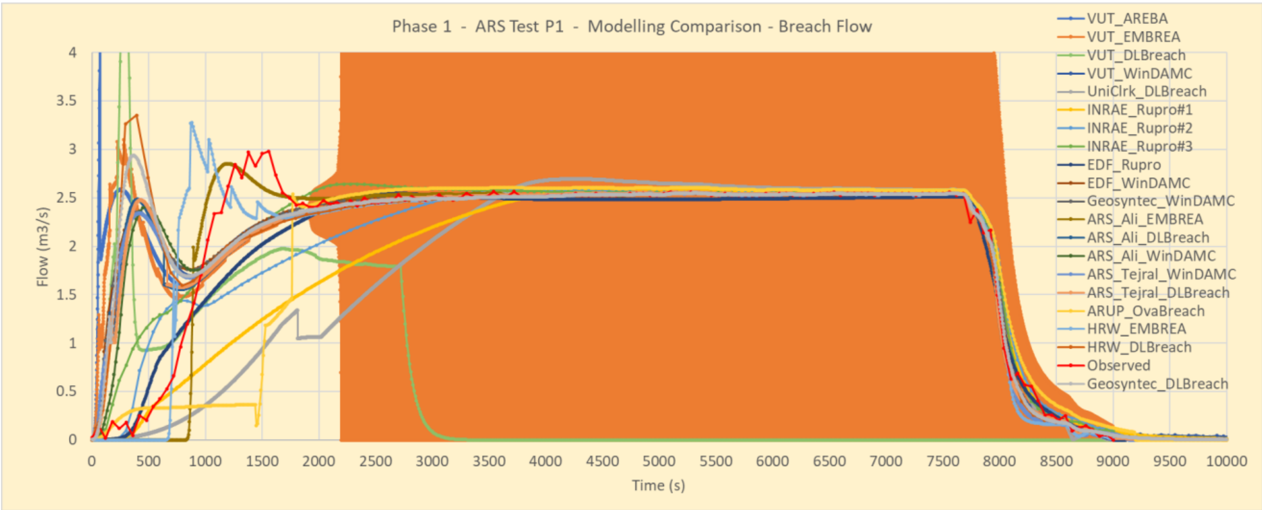


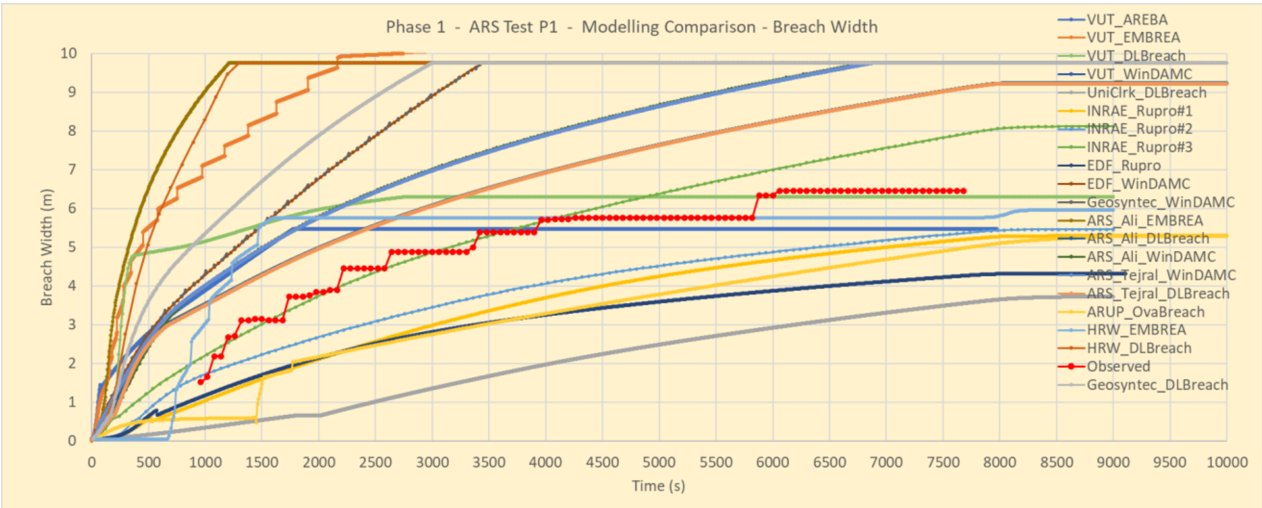
Figure E.1: ARS P1 internal erosion test case

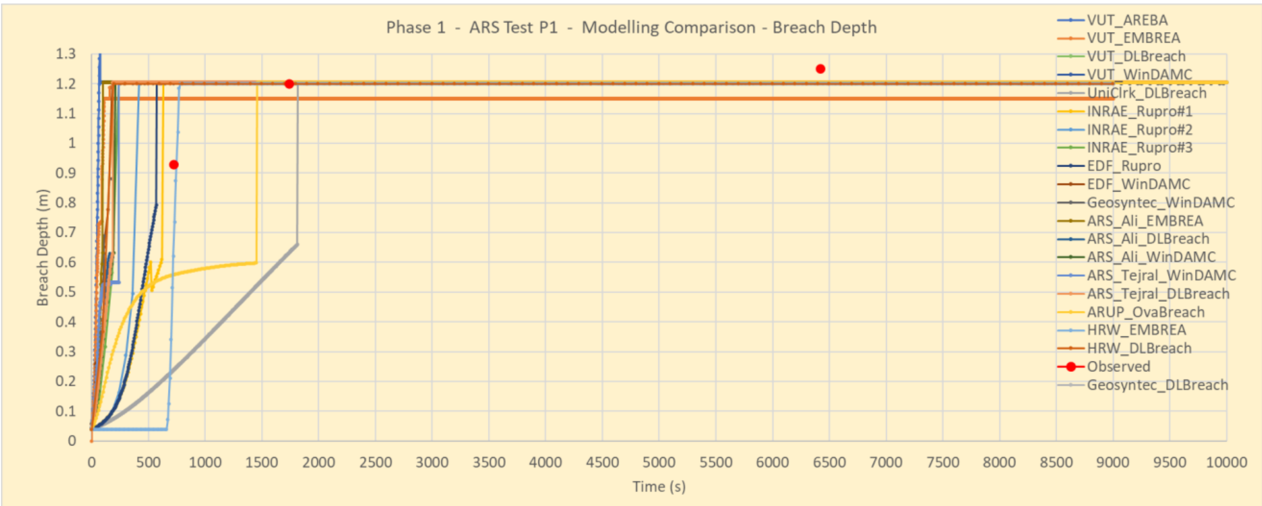
Table E.1: Phase 1 – ARS P1: Modeller Assumptions

Models & Modellers:		Structure		Initiation			Soil Parameters								Flow	Computational					Reported Problems or observations
		Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	Ron Tejral & Ali Abdelfatah																				
	Ron Tejral																				
Blind	WinDAMC	c_u = 13 kPa	9.29	0.04 × 0.04 m rect	9	0 sec	210	27867	#N/A	#N/A	#N/A	#N/A	#N/A	6.89	0.02	36000.0	#N/A	Conceptually the breach dimensions apply from invert at u/s to headcut at d/s.	#N/A	#N/A	WinDAM has option of dividing user-entered timestep to limit %change in peak discharge and maximum water surface elevation. I had entered 3.6 s (0.001 hrs), but all time steps were subdivided at least once. 0.9 ≤ timestep ≤ 1.8 s.
Blind	DL Breach	Friction angle estimated from soil class; cohesion approximated from angle and undrained shear strength.	0.00	0.04 × 0.04 m square	center	0	120	N/A	7	32	0.34	N/A	N/A	0.144	0.016	0.2	N/A	From pipe or open-channel breach invert at u/s to outlet or headcut at d/s.	N/A	N/A	Spillways and tailwater stage-discharge relationships cannot be defined by tables; complicates data entry. No elevation setting for tailwater by Manning formula.
Aware	None																				
	Ali Abdelfatah																				
Blind	WinDAMC	Cu = 13 Kpa	30.48	0.04X0.04 m Rectangle	5	0 Sec.	120	1906	N/A	N/A	N/A	N/A	N/A	0.14	0.02	0.017	N/A	Theoretically, the breach	N/A	N/A	
Blind	DL Breach	Cu = 13 Kpa	30.48	0.04X0.04 m Rectangle	5, at the center, and at	0 Sec.	120	1906	N/A	32	0.34	N/A	N/A	0.14	0.016	0.017	N/A	Theoretically, the breach	N/A	N/A	The kd value of 120 cm^3/Ns is much larger than the range of kd values calibrated in DLBreach manual. But I used the given Kd and the model run perfact.
Blind	EMBREA	Cu = 13 Kpa	30.48	0.04X0.04 m Rectangle	5	0 Sec.	120	1906	N/A	32	0.34	N/A	N/A	0.14	0.02	0.017	N/A	Theoretically, the breach dimensions spread from invert at up/s to headcut at d/s.	N/A	N/A	
Aware	WinDAMC	Results for Q look identical?					50														
Aware	DL Breach						50														
Aware	EMBREA	Results/ analysis incomplete - unable to plot																			
ARUP	Veronika Stoyanova																				
	ARUP OvaBreach	Homogenious, tail water depth ignored	30.48	0.04	as per diagram	0.00	120.00	1900	7	32	0.34	n/a	n/a	0.144	0.009378052	10s	n/a (1D model)		0.00012	0.00225	Fot this run I changed the breach wdth relationship to a trapesoidal shape width the top width being a function of the effective angle of friction and the cohesion. It was ralatively straightforward run, with not many of the parameters left fo rinterpretation. The Manning's n is a function of d50.
Aware	None																				
HRW	Mohamed Hassan																				
	DL BREACH	Parameters were taken as provided in the test case description. I had to fit a curve for the spillway outflow as the equation exponent in DL Breach model is a fixed value.	30.48	0.04	30.76	0	120	1740	7	32	0.34	NA	NA	0.144	0.025	10	NA	Critical section which moves with time and is not fixed	NA	NA	I was not able to limit the breach width to 5 times the depth as I did in EMBREA.
	EMBREA	Parameters were taken as provided in the test case description. No changes were made	30.48	0.04	30.76	0	120	1740	7	32	0.34	NA	NA	0.144	0.025	10	5	Critical section which moves with time and is not fixed	NA	NA	*Breach width was assumed not to exceed 5 time the depth (i.e. 6 meters). A sensitivity run was undertaken to check this assumption. The results of this run showed that the peak value and timing were not affected but the final breach width was 9 m instead of 5.95 m.*
Aware	None																				
ERAU	Ghada Elithy																				
	DL BREACH	homogenous dam	364.81	0.2	mid	0	N/A	2770	20	45.6	0.244										
Aware	None																				
VUT	Stanislav Kotaska																				
Blind	TUD AREBA	Homogenous dam without protection	30.48	0.04	middle	-	120	1900	7	32	0.34	-	-	0.144	0.03	1	-	-	-	-	
Blind	EMBREA	Homogenous dam without protection	30.48	0.04	middle	-	120	1900	7	32	0.34	-	-	0.144	0.03	1	-	-	-	-	Oscillation when the downstream condition is affected
Blind	WinDAM	Homogenous dam without protection	30.48	0.04	middle	-	120	1900	7	32	0.34	-	-	0.144	0.03	-	-	-	-	-	
Blind	DL BREACH	Homogenous dam without protection	30.48	0.04	middle	-	120	1900	7	32	0.34	-	-	0.144	0.03	1	-	-	-	-	
Aware	TUD AREBA						6							0.144							
Aware	EMBREA						8.5							0.144							
Aware	WinDAM						8.5							0.144							
Aware	DL BREACH						20							0.6							
Geosyntec	Al Preston																				
Blind	WinDAM		30.48	0.04	4.9	0	120	1900	7	32	0.34			0.144	0.03	60.12					
Blind	DL BREACH			0.04	center	0	120		7	32	0.34			0.144	0.02	0.5					
Aware	None																				
	André Paquier																				
Blind	Rupro #1			0.04		0		2650			0.34				0.033	10s					calculation 1 using castorDigue
Blind	Rupro #2			0.04		0		2650			0.34				0.033	1 s					calculation 2 using Rubar 20 same assumptions as CastorDigue
Blind	Rupro #3			0.04		0	120	2650			0.34			0.144	0.033	1 s					Calculation 3 using Rubar 20 and provided erodibility value
Aware	Rupro #1																				
Aware	Rupro #2																				
Aware	Rupro #3																				
UniClrk	Weiming Wu																				
Blind	DL BREACH	trapezodal cross-section: dam is 1.204 m high, dam crest is 1.98 m wide, upstream slope 1V:3.22H and downstream slope 1V:2.95H. The dam foundation is assumed nonerodible.	30.48	0.04	middle, 0.92 m below dam crest	at t=0 s	10.3	2650	7	32	0.343			0.144	0.016	0.2		dam crest			The given kd value of 120 cm^3/Ns is much larger than the range of kd values calibrated in DLBreach manual. A value of 10.3 cm^3/Ns is used in this blind test. This value was used for a similar SM soil in DLBreach manual.
Aware	DL BREACH						60														The kd is changed to 60 cm^3/Ns. The pipe entrance head loass coefficient is adjusted from 0.05 to 1.5. This is done by using the card: Pipe_Entrance_Head_Loss_Coef 1.5 The downstream backwater effect is significant. DLBreach does not use the rating curve, so the measured downstream water level is used as boundary condition.
EDF	Pierre Squillari (Geophy)																				
Blind	Rupro	slopes are averaged (they are not defined in rupro, only crest and bottom width) Small adjustment for the side discharge weir as Rupro's weir formula is slightly different than the one used here (^3/2 VS ^1.62)	30.48	0.04	Middle	no delay	-	2650 (grain density)	-	-	0.34	-	-	-	0.03	5 s		rectangular shape			
Blind	WinDAM		100 ft	0.13 ft	middle of the dam	no delay	70 (ft/hr)/(psf)	119 lbs/ft3	-	-	-	-	-	0.003 psf	Identical number for slopes and crest = 0.03	< 0.01			-	-	

E.3 Phase 1 – ARS P1 Modelling Results







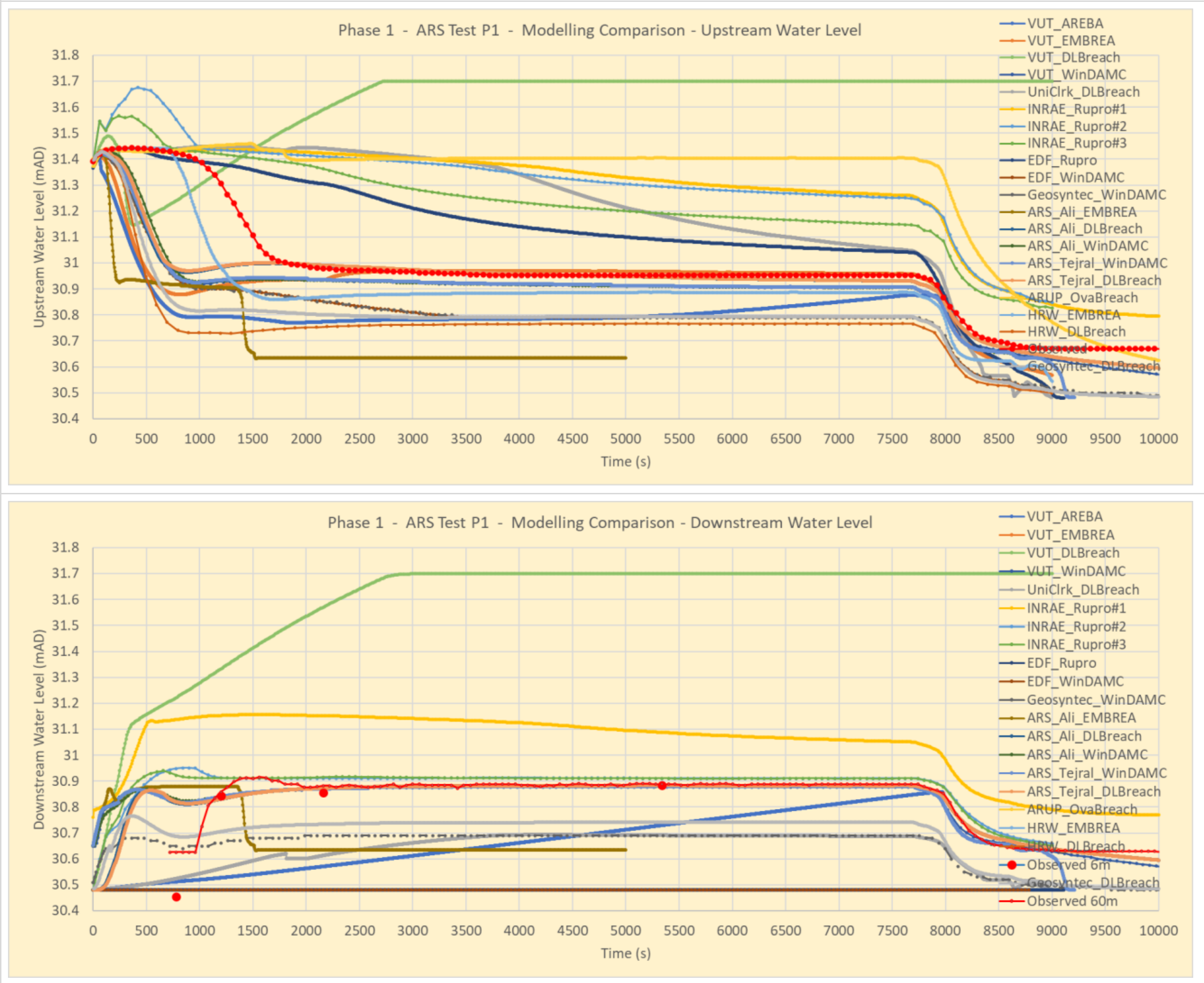


Figure E.2: Phase 1 – ARS P1: All modelling results

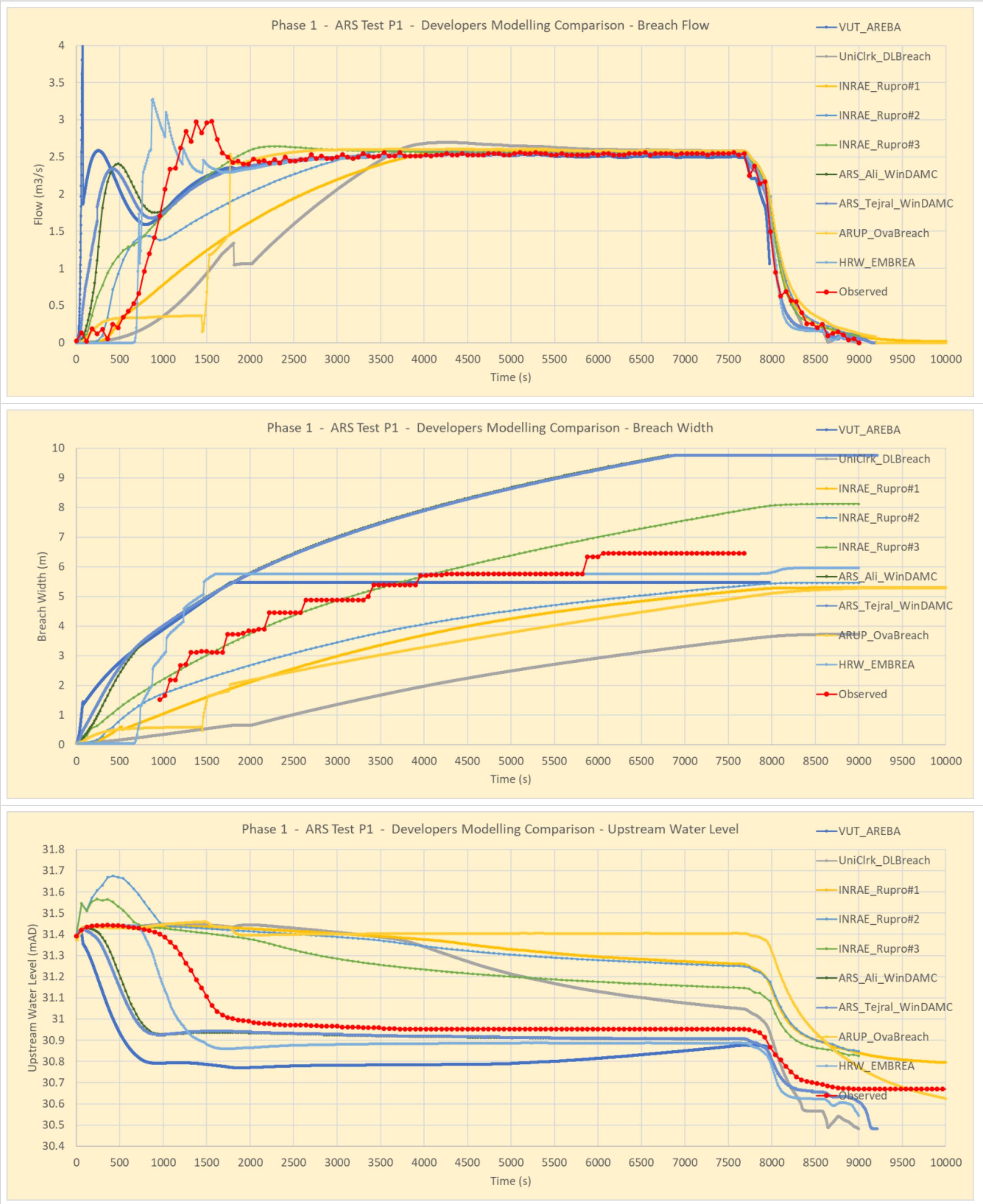


Figure E.3: Phase 1 – ARS P1: Developers modelling results

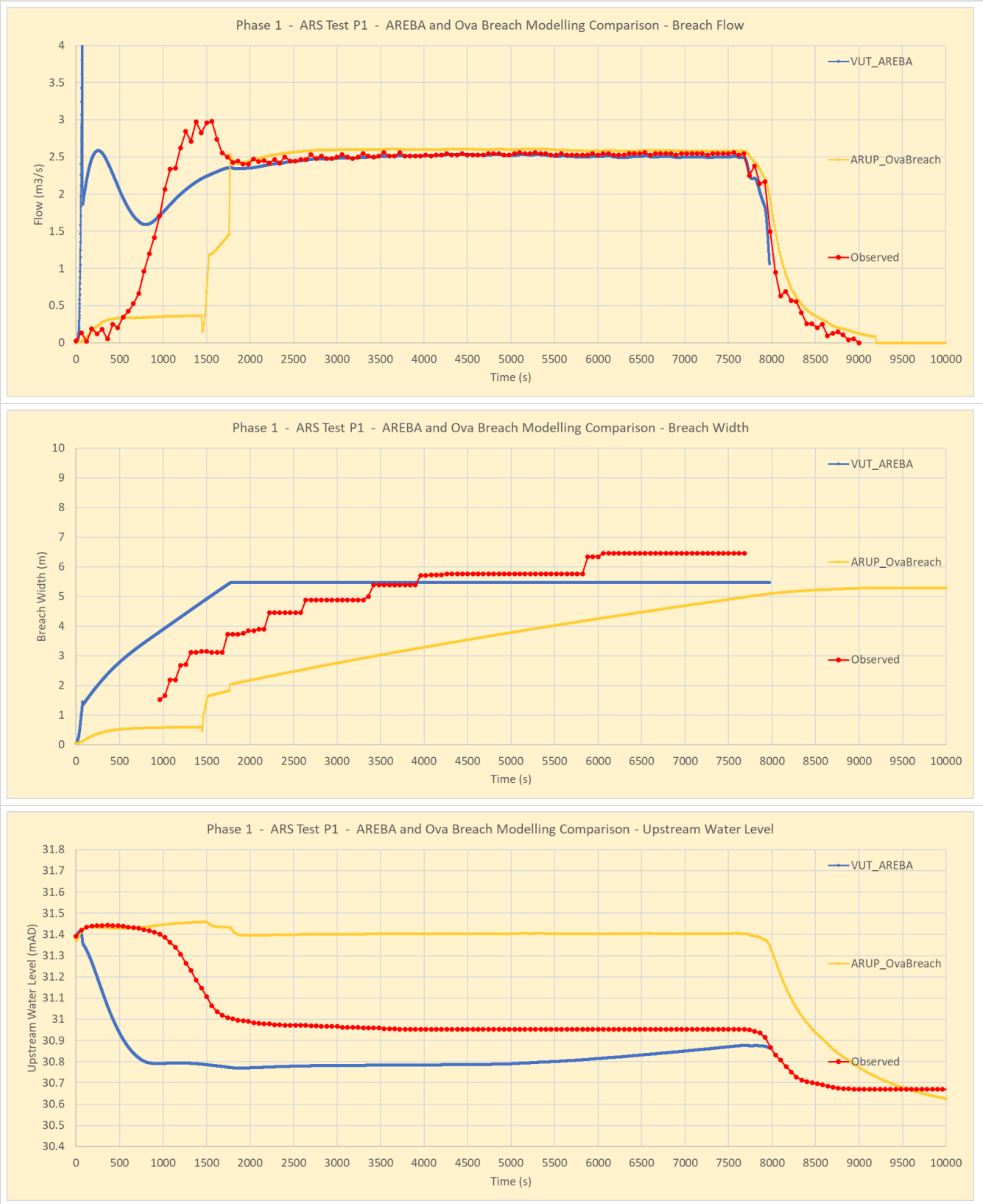


Figure E.4: Phase 1 – ARS P1: Modelling results using AREBA and OvaBreach

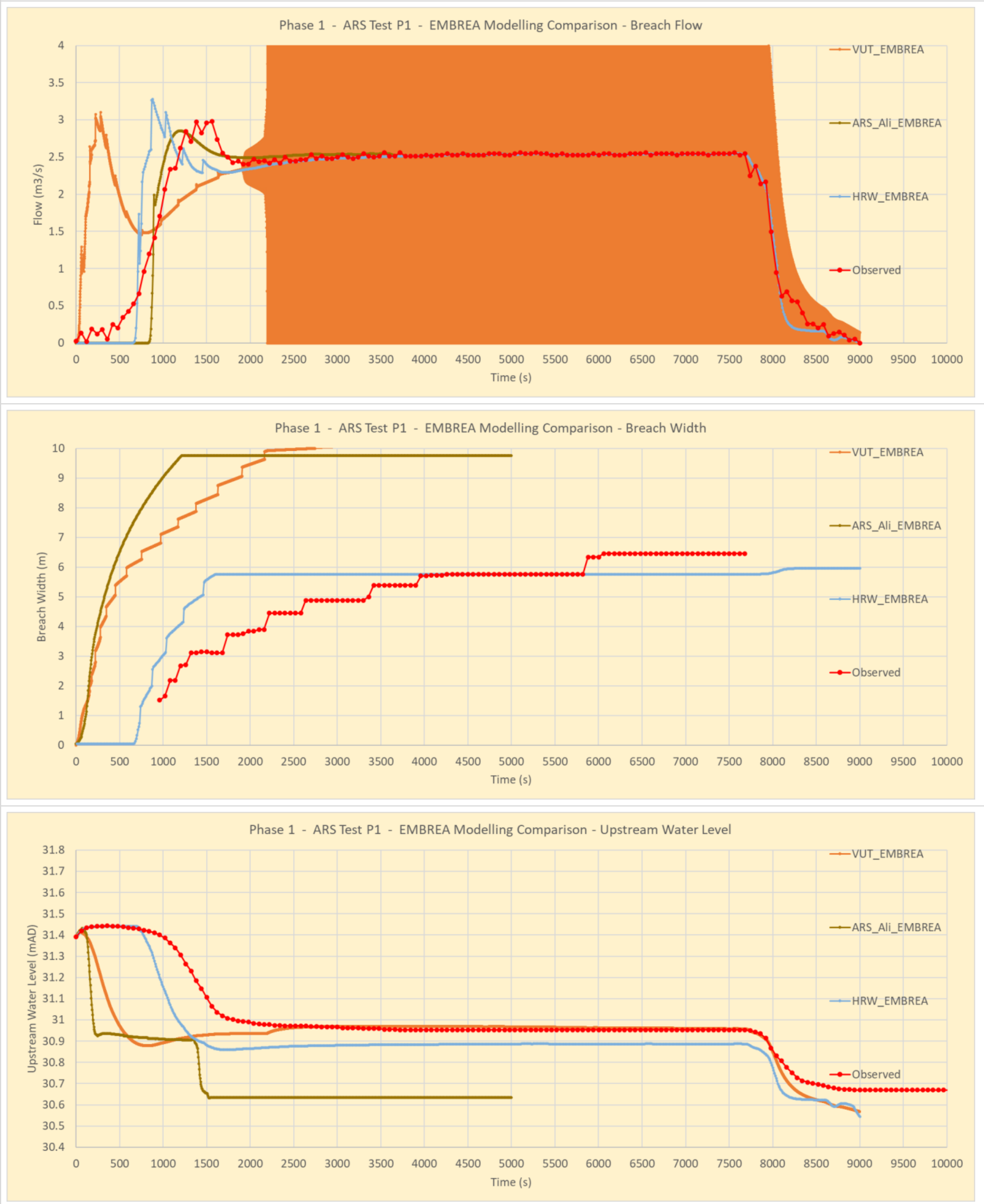


Figure E.5: Phase 1 – ARS P1: Modelling results using EMBREA

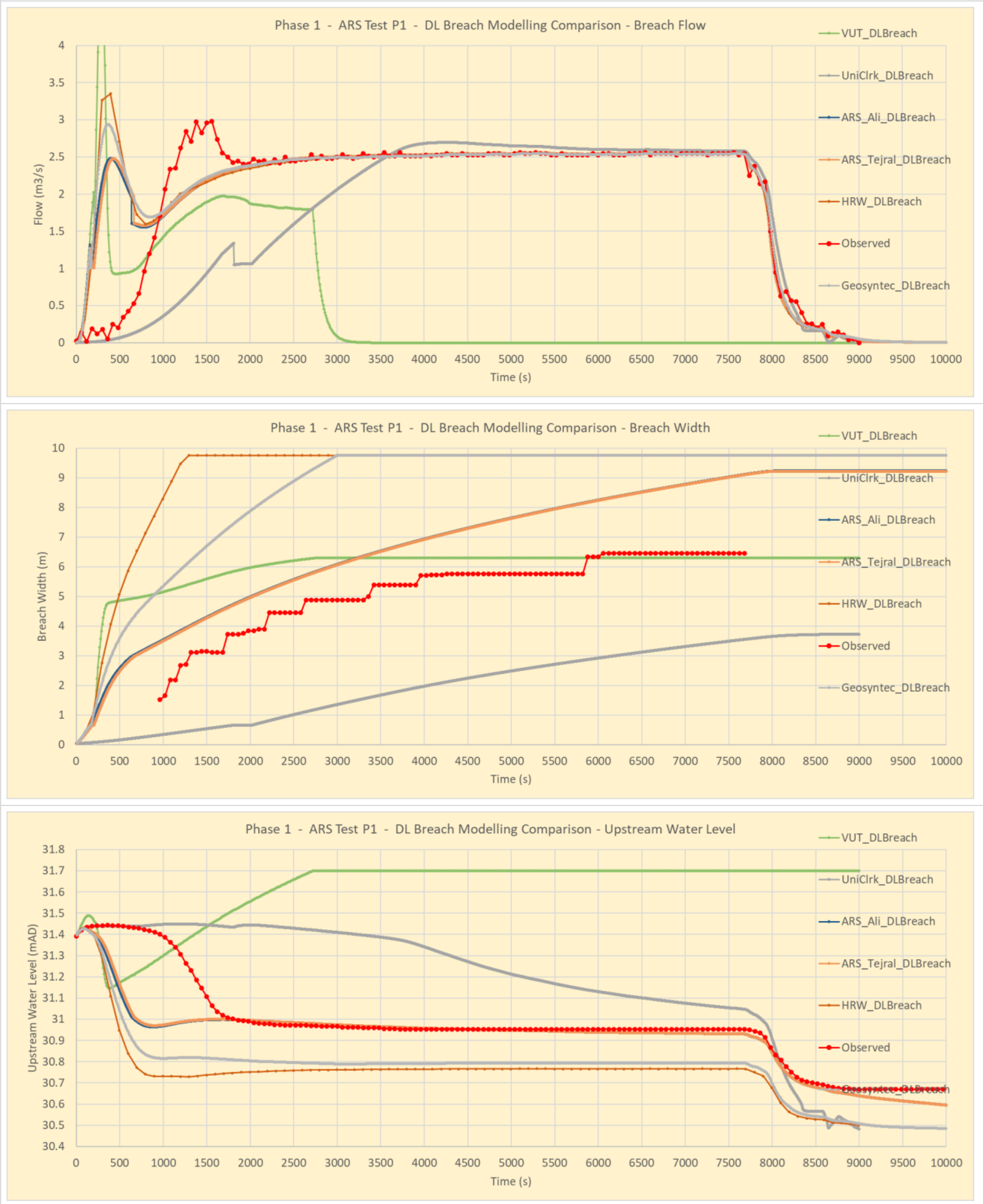


Figure E.6: Phase 1 – ARS P1: Modelling results using DLBreach

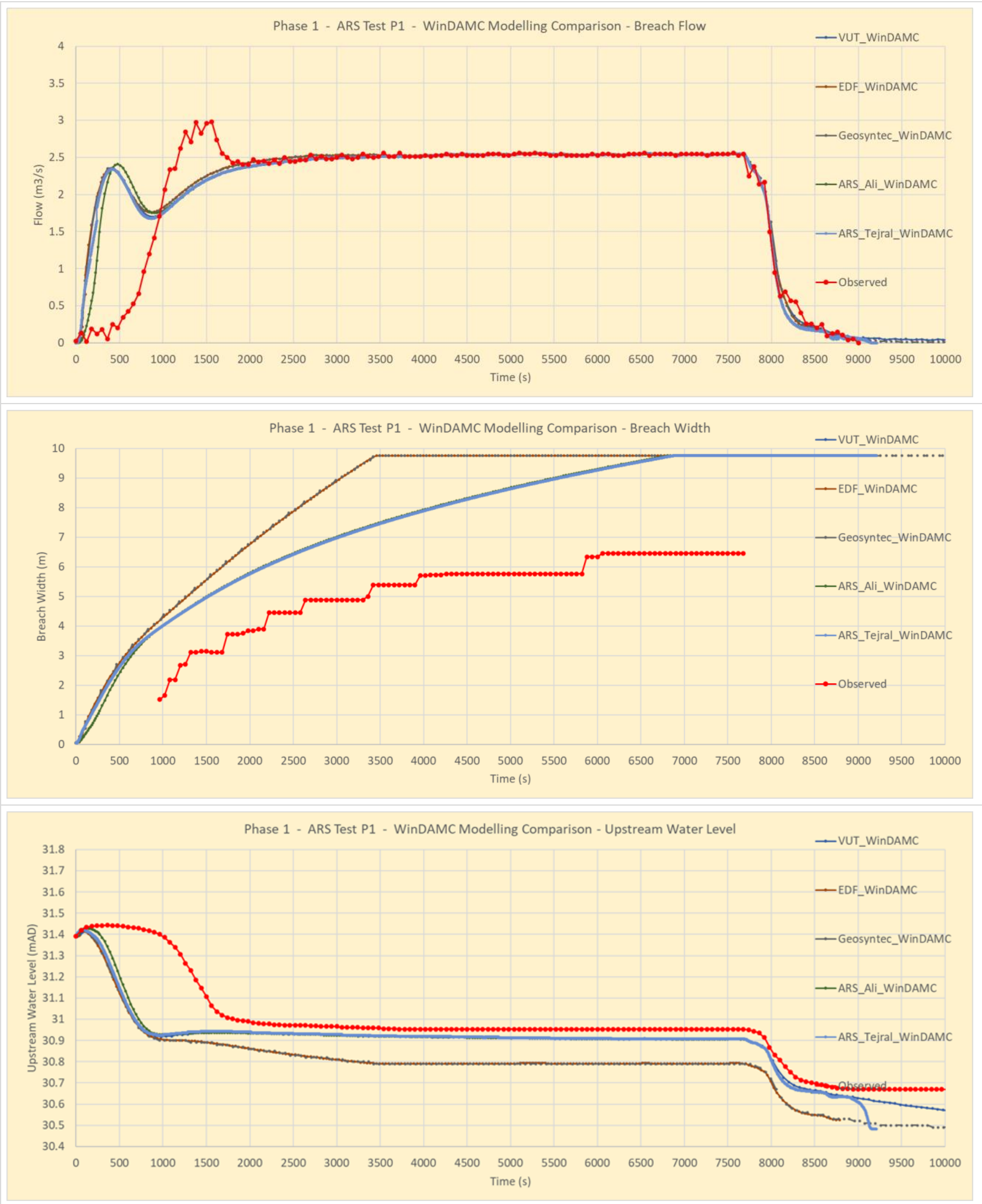


Figure E.7: Phase 1 – ARS P1: Modelling results using WinDAM C

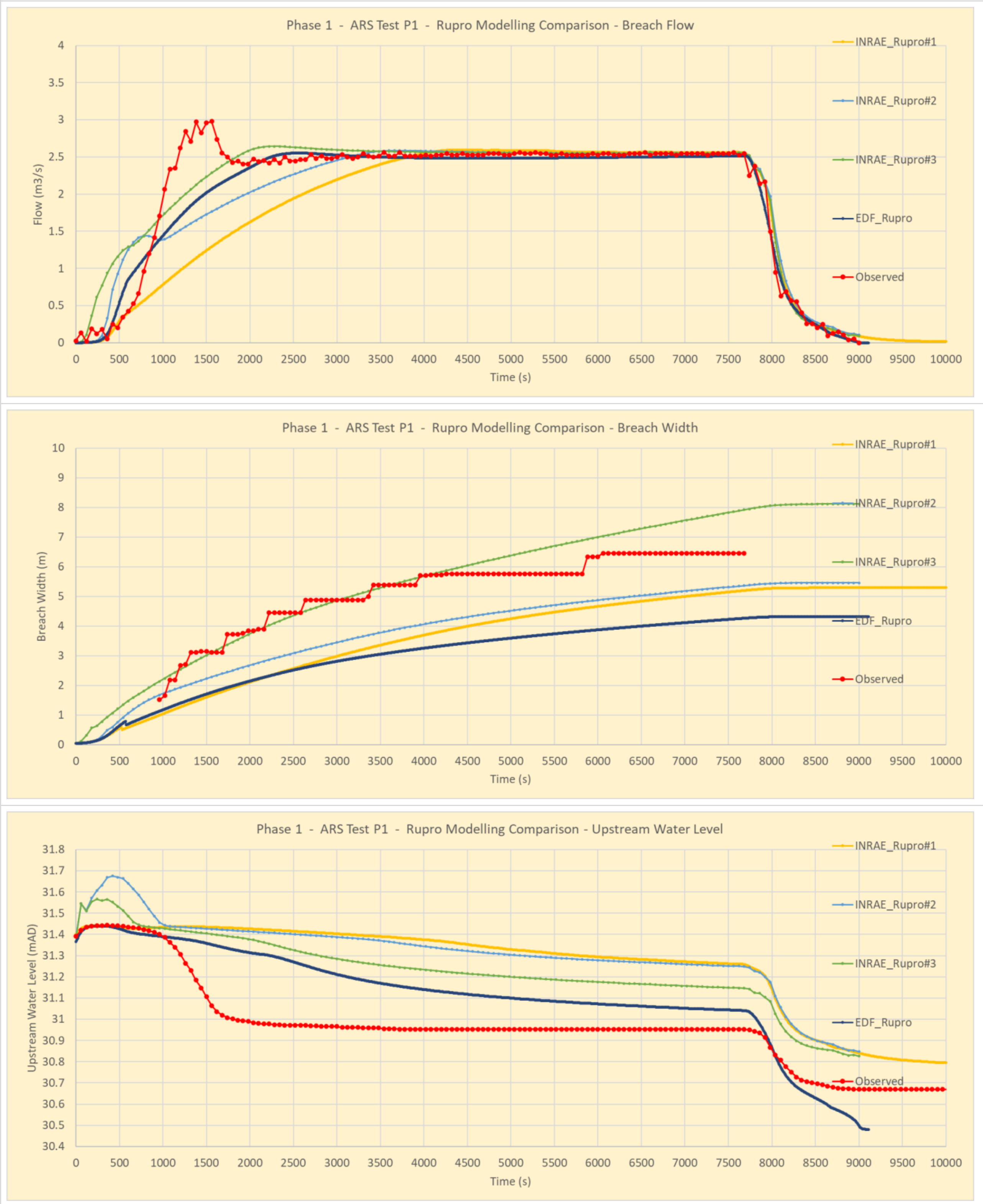


Figure E.8: Phase 1 – ARS P1: Modelling results using Rupro

E.4 Phase 1 – P1 Aware Modelling Results

Aware modelling results were submitted by USDA ARS, BUT and UniClrk for this test case:

ARS Ali – WinDAMC & DLBreach

Modelling using K_d=50 instead of 120 resulted in a very similar result for WinDAM C and no significant improvement using DLBreach, as shown in the Figure below. Here the reduction in soil erodibility results in a slower breach, with smaller surge in the hydrograph, but the failure remains predicted too quickly within the simulation.

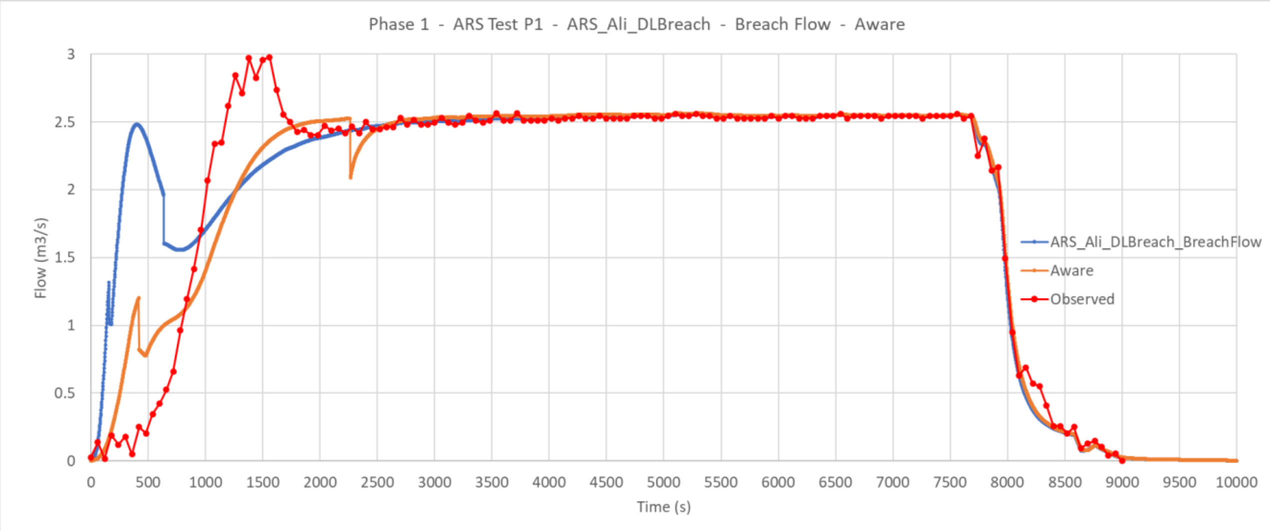


Figure E.9: Phase 1 – ARS P1 Aware Modelling results using DLBreach

BUT – TUD AREBA, EMBREA, WinDAM C and DLBreach

Here the parameter values for soil erodibility and critical shear stress were modified to improve performance:

	Erodibility	Critical shear Stress
Blind (Observed) value	120	0.144
TUD AREBA	6	0.144
EMBREA	8.5	0.144
WinDAM C	8.5	0.144
DLBreach	20	0.5

BUT – TUD AREBA

A significant improvement in performance was gained by adjusting these parameters (Figure E.10).

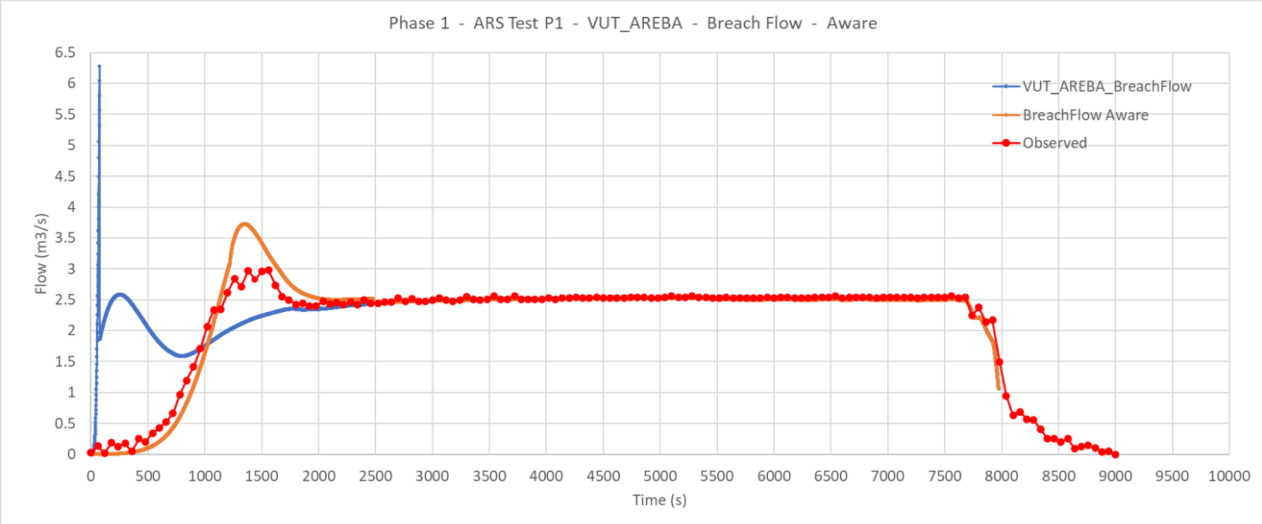


Figure E.10: Phase 1 – BUT P1 Aware Modelling results using AREBA

BUT – EMBREA

An improvement in timing was achieved (Figure E.11), but a better fit can be achieved by a different model setup – see Figure E.12 from HRW blind modelling using EMBREA.

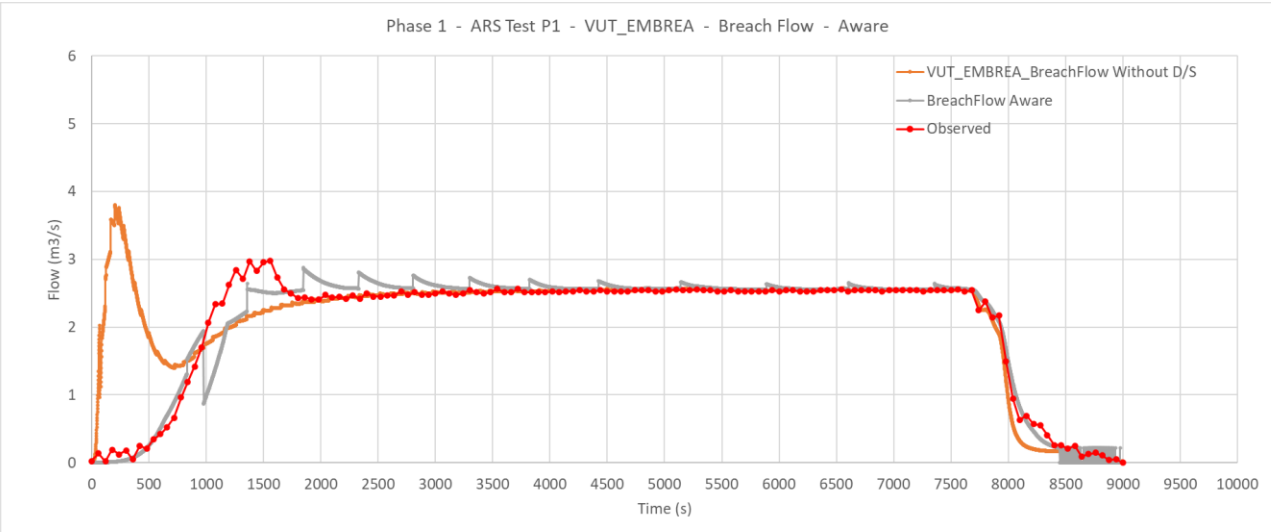


Figure E.11: Phase 1 – BUT P1 Aware Modelling results using EMBREA

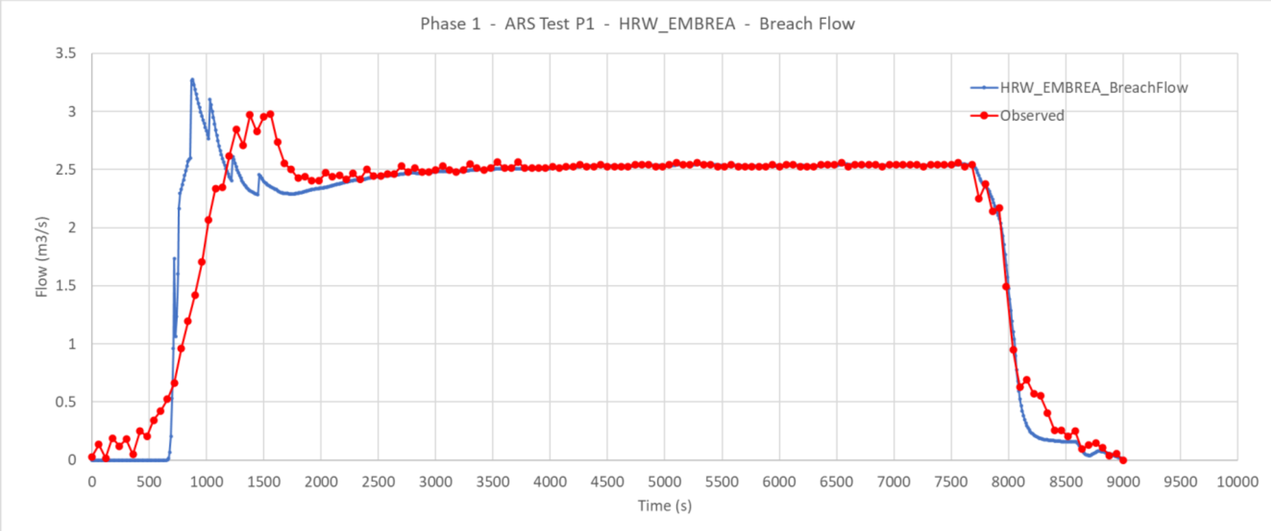


Figure E.12: Phase 1 – HRW P1 Blind Modelling results using EMBREA

HRW did not undertake aware modelling for this test case, considering their blind modelling results to be as close as reasonably expected.

BUT – WinDAM C

In this example, using the different parameters did not improve the modelling results (Figure E.13).

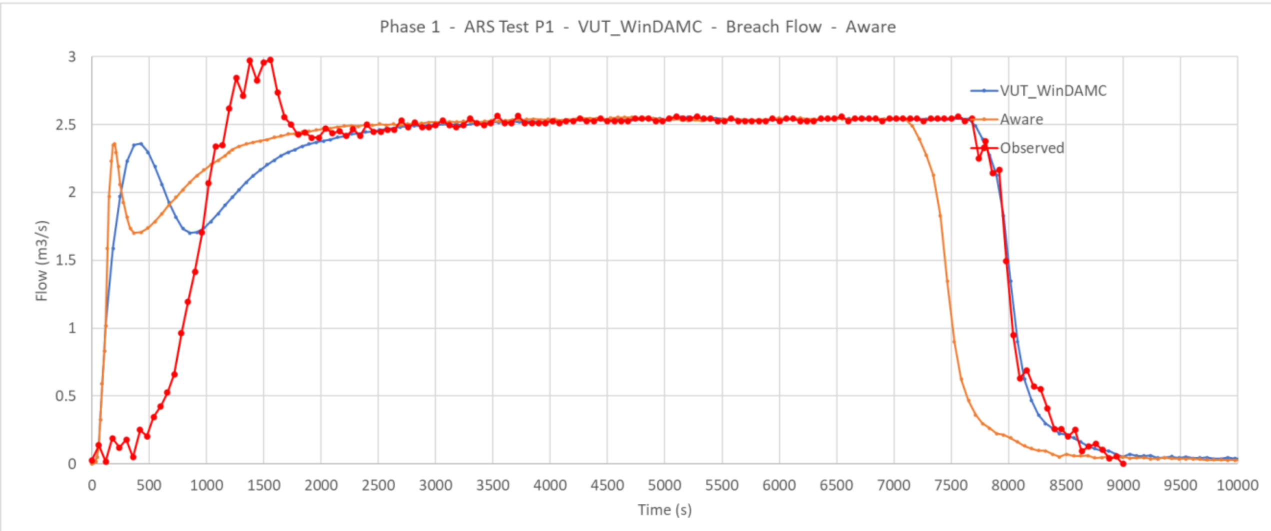


Figure E.13: Phase 1 – BUT P1 Aware Modelling results using WinDAM C

BUT – DLBreach

Using the different parameters here make a significant improvement to the modelling results (Figure E.14).

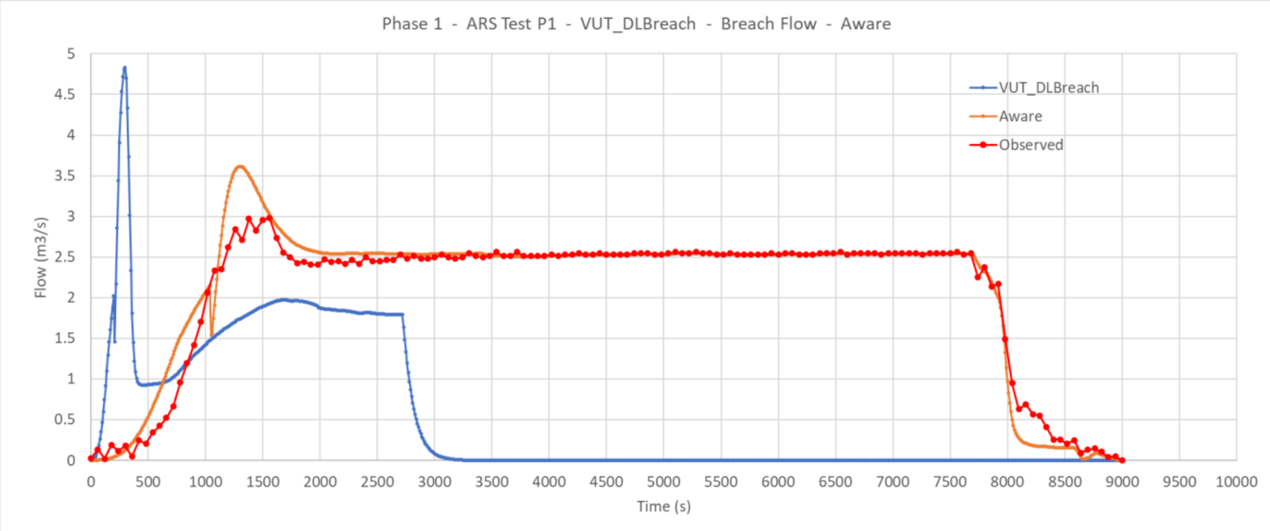


Figure E.14: Phase 1 – BUT P1 Aware Modelling results using DLBreach

UniClrk – DLBreach

A comparison of blind versus aware results using DLBreach can be seen in the Figure below.

It should be noted that the blind test results did not follow the defined parameter values for erodibility. A value of 10.3 instead of 120 cm^3/Ns was used with the statement "The given K_d value of 120 cm^3/Ns is much larger than the range of K_d values calibrated in DLBreach manual. A value of 10.3 cm^3/Ns is used in this blind test. This value was used for a similar SM soil in DLBreach manual". It can be seen that the modelling results are not close to the observed conditions.

For the aware modelling:

- The K_d is changed to 60 cm^3/Ns
- The pipe entrance head loss coefficient is adjusted from 0.05 to 1.5. This is done by using the card: Pipe_Entrance_Head_Loss_Coef 1.5
- The downstream backwater effect is significant. DLBreach does not use the rating curve, so the measured downstream water level is used as boundary condition.

This results in a closer overall hydrograph, however the actual breach process (rather than simply routing the test flow) is predicted too early and too small.

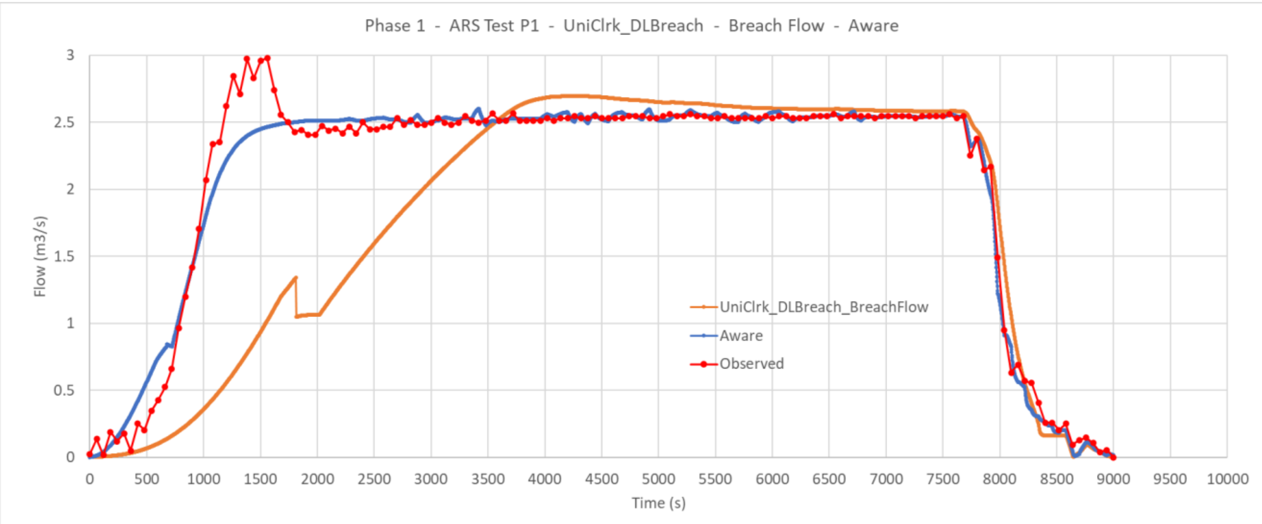


Figure E.15: Phase 1 – UniClrk P1 Aware Modelling results using DLBreach

F Phase 1 – ARS P4 Test Case

F.1 Phase 1 – ARS P4 Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	USDA-ARS-P4_Blind_v4.xlsx USDA-ARS-P4_Aware_v1.xlsx
Analysis & comparison of modelling results	Phase1_ModellingComparison_P4_21_03_01.xlsx

F.2 Test Case Description

This test case was performed at the USDA ARS site in Stillwater Oklahoma and consisted of a homogeneous earth embankment 1.24 m high, 9.75 m long, with a crest width of 1.98 m and slopes of approximately 1 in 3. A pipe of 0.04 m diameter was created through the levee 0.23 m from the base, by removing a rigid pipe of that diameter, which had been constructed into the levee.

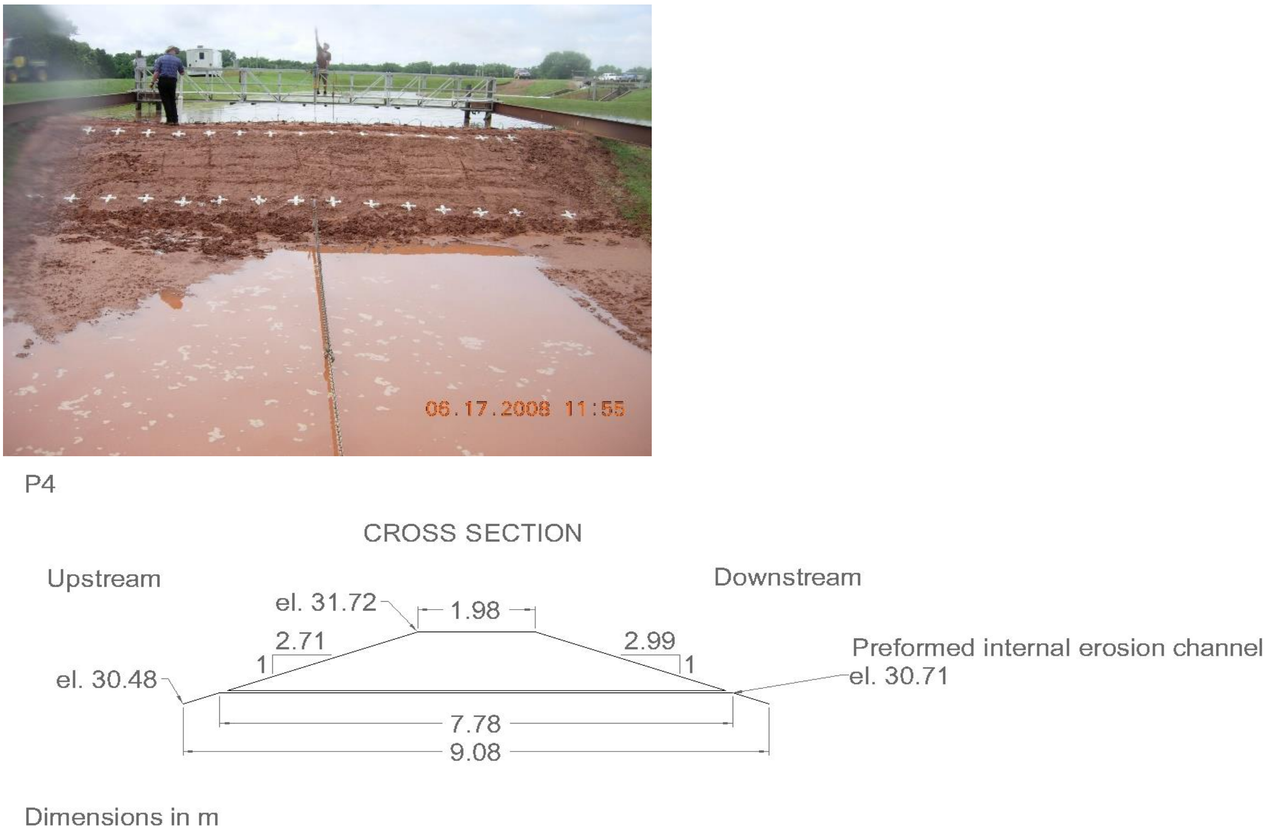
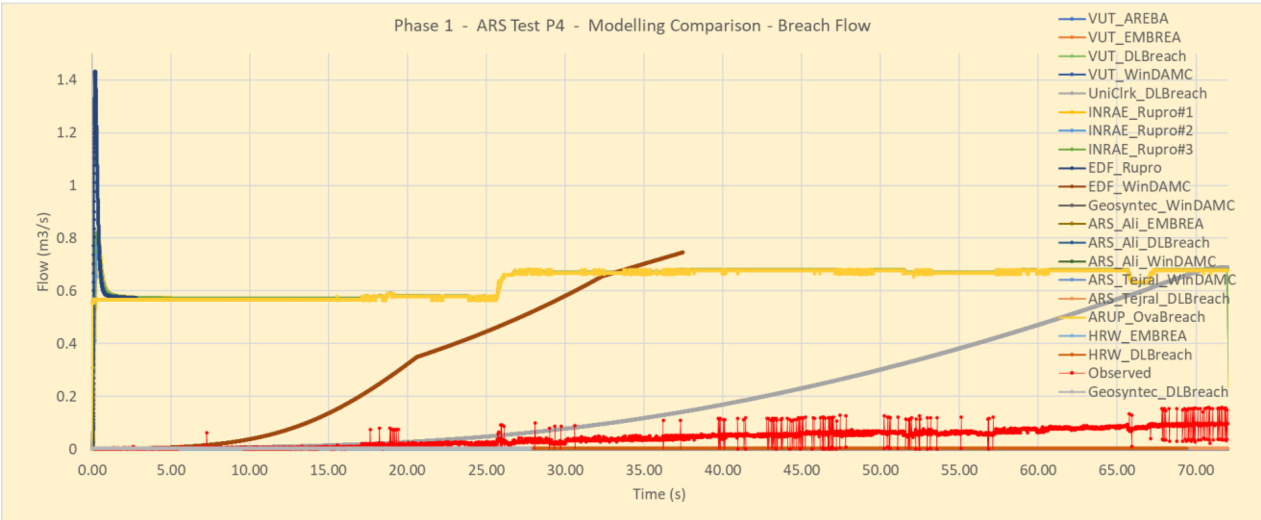


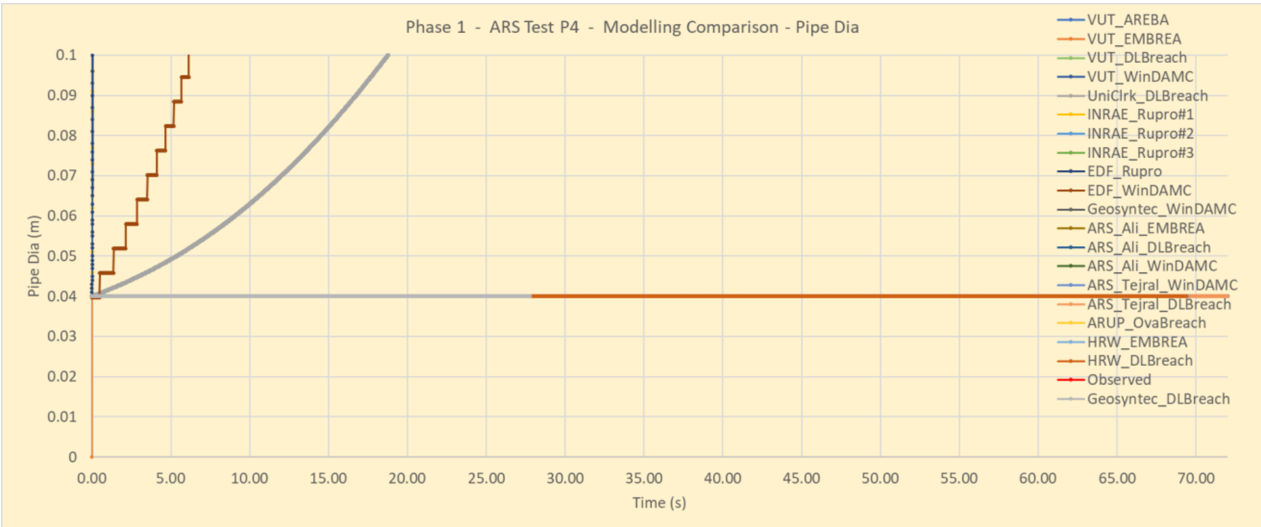
Figure F.1: ARS P4 internal erosion test case

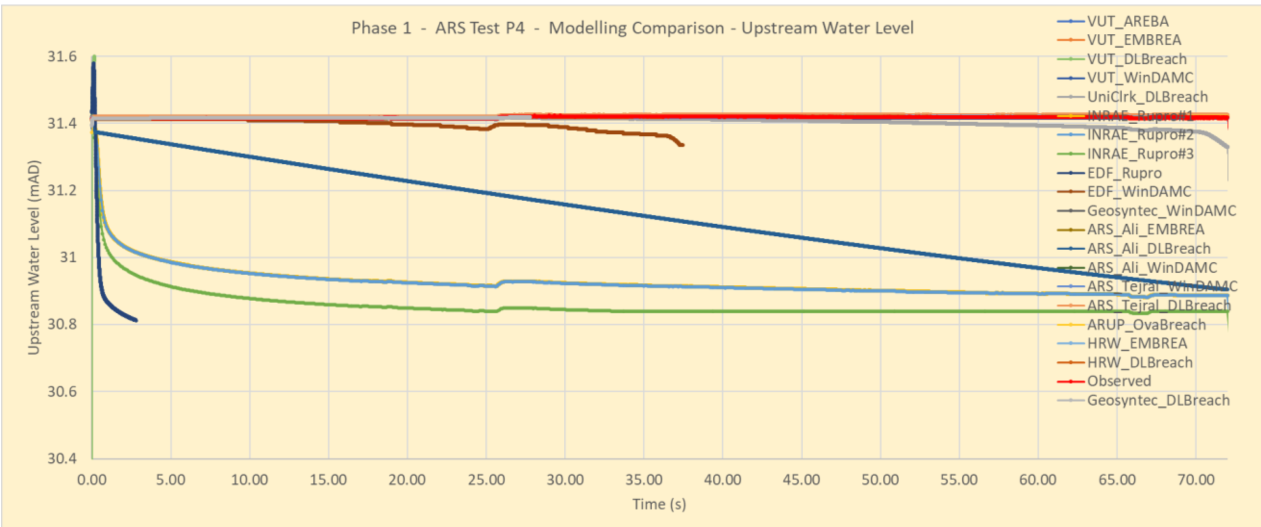
Table F.1: Phase 1 – ARS P4: Modeller Assumptions

Models & Modellers:			**Structure**		**Initiation**			**Soil Parameters**								**Flow**	**Computational**					**Reported Problems or observations**
		Variables	Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	Ron Tejral & Ali Abdelfatah																					
	Ron Tejral																					
Blind	WinDAMC	missing																				
Blind	DL Breach	✓	Friction angle estimated from soil class; cohesion approximated from angle and undrained shear strength.	0.00	0.04 × 0.04 m square	center	0 sec	0.1	N/A	34	27	0.35	N/A	N/A	35	0.016	5	N/A	From pipe or open-channel breach invert at u/s to outlet or headcut at d/s.	N/A	N/A	Spillways and tailwater stage-discharge relationships cannot be defined by tables; complicates data entry. No elevation setting for tailwater by Manning formula.
Aware	None	×																				
	Ali Abdelfatah																					
Blind	WinDAMC	Problem - zero data extracted	Cu = 55 Kpa	30.48	0.04 X 0.04 m Rectangle	5	0 Sec.	0.1	2050	N/A	N/A	N/A	N/A	N/A	35	0.02	60 sec.	N/A	Theoretically, the breach dimensions spread from	N/A	N/A	WinDAM has option of dividing user-entered timestep to limit % change in peak discharge and maximum water surface elevation. I had entered 60.2 s (0.017 hrs),
Blind	DL Breach	✓	Cu = 55 Kpa	30.48	0.04 X 0.04 m Rectangle	5	0 Sec.	0.1	2050	34	27	0.35	N/A	N/A	35	0.16	60 sec.	N/A	Theoretically, the breach	N/A	N/A	WinDAM has option of dividing user-entered timestep to limit % change in peak discharge and maximum water surface elevation. I had entered 60.2 s (0.017 hrs),
Blind	EMBREA	✓	Cu = 55 Kpa	30.48	0.04 X 0.04 m Rectangle	5	0 Sec.	0.1	2050	34	27	0.35	N/A	N/A	35	0.02	60 sec.	N/A	Theoretically, the breach dimensions spread from invert at up/s to headcut at d/s.	N/A	N/A	WinDAM has option of dividing user-entered timestep to limit % change in peak discharge and maximum water surface elevation. I had entered 60.2 s (0.017 hrs),
ARUP	Veronika Stoyanova																					
	ARUP OvaBreach	✓	Homogenious, tail water depth ignored	30.48	0.04	as per diagram	0.00	0.1	2050	34	27	0.35	n/a	n/a	35	0.007705463	60s	n/a (1D model)		0.0000001	0.000205	The Manning's n and ks values are functions of d50.
Aware	None	×																				
HRW	Mohamed Hassan																					
	DL BREACH	✓	Parameters were taken as provided. No changes were made I was not able to model the rating curve downstream boundary as this option is not available in the DLBreach model. I had also to fit a curve for the spillway outflow as the equation exponent in DL Breach model is a fixed value.	30.48	0.04	30.71	0	0.1	1740	34	27	0.35	NA	NA	35	0.03	10	NA	Critical section which moves with time and is not fixed	NA	NA	Run showed no erosion of the pipe in either the vertical or the lateral directions. Top of the pipe did not collapse within the simulation which was about 72 hours. output file showed a breach outflow, width and area of zero which needs explanation. A number of runs were undertaken to determine the critical shear stress at which erosion can be initiated. A value of 6 Pa was low enough to do so. The results of this run (i.e. with a critical shear stress value of 6 Pa) are shown in sheet BLIND Modelling Results (2).
	EMBREA	✓	Parameters were taken as provided in the test case description. No changes were made	30.48	0.04	30.71	0	0.1	1740	34	27	0.35	NA	NA	35	0.03	5	5	Critical section which moves with time and is not fixed	NA	NA	Run showed no erosion of the pipe in either the vertical or the lateral directions. Top of the pipe did not collapse within the simulation which was about 72 hours. A number of runs were undertaken to determine the critical shear stress at which erosion can be initiated. A value of 7.5 Pa was low enough to do so. The results of this run (i.e. with a critical shear stress value of 7.5 Pa) are shown in sheet BLIND Modelling Results (2).
			Run with critical shear stress of 7.5 Pa												**7.5**							Erosion of pipe takes place but no pipe collapse. Some oscillations occurred at the end of the run due to either changes between free and pipe flow and/or downstream drowning. These will be investigated later.
Aware	None	×																				
ERAU	Ghada Elithy																					
Blind	None	×																				
Aware	None	×																				
VUT	Stanislav Kotaska																					
Blind	TUD AREBA	✓	Homogenous dam without protection	30.48	0.04	middle	-	0.1	2010	34	27	0.35	-	-	35	0.03	1	-	-	-	-	
Blind	EMBREA	✓	Homogenous dam without protection	30.48	0.04	middle	-	0.1	2010	34	27	0.35	-	-	35	0.03	1	-	-	-	-	
Blind	WinDAM	✓	Homogenous dam without protection	30.48	0.04	middle	-	0.1	2010	34	27	0.35	-	-	35	0.03	-	-	-	-	-	
Blind	DL BREACH	✓	Homogenous dam without protection	30.48	0.04	middle	-	0.1	2010	34	27	0.35	-	-	35	0.03	1	-	-	-	-	
Aware	TUD AREBA	✓																				
Aware	EMBREA	✓	Results file empty																			
Aware	WinDAM	✓																				
Aware	DL BREACH	✓																				
Geosyntec	Al Preston																					
Blind	WinDAM	✓		30.48	0.04	4.9	0	0.1	2050	34	27	0.35			35	0.03	60.12					
Blind	DL BREACH	✓		30.48	0.04	middle	0	0.10	2050	34	27	0.35			35	0.03	0.5					
Aware	None	×																				
	André Paquier																					
Blind	Rupro #1	✓			0.04		0		2670			0.35				0.033	10s					calculation 1 using castorDigue
Blind	Rupro #2	✓			0.04		0		2670			0.35				0.033	0.5 s					calculation 2 using Rubar 20 same assumptions as CastorDigue
Blind	Rupro #3	✓			0.04		0	0.1	2670			0.35			35	0.033	0.5 s					Calculation 3 using Rubar 20 and provided erodibility value
Aware	Rupro #1	×																				
Aware	None	×																				
Aware	Rupro #3	×																				
UniClrk	Weiming Wu																					
Blind	DL BREACH	✓	trapezodal cross-section: dam is 1.24 m high, dam crest is 1.98 m wide, upstream slope 1V:2.71H and downstream slope 1V:2.99H. The dam foundation is assumed nonerodible.	30.48	0.04	middle, 1.01 m below dam crest	at t=0 s	0.1	2.67	34	27	0.348			5	0.016	0.2		dam crest			The critical shear stress for incipient erosion was measued as 35 Pa by using JET test. This value does not allow any erosion in the pipe, since the applied shear stress is less than this value. Instead, the critical shear stress is set as 5.0 Pa in this blind test. The pipe roof does not collapse in the entire simulation period. The breach is not fully developed
Aware	DL BREACH	✓													3.7							The critical shear stress is calibrated as 3.7 Pa The pipe entrance head loss coefficient is adjusted to 1.5 by using the following card in the input file: Pipe_Entrance_Head_Loss_Coef 1.5
EDF	Pierre Squillari (Geophy)																					
Blind	Rupro	✓		30.48	0.04	middle of the dam	no delay in initiation	-	2650 (grain)	-	-	0.35	-	-	-	0.03	5s					grain diameter taken as D50 = 0.02 mm
Blind	WinDAM	✓		30.48	0.04	middle of the dam	no delay	0.1	2010	34	27	0.35			35	0.03	< 0.01 h					
Aware		×																				

F.3 Phase 1 – ARS P4 Modelling Results







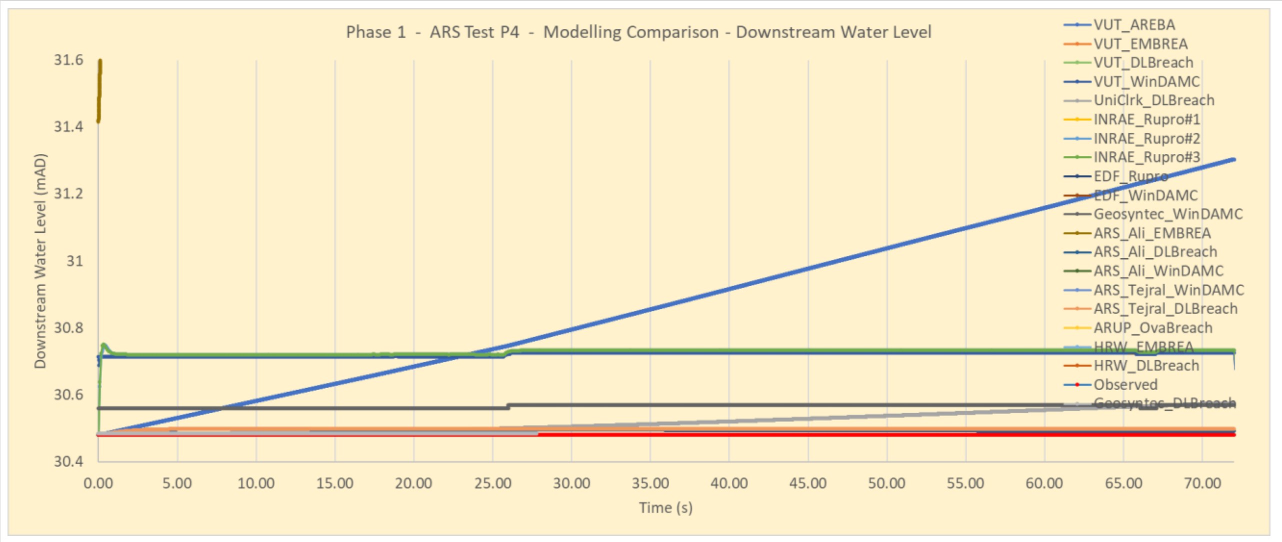


Figure F.2: Phase 1 – ARS P4: All modelling results

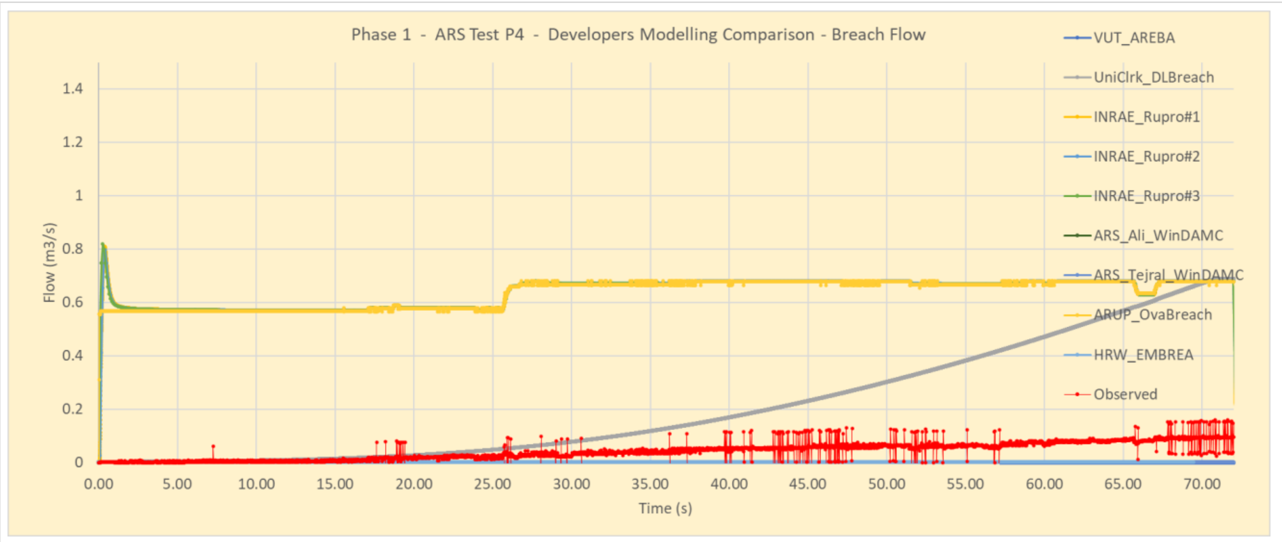


Figure F.3: Phase 1 – ARS P4: Developers modelling results

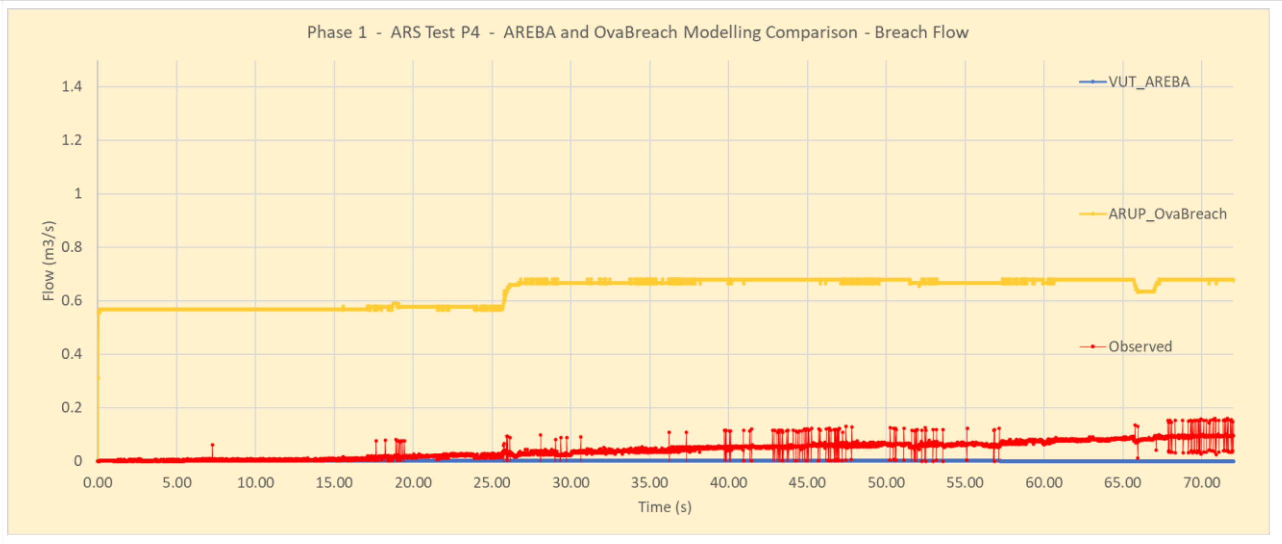


Figure F.4: Phase 1 – ARS P4: Modelling results using AREBA and OvaBreach

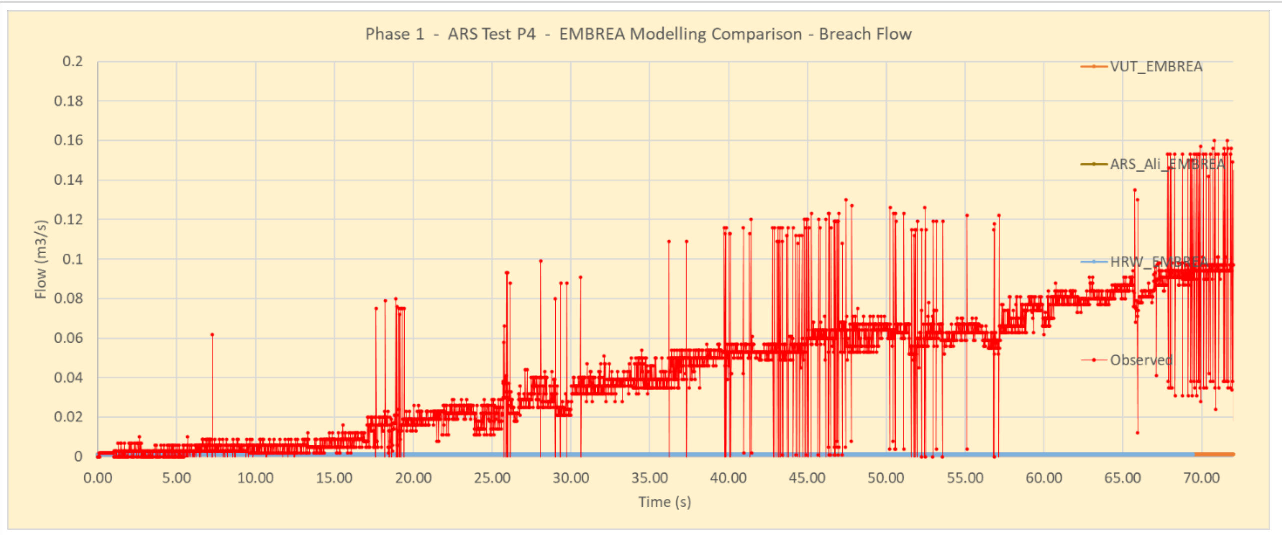


Figure F.5: Phase 1 – ARS P4: Modelling results using EMBREA

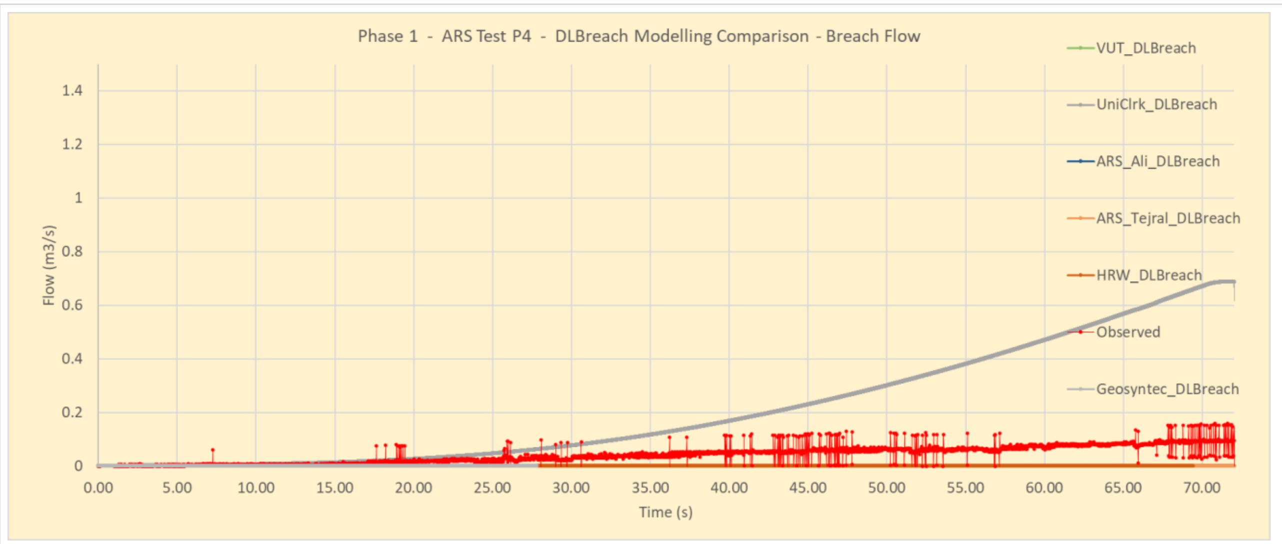


Figure F.6: Phase 1 – ARS P4: Modelling results using DLBreach

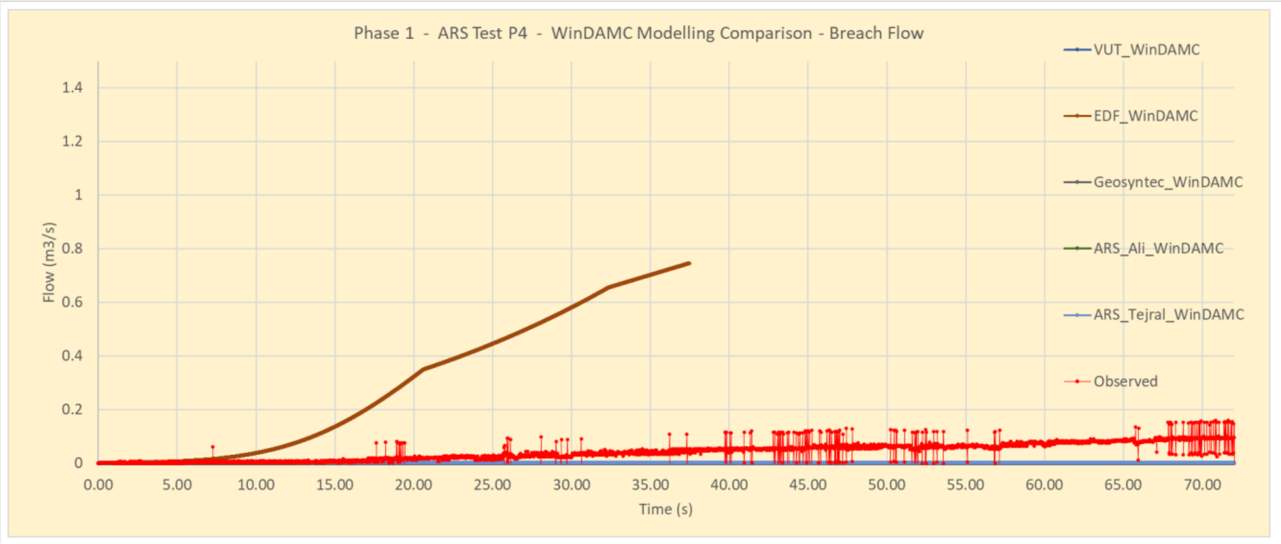


Figure F.7: Phase 1 – ARS P4: Modelling results using WinDAM C

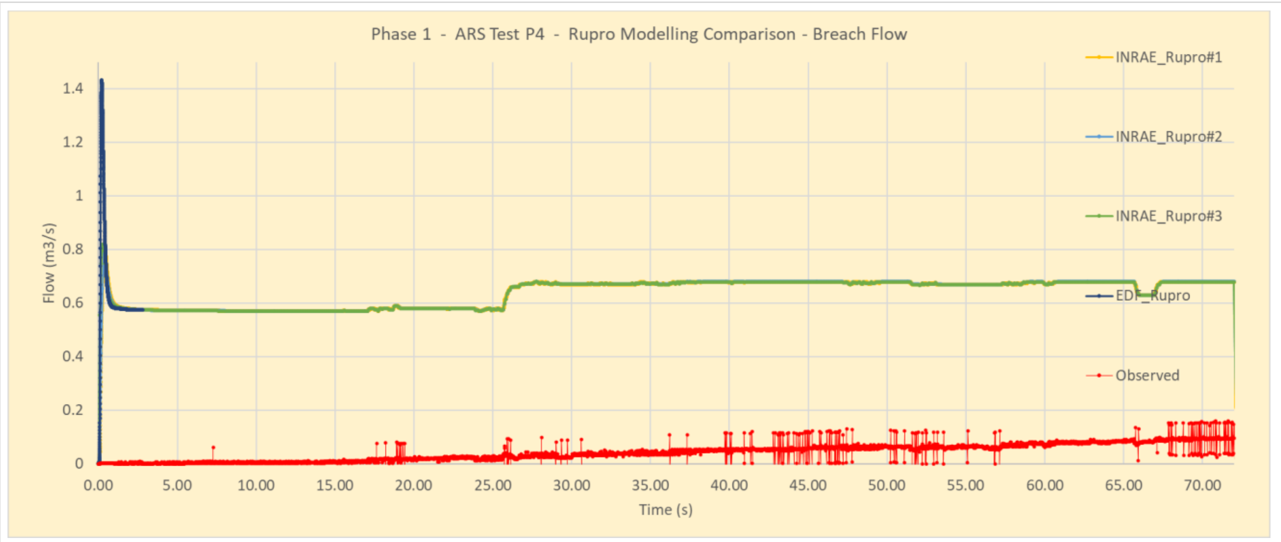


Figure F.8: Phase 1 – ARS P4: Modelling results using Rupro

F.4 Phase 1 – P4 Aware Modelling Results

Aware modelling results were submitted by BUT, UniClrk, HRW and INRAE for this test case:

UniClrk – DLBreach

It should be noted that the blind test results did not follow the defined parameter values for the critical shear stress which were measured on site as being 35 Pa.

For the blind test, UniClrk noted:

- The critical shear stress for incipient erosion was measured as 35 Pa by using JET test. This value does not allow any erosion in the pipe, since the applied shear stress is less than this value. Instead, the critical shear stress is set as 5.0 Pa in this blind test
- The pipe roof does not collapse in the entire simulation period. The breach is not fully developed."

For the aware test, modifications were made as follows:

- The critical shear stress is calibrated as 3.7 Pa
- The pipe entrance head loss coefficient is adjusted to 1.5 by using the following card in the input file: Pipe_Entrance_Head_Loss_Coef 1.5.

Results for flow and breach width are show in Figure F.9 below. It can be seen that whilst the aware adjustments gave a better match to the flow, the prediction of breach dimensions became worse.

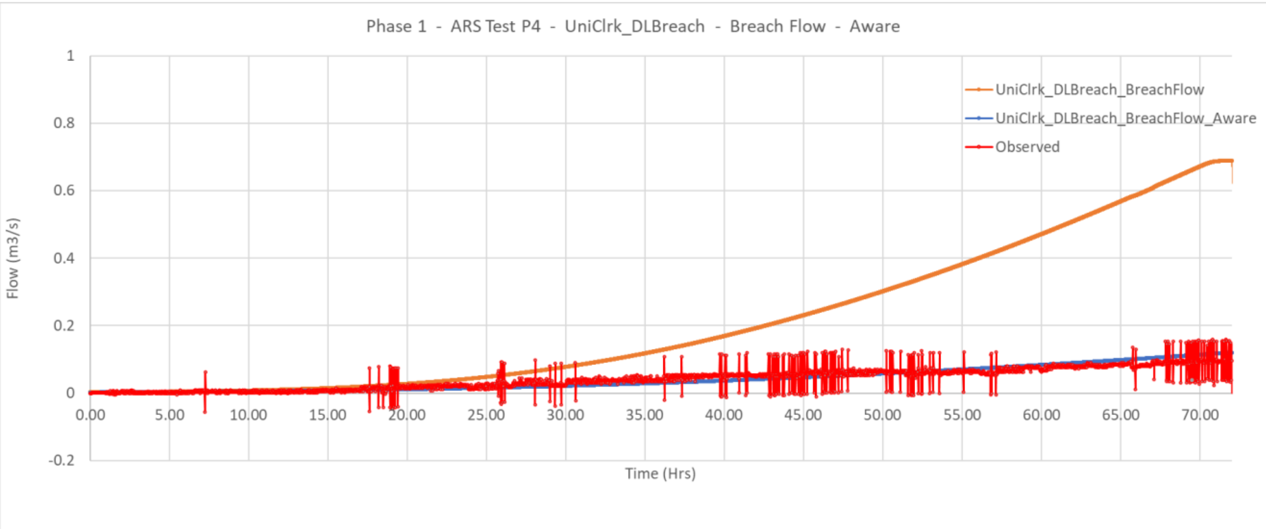


Figure F.9: Phase 1 – UniClrk P4 aware flow modelling results using DLBreach

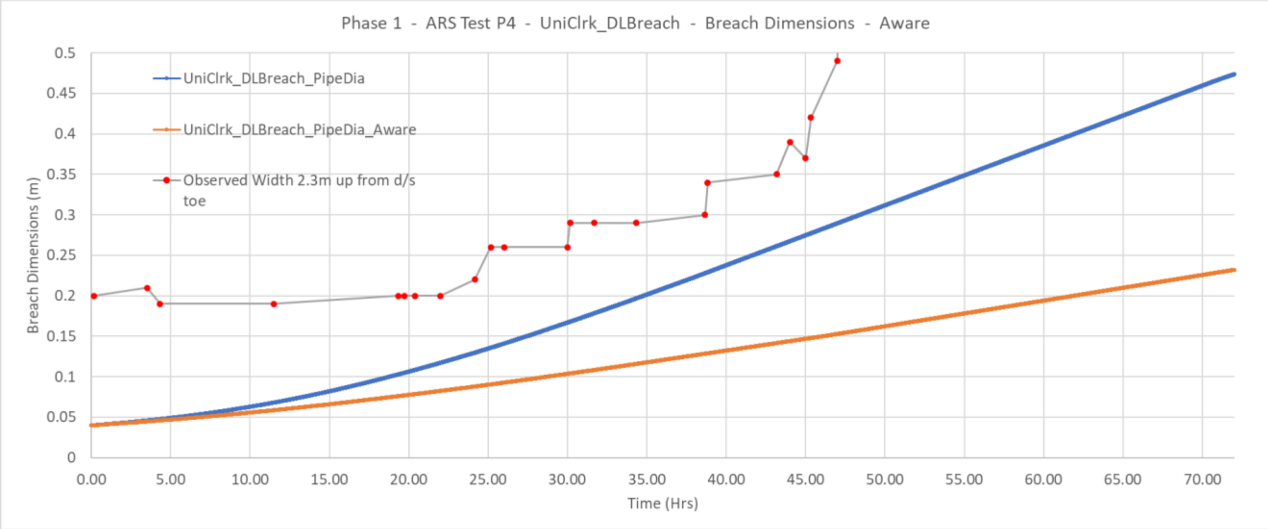


Figure F.10: Phase 1 – UniClrk P4 aware breach width modelling results using DLBreach

BUT – AREBA

Adjusting modelling parameters allowed BUT with AREBA to improve both flow and breach dimension predictions.

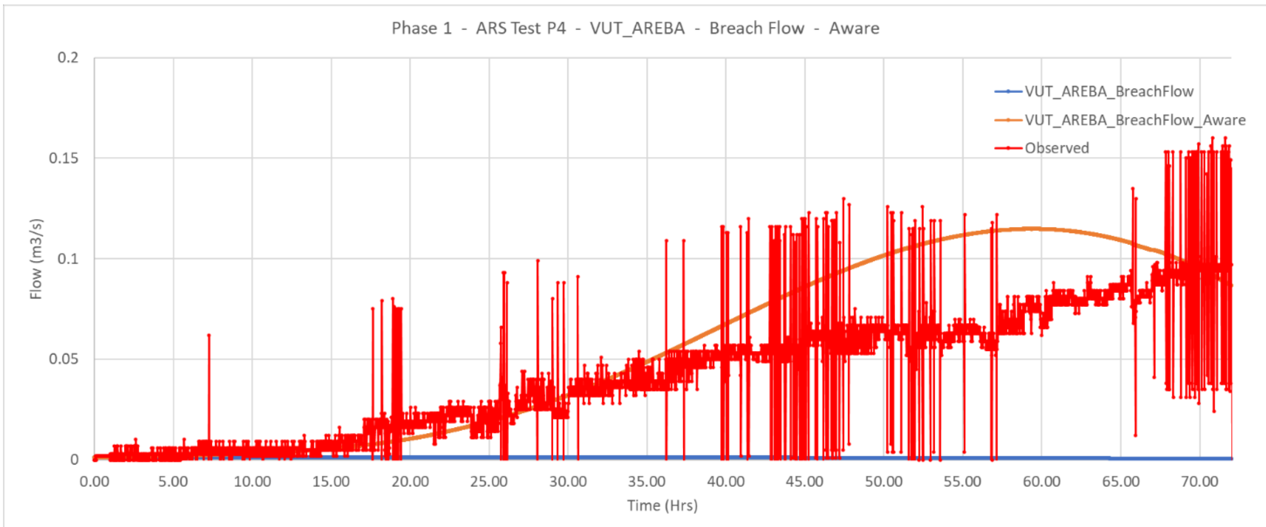


Figure F.11: Phase 1 – BUT P4 aware flow modelling results using AREBA

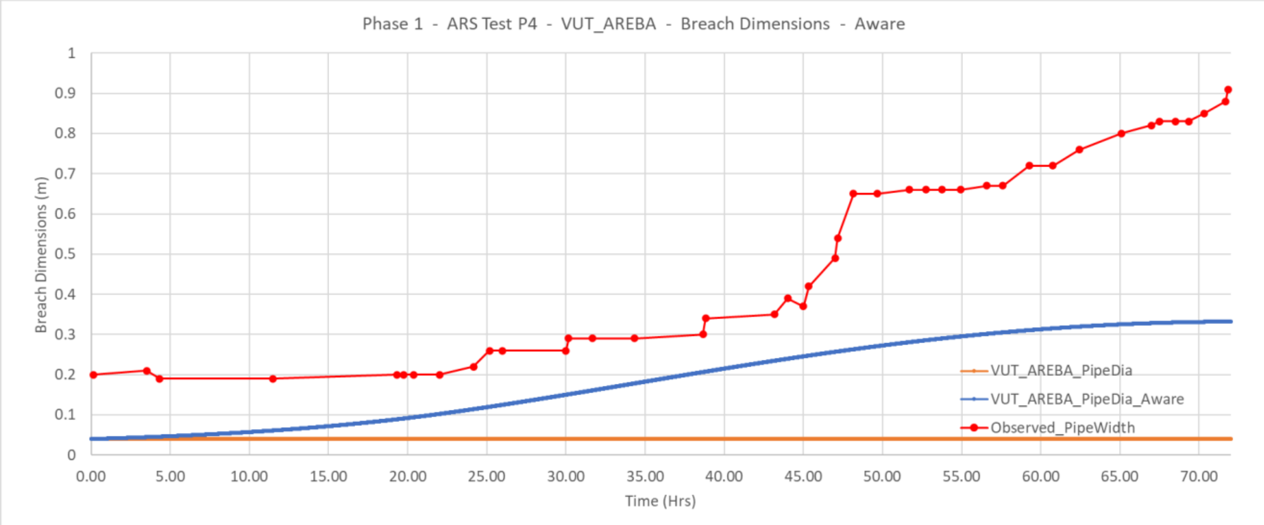


Figure F.12: Phase 1 – BUT P4 aware breach width modelling results using AREBA

BUT - DLBreach

A similar trend was achieved with DLBreach.

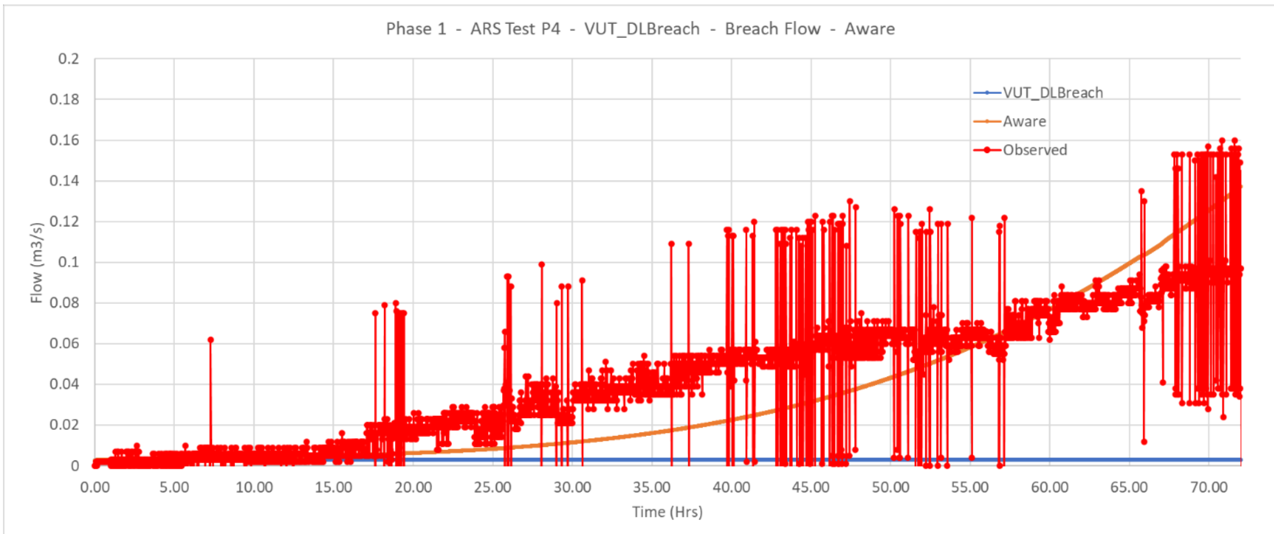


Figure F.13: Phase 1 – BUT P4 aware flow modelling results using DLBreach

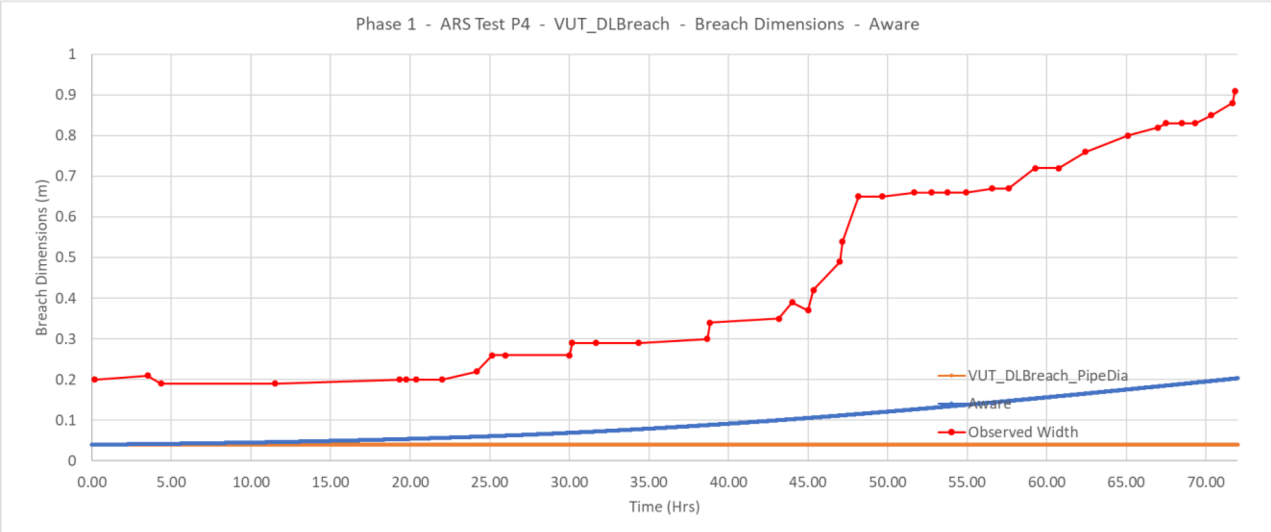


Figure F.14: Phase 1 – BUT P4 aware breach width modelling results using DLBreach

HRW – DLBreach

HRW provided two sets of run data after analysing an initial run where no erosion took place.

Following the initial blind run, the following conclusions were drawn and used to define parameters for a second run:

- Run showed no erosion of the pipe in either the vertical or the lateral directions. Top of the pipe did not collapse within the simulation which was about 72 hours. output file showed a breach outflow, width and area of zero which needs explanation
- A number of runs were undertaken to determine the critical shear stress at which erosion can be initiated. A value of 6 Pa was low enough to do so. The results of this run (i.e. with a critical shear stress value of 6 Pa) are shown as a second run (aware run).

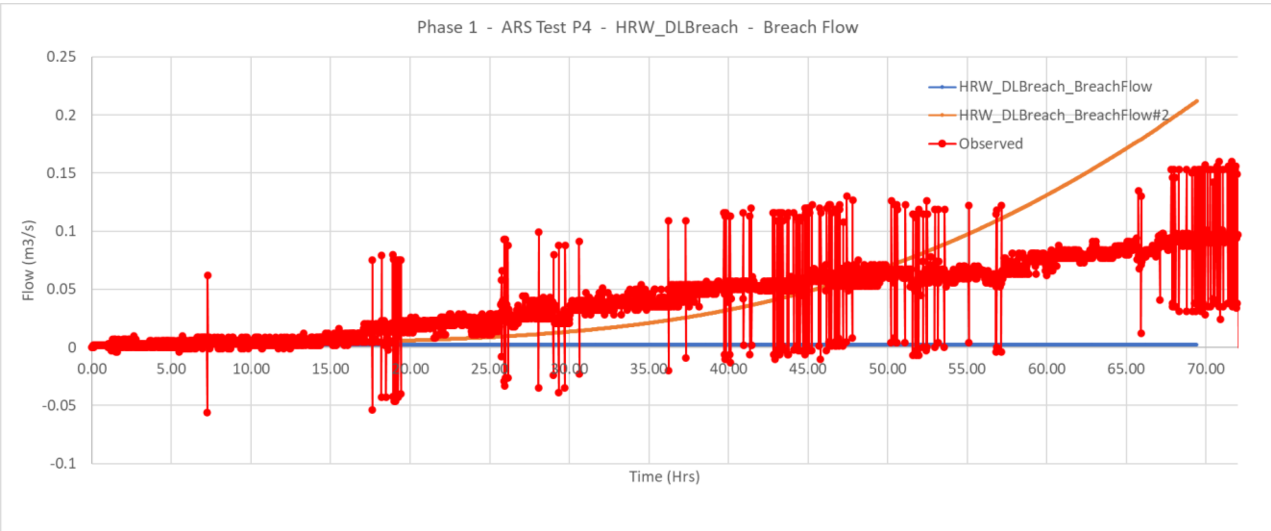


Figure F.15: Phase 1 – HRW P4 aware flow modelling results using DLBreach

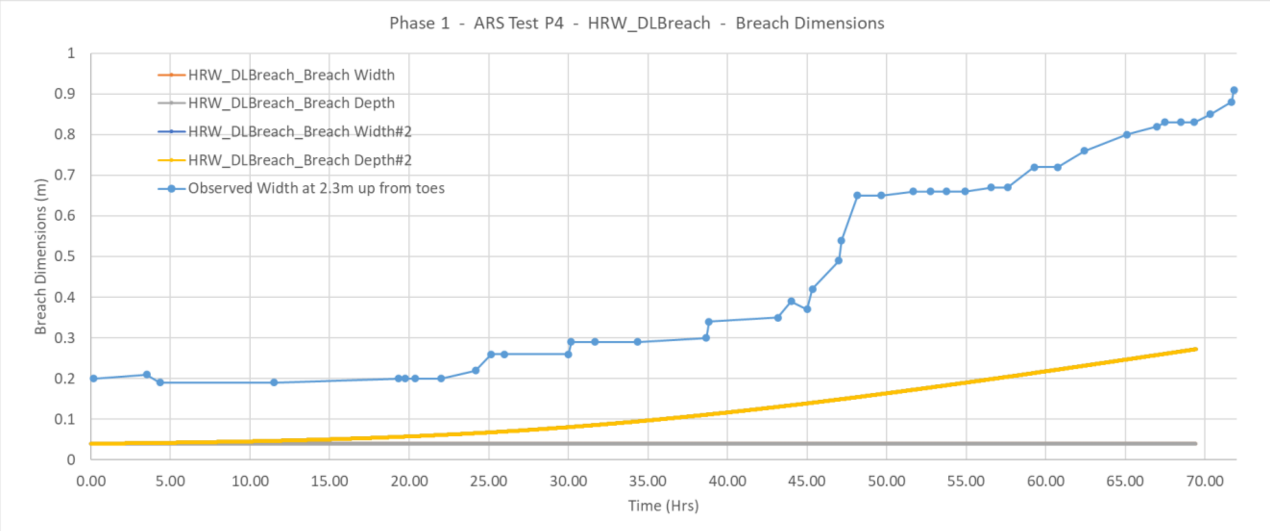


Figure F.16: Phase 1 – HRW P4 aware breach width modelling results using DLBreach

HRW – EMBREA

HRW provided two sets of run data after analysing an initial run where no erosion took place.

Following the initial blind run, the following conclusions were drawn and used to define parameters for a second run:

- Run showed no erosion of the pipe in either the vertical or the lateral directions. Top of the pipe did not collapse within the simulation which was about 72 hours
- A number of runs were undertaken to determine the critical shear stress at which erosion can be initiated. A value of 7.5 Pa was low enough to do so. The results of this run (i.e. with a critical shear stress value of 7.5 Pa) are shown as a second run (aware run).

For the second (aware) run:

- Erosion of pipe takes place but no pipe collapse. Some oscillations occurred at the end of the run due to either changes between free and pipe flow and/or downstream drowning. These will be investigated later
- It can be seen that whilst the aware run predicts a breach width close to observed, it over predicts the breach discharge required to produce this.

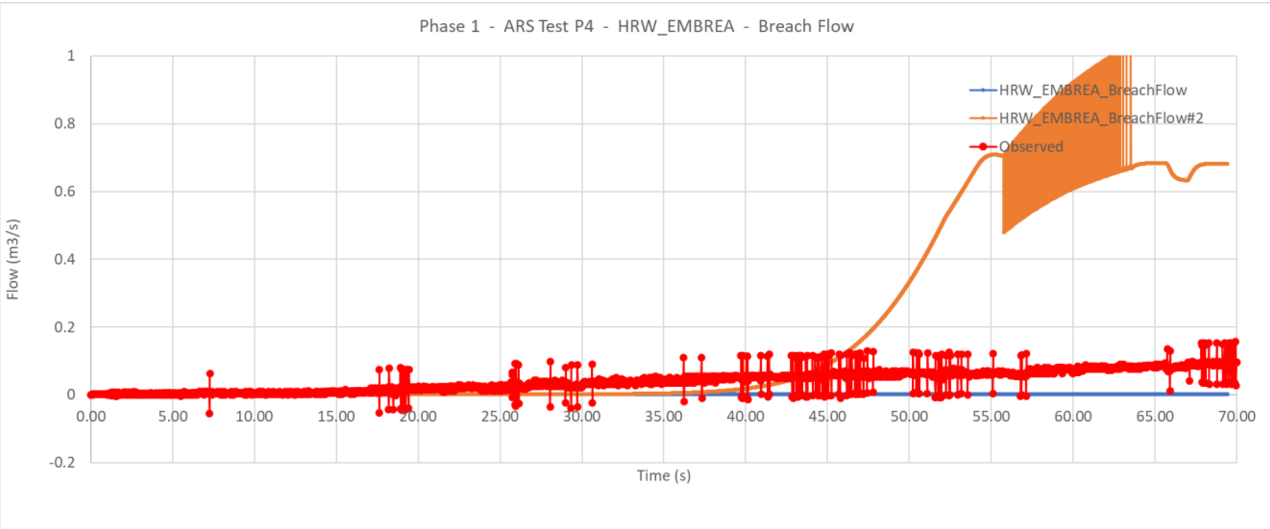


Figure F.17: Phase 1 – HRW P4 aware flow modelling results using EMBREA

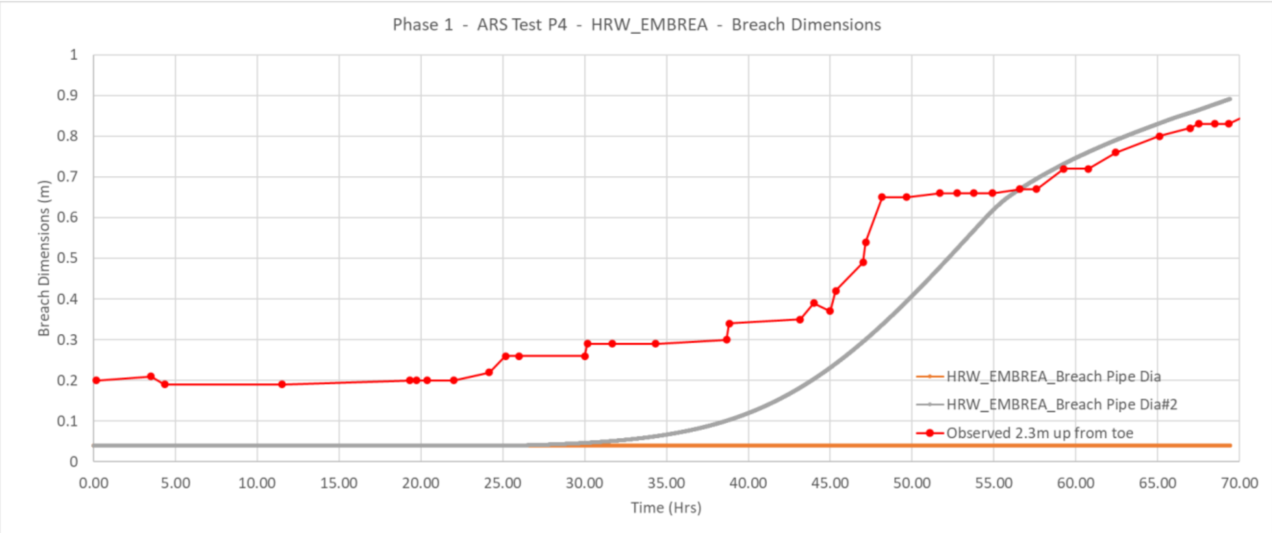


Figure F.18: Phase 1 – HRW P4 aware breach width modelling results using EMBREA

INRAE – Rupro#1 and Rupro#3

INRAE undertook aware modelling to see whether the initial predictions – which predicted quick breach – could be improved. The aware modelling showed improvements towards the observed data, but results were still significantly away from the observed data.

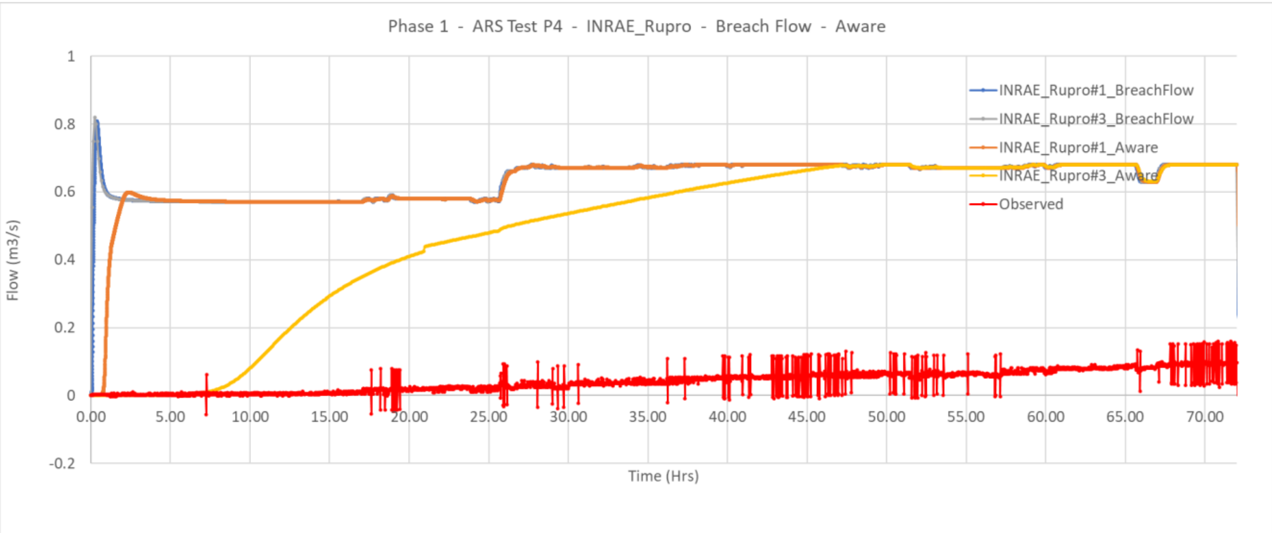


Figure F.19: Phase 1 – INRAE P4 aware flow modelling results using Rupro

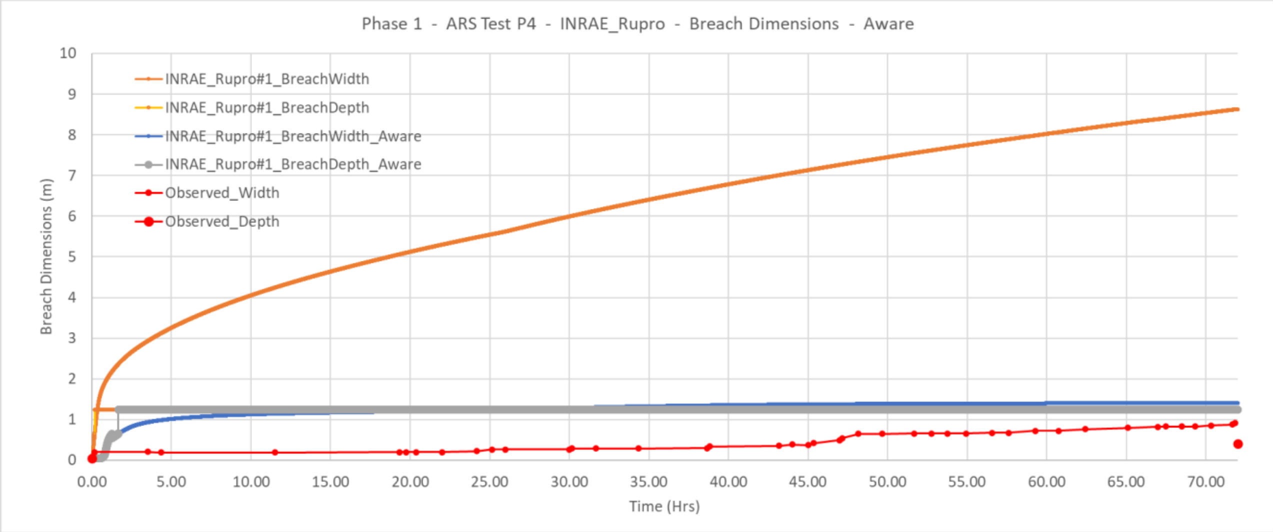


Figure F.20: Phase 1 – INRAE P4 aware breach width modelling results using Rupro

G Phase 2 – Lawn Lake Dam Failure Case Study

G.1 Phase 2 – Lawn Lake Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	Lawn_Lake_Blind.xlsx Lawn_Lake_Aware.xlsx
Analysis & comparison of modelling results	Phase2_ModellingComparison_LawnLake_21_07_15.xlsx

G.2 Case Study Description

On Thursday, July 15, 1982, campers report hearing roar around 02:00 a.m. (supposed to be the time of the beginning of the piping progression). Just before sunrise, at about 5:30 a.m., the privately-owned Lawn Lake dam, a 7,9 m high earthen structure, located at an elevation of about 3351.7 m in the Rocky Mountain National Park, breached due to a piping failure, releasing 0,83 Mm^3 and an estimated peak discharge of 504 m^3/s of water down the Roaring River.

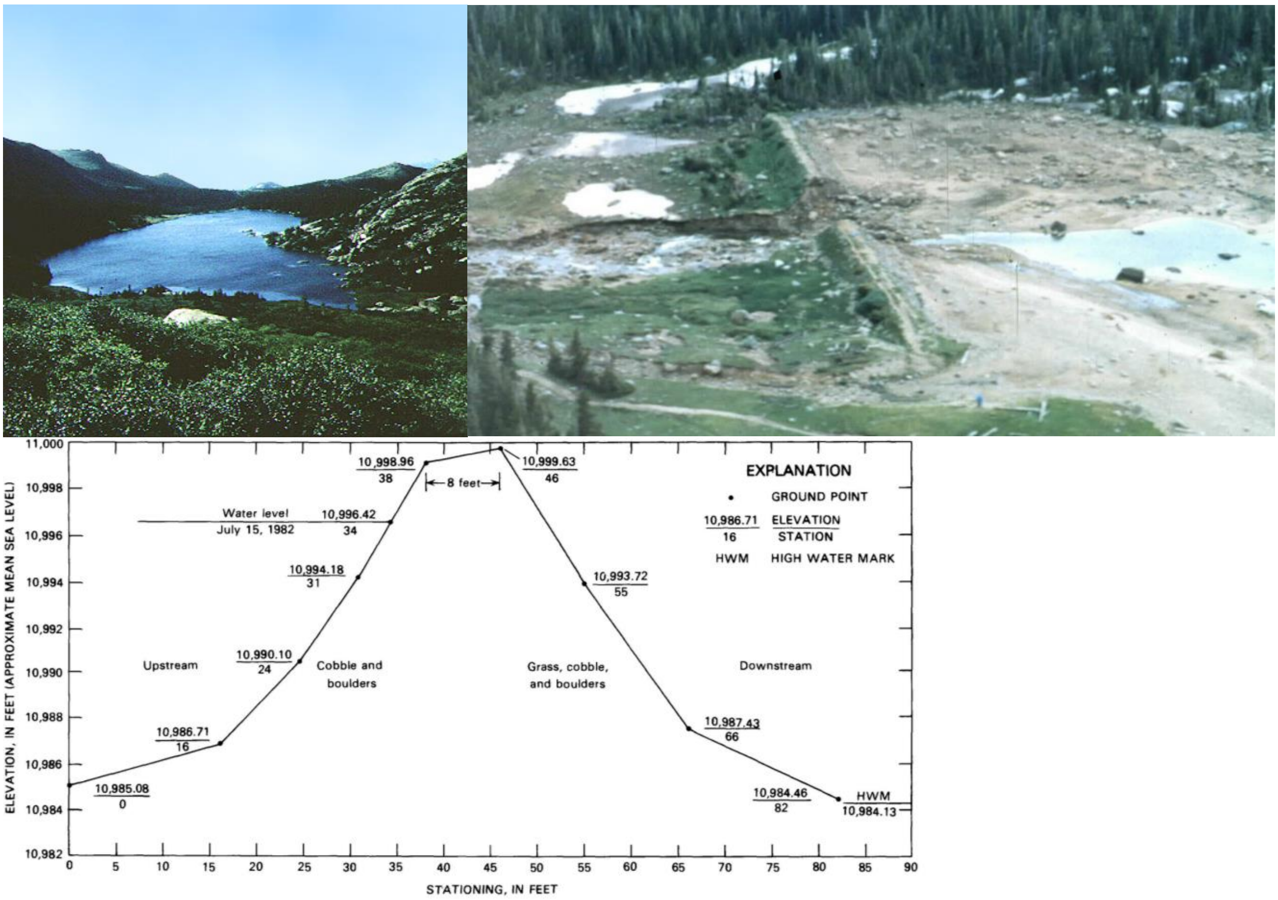
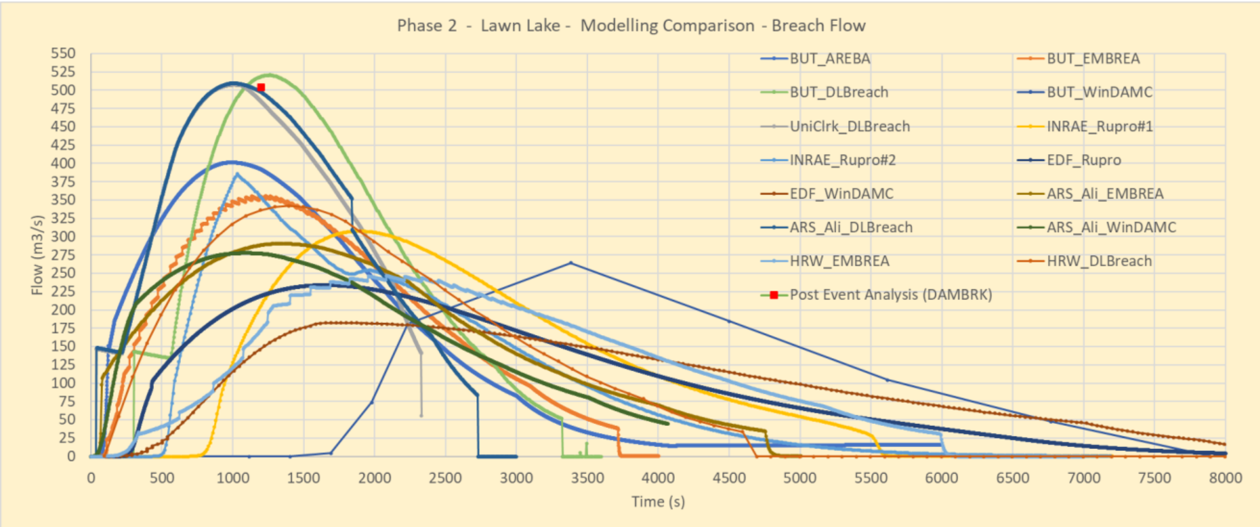


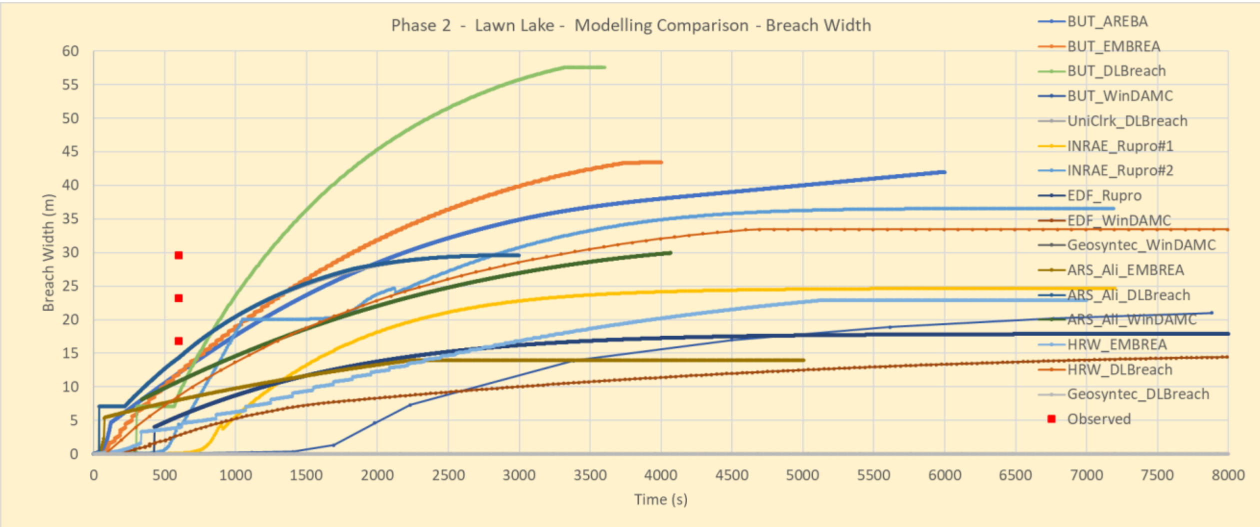
Figure G.1: Lawn Lake Dam Case Study

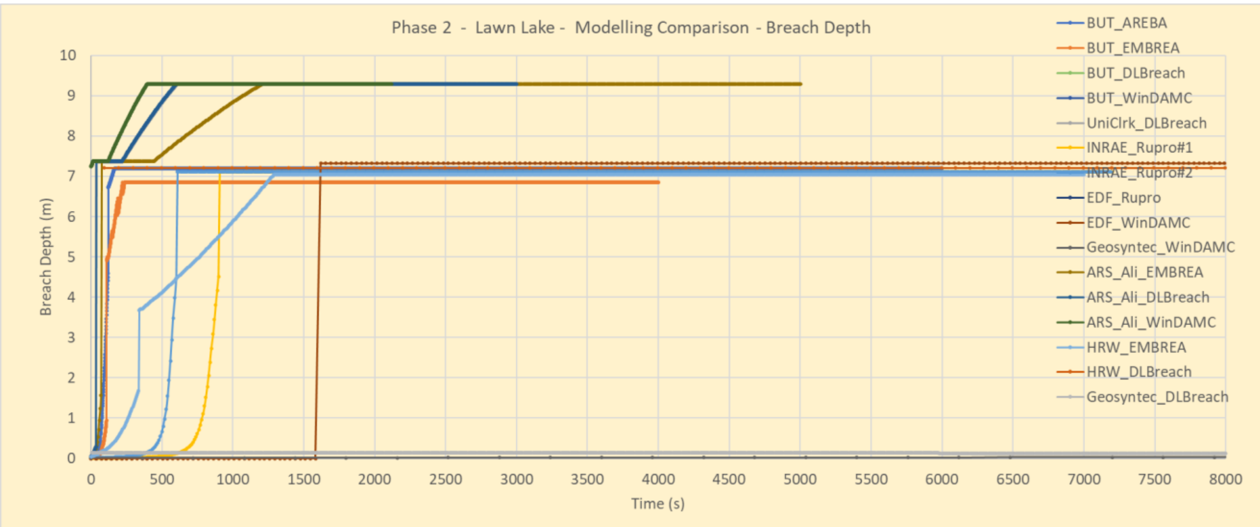
Table G.1: Phase 2 – Lawn Lake: Modeller Assumptions

Models & Modellers:			**Structure**		**Initiation**				**Soil Parameters**							**Flow**	**Computational**					**Reported Problems or observations**
		Variables	Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	Ali Abdelfatah																					
Blind	WinDAMC			3345.10	0.05*0.05 m Rec.	middle dam base	0 Sec	50	2650	3.5	29.9	0.4455			0.15	0.016	0.01					
Blind	DL Breach			3345.10	0.05*0.05 m Rec.	middle dam base	0 Sec	50	2650	3.5	29.9	0.4455			0.15	0.016	0.01					
Blind	EMBREA			3345.10	0.05*0.05 m Rec.	middle dam base	0 Sec	50	2650	3.5	29.9	0.4455			0.15	0.016	0.01					
HRW	Mohamed Hassan																					
	DL BREACH		Homogeneous structure with kd = 15 cm3/N.s and critical shear stress = 1 pa.	3345.10	0.05	3345.18	Start of the simulation	15	1416 (dry)	3.5	29.9	0.44	NA	NA	1	0.03	5	5	Middle section	NA	NA	Estimating Kd is tricky for a case such as this one with limited information on compaction and water content. The used value in the model can be quite different from the actual erodibility. Aware testing will reveal this.
	EMBREA		Homogeneous structure with kd = 15 cm3/N.s and critical shear stress = 1 pa.	3345.10	0.05	3345.18	Start of the simulation	15	1416 (dry)	3.5	29.9	0.44	NA	NA	1	0.03	5	5	Critical section which moves with time and is not fixed	NA	NA	Estimating Kd is tricky for a case such as this one with limited information on compaction and water content. The used value in the model can be quite different from the actual erodibility. Aware testing will reveal this.
Aware	EMBREA Aware 1		The breach depth was allowed to erode below the foundation in this aware run. Variable Kd was also used in this case rathar than same Kd.					A variable Kd factor = 3 was assumed for this case which means for the overtopping case of the failure Kd = 15*3 = 45 cm3/N.s. Kd for the piping part of the breach was kept as is (i.e. 15 cm3/N.s)														Breach depth factor was restored to the default value of 1.6 instead of 1.0 which was used in the blind run.
	EMBREA Aware 2		This run is identical to Aware Runs 1 except that the breach width was restricted to the average breach width value (i.e. 23.5m).					A variable Kd factor = 3 was assumed for this case which means for the overtopping case of the failure Kd = 15*3 = 45 cm3/N.s. Kd for the piping part of the breach was kept as is (i.e. 15 cm3/N.s)														Breach depth factor was restored to the default value of 1.6 instead of 1.0 which was used in the blind run.
BUT	Stanislav Kotaska																					
Blind	TUD AREBA		Homogenous dam without protection	3345.1	0.01	middle	-	27	2050	3.5	29.9	0.45	0.5	2.31481E-06	4.5	0.039	1	-	-	-	-	coefficient M = 1.4
Blind	EMBREA		Homogenous dam without protection	3345.1	0.01	middle	-	30	2050	3.5	29.9	0.45	-	-	4.5	0.039	1	1	-	-	-	
Blind	WinDAM		Homogenous dam without protection	3345.1	0.01	middle	-	17.68	2050	3.5	29.9	0.45	-	-	6.2	0.039	-	-	-	-	-	
Blind	DL BREACH		Homogenous dam without protection	3345.1	0.01	middle	-	17.68	2650	3.5	29.9	0.45	-	-	6.2	0.039	1	-	-	-	-	
Aware	TUD AREBA							45														
Aware	EMBREA							59														
Aware	DL BREACH							129														
UniClrk	Weiming Wu																					
Blind	DL BREACH		trapezodal cross-section: dam is 7.3 m high, dam crest is 2.4 m wide, upstream slope 1V:1.5H and downstream slope 1V:1.5H. The dam foundation is assumed to have a 2 m thick erodible layer	3345.10	0.05	middle, dam base	at t=0 s	50	2650	3.5	29.9	0.465			0.15	0.016	0.1		dam crest			The pipe entrance head loass coefficient is 1.5 .
Aware	DL BREACH																					
EDF	Pierre Squillari (Geophy)																					
Blind	Rupro		The non-erodible foundation is below the dame base	3345.10	0.03 x 0.03	middle	none		Grain density : 2650 kg/m3			0.45				0.033	1s					Grain diameter = 4 mm
			best estimates of key values are taken.		3.00E-02																	
Blind	WinDAM		below the dam base, soil is non-erodible	3345.10	3.00E-02	middle	none	10	1692.6						2							
Blind			best estimates of key values are taken.																			
Blind			US units	10974.70	0.1			5.65	105.7						0.04	0.03						

G.3 Phase 2 – Lawn Lake Modelling Results







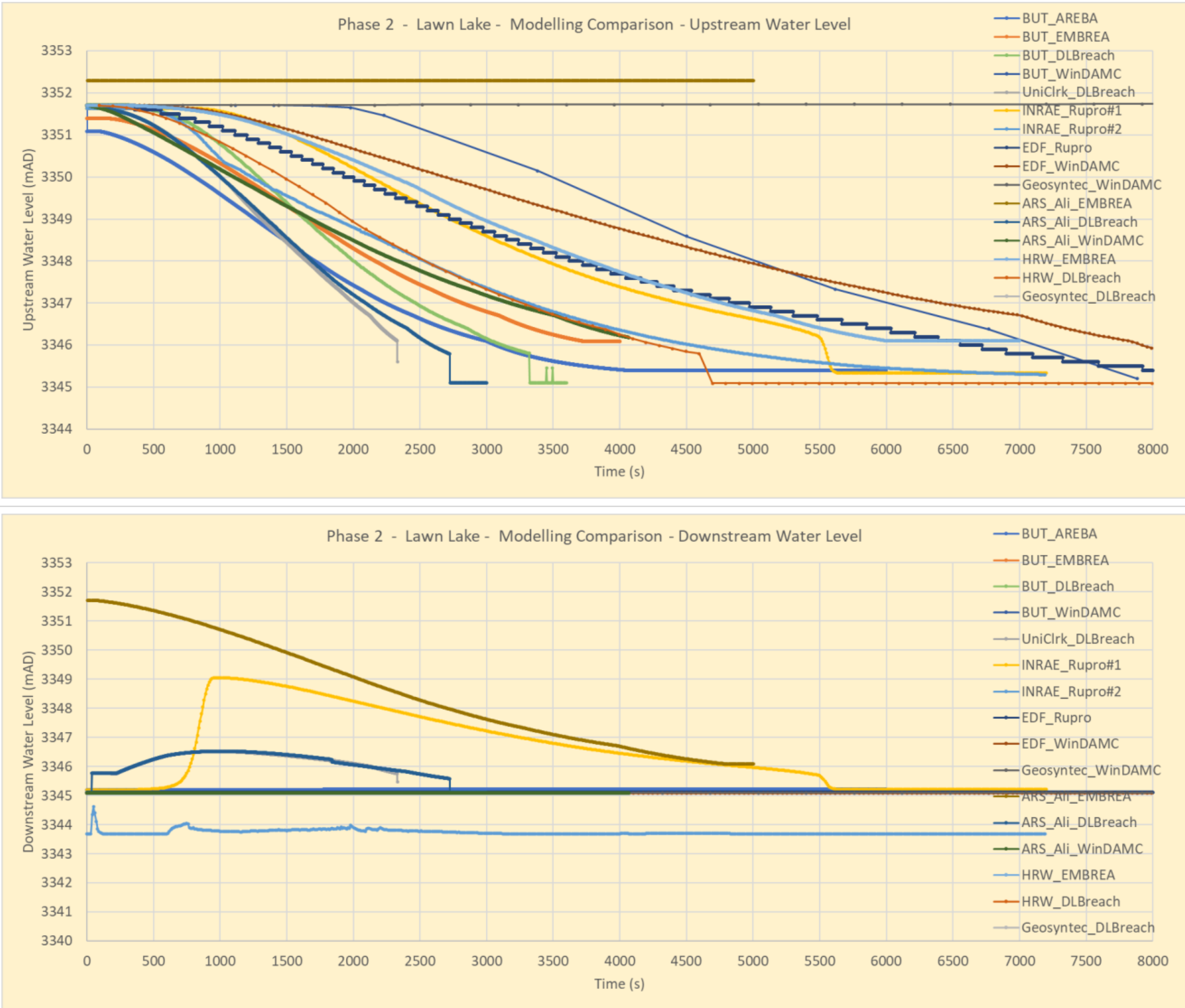
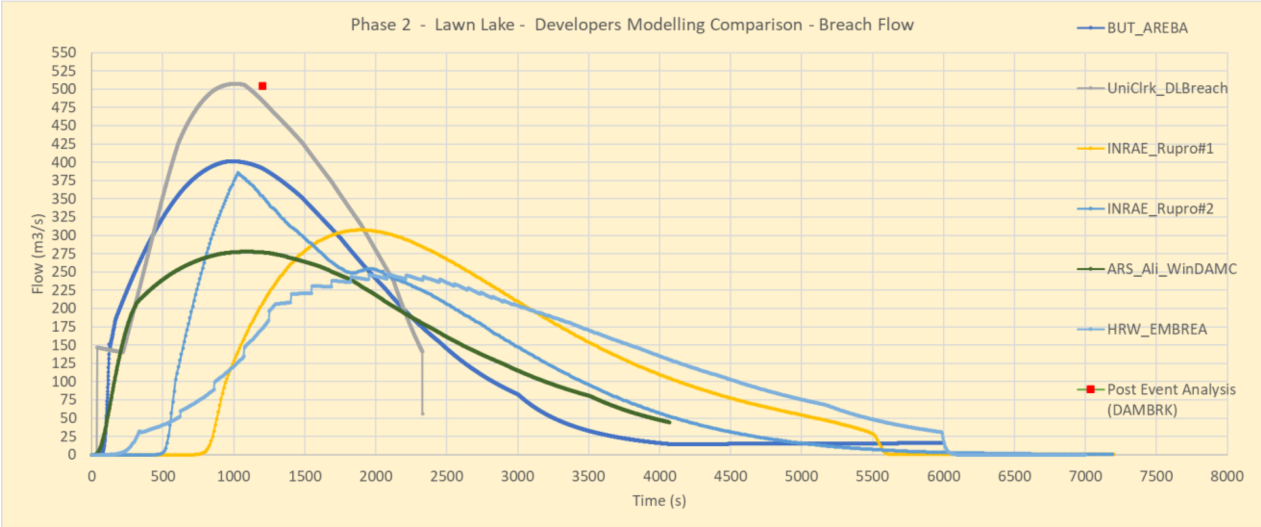
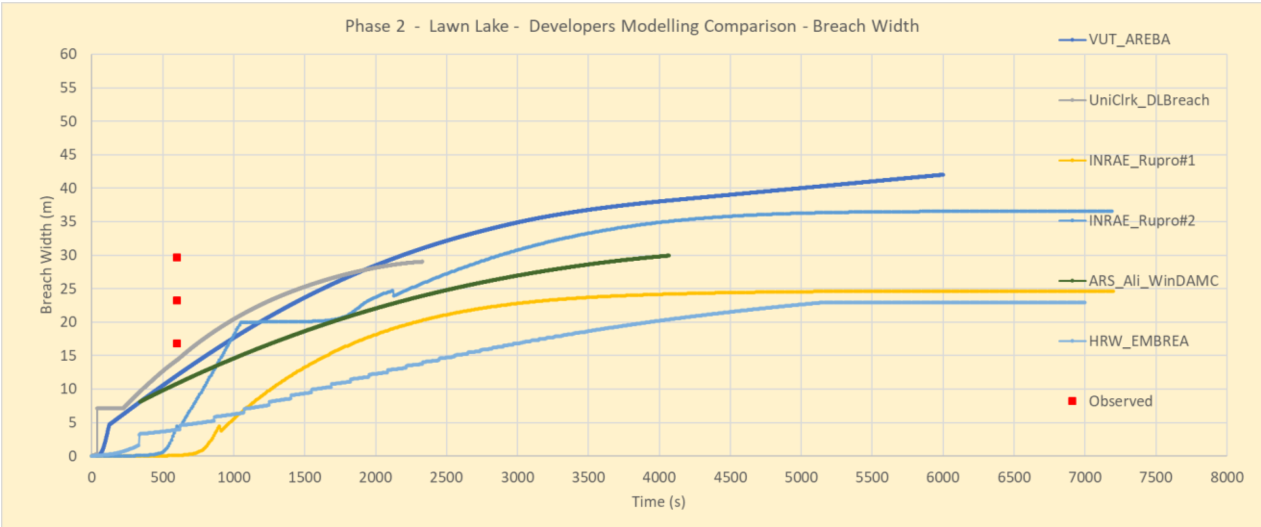
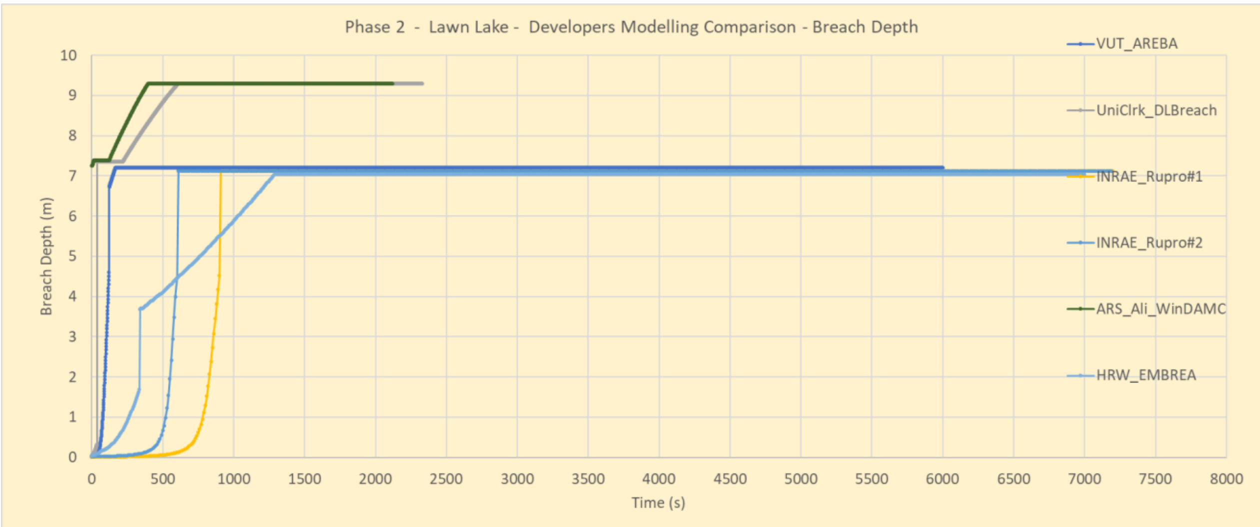


Figure G.2: Phase 2 – Lawn Lake: All modelling results


Phase 2 - Lawn Lake - Developers Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_AREBA
UniClrk_DLBreach
INRAE_Rupro#1
INRAE_Rupro#2
ARS_Ali_WinDAMC
HRW_EMBREA
Post Event Analysis (DAMBRK)


Phase 2 - Lawn Lake - Developers Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
VUT_AREBA
UniClrk_DLBreach
INRAE_Rupro#1
INRAE_Rupro#2
ARS_Ali_WinDAMC
HRW_EMBREA
Observed


Phase 2 - Lawn Lake - Developers Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
VUT_AREBA
UniClrk_DLBreach
INRAE_Rupro#1
INRAE_Rupro#2
ARS_Ali_WinDAMC
HRW_EMBREA

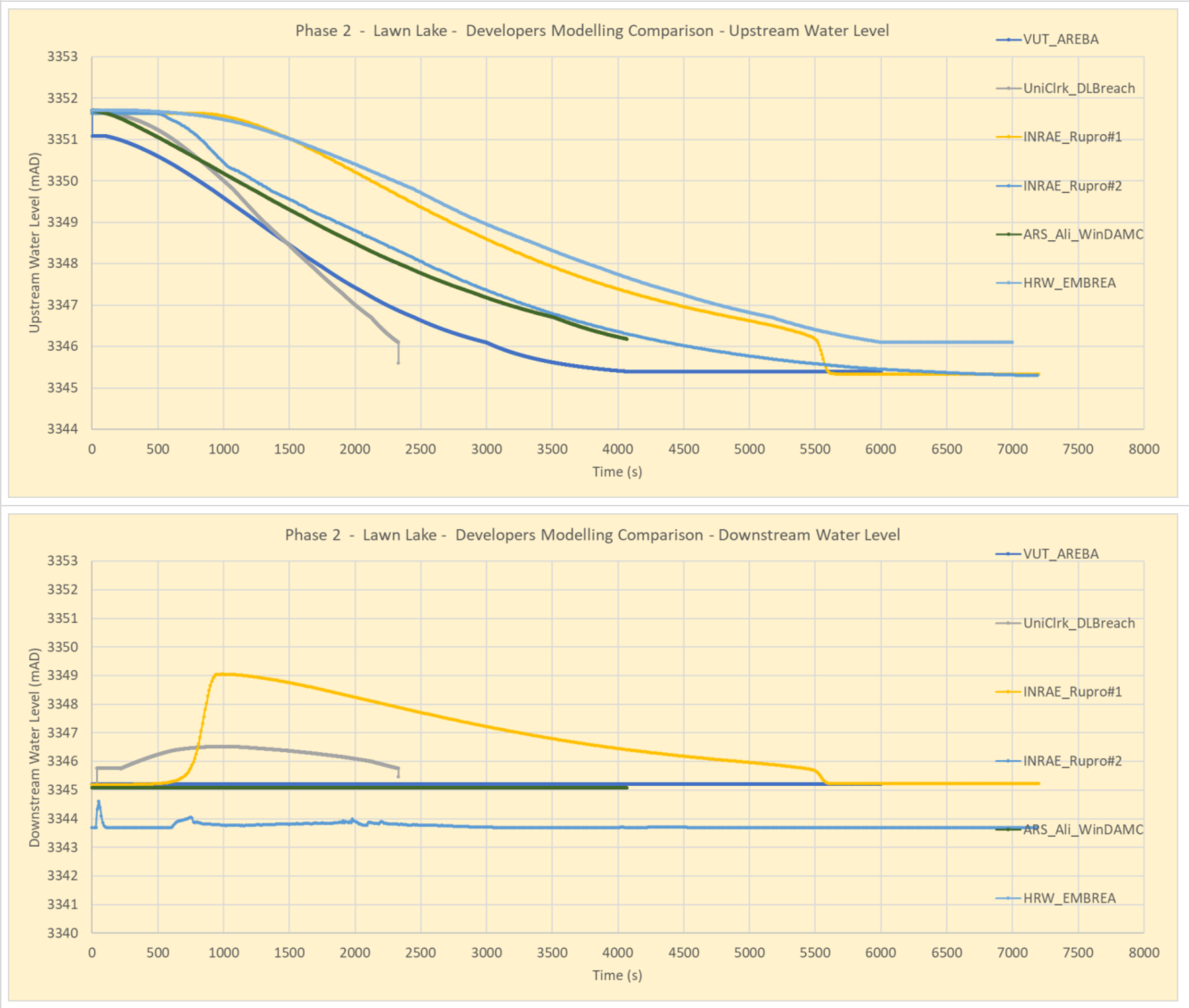
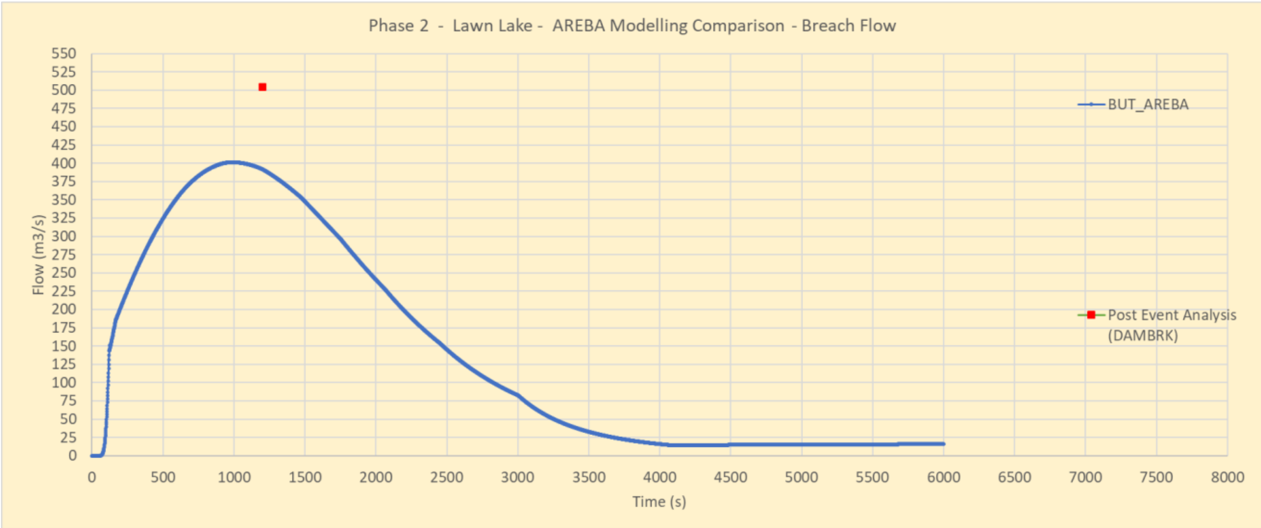
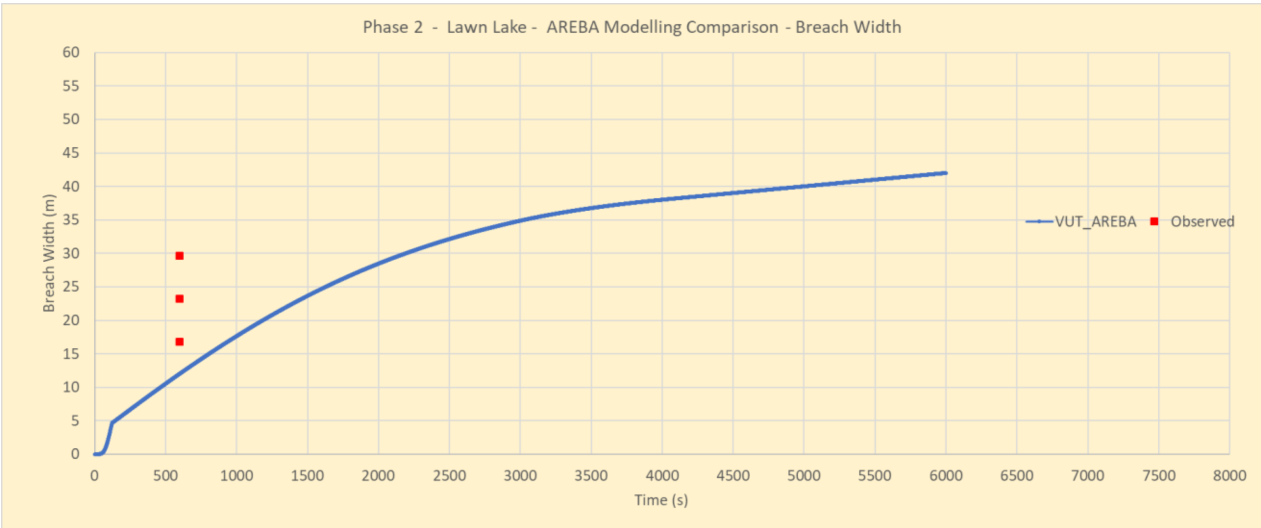
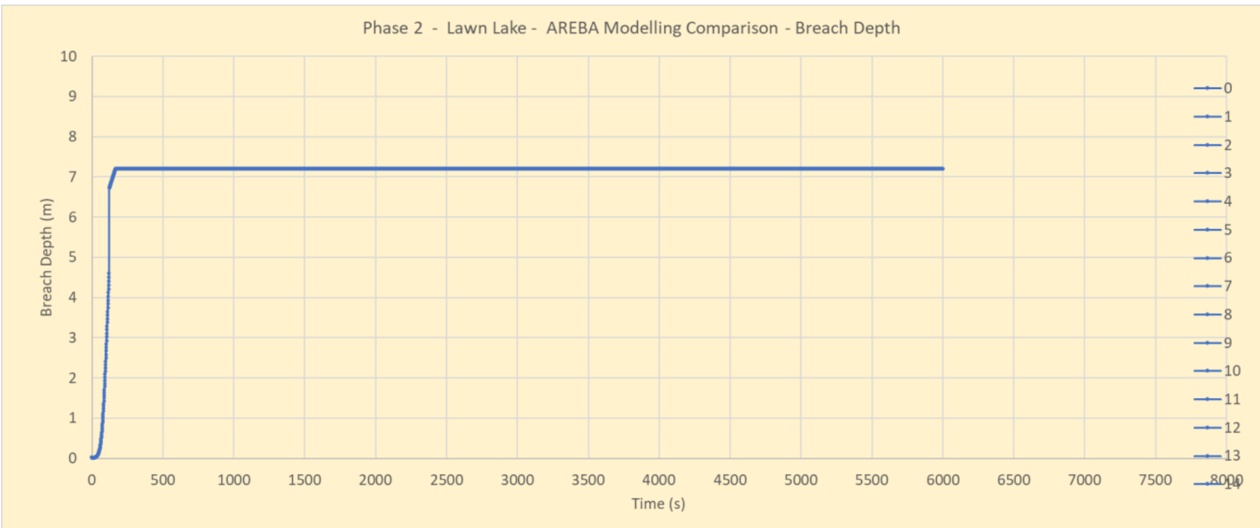


Figure G.3: Phase 2 – Lawn Lake: Developers modelling results


Phase 2 - Lawn Lake - AREBA Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_AREBA
Post Event Analysis (DAMBRK)


Phase 2 - Lawn Lake - AREBA Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
VUT_AREBA
Observed


Phase 2 - Lawn Lake - AREBA Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)

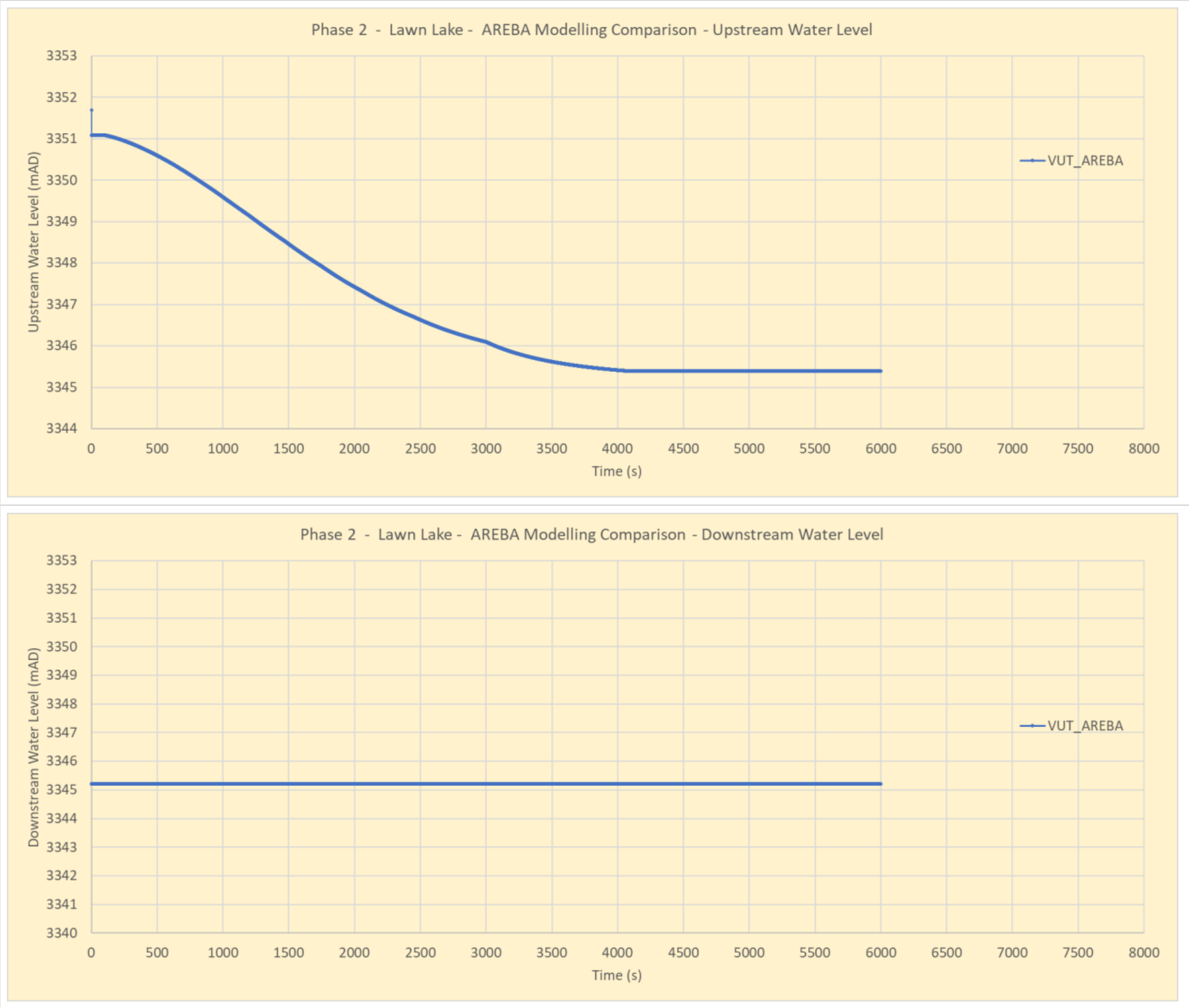
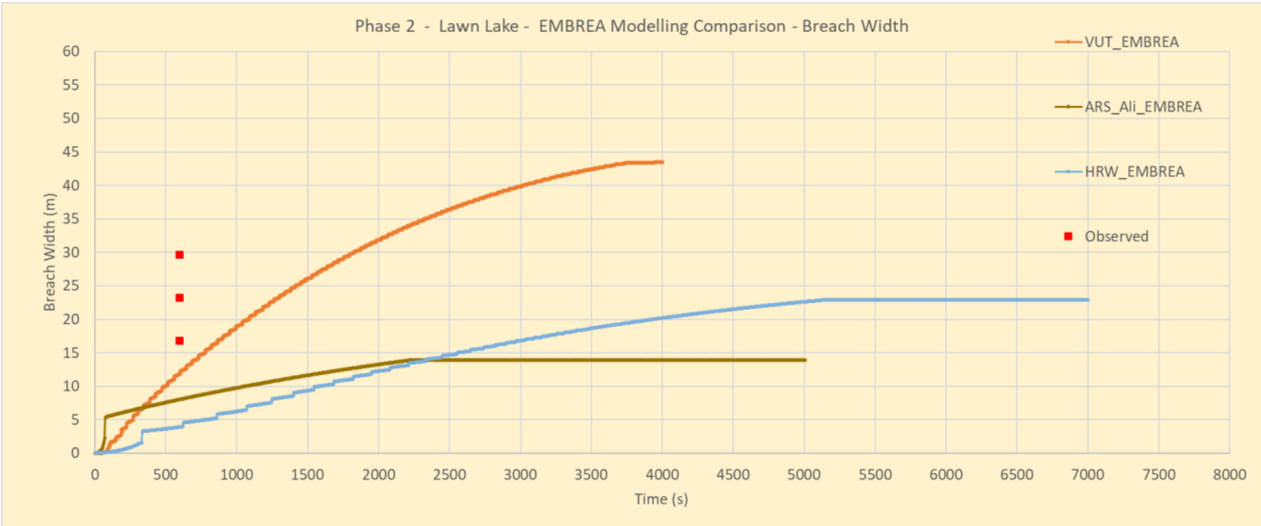
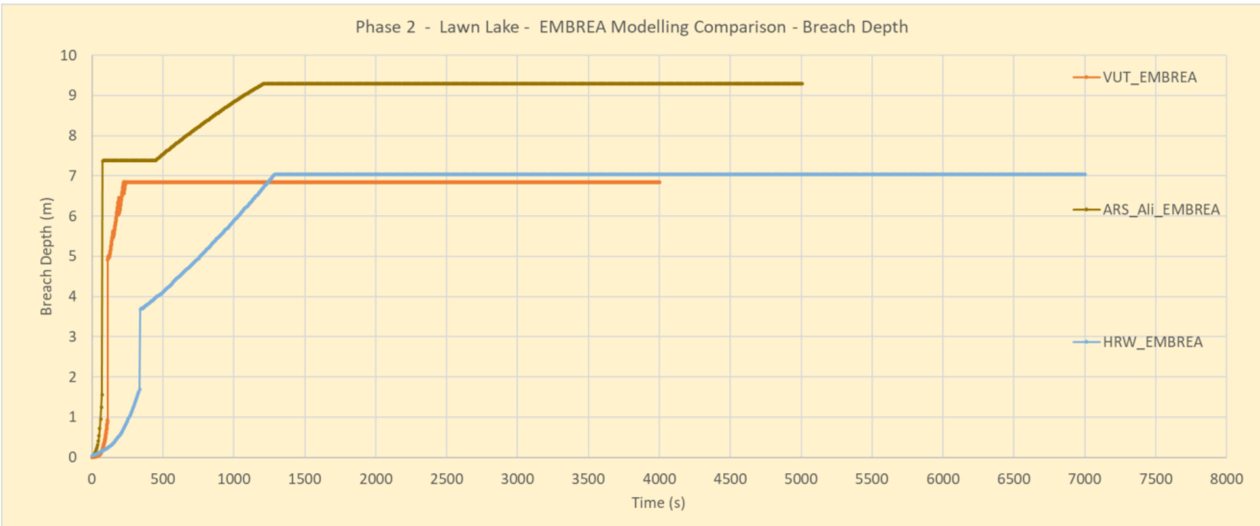


Figure G.4: Phase 2 – Lawn Lake: Modelling results using AREBA


Phase 2 - Lawn Lake - EMBREA Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_EMBREA
ARS_Ali_EMBREA
HRW_EMBREA
Post Event Analysis (DAMBRK)


Phase 2 - Lawn Lake - EMBREA Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
VUT_EMBREA
ARS_Ali_EMBREA
HRW_EMBREA
Observed


Phase 2 - Lawn Lake - EMBREA Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
VUT_EMBREA
ARS_Ali_EMBREA
HRW_EMBREA

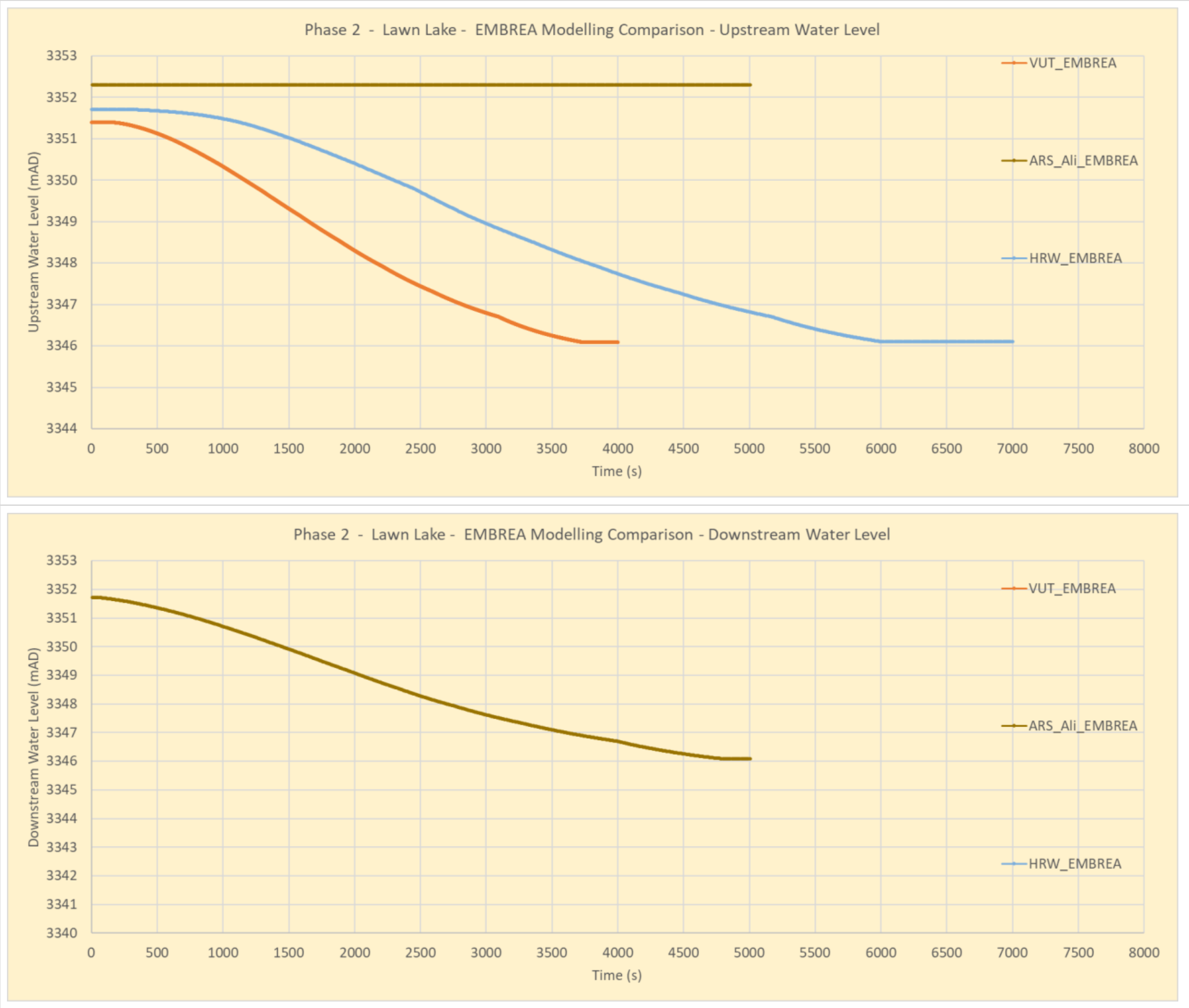
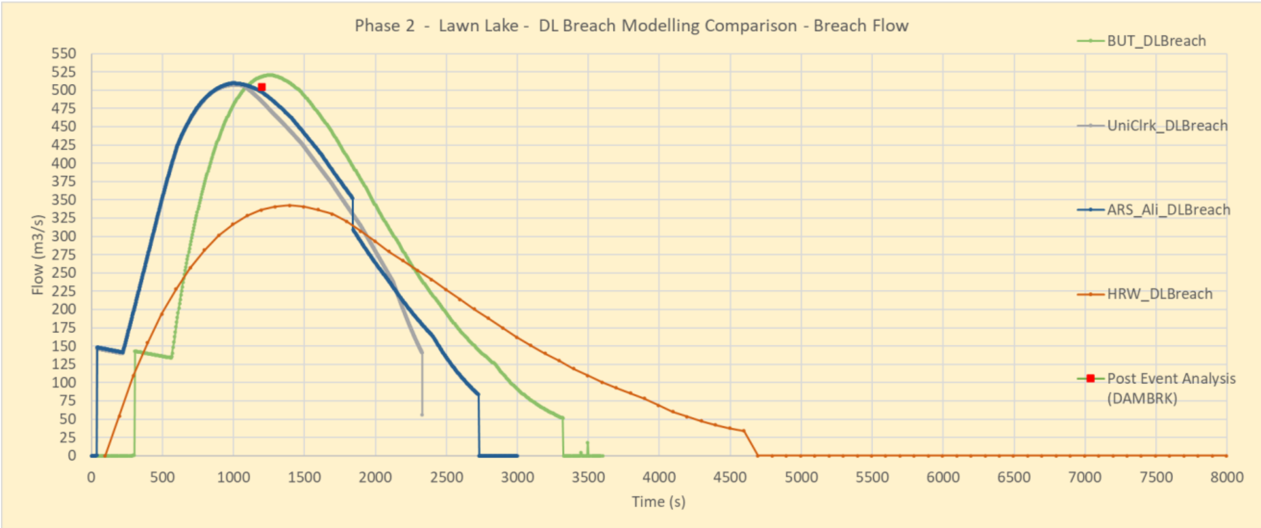
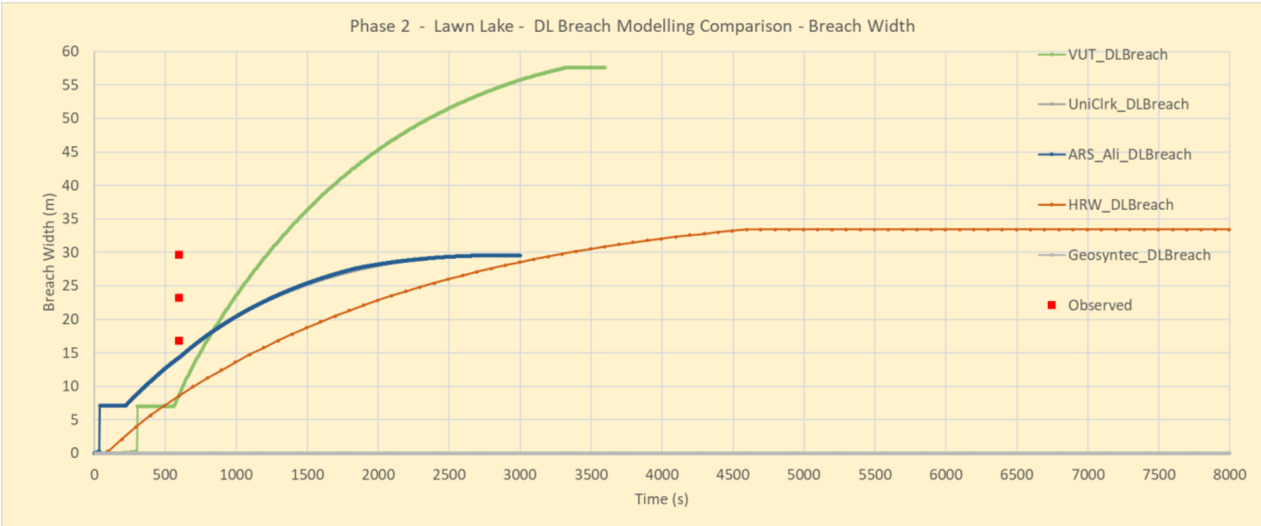
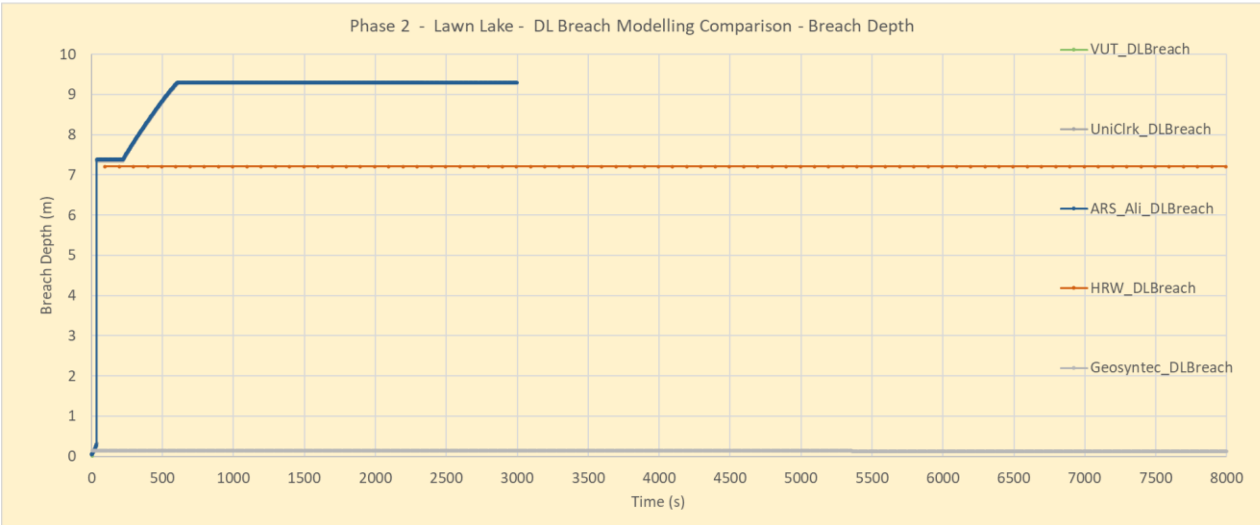
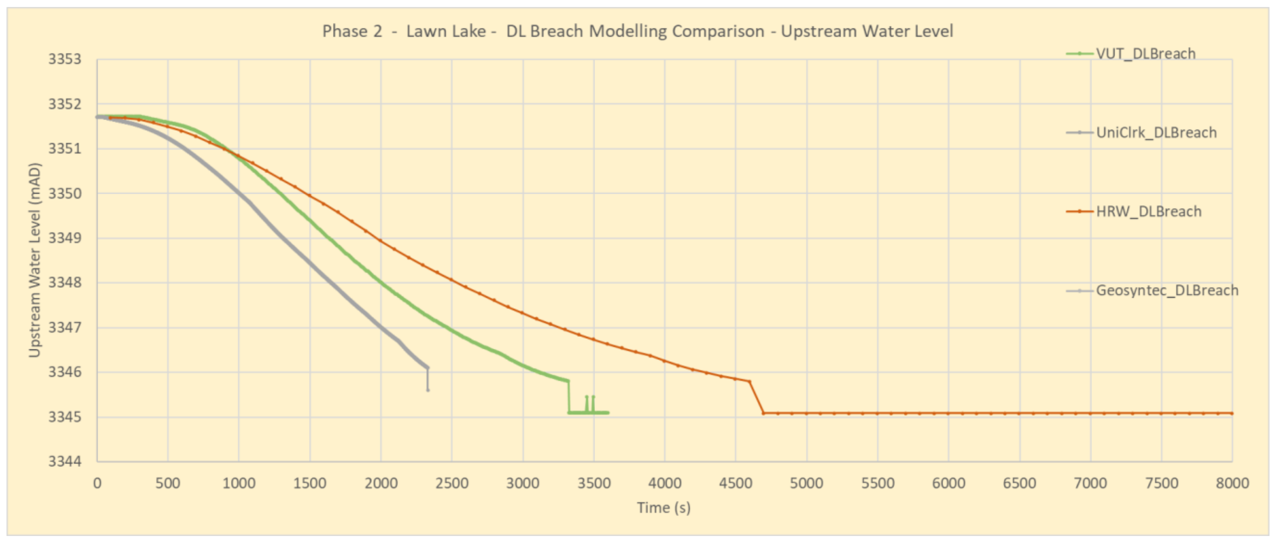


Figure G.5: Phase 2 – Lawn Lake: Modelling results using EMBREA


Phase 2 - Lawn Lake - DL Breach Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_DLBreach
UniClrk_DLBreach
ARS_Ali_DLBreach
HRW_DLBreach
Post Event Analysis (DAMBRK)


Phase 2 - Lawn Lake - DL Breach Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
VUT_DLBreach
UniClrk_DLBreach
ARS_Ali_DLBreach
HRW_DLBreach
Geosyntec_DLBreach
Observed


Phase 2 - Lawn Lake - DL Breach Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
VUT_DLBreach
UniClrk_DLBreach
ARS_Ali_DLBreach
HRW_DLBreach
Geosyntec_DLBreach



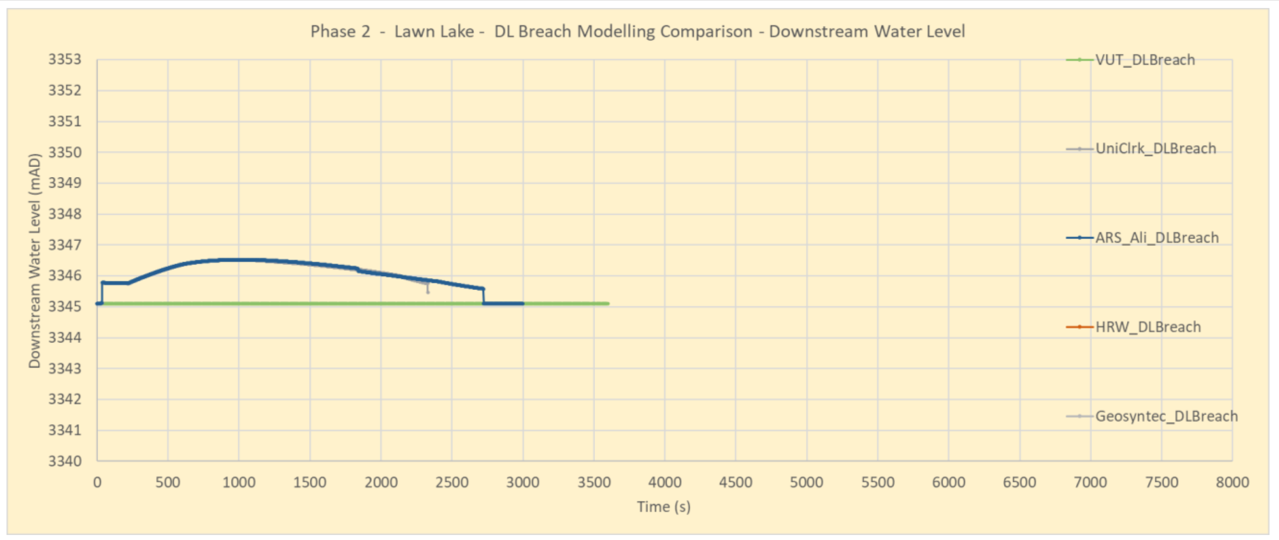
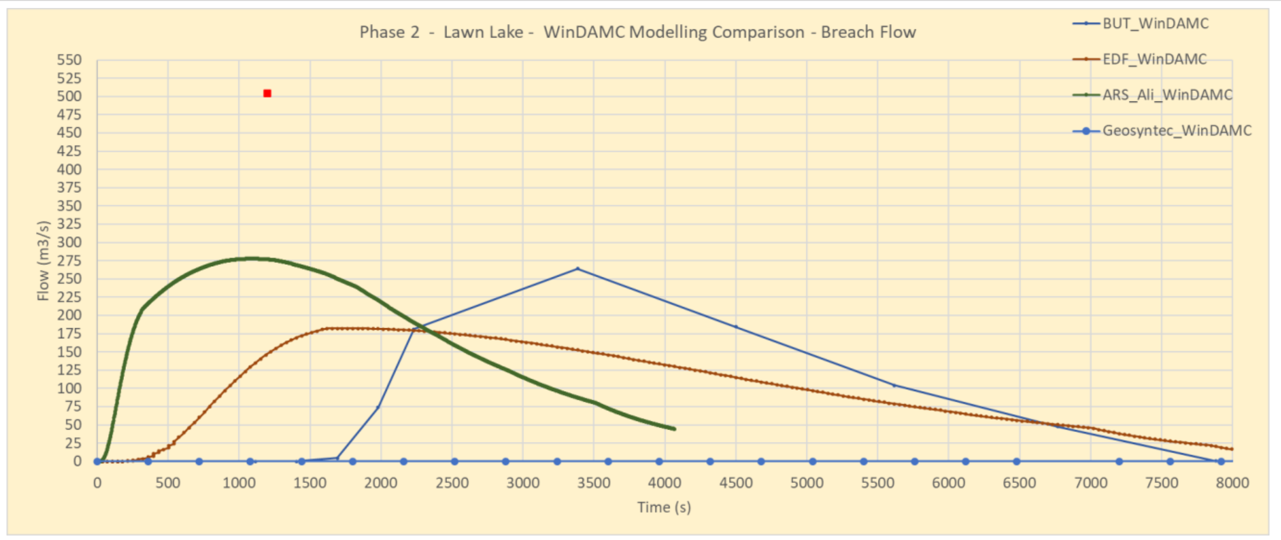
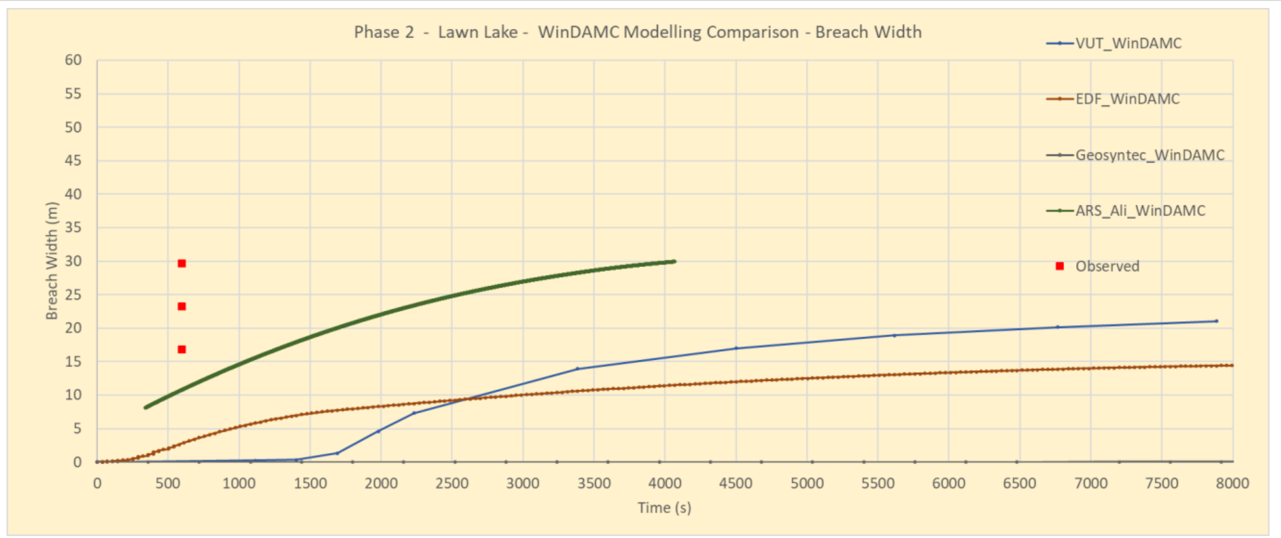


Figure G.6: Phase 2 – Lawn Lake: Modelling results using DLBreach


Phase 2 - Lawn Lake - WinDAMC Modelling Comparison - Breach Flow
BUT_WinDAMC
EDF_WinDAMC
ARS_Ali_WinDAMC
Geosyntec_WinDAMC
Flow (m3/s)
Time (s)


Phase 2 - Lawn Lake - WinDAMC Modelling Comparison - Breach Width
VUT_WinDAMC
EDF_WinDAMC
Geosyntec_WinDAMC
ARS_Ali_WinDAMC
Observed
Breach Width (m)
Time (s)

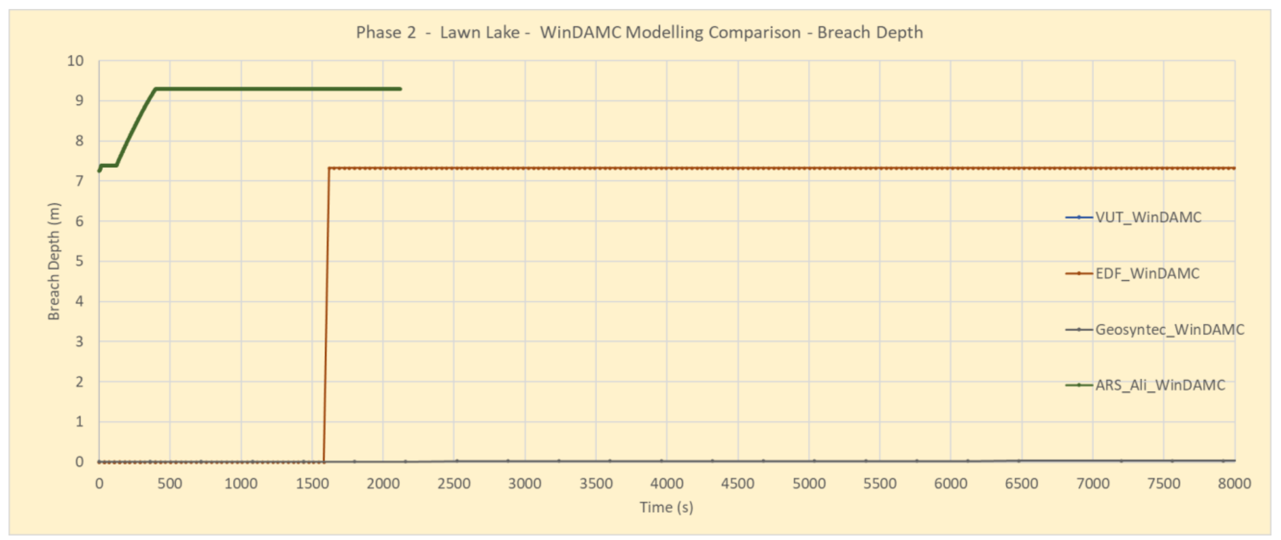
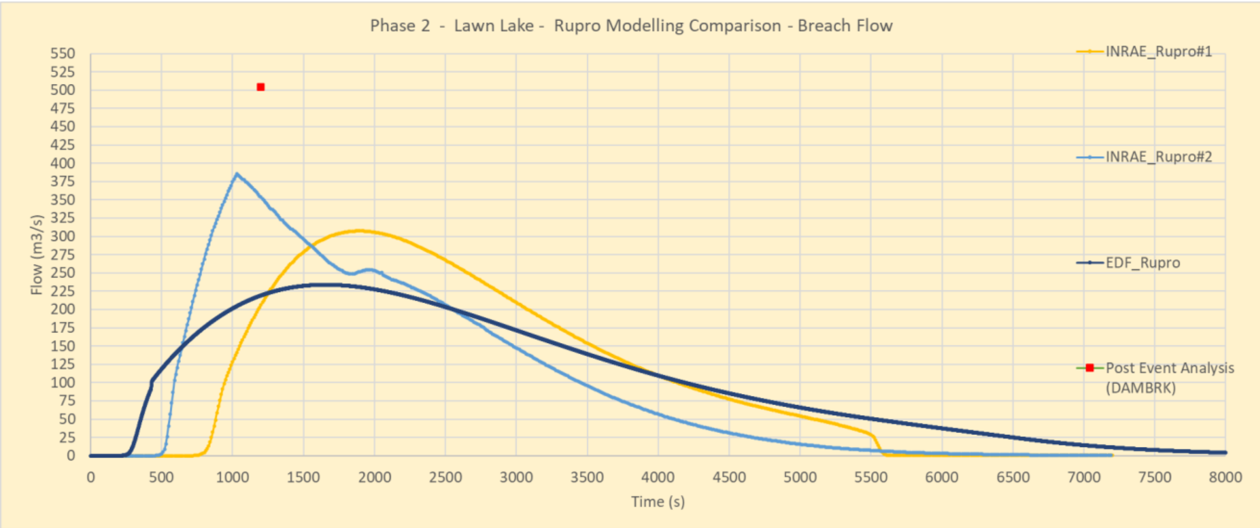
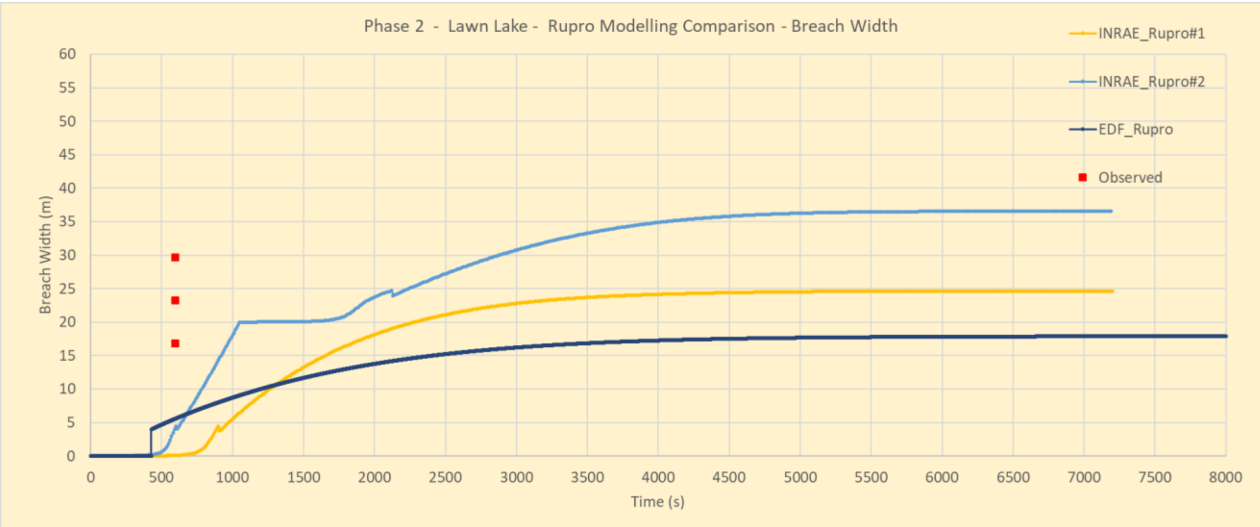
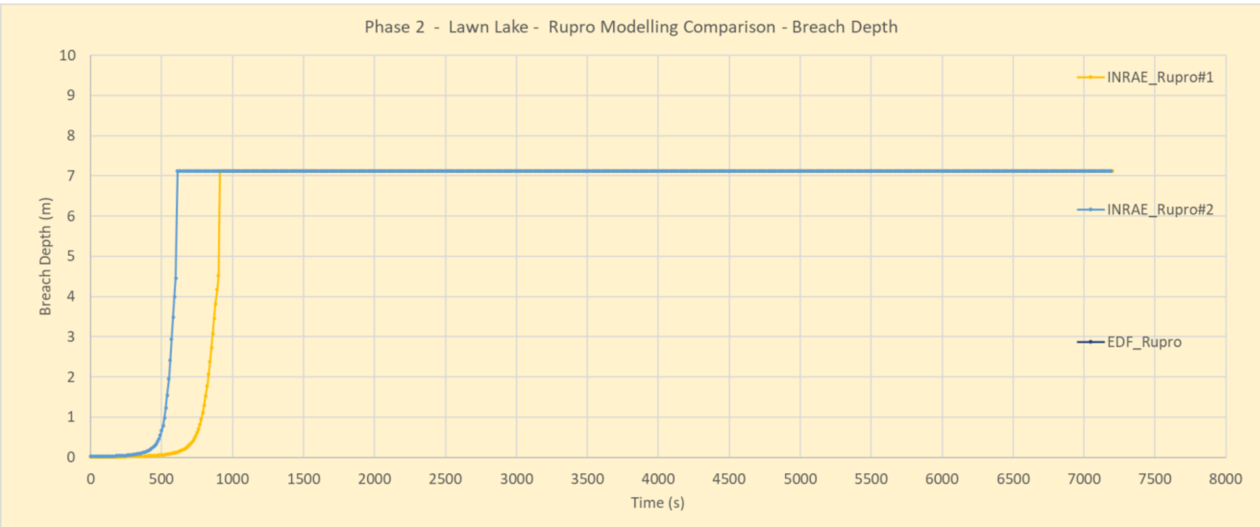

Phase 2 - Lawn Lake - WinDAMC Modelling Comparison - Breach Depth
VUT_WinDAMC
EDF_WinDAMC
Geosyntec_WinDAMC
ARS_Ali_WinDAMC
Breach Depth (m)
Time (s)

Figure G.7: Phase 2 – Lawn Lake: Modelling results using WinDAM C


Phase 2 - Lawn Lake - Rupro Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
INRAE_Rupro#1
INRAE_Rupro#2
EDF_Rupro
Post Event Analysis (DAMBRK)


Phase 2 - Lawn Lake - Rupro Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
INRAE_Rupro#1
INRAE_Rupro#2
EDF_Rupro
Observed


Phase 2 - Lawn Lake - Rupro Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
INRAE_Rupro#1
INRAE_Rupro#2
EDF_Rupro

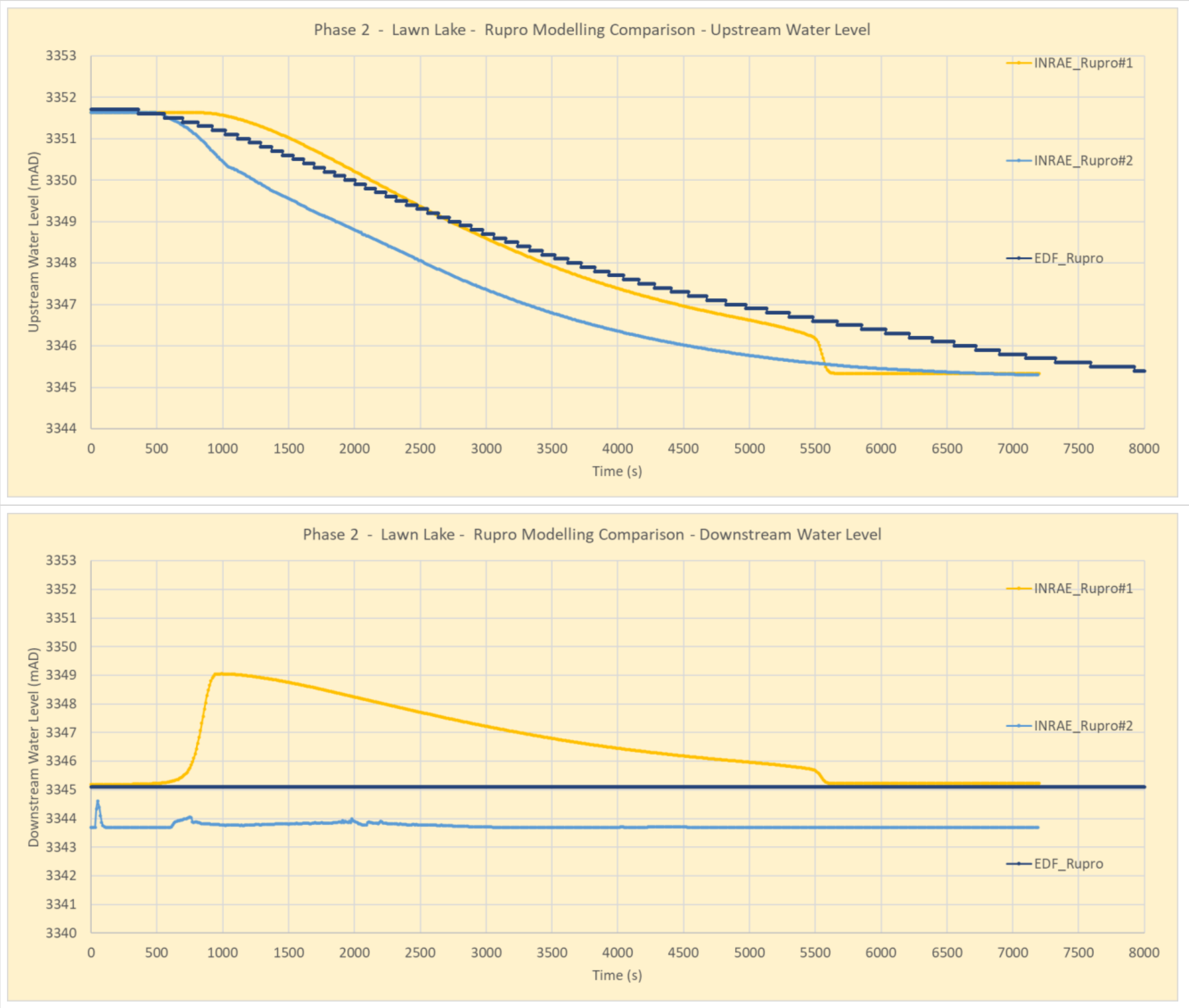


Figure G.8: Phase 2 – Lawn Lake: Modelling results using Rupro

G.4 Phase 2 – Lawn Lake Aware Modelling Results

HRW and BUT undertook aware modelling for the Lawn Lake case.

HRW – EMBREA

Two aware runs were undertaken as shown in the Figure G.9 below.

For Aware 1:

- The breach depth was allowed to erode below the foundation in this aware run. Variable K_d was also used in this case rather than same K_d
- A variable K_d factor=3 was assumed for this case which means that for the overtopping part of the failure K_d=15*3=45 cm^3/N.s. K_d for the piping part of the breach was kept as is (i.e. 15 cm^3/N.s)
- Breach depth factor was restored to the default value of 1.6 instead of 1.0 which was used in the blind run.

For Aware 2:

- This run is identical to Aware Runs 1 except that the breach width was restricted to the average breach width value (i.e. 23.5 m).

Both run results gave results that matched the observed estimate extremely well.

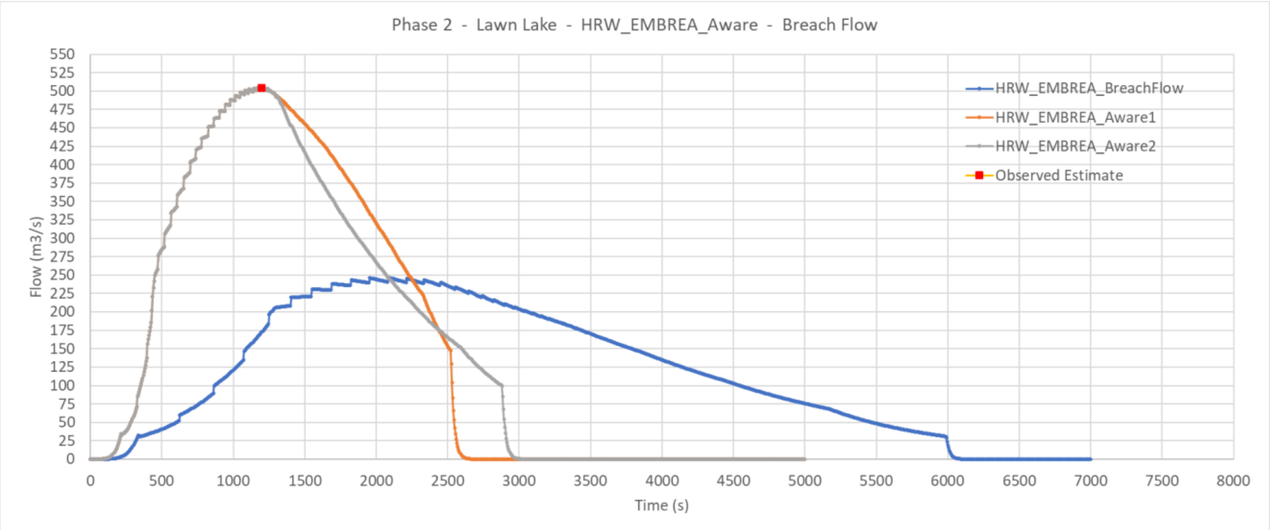


Figure G.9: Phase 2 – Lawn Lake: HRW aware modelling results using EMBREA

BUT – WinDAM C

A significantly better result was achieved by modifying the soil erodibility (129 instead of 17.68 $cm^3/N.S$).

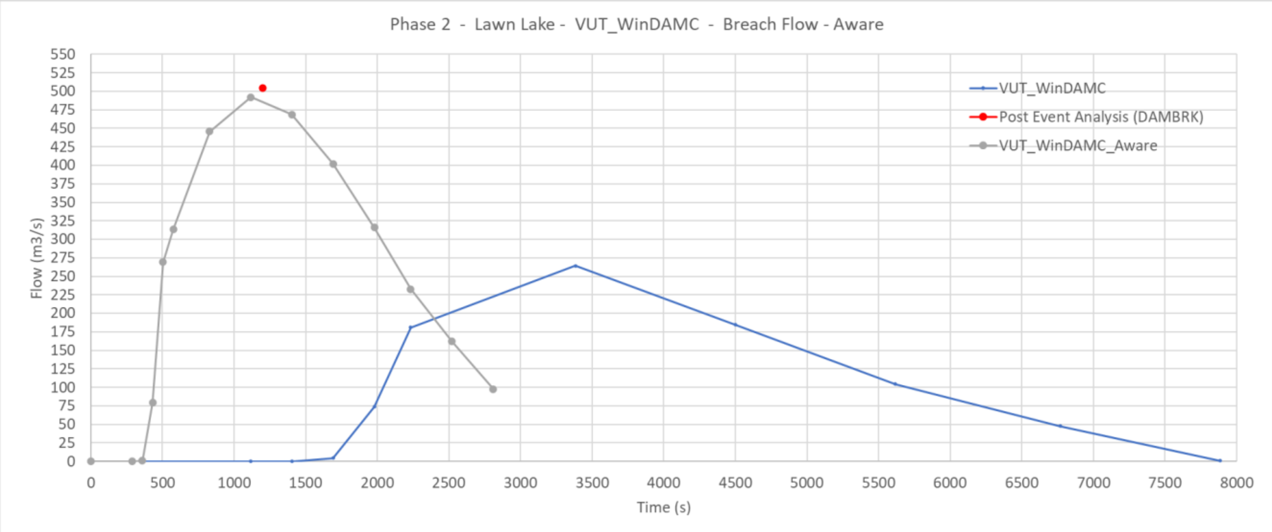


Figure G.10: Phase 2 – Lawn Lake: BUT aware modelling results using WinDAM C

BUT – EMBREA

A significantly better result was achieved by modifying the soil erodibility (59 instead of 30 $cm^3/N.S$).

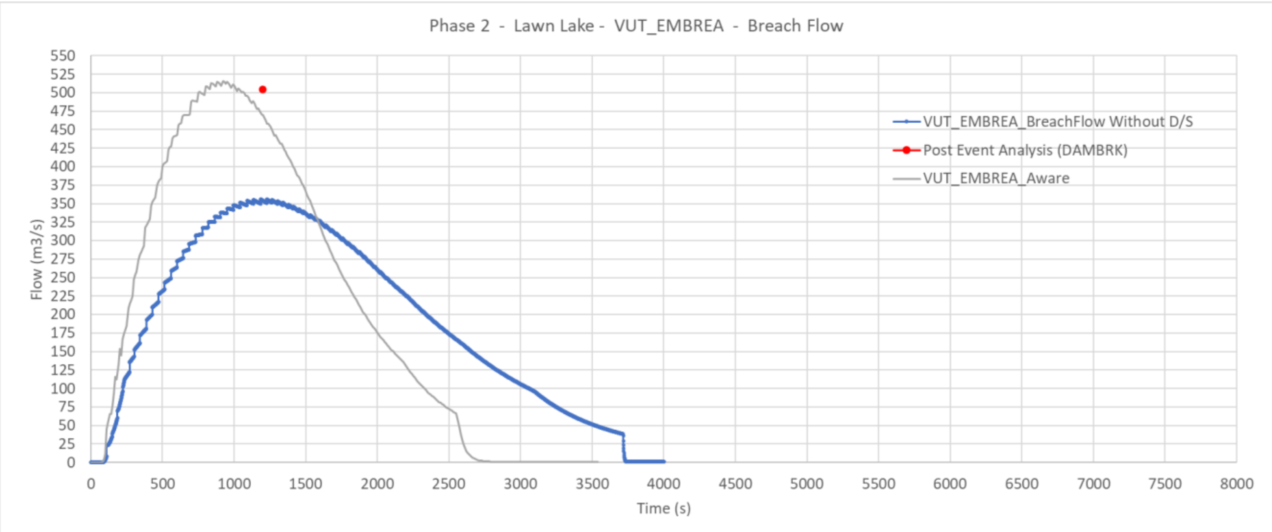


Figure G.11: Phase 2 – Lawn Lake: BUT aware modelling results using EMBREA

BUT – AREBA

A significantly better result was achieved by modifying the soil erodibility (45 instead of 27 $cm^3/N.S$).

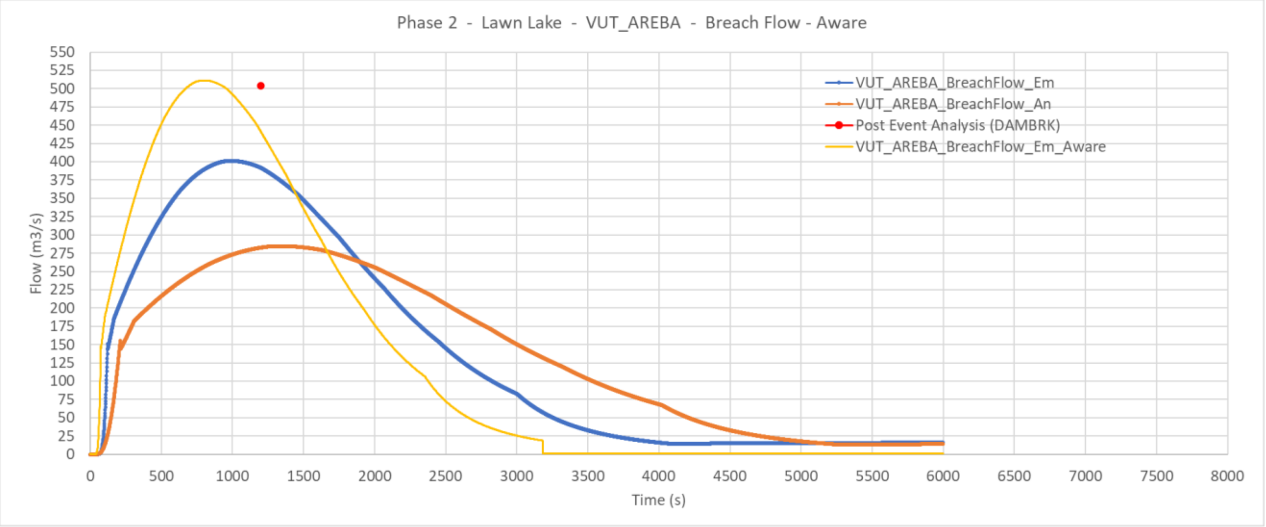


Figure G.12: Phase 2 – Lawn Lake: BUT aware modelling results using AREBA

H Phase 2 – Big Bay Dam Failure Case Study

H.1 Phase 2 – Big Bay Test Case Data Files

File Description	Filename
Test case description (for modellers blind test)	Big_Bay_Blind.xlsx Big_Bay_Aware_v2.xlsx
Analysis & comparison of modelling results	Phase2_ModellingComparison_BigBay_21_07_22.xlsx

H.2 Case Study Description

The Big Bay Dam was constructed in 1991, and failure occurred on 12th March 2004 through piping. The dam was a homogeneous earthen embankment dam with a cutoff wall made from the on-site materials with added bentonite. The initial pipe was located close to the outlet conduit, between the foundation and the conduit. Borings showed that the cutoff wall was of similar permeability to the rest of the dam and did not reach the low permeability foundation but instead stopped in the alluvium layer. Dam materials were mostly classified as SC (clayey sand) with some samples showing traces of coarse sand and/or gravel. The downstream face was covered with grass.

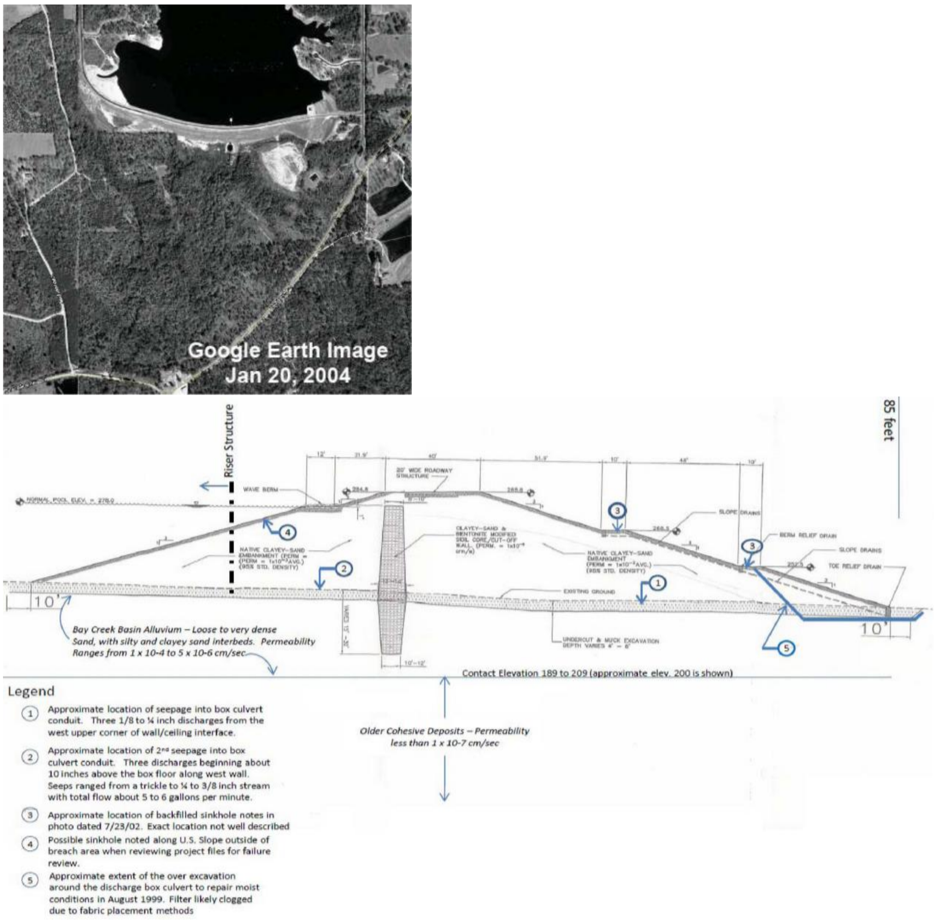
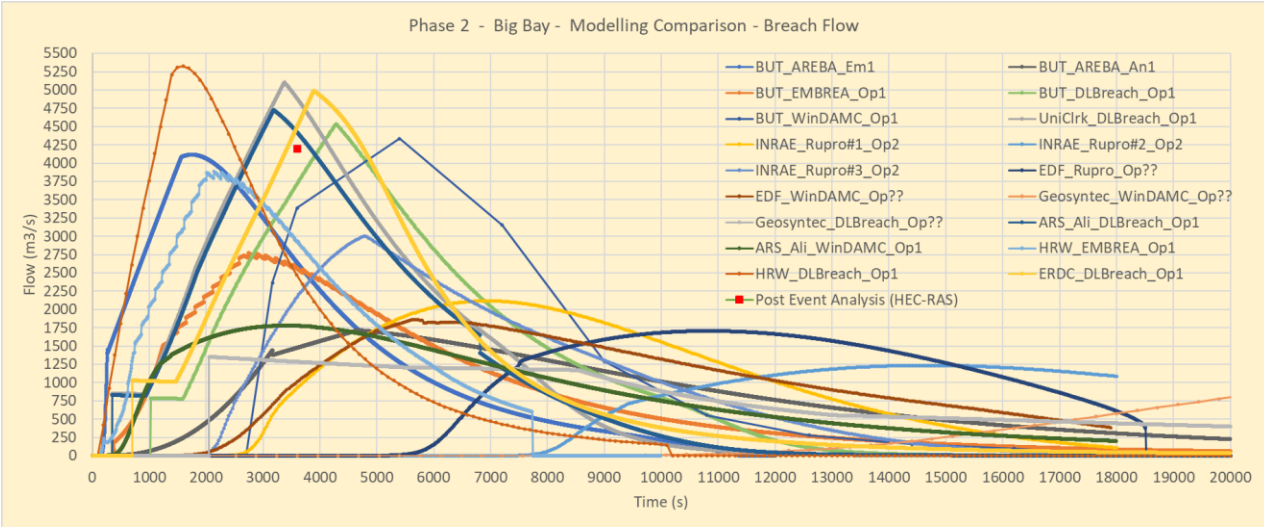


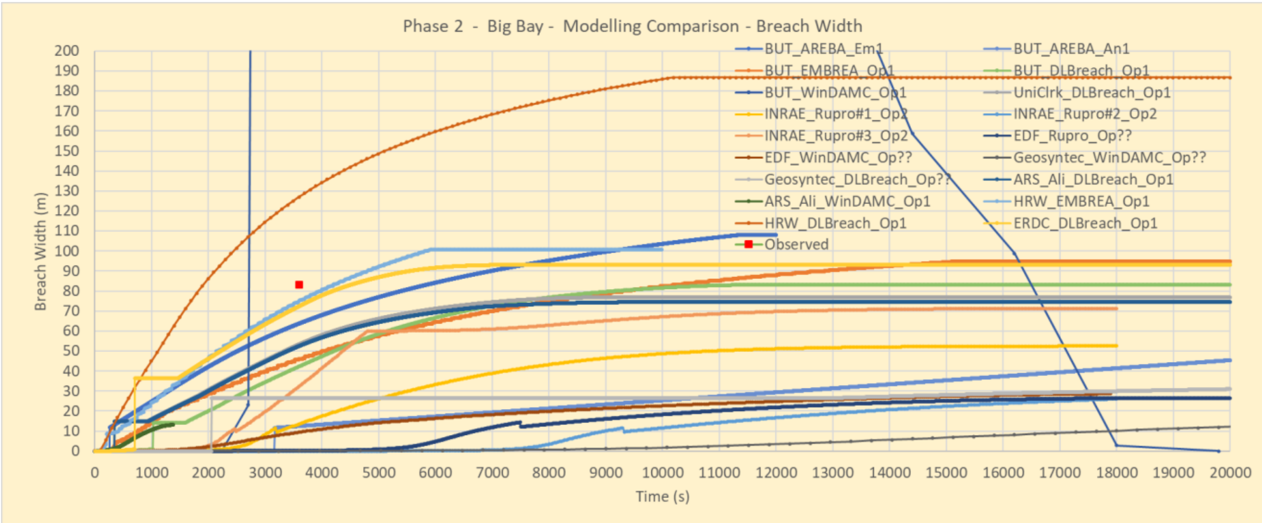
Figure H.1: Big Bay Dam Case Study

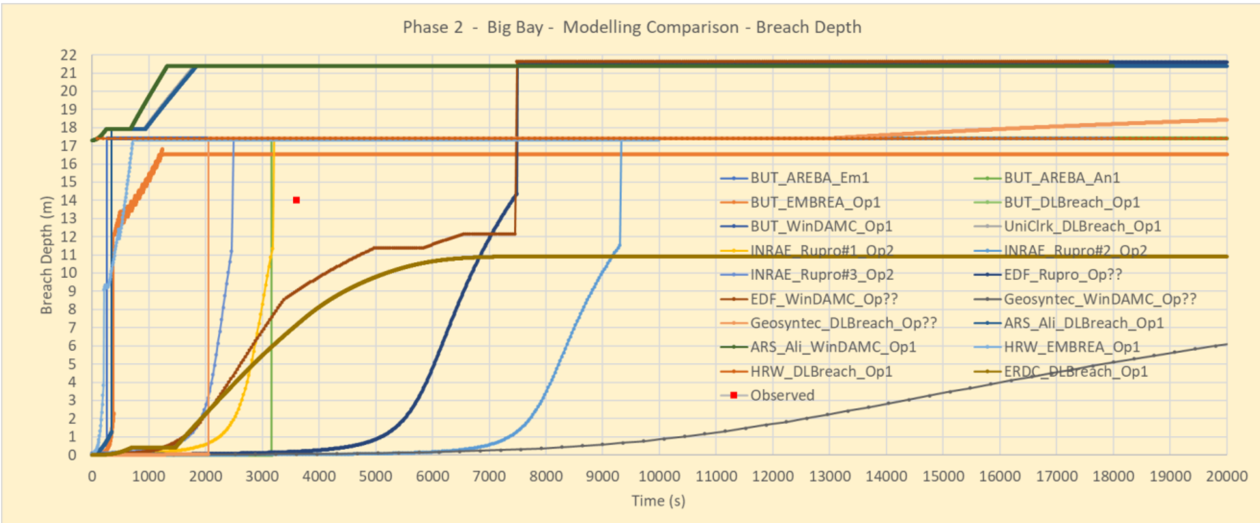
Table H.1: Phase 2 – Big Bay: Modeller Assumptions

Models & Modellers:			Structure		Initiation			Soil Parameters								Flow	Computational					Reported Problems or observations
		Variables	Structure Assumptions For Modelling Approach	Dam Foundation mAD	Initiating Diameter m	Location along dam	Initiating Timing?	Erodibility Kd cm3/NS	Density kg/m3	Cohesion Kpa	Friction Angle	Porosity	Critical Porosity	Hydraulic Conductivity m/s	Critical Shear Stress Pa	Mannings	timestep	section spacing m	Location of breach width parameter	Headcut Erodibility coeficient K m3/s	Headcut parameter C	
USDA	Ali Abdelfatah																					
Blind	WinDAMC			71.30	0.05*0.05 m Rec.	middle dam base	0 Sec	25	2650	15	31	0.422	N/A	N/A	0.15	0.02	0.01	N/A		N/A	N/A	
Blind	DL Breach			71.30	0.05*0.05 m Rec.	middle dam base	0 Sec	25	2650	15	31	0.422	N/A	N/A	0.15	0.016	0.01	N/A		N/A	N/A	
	EMBREA Pro			71.30	0.05*0.05 m Rec.	middle dam base	0 Sec	25	2650	15	31	0.422	N/A	N/A	0.15	0.02	0.01	N/A		N/A	N/A	
HRW	Mohamed Hassan																					
	DL BREACH		Homogeneous structure with kd = 50 cm3/N.s and critical shear stress = 1 pa.	3.00	0.05	0.1	Start of the simulation	50	1667 (dry_estimated)	10 (judgment)	30 (judgement)	0.3	NA	NA	1	0.025	10	10	Middle section	NA	NA	Estimating Kd is tricky for a case such as this one with limited information on compaction and water content. The used value in the model can be quite different from the actual erodibility. Aware testing will reveal this.
	EMBREA		Homogeneous structure with kd = 50 cm3/N.s and critical shear stress = 1 pa.	0.00	0.05	0.1	Start of the simulation	50	1667 (dry_estimated)	10 (judgment)	30 (judgement)	0.3	NA	NA	1	0.025	10	10	Critical section which moves with time and is not fixed	NA	NA	Estimating Kd is tricky for a case such as this one with limited information on compaction and water content. The used value in the model can be quite different from the actual erodibility. Aware testing will reveal this.
Aware	EMBREA Aware 1		The breach depth was allowed to erode below the foundation in this aware run. Variable Kd was also used in this case rathar than same Kd.					A variable Kd factor = 3 was assumed for this case which means for the overtopping case of the failure Kd = 15*3 = 45 cm3/N.s. Kd for the piping part of the breach was kept as is (i.e. 15 cm3/N.s)														Breach depth factor was restored to the default value of 1.6 instead of 1.0 which was used in the blind run.
	EMBREA Aware 2		This run is identical to Aware Runs 1 except that the breach width was restricted to the average breach width value (i.e. 23.5m).					A variable Kd factor = 3 was assumed for this case which means for the overtopping case of the failure Kd = 15*3 = 45 cm3/N.s. Kd for the piping part of the breach was kept as is (i.e. 15 cm3/N.s)														Breach depth factor was restored to the default value of 1.6 instead of 1.0 which was used in the blind run.
ERAU	Ghada Elithy																					
Blind	DL BREACH		Homogenous dam without protection	71.30	0.02	middle	0	14	1866*	10	28	0.3	-	-	3	0.02	0.2	-	-	-	-	
Aware	DL Breach		Homogenous dam without protection	71.30	0.05	middle	0	14	1866*	10	28	0.3			0.24	0.016	0.2					
BUT	Stanislav Kotaska																					
Blind	TUD AREBA		Homogenous dam without protection	71.30	0.01	middle	1	27	2020	11	35	0.3	0.35	2.20E-07	1.6	0.035	1	-	-	-	-	
Blind	EMBREA		Homogenous dam without protection	71.30	0.01	middle	1	27	2020	11	35	0.3	-	-	1.6	0.025	1	1	-	-	-	
Blind	WinDAM		Homogenous dam without protection	71.30	0.01	middle	1	84	2020	11	35	0.3	-	-	2.4	0.025	-	-	-	-	-	
Blind	DL BREACH		Homogenous dam without protection	71.30	0.01	middle	1	10	2400	11	35	0.3	-	-	0.5	0.025	1	-	-	-	-	
Aware	TUD AREBA				30																	
	TUD AREBA				50																	
Aware	EMBREA				73																	
	EMBREA				140																	
Aware	WinDAM				86																	
	WinDAM				117																	
Aware	DL BREACH				9																	
	DL BREACH				13.6																	
	André Paquier																					
Blind	Rupro #1		Homogeneous dam without protection	71.30	0.05				2420			0.423				0.05						erosion rate with Meyer Peter Muller equation
Blind	Rupro #2		Homogeneous dam without protection	71.30	0.01				2700			0.423				0.033						erosion rate with Meyer Peter Muller equation; also changed the diamter from 0.2mm in other runs to 0.3mm here
Blind	Rupro #3		Homogeneous dam without protection	71.30	0.05				2420			0.423				0.05						erosion rate with MPM equation equivalent to Kd=5; difficulties to set cross sections up and downstream
Aware	Rupro #1		Homogeneous dam without protection	71.30	0.05				2420			0.423				0.067						Same as Rupro 1 except manning
UniClrk	Weiming Wu																					
Blind	DL BREACH		trapezodal cross-section: dam is 17.4 m high, dam crest is 12 m wide, upstream slope 1V:3H and downstream slope 1V:3H. The dam foundation is assumed to have a 4 m thick erodible layer	71.30	0.05	middle, dam base	at t=0 s	25	2650	15	31	0.3			0.15	0.016	0.5		dam crest			The pipe entrance head loass coefficient is 1.5 . The reservoir storage capacity oprions 1 and 2 are used
Aware	DL BREACH																					
EDF	Pierre Squillari (Geophy)																					
Blind	WinDAM		The non-erodible foundation is below the dame base	67 m	0.03 x 0.03	middle	none	5 cm3/N/s	total weight = 20 kN/m3 = 127 lbs/ft3							0.03 for crest and slopes (no impact on results)	0.01 hr					Cu = 1000 psf
Blind			best estimates of key values are taken.	220 ft				2.8 ft3/hr/lbs														Tau in the initial conduit (Pa) =

H.3 Phase 2 – Big Bay Modelling Results







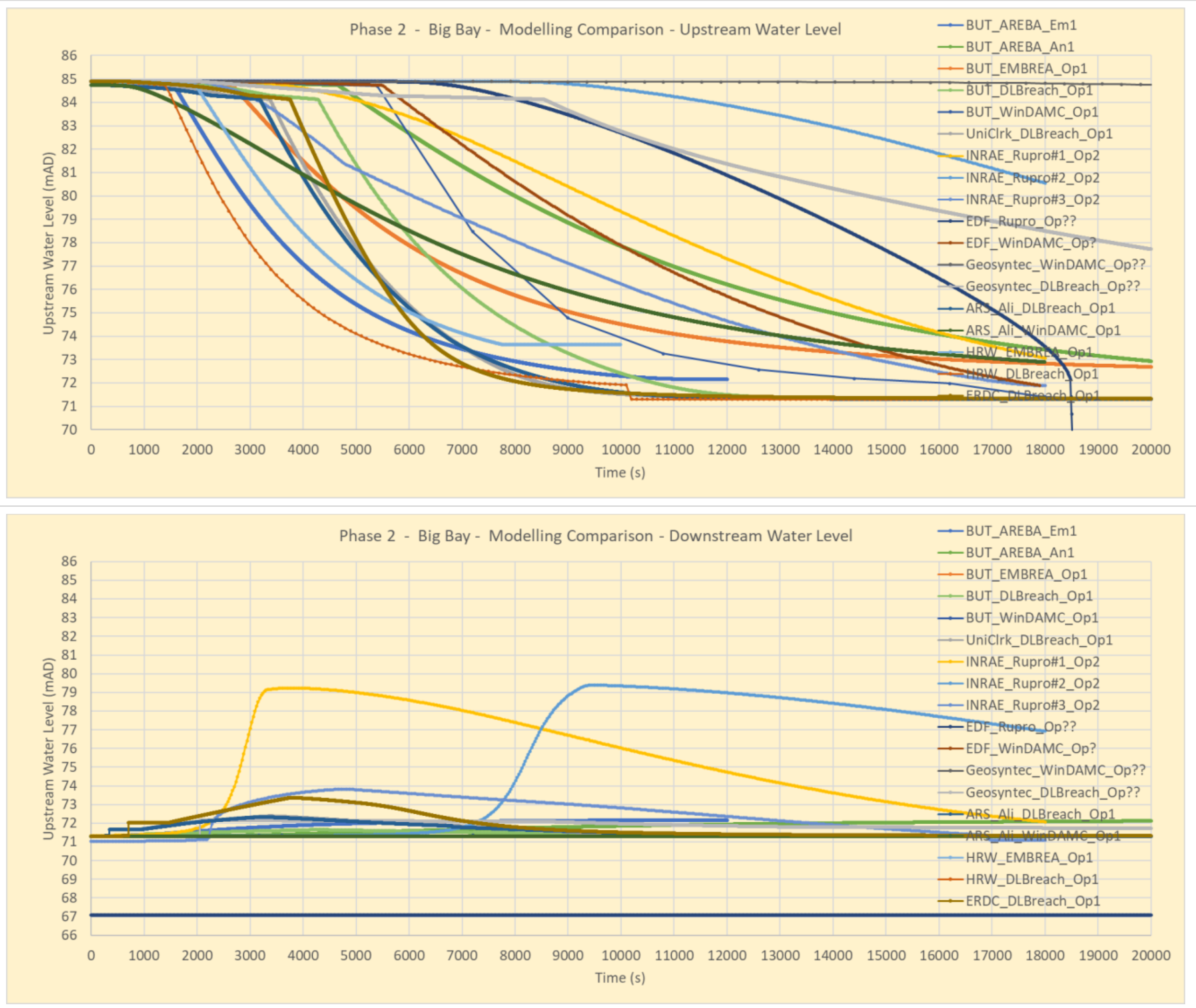
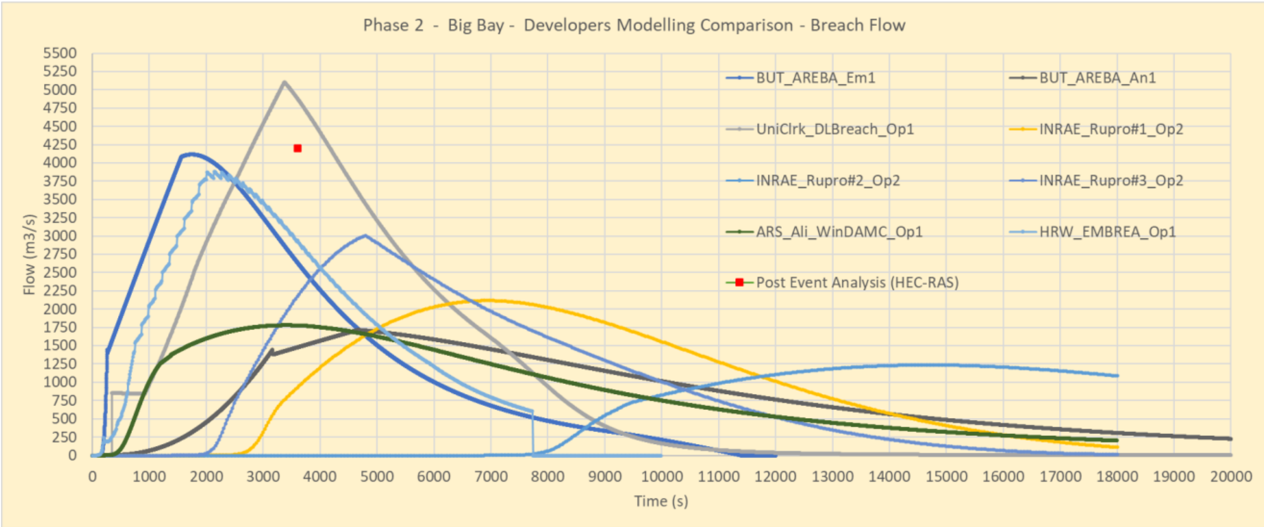
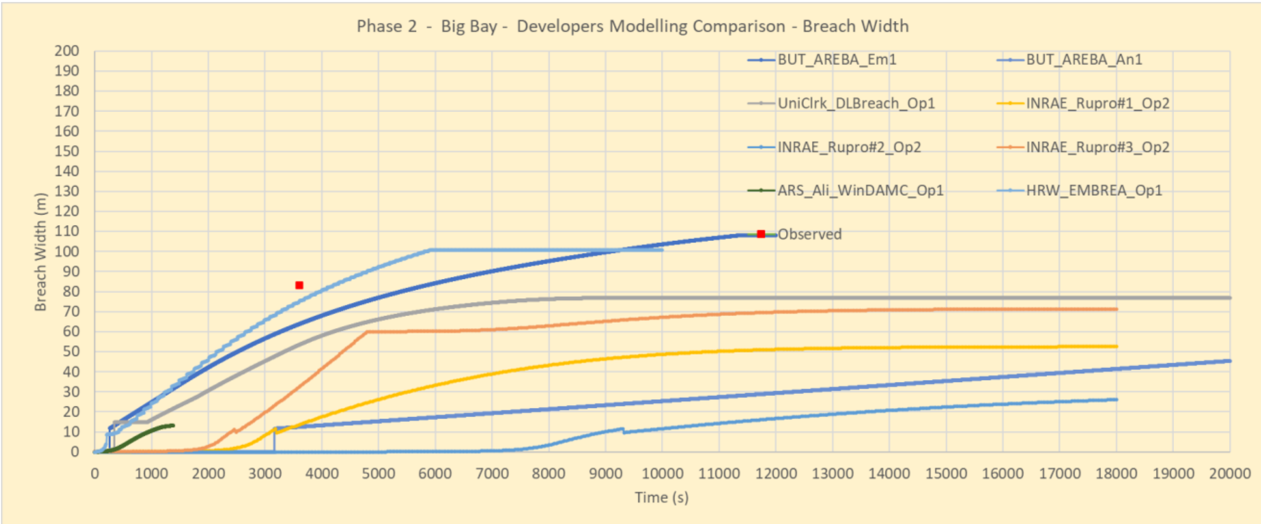
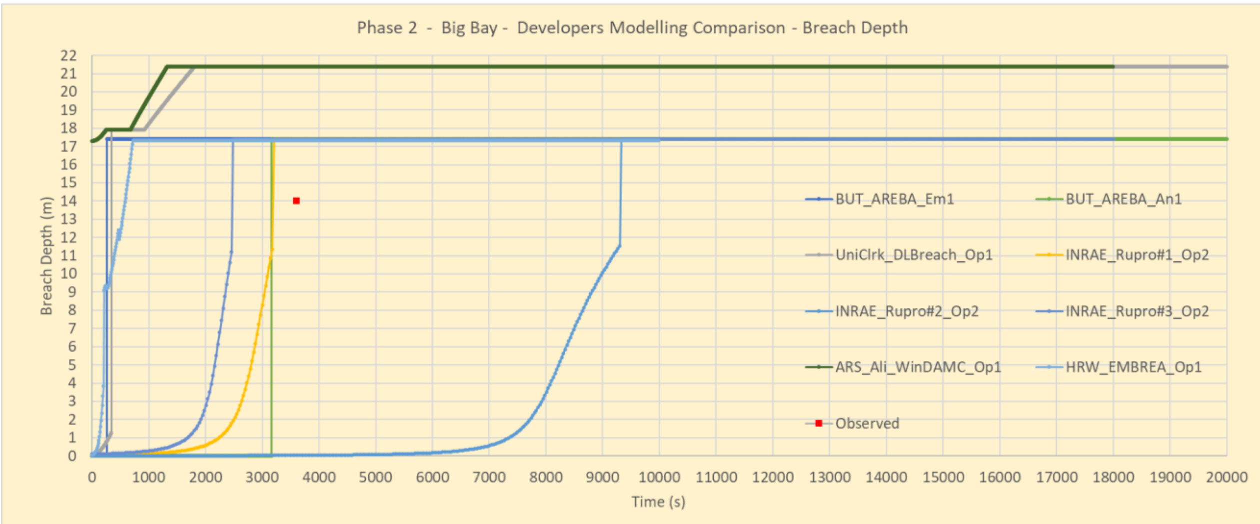


Figure H.2: Phase 2 – Big Bay: All modelling results


Phase 2 - Big Bay - Developers Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
UniClrk_DLBreach_Op1
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
ARS_Ali_WinDAMC_Op1
HRW_EMBREA_Op1
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - Developers Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
UniClrk_DLBreach_Op1
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
ARS_Ali_WinDAMC_Op1
HRW_EMBREA_Op1
Observed


Phase 2 - Big Bay - Developers Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
UniClrk_DLBreach_Op1
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
ARS_Ali_WinDAMC_Op1
HRW_EMBREA_Op1
Observed

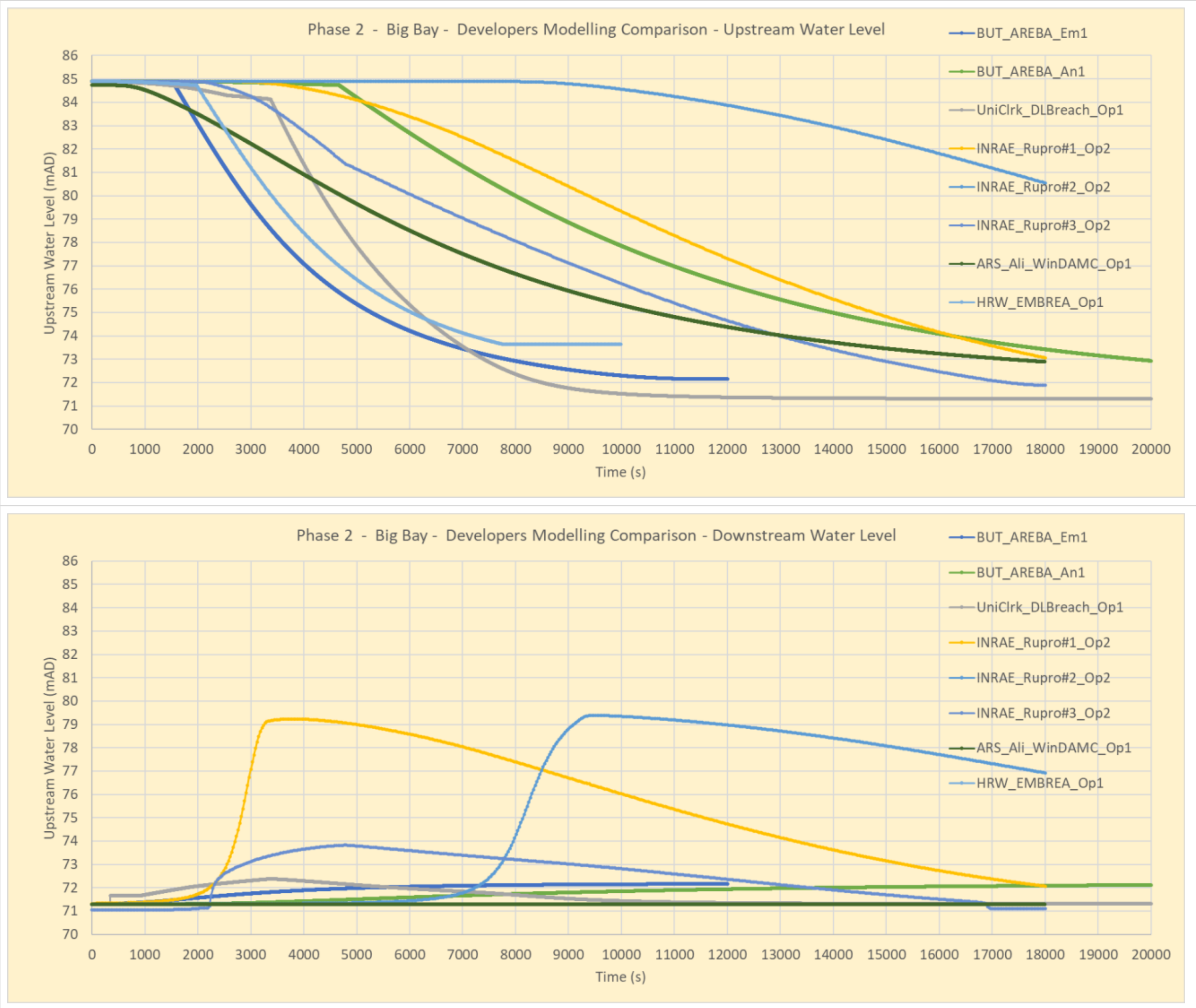
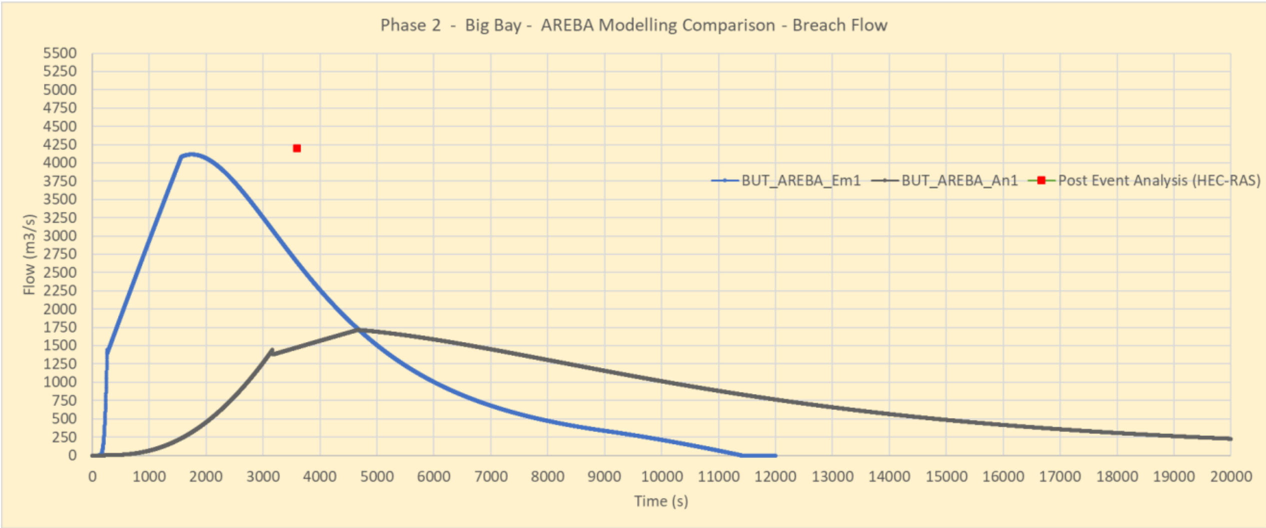
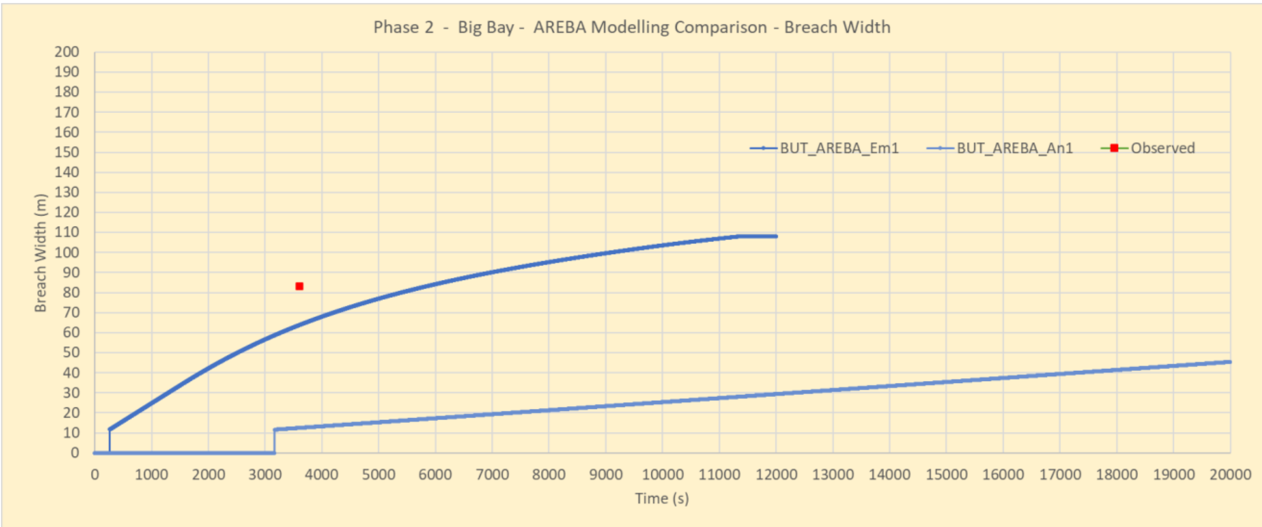
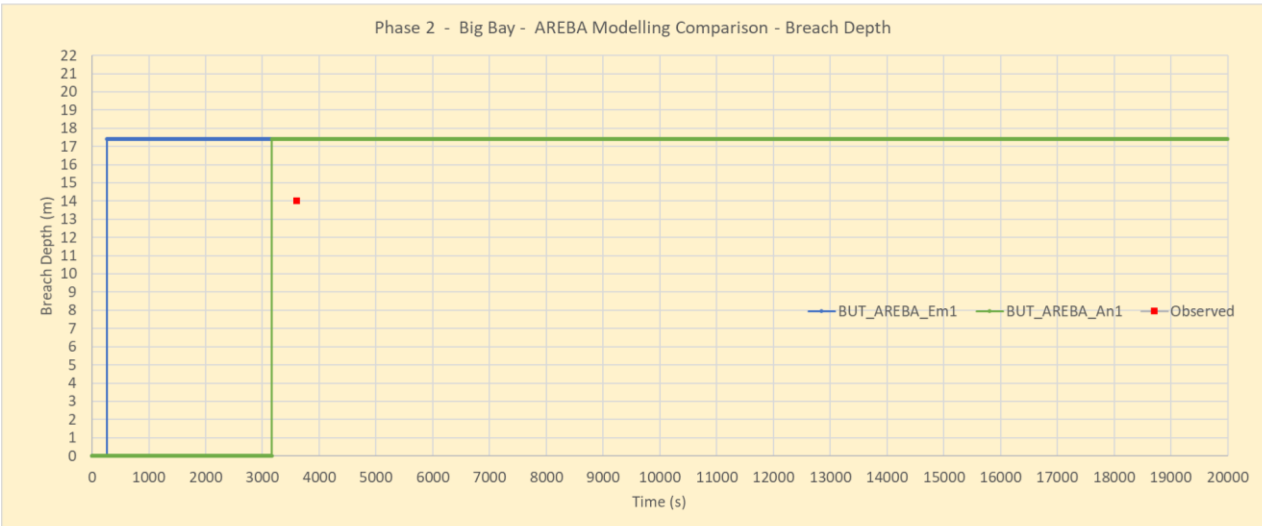


Figure H.3: Phase 2 – Big Bay: Developers modelling results


Phase 2 - Big Bay - AREBA Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - AREBA Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
Observed


Phase 2 - Big Bay - AREBA Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
BUT_AREBA_Em1
BUT_AREBA_An1
Observed

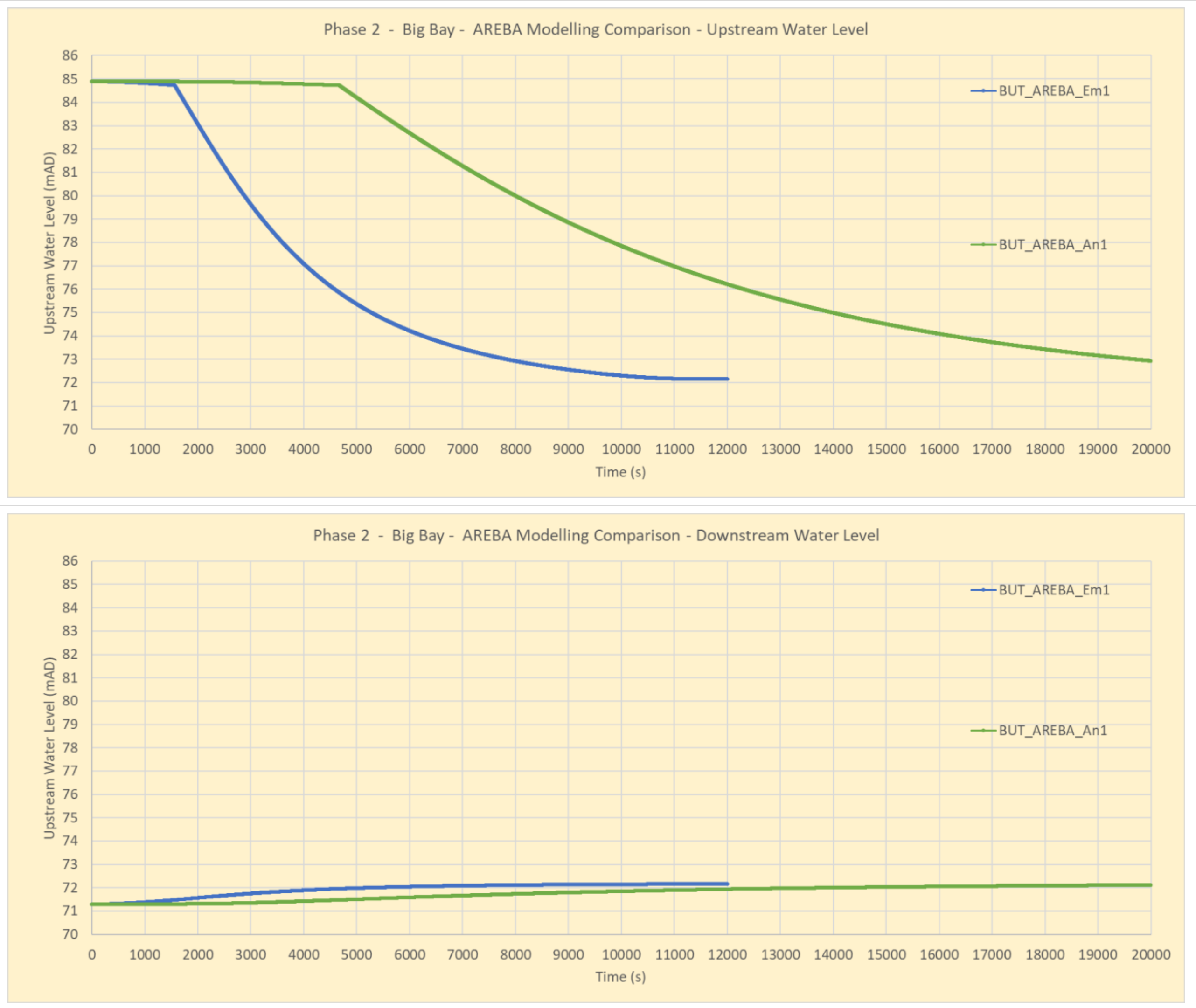
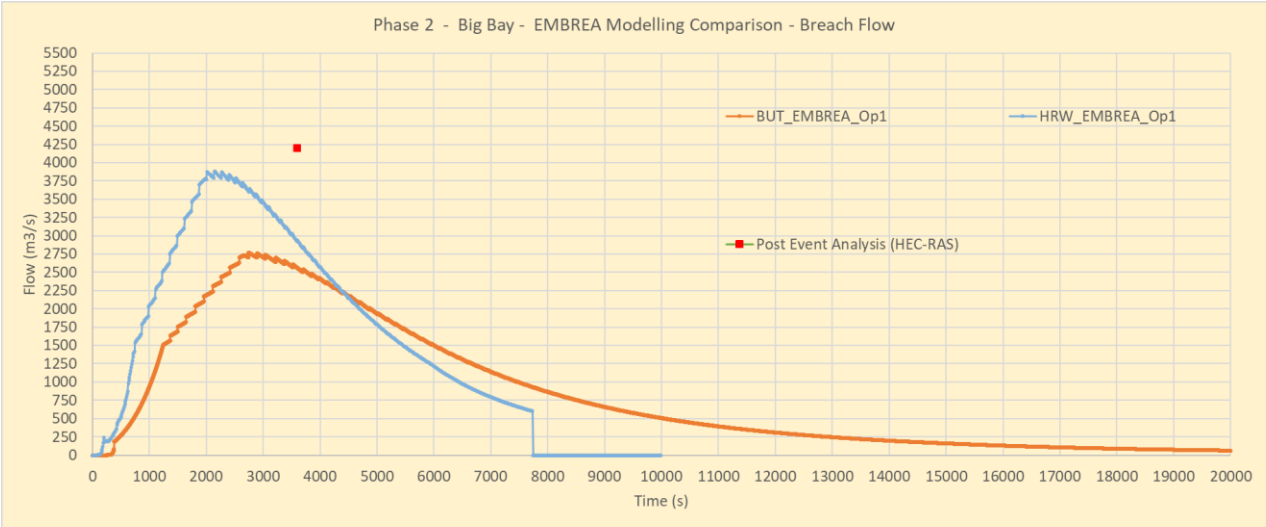
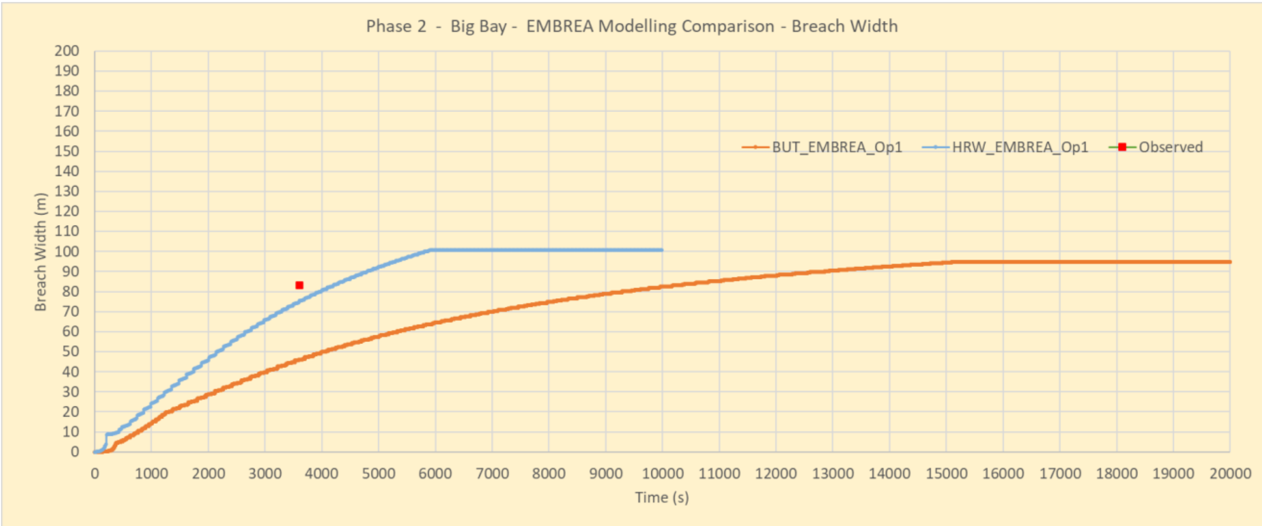
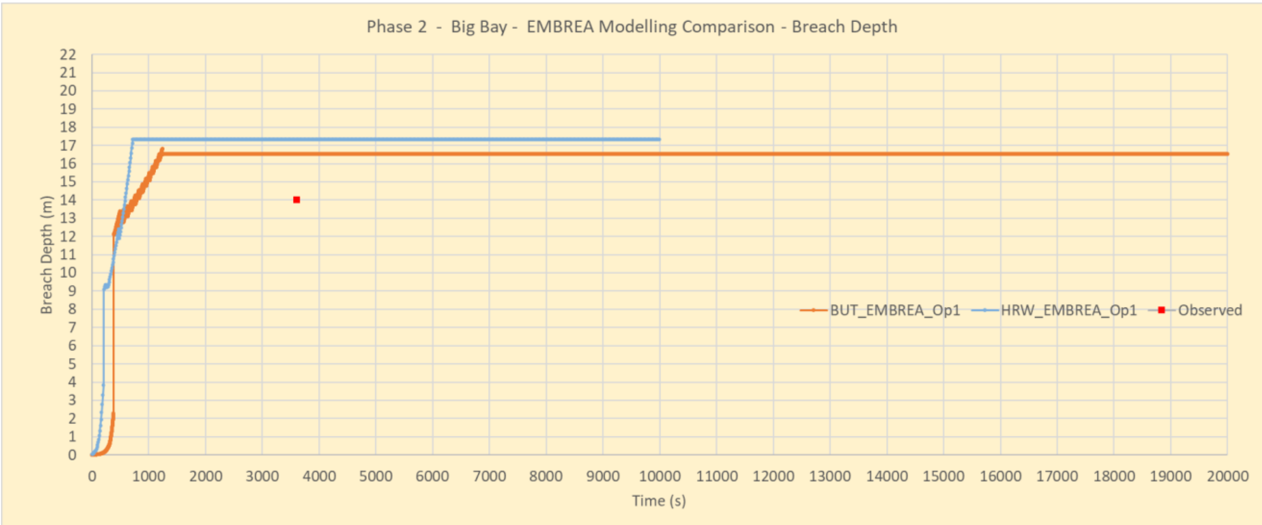
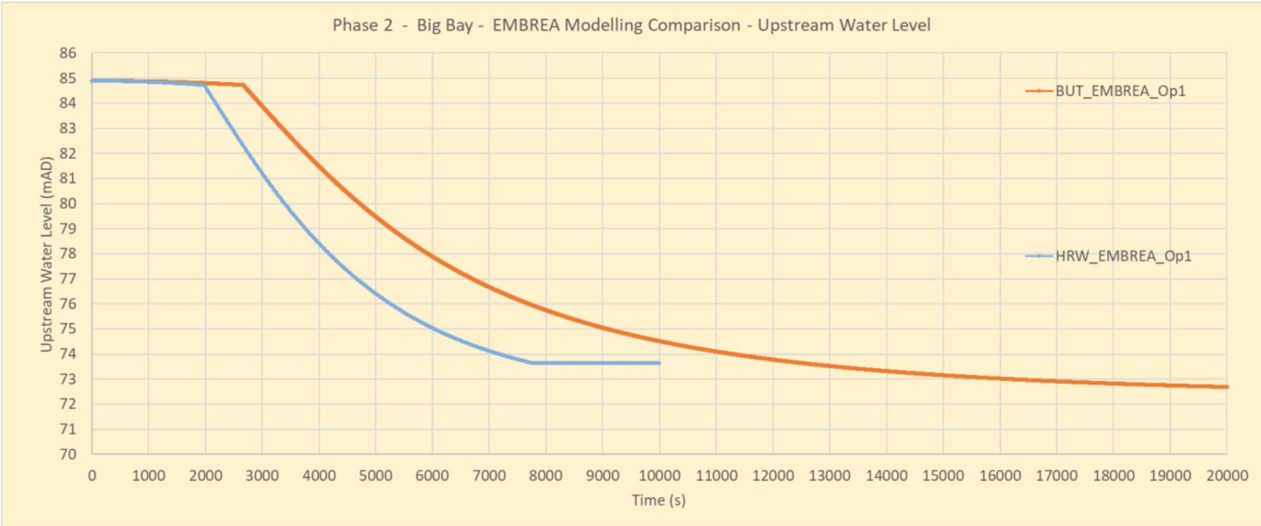


Figure H.4: Phase 2 – Big Bay: Modelling results using AREBA


Phase 2 - Big Bay - EMBREA Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_EMBREA_Op1
HRW_EMBREA_Op1
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - EMBREA Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
BUT_EMBREA_Op1
HRW_EMBREA_Op1
Observed


Phase 2 - Big Bay - EMBREA Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
BUT_EMBREA_Op1
HRW_EMBREA_Op1
Observed



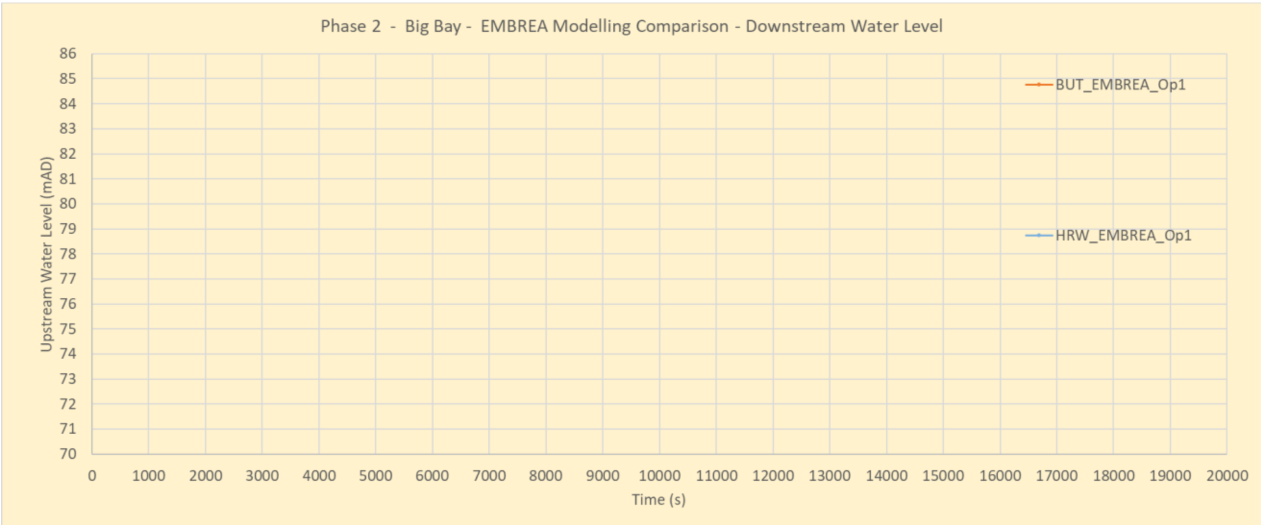
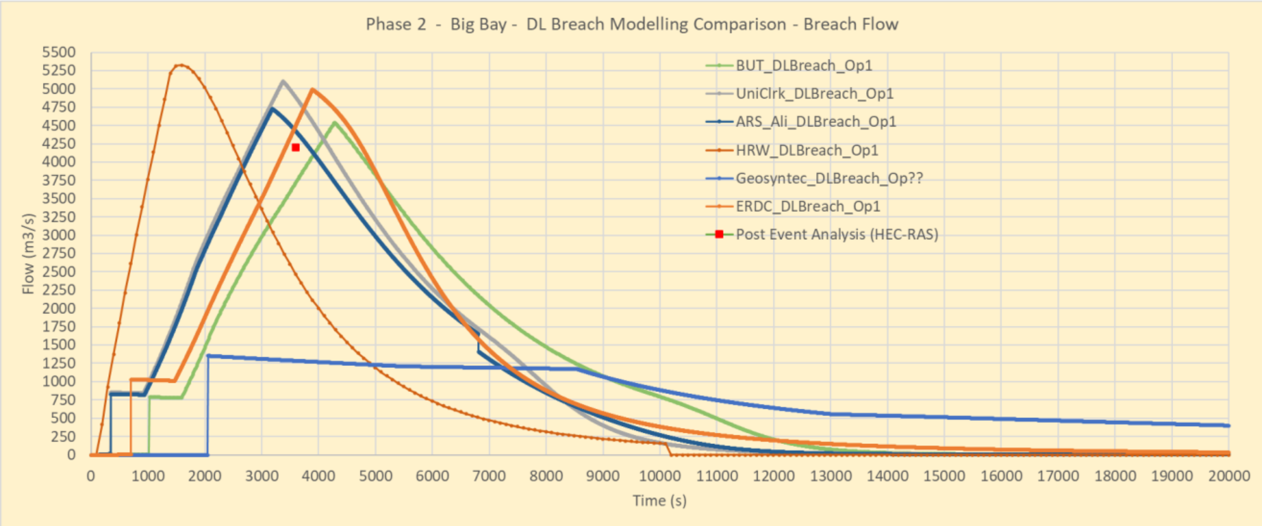
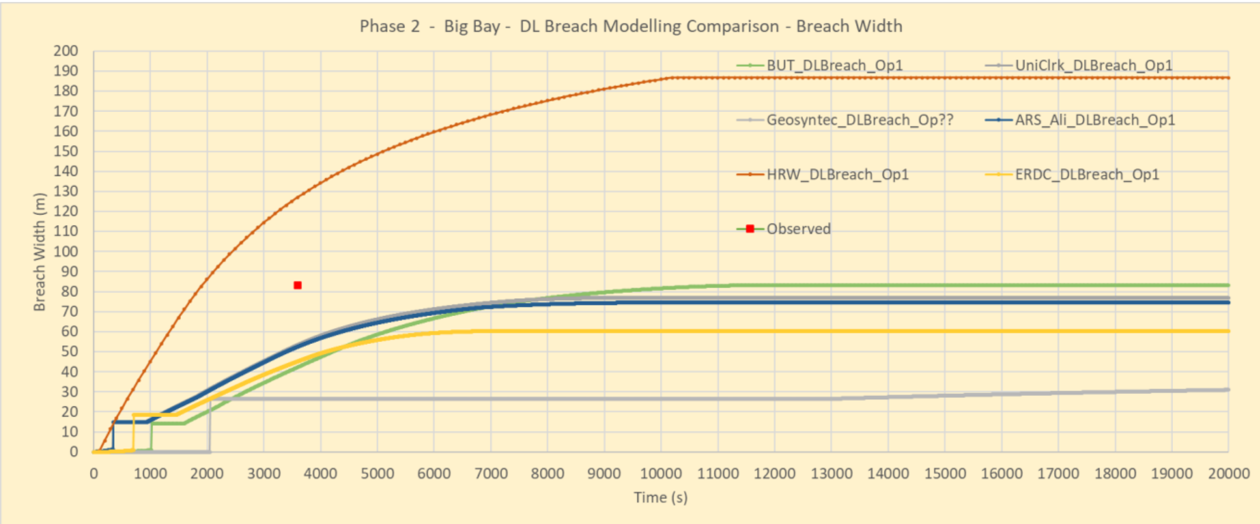
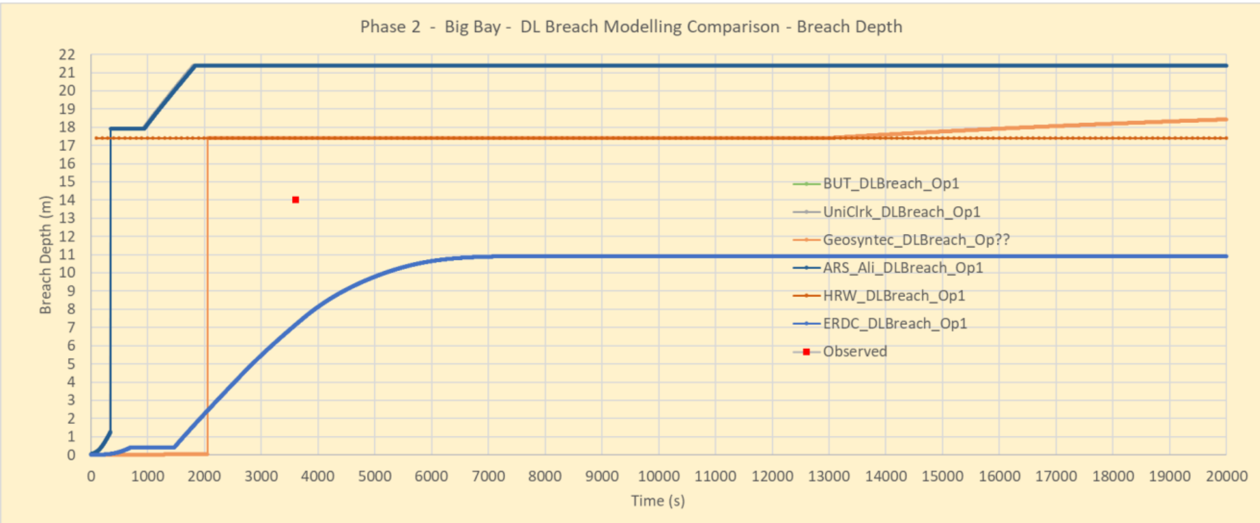


Figure H.5: Phase 2 – Big Bay: Modelling results using EMBREA


Phase 2 - Big Bay - DL Breach Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_DLBreach_Op1
UniClrk_DLBreach_Op1
ARS_Ali_DLBreach_Op1
HRW_DLBreach_Op1
Geosyntec_DLBreach_Op??
ERDC_DLBreach_Op1
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - DL Breach Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
BUT_DLBreach_Op1
UniClrk_DLBreach_Op1
Geosyntec_DLBreach_Op??
ARS_Ali_DLBreach_Op1
HRW_DLBreach_Op1
ERDC_DLBreach_Op1
Observed


Phase 2 - Big Bay - DL Breach Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
BUT_DLBreach_Op1
UniClrk_DLBreach_Op1
Geosyntec_DLBreach_Op??
ARS_Ali_DLBreach_Op1
HRW_DLBreach_Op1
ERDC_DLBreach_Op1
Observed

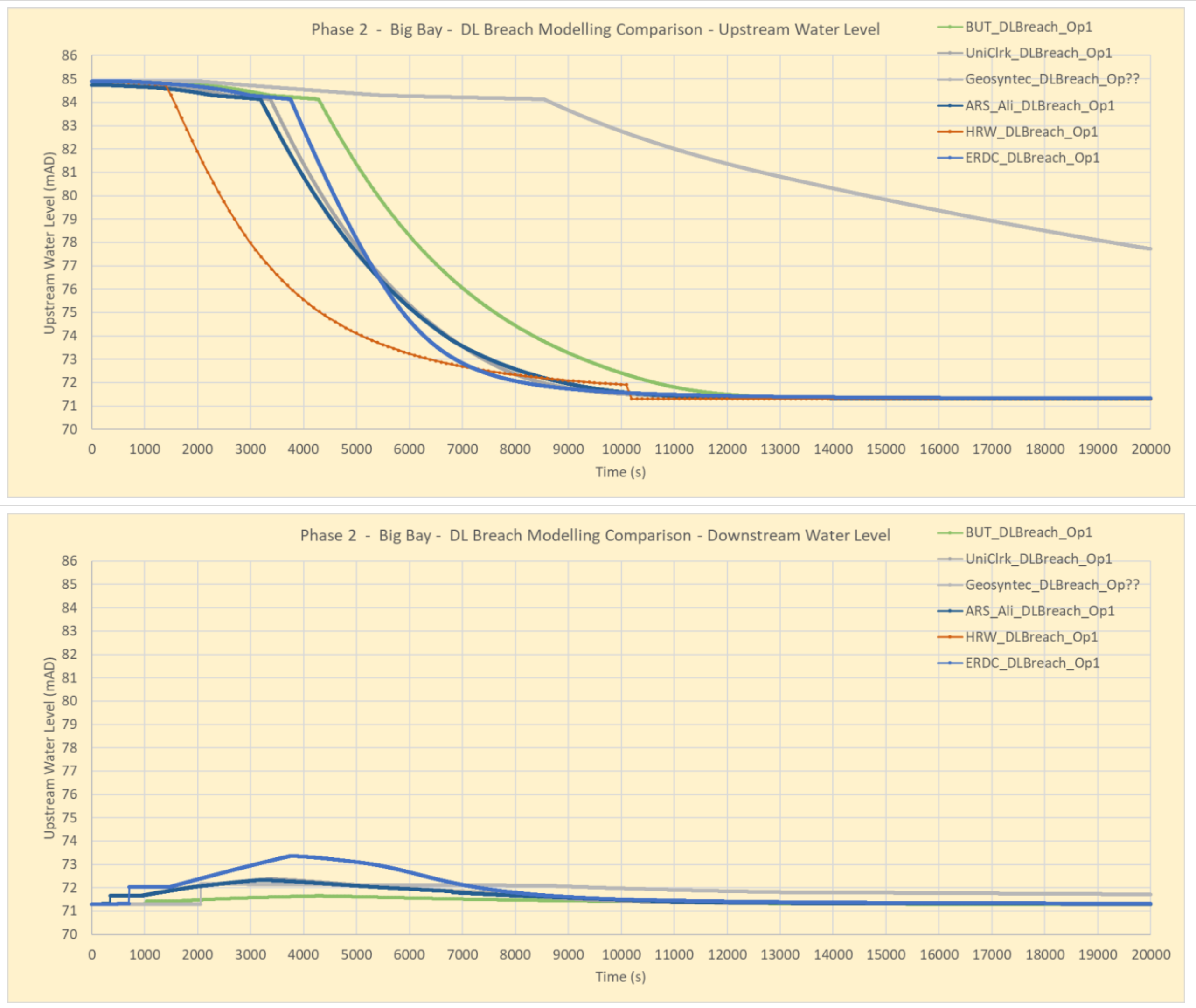
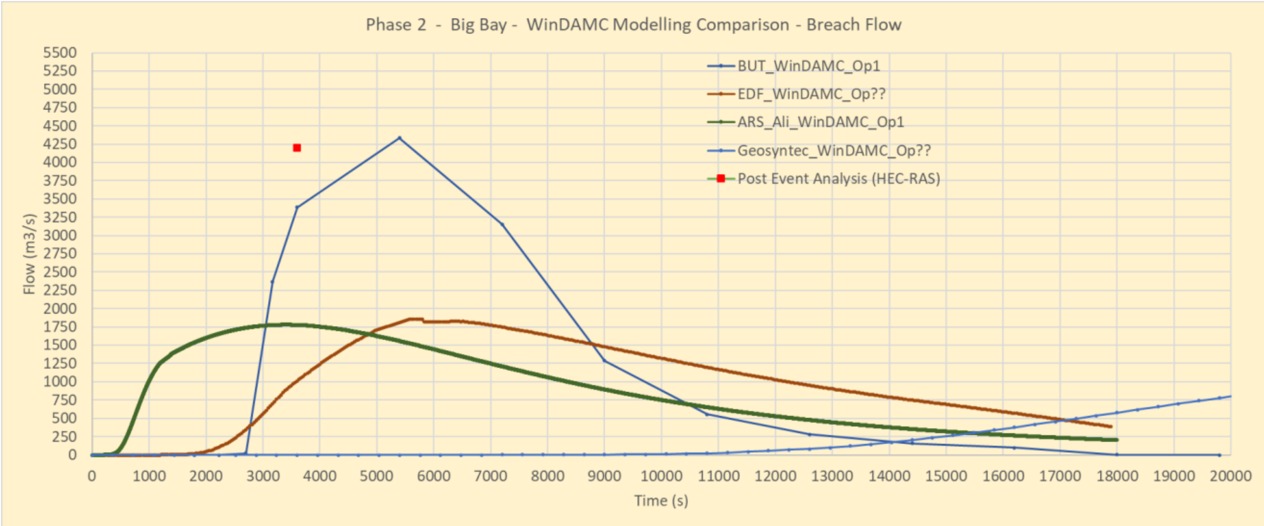
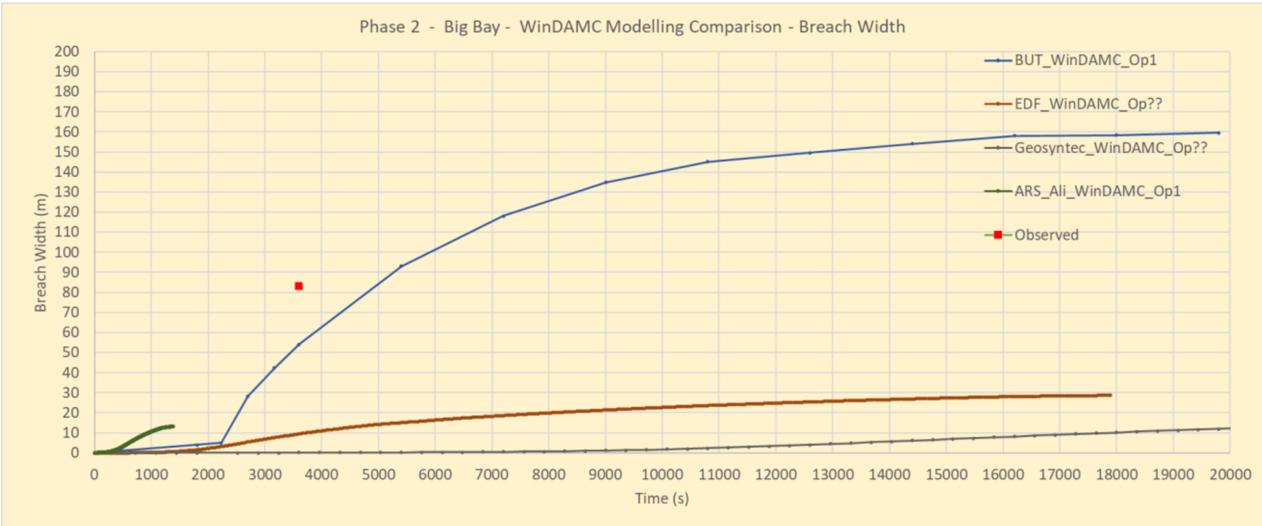
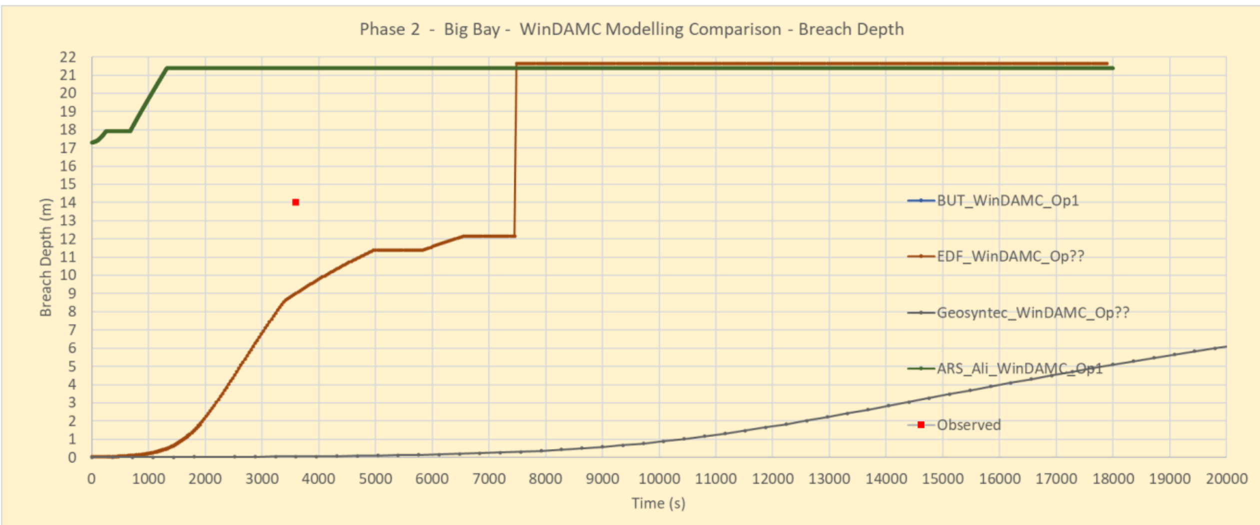
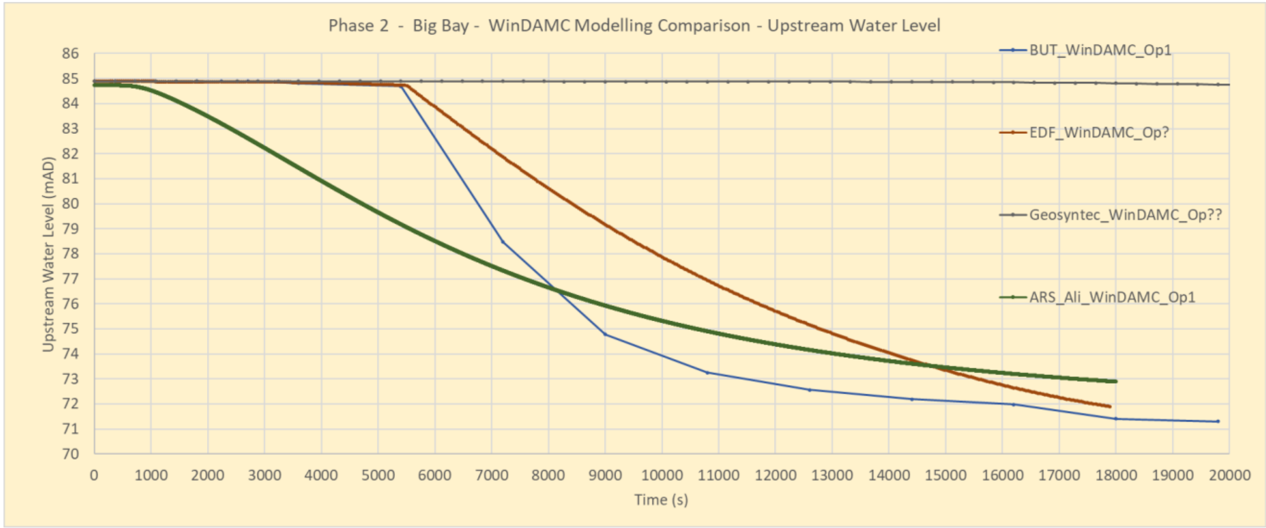


Figure H.6: Phase 2 – Big Bay: Modelling results using DLBreach


Phase 2 - Big Bay - WinDAMC Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
BUT_WinDAMC_Op1
EDF_WinDAMC_Op??
ARS_Ali_WinDAMC_Op1
Geosyntec_WinDAMC_Op??
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - WinDAMC Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
BUT_WinDAMC_Op1
EDF_WinDAMC_Op??
Geosyntec_WinDAMC_Op??
ARS_Ali_WinDAMC_Op1
Observed


Phase 2 - Big Bay - WinDAMC Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
BUT_WinDAMC_Op1
EDF_WinDAMC_Op??
Geosyntec_WinDAMC_Op??
ARS_Ali_WinDAMC_Op1
Observed



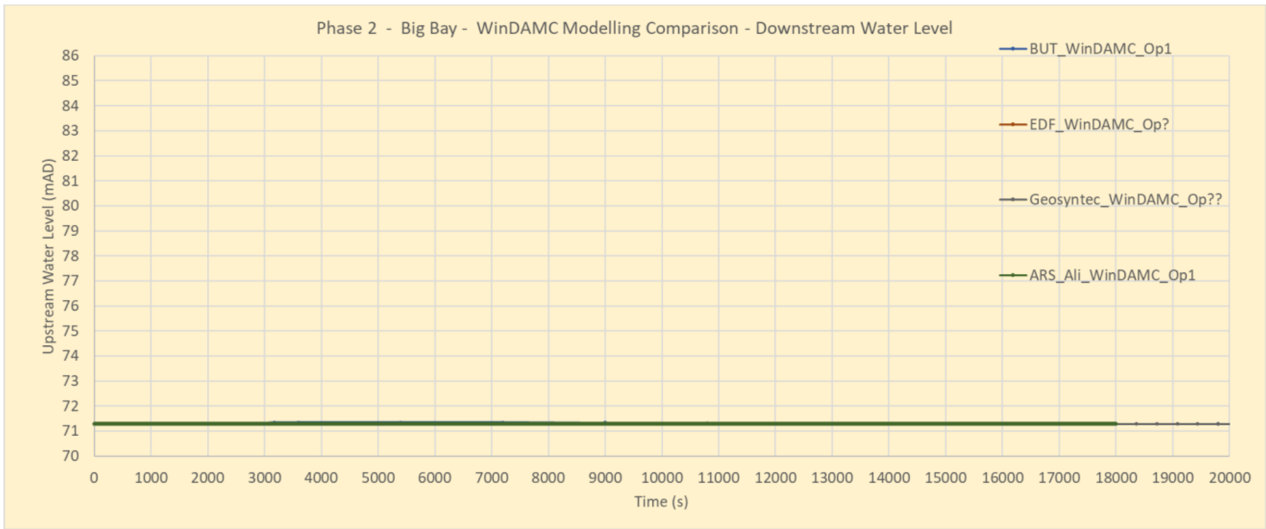
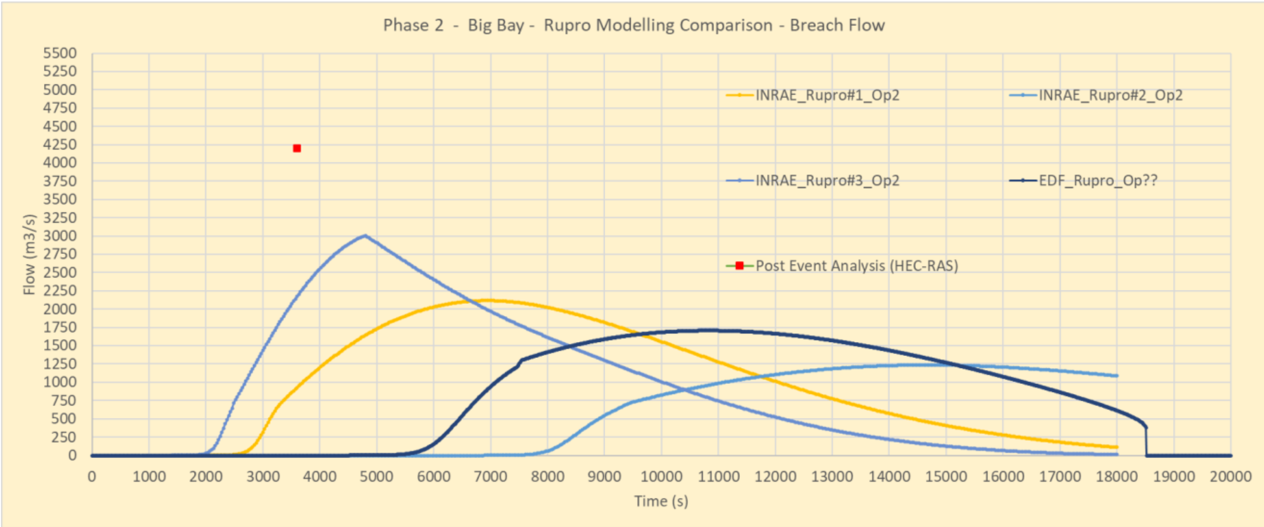
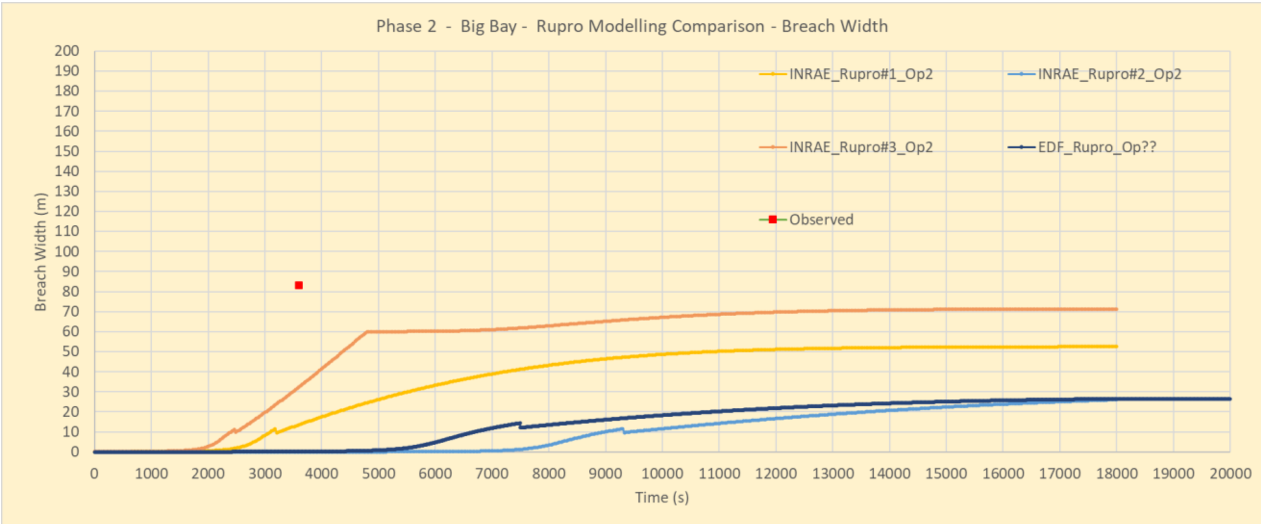
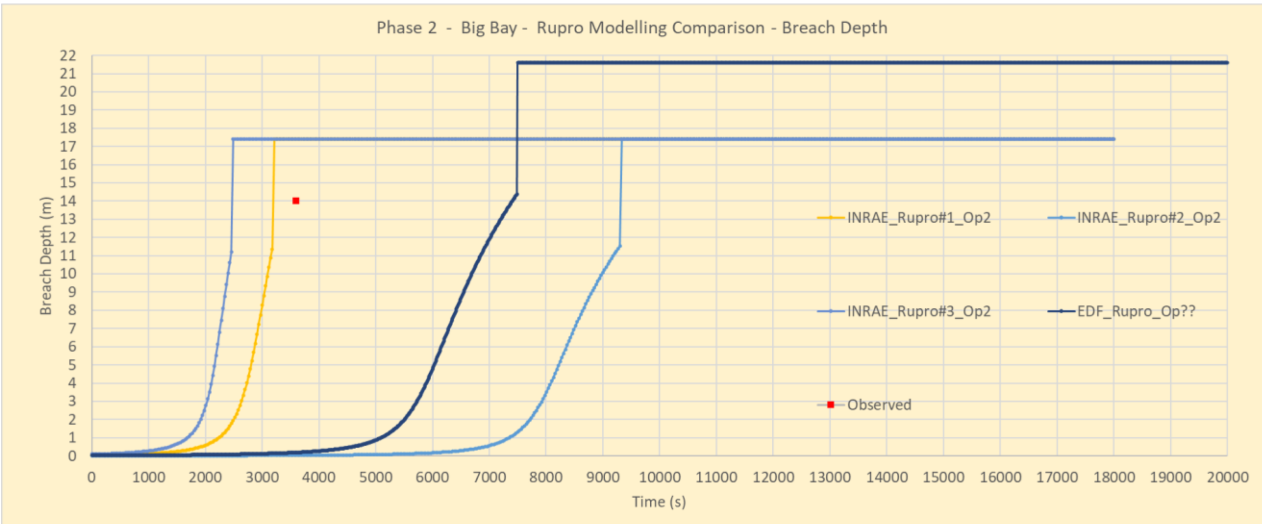


Figure H.7: Phase 2 – Big Bay: Modelling results using WinDAM C


Phase 2 - Big Bay - Rupro Modelling Comparison - Breach Flow
Flow (m3/s)
Time (s)
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
EDF_Rupro_Op??
Post Event Analysis (HEC-RAS)


Phase 2 - Big Bay - Rupro Modelling Comparison - Breach Width
Breach Width (m)
Time (s)
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
EDF_Rupro_Op??
Observed


Phase 2 - Big Bay - Rupro Modelling Comparison - Breach Depth
Breach Depth (m)
Time (s)
INRAE_Rupro#1_Op2
INRAE_Rupro#2_Op2
INRAE_Rupro#3_Op2
EDF_Rupro_Op??
Observed

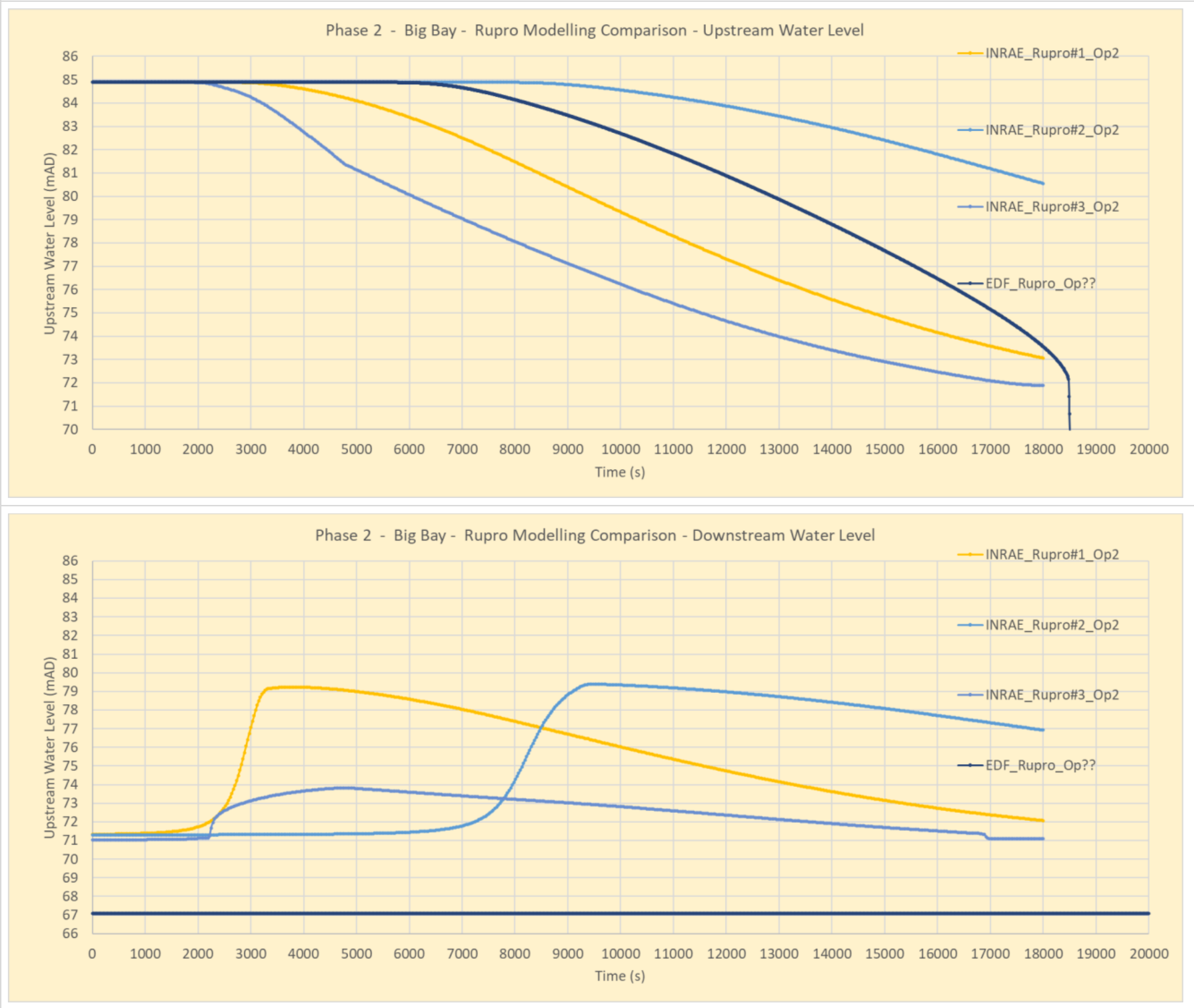


Figure H.8: Phase 2 – Big Bay: Modelling results using Rupro

H.4 Phase 2 – Big Bay Aware Modelling Results

USDA-ARS, ERAU, HRW, INRAE and BUT all undertook aware modelling for the Big Bay Dam failure case.

USDA-ARS investigated the performance of:	EMBREA Pro and DLBreach
ERAU investigated the performance of:	DLBreach
HRW investigated the performance of:	EMBREA
INRAE investigated the performance of:	Rupro
BUT investigated the performance of:	AREBA, DLBreach, EMBREA and WinDAM C.

ARS_Ali - EMBREA Pro

Blind modelling resulted in under prediction of the breach flow, albeit the timing of the peak was not far from observed.

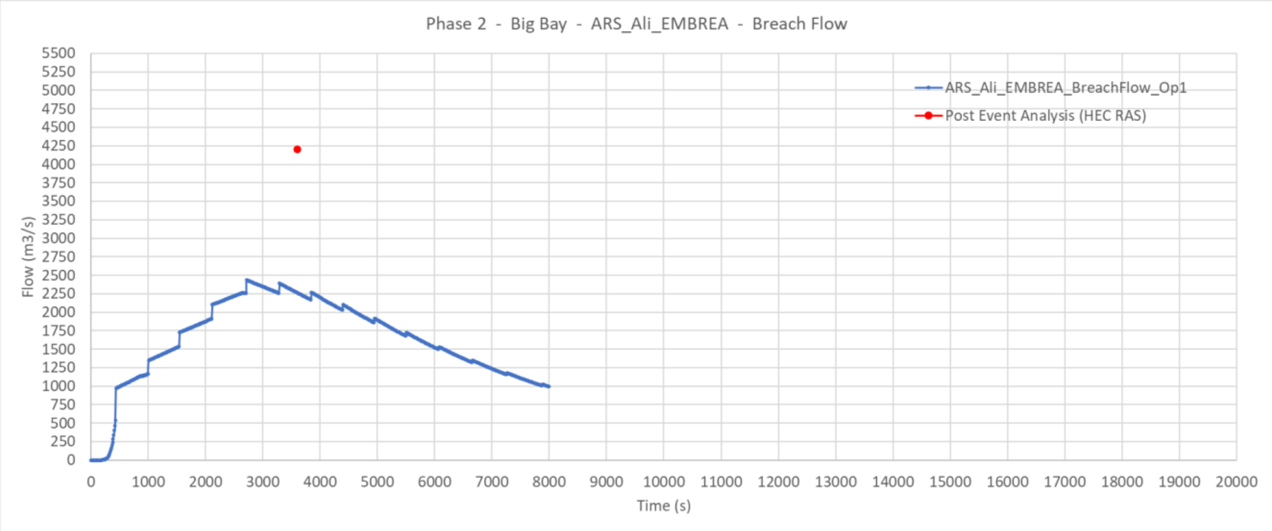


Figure H.9: Phase 2 – Big Bay: ARS blind modelling results using EMBREA

ARS_Ali then undertook an analysis of how varying K_d affected the predictions with the following results.

Impact of varying K_d:

- K_d = 25 cm³/N.s Time to peak = 3170 s Q peak = 2435 m³/s
  T_c = 0.15 Pa
- K_d = 45 cm³/N.s Time to peak = 2310 s Q peak = 3089 m³/s
  T_c = 5 Pa
- K_d = 100 cm³/N.s Time to peak = 1555 s Q peak = 4401 m³/s.
  T_c = 5 Pa.

Hence increasing erodibility creates a predicted peak flow closer to the observed value, but earlier than the observed timing.

ARS_Ali - DLBreach

Blind modelling resulted in close prediction of the breach flow, as shown below.

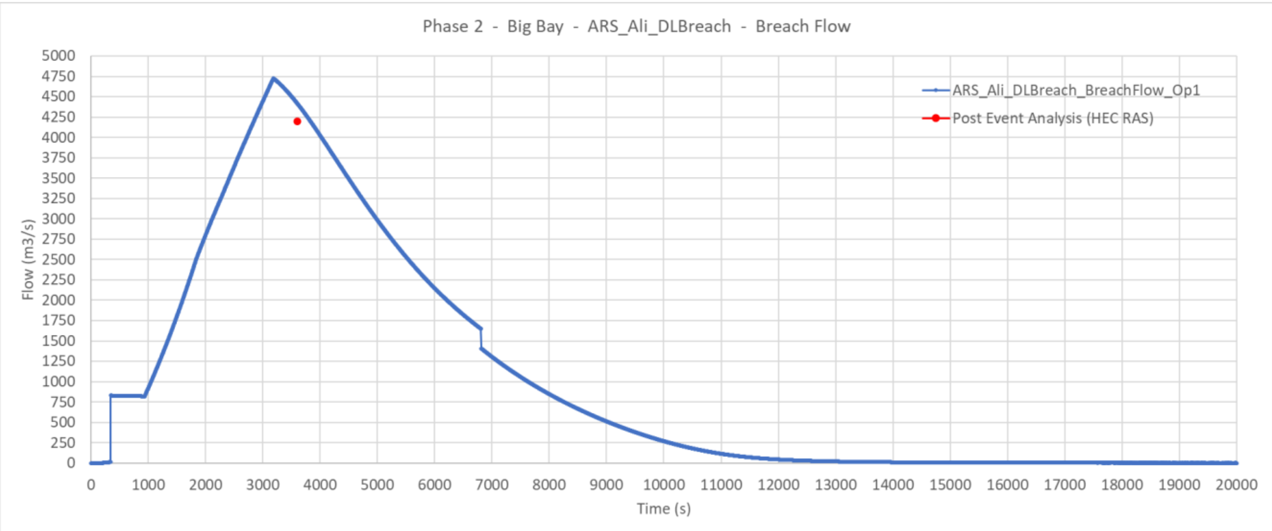


Figure H.10: Phase 2 – Big Bay: ARS blind modelling results using DLBreach

ARS then undertook an analysis of how varying K_d affected the predictions with the following results.

Impact of K_d:

- K_d = 14.4 ft³/lbs/hr = 25 cm³/N/s à Time peak = 0.88 h, Q peak = 166934 cfs (4727 m³/s)
- K_d = 25.45 ft³/lbs/hr = 45 cm³/N/s à Time peak = 0.65 h, Q peak = 219164 cfs (6206 m³/s)
  Tau=0.1 psf=5 pa
- K_d = 56.56 ft³/lbs/hr = 100 cm³/N/s → Time peak = 0.432 h, Q peak = 321331 cfs (9099 m³/s)
  Tau=0.1 psf=5 pa.

ARS_Ali then undertook an analysis of how varying K_d affected the predictions with the following results:

Impact of varying K_d:

- K_d = 25 cm³/N.s Time to peak = 3170 s Q peak = 4727 m³/s
- K_d = 45 cm³/N.s Time to peak = 2340 s Q peak = 6206 m³/s
  T_c = 5 Pa
- K_d = 100 cm³/N.s Time to peak = 1555 s Q peak = 9099 m³/s
  T_c = 5 Pa.

Hence changing erodibility changes the predicted peak flow but also moves the timing earlier. This behaviour is similar to the trends seen with EMBREA, but absolute values differ.

ERAU – DLBreach

ERAU undertook modelling using DLBreach and obtained similar, albeit slightly different results for the blind modelling as compared to ARS.

Subsequent Aware modelling used a 5cm initiation pipe (instead of 2 cm) and reduced the critical shear stress to 0.24 Pa (instead of 3 Pa).

The aware results gave a closer prediction to peak discharge, but later than observed.

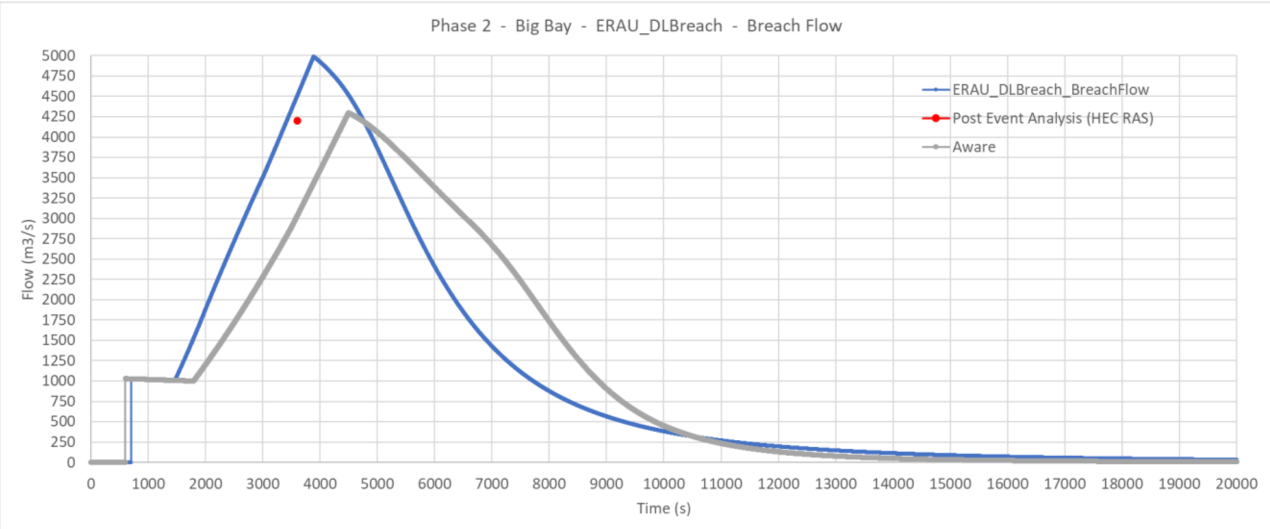


Figure H.11: Phase 2 – Big Bay: ERAU aware modelling results using DLBreach

HRW – EMBREA

Varying a number of parameters and the approach to modelling was undertaken, using both reservoir bathymetry options (Option 1 and Option 2). Significant improvements to modelling accuracy were achieved.

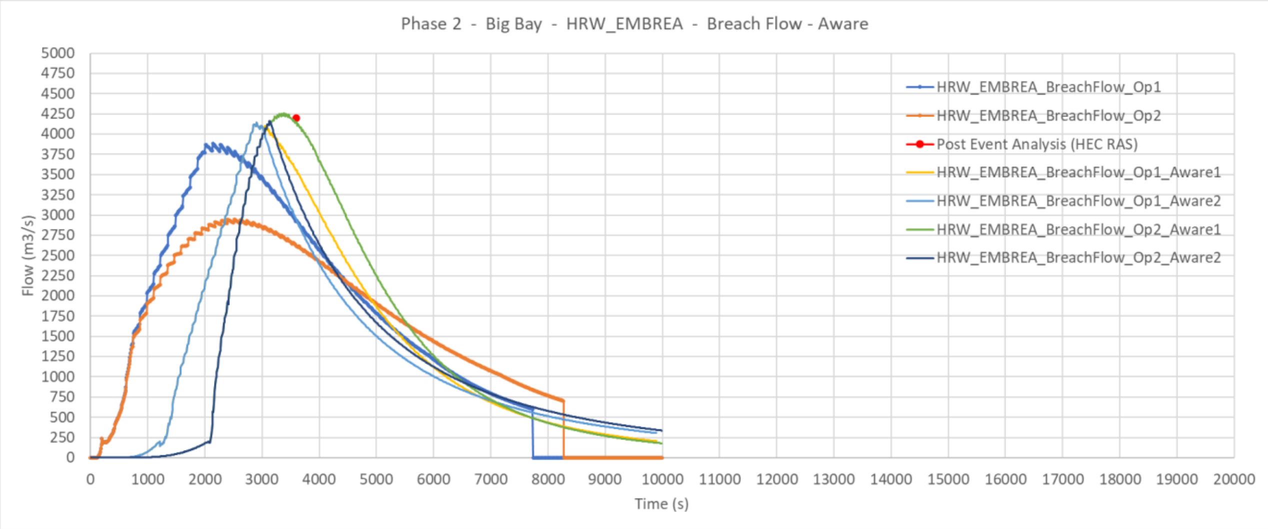


Figure H.12: Phase 2 – Big Bay: HRW aware modelling results using EMBREA

Run 'Option 2 Aware 1' appeared to offer the closest fit to observed, although all 4 additional runs were a significant improvement on the blind modelling results. The aware runs were undertaken by adapting the modelling parameters and approach as follows:

Op1Aware1:

- The observed breach depth was restricted to 14 m, therefore the breach depth was restricted to 14 m in this aware run
- Erodibility coefficient was also reduced to 10 cm^3/N.s for the piping part of the failure
- A variable K_d factor = 2 was also assumed for this case which means for the overtopping case of the failure K_d = 10*2 = 20 cm^3/N.s
- Friction coefficient was increased from 0.05 to 1.5 for the piping part of the failure mode and from 0.025 to 0.062 for the overtopping part
- Critical shear stress was increased from 1 to 10 Pa.

Op1Aware2:

- This run is identical to Run 1 (see above) but the breach width restricted to average observed value (i.e. 83.2 m). It was undertaken to check if restricting the breach width will have an impact on the breach peak outflow value and timing or not.

Op2Aware1:

For the blind run of this option, EMBREA estimated a breach peak outflow that is lower than the estimated peak outflow for this case (i.e. 2950 m^3/s). At time 3500 seconds, the estimated breach dimensions were 17.35 m and 73.81 m which are also slightly different than the observed ones which are 14 m and 83.2 m. Same changes that were made in the aware run of option did not change the results much. So again here, the breach depth was restricted to 14 m. A number of changes have also been made to the timing of the peak outflow value, namely:

- Erodibility coefficient was also reduced to 5 cm^3/N.s for the piping part of the failure. A variable K_d factor = 28 was also assumed for this case which means for the overtopping case of the failure K_d = 5*28 = 140 cm^3/N.s
- Friction coefficient was increased from 0.05 to 1.5 for the piping part of the failure mode and from 0.025 to 0.03 for the overtopping part
- Weir coefficient was increased from 1.7 to 2.2.

Op2Aware2:

- This run is identical to the Op2Aware1 run but the breach width restricted to average observed value (i.e. 83.2 m). It was undertaken to check if restricting the breach width will have an impact on the breach peak outflow value and timing or not.

INRAE – Rupro

Assessment of Rupro#1 performance increasing Manning's n to 0.067 instead of 0.05 – showed an improvement in Q_p estimation, but worsening of the timing.

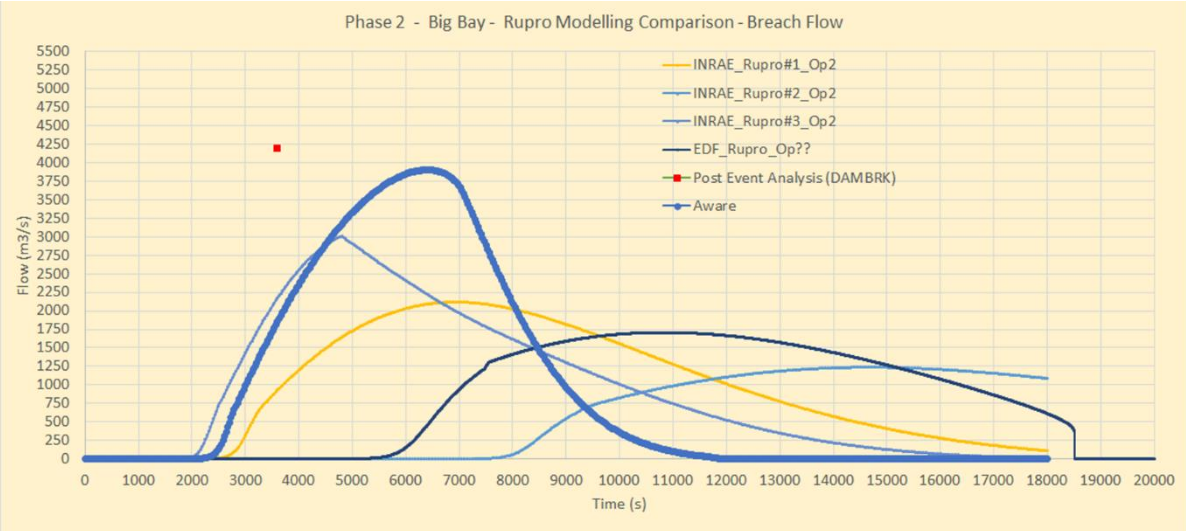


Figure H.13: Phase 2 – Big Bay: INRAE aware modelling results using Rupro

BUT – AREBA

Two tests were undertaken investigating the performance achieved by changing the erodibility value from 27 to 30 (Op 1) and 50 (Op 2) cm³/N.s.

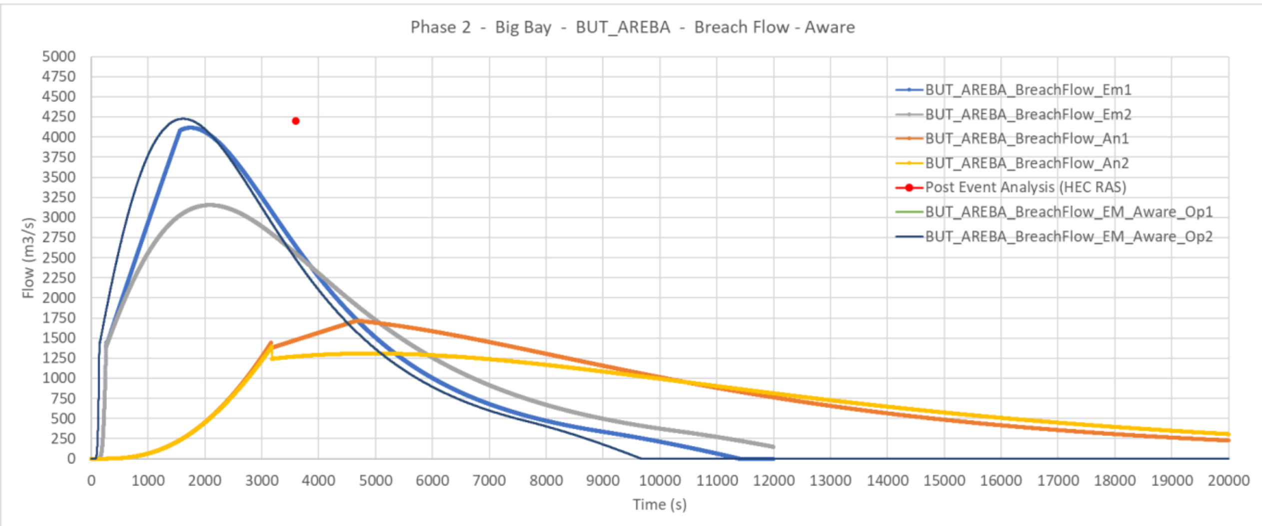


Figure H.14: Phase 2 – Big Bay: BUT aware modelling results using AREBA

BUT – DLBreach

Two tests were undertaken investigating the performance achieved by changing the erodibility value from 10 to 9 (Op 1) and 13.6 (Op 2) cm³/N.s.

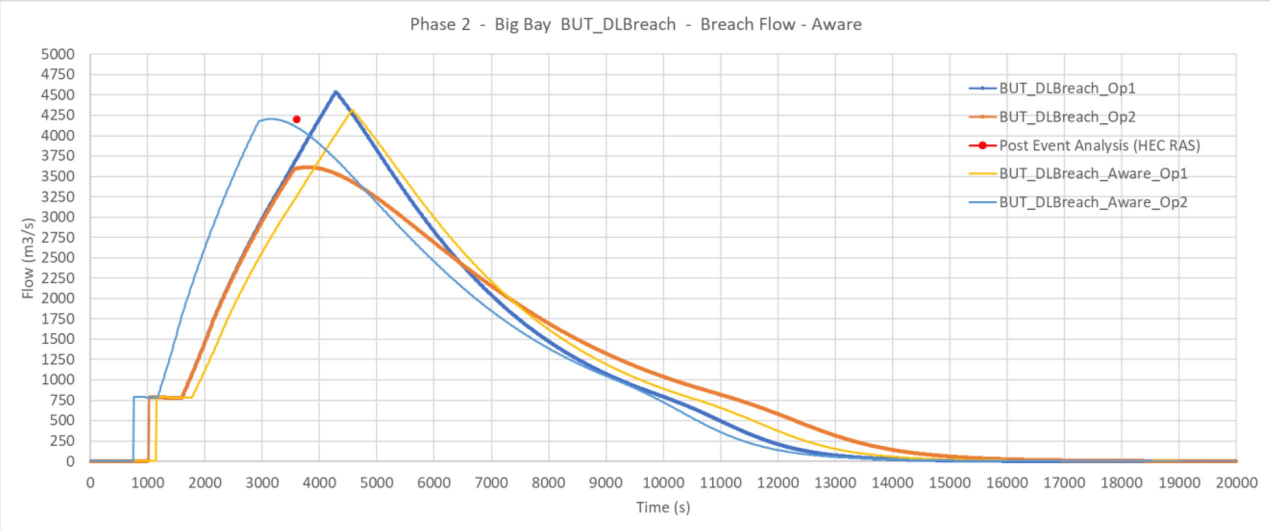


Figure H.15: Phase 2 – Big Bay: BUT aware modelling results using DLBreach

BUT – EMBREA

Two tests were undertaken investigating the performance achieved by changing the erodibility value from 27 to 73 (Op 1) and 140 (Op 2) cm³/N.s.

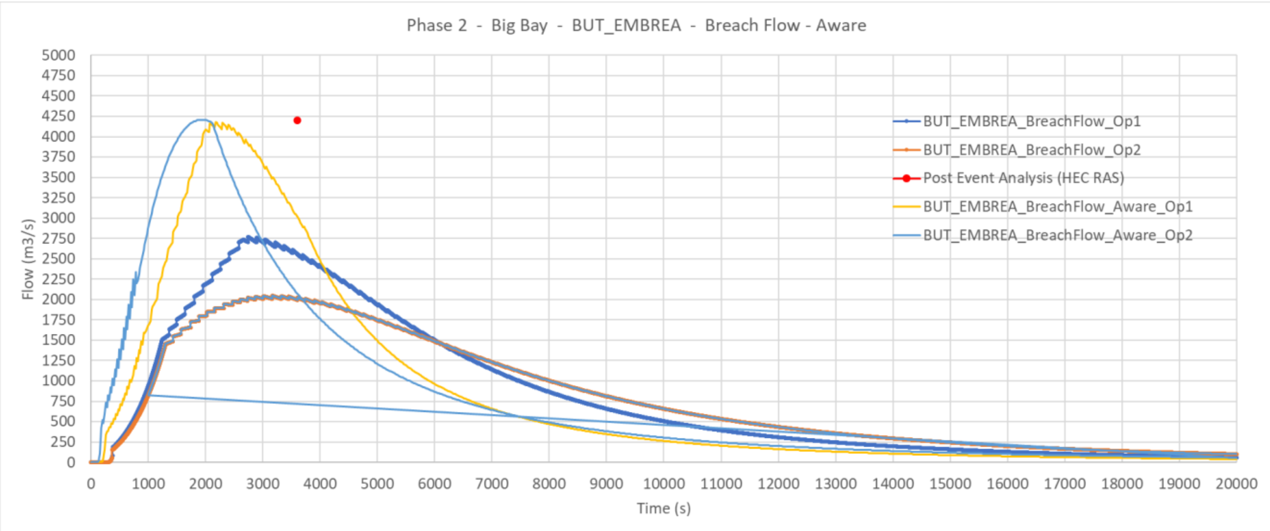


Figure H.16: Phase 2 – Big Bay: BUT aware modelling results using EMBREA

BUT – WinDAM C

Two tests were undertaken investigating the performance achieved by changing the erodibility value from 84 to 86 (Op 1) and 117 (Op 2) cm³/N.s.

Figure H.17: Phase 2 – Big Bay: BUT aware modelling results using WinDAM C

I Phase 3 – ARS P1 Test Case and Big Bay Dam Failure Uncertainty Data Specification

I.1 Introduction

This note provides an approach for quantifying the uncertainties associated with the USDA ARS P1 test input and validation data. The data in this note comes mainly from the following sources:

1. Blind and aware data provided to modellers as part of this project
2. Ali, A. K., Hunt, S., Tejral, R. D., 2021, Embankment Breach Research: Observed Internal Erosion Processes, Transactions of the ASABE. 64(2): 745-760. (DOI: 10.13031/trans.13701)
3. Hanson, G., Tejral, R., Hunt, S. and Temple, D., 2010, Internal Erosion and Impact of Erosion Resistance. 30th Annual USSD Conference, Sacramento, 12-16 April 2010, 773-784.

The Modellers Edition of this note contains only the information required to simulate the ARS P1 breaching process, without reference to the uncertainties that may exist in estimation of the 'observed' failure data. This is to encourage the modelling process to be as close as possible to a 'blind' simulation (whilst recognising that modellers have already analysed the test case deterministically).

I.2 Description of the USDA ARS P1 test

As previously provided in modelling Phase 2. See file: USDA-ARS-P1_Blind_v6.xlsx or USDA-ARS-P1_Aware_v2.xlsx.

I.3 Quantification of uncertainty in test case data:

To quantify the uncertainties associated with the test case data, the following was undertaken:

1. Relevant data was collected from the above-mentioned sources
2. An analysis was undertaken to assess the validity of the collected data amongst the above data sources with USDA ARS colleagues
3. Based on the above, a best estimate of data uncertainties was made and a recommended range for each input/output parameter suggested (see details below).

Breach outflow.

Information withheld from modellers edition.

Peak reservoir water levels.

Information withheld from modellers edition.

Final breach depth.

Information withheld from modellers edition.

Failure time.

Information withheld from modellers edition.

Volume of released water.

Information withheld from modellers edition.

I.4 Uncertainty in Modelling Input data

Various input data was provided for this test case, namely:

I.4.1 Embankment geometry data

a. Embankment height

Data Sources

Source	Data
Blind data	3.95 ft (approx. 1.2 m)
Aware data	
Ali et al. (2021)	1.3 m
Hanson et al. (2010)	1.3 m

Data Review

The 1.2 m reported in the blind and aware data seems to be the nominal value or due to difference in interpretation of survey. The 1.3 m value in the other two sources is from the fitted cross section.

Conclusions & Recommendations

1. Best estimate: height is 1.30 m
2. Recommended range: based on the cross section from 1.2 to 1.4 m.

b. Embankment crest width

Data Sources

Source	Data
Blind data	6.5 ft (approx. 1.98 m)
Aware data	
Ali et al. (2021)	1.8 m
Hanson et al. (2010)	1.8 m

Data Review

The 1.8 m reported in the Ali et al. (2021) and Hanson et al. (2010) seems to be the nominal value. The 1.98 m value in the aware data is from the fitted cross section.

Conclusions & Recommendations

1. Best estimate: crest width is 1.98 m
2. Recommended range: 1.98 m ± 0.2 m.

c. Embankment crest length

Data Sources

Source	Data
Blind data	32 ft (approx. 9.75 m)
Aware data	
Ali et al. (2021)	NA
Hanson et al. (2010)	NA

Data Review

The length reported in the blind and aware data is the only source for crest length.

Conclusions & Recommendations

1. Best estimate: crest length is 9.75 m

2. Recommended range: No range is considered since breach does not reach full embankment length and there was no flow over the crest.

d. Embankment upstream and downstream slopes

Data Sources

Source	Data
Blind data Aware data	3.22H:1V for the upstream slope and 2.95H:1V for the downstream slope
Ali et al. (2021)	3H:1V for both slopes
Hanson et al. (2010)	3H:1V for both slopes

Data Review

The 3H:1V reported in the Ali et al. (2021) and Hanson et al. (2010) seems to be the nominal value. The slope values in the blind and aware data are from the fitted cross section.

Conclusions & Recommendations

1. Best estimate: upstream and downstream slopes are 3.22H:1V and 2.95H:1V respectively
2. Recommended range: No range.

I.4.2 Embankment soil data

e. Soil type

Data Sources

Source	Data
Blind data Aware data	SM (silty sand) (Sand = 74% - Silt = 19% - Clay = 8% - AASHTO) - Note Sum is > 100
Ali et al. (2021)	Silty sand (Sand > 0.075 mm = 74% - Fines > 0.002 mm = 20% - Fines < 0.002 mm = 6%) Based on ASTM Standard D2487 (ASTM, 2000a)
Hanson et al. (2010)	SM (Silty sand - Sand > 0.075 mm = 64% - Fines > 0.002 mm = 29% - Fines < 0.002 mm = 7%) Based on ASTM Standard D 2487

Data Review

All sources agree on the soil type (i.e. Silty sand). Based on USDA ARS records, the data reported in Ali et al. (2021) were from 11 field samples taken, and they represent the median of those samples. Data in the blind and aware data is similar to that reported in Ali et al. (2021). Data in Hanson et al. (2010) differs in the percentages of sand and Fines > 0.002 but do not significantly differ for the percentage of Fines < 0.002.

Conclusions & Recommendations

1. Best estimate: Silty sand with 6-8% Fines < 0.002 (i.e. clay).

f. Grading size distribution (or simply D_{50} if data not available)

Data Sources

Source	Data
Blind data Aware data	D50 = 0.13 mm
Ali et al. (2021)	Not explicitly mentioned but D50 is > 0.075 mm (See above section)
Hanson et al. (2010)	Not explicitly mentioned but D50 is > 0.075 mm (See above section)

Data Review

Based on USDA ARS records, the 0.13 mm value reported in the blind and aware data represents the median D50 of the 11 field samples taken for this test.

Conclusions & Recommendations

1. Best estimate: D_{50} = 0.13 mm
2. Recommended range: D_{50} = 0.1-0.15 mm based on ±10% on percentage passing.

Potential Uses of this Data

This parameter can be used in breach model in several ways, namely:

- Estimate critical shear stress for coarse grained material (eg Shields number)
- Estimate erosion and/or sedimentation if an equilibrium sediment transport equation is used.

g. Plasticity index

Data Sources

Source	Data
Blind data	Non-Plastic
Aware data	Non-Plastic
Ali et al. (2021)	Non-Plastic Based on ASTM Standard D4318
Hanson et al. (2010)	Non-Plastic Based on ASTM Standard D4318

Data Review

All sources report a non-plastic soil.

Conclusions & Recommendations

1. Best estimate: non-plastic
2. Recommended range: none.

Potential Uses of this Data

This parameter can be used in breach model in several ways, namely:

- Estimate critical shear stress for fine grained material
- Estimate other soil properties as a proxy parameter.

h. Soil densities and/or unit weights (eg dry, unsaturated and saturated)

Data Sources

Source	Data
Blind data	Average dry density = 1.74 g/cm³ and average density = 1.9 g/cm³
Aware data	Average dry density = 1.74 g/cm³ and average density = 1.9 g/cm³
Ali et al. (2021)	The post-breach average dry bulk density was 1.79 g/cm³. No data on average density
Hanson et al. (2010)	Dry unit weight = 1.79 g/cm³ (Figure 4). No data on average density

Data Review

According to the USDA ARS records, 2 and 3 inch soil samples were taken during construction to measure the dry density. The average dry density of the blind and aware data represents the average of the 2 inch diameter samples taken during construction. The average of the 3 inch diameter samples taken during construction is 1.70 g/cm³. The average of the 2 and 3 inch diameter samples during construction is 1.70 g/cm³. Range of the construction samples was 1.6 to 1.77 g/cm³. Post breach samples ranged from 1.72 to 1.82 g/cm³ with an average of 1.79 g/cm³.

Conclusions & Recommendations

1. Best estimate: use during construction average which is 1.7 g/cm^3
2. Recommended range: use during construction range which is 1.6-1.82 g/cm^3.

Potential Uses of this Data

- Used if a model performs block and roof stability calculations
- Used to calculate the void ratio of the embankment.

i. Grain density

Data Sources

Source	Data
Blind data	2650 Kg/m^3
Aware data	
Ali et al. (2021)	No data
Hanson et al. (2010)	No data

Data Review

A typical range for grain density is 2600-2700 Kg/m^3. An average value was used in this case.

Conclusions & Recommendations

1. Best estimate: 2650 Kg/m^3
2. Recommended range: 2600-2700 Kg/m^3.

Potential Uses of this Data

- Not directly used (may guide the choice of other parameters).

j. Void ratio

Data Sources

Source	Data
Blind data	0.52
Aware data	1.89
Ali et al. (2021)	NA
Hanson et al. (2010)	NA

Data Review

Based on USDA ARS record, void ratio was calculated from grain density and dry unit weight, hence its uncertainty depends upon variations in those values. Based on a review of the USDA ARS data sheets, the aware data was incorrectly calculated, and the blind data reflects the correct information on porosity and void ratio values.

Conclusions & Recommendations

1. Best estimate: 0.52
2. Recommended range: 0.5-0.65 (based on ranges for dry unit weight and grain density).

Potential Uses of this Data

Void ratio and/or porosity can be used for:

- Calculating the volume of material transported if an equilibrium sediment transport equation is used
- Calculating the stability of soil from a block failure.

k. Porosity

Data Sources

Source	Data
Blind data	0.34
Aware data	0.65
Ali et al. (2021)	NA
Hanson et al. (2010)	NA

Data Review

Based on USDA ARS record, void ratio was calculated from grain density and dry unit weight, hence its uncertainty depends upon variations in those values. Based on a review of the USDA ARS data sheets, the aware data was incorrectly calculated, and the blind data reflects the correct information on porosity and void ratio values.

Conclusions & Recommendations

1. Best estimate: 0.34
2. Recommended range: 0.33-0.40 (based on the range for void ratio).

Potential Uses of this Data

- Not directly used (may guide the choice of other soil parameters).

l. Friction Angle

Data Sources

Source	Data
Blind data	32
Aware data	
Ali et al. (2021)	No data
Hanson et al. (2010)	No data

Data Review

According to the USDA ARS records, friction angle was not measured. Value in the aware data appears to be the average value of typical range from 30 to 34 of dense silty sand (see https://www.geotechdata.info/parameter/angle-of-friction).

Conclusions & Recommendations

1. Best estimate: 32
2. Recommended range: 30-34.

Potential Uses of this Data:

- Used if a model performs block or roof stability calculations.

m. Cohesion

Data Sources

Source	Data
Blind data	7 kPa
Aware data	
Ali et al. (2021)	No data
Hanson et al. (2010)	No data

Data Review

According to the USDA ARS records, cohesion was not measured. Value in the aware data appears to be average value for cohesion of silty fine sands with slight plasticity (ML), which is 7 kPa. Materials usually range from 4 to 9 kPa.

Conclusions & Recommendations

1. Best estimate: 7 kPa
2. Recommended range: 4-9 kPa.

Potential Uses of this Data:

- Used if a model performs block or roof stability calculations.

n. Jet Test Erodibility Coefficient

Data Sources

Source	Data
Blind data	120 cm^3/N.s
Aware data	
Ali et al. (2021)	The post-breach soil erodibility range from the in situ tests on the embankment and undisturbed samples was 23 cm^3/N.s to 270 cm^3/N.s
Hanson et al. (2010)	Greater than 100 cm^3 /N.s

Data Review

According to the USDA ARS records, The Blind and aware data value is the average of the post-breach soil samples whereas the soil erodibilities provided in Ali et al. (2021) represent the range of values from three undisturbed samples and four in-situ tests post-breach. No evidence of change with depth found in data or notes.

Conclusions & Recommendations

1. Best estimate: 120 cm^3/N.s
2. Recommended range: 23-270 cm^3/N.s.

Potential uses of this data:

- Used to estimate the eroded material if an erodibility equation is used
- Can be used to estimate the headcut migration coefficient if a model performs headcut erosion.

o. Critical shear stress

Data Sources

Source	Data
Blind data	0.144 Pa
Aware data	
Ali et al. (2021)	NA
Hanson et al. (2010)	NA

Data Review

According to the USDA ARS staff, several methods (eg Blaisdell, Scour Depth approach, etc) exist for arriving at the critical shear stress using the JET erosion methodology. Using the Blaisdell Method[1], critical shear stress was 0.16. For this test, USDA ARS staff discussed this with Greg Hanson who stated that it is correct that when you could not get a stable solution with highly erodible materials a constant critical shear stress value of 0.144 or 0.16 (Blasidell) or even zero can be used. The thought being that the k_d value was dominating the process in this situation, and the critical shear stress was playing a small part.

Conclusions & Recommendations

1. Best estimate: 0.144 Pa

[1] Blaisdell, F.W., Clayton, L.A., and Hebaus, G. G. 1981. Ultimate Dimension of Local Scour. Journal of Hydraulics Division, Vol. 107, No. 3, pp. 327-337.

2. Recommended range: 0 – 0.16 Pa.

Potential Uses of this Data:

- Used to estimate the excess shear stress for a number of erodibility and equilibrium sediment transport equations.

p. Unconfined compressive strength

Data Sources

Source	Data
Blind data	26 kPa
Aware data	26 kPa
Ali et al. (2021)	NA
Hanson et al. (2010)	NA

Data Review

According to the USDA ARS records, the reported value in the blind and aware data is the mean value of six samples. Samples ranged from 457 to 628 psf, which converts to 22 kPa to 30 kPa.

Conclusions & Recommendations

1. Best estimate: 26 kPa
2. Recommended range: 22-30 kPa.

q. Water content

Data Sources

Source	Data
Blind data	Average water content at construction = 11.2%
Aware data	Average water content at construction = 11.2%
Ali et al. (2021)	The post-breach average compacted moisture content was 10.3%. Figure 5 gives an optimum range from (approx.) 9-13%
Hanson et al. (2010)	Approx. 10%

Data Review

According to the USDA ARS records, the reported value in the blind and aware data is at construction and Ali et al. (2021) is post breach data. The reported value in Hanson et al. (2010) is in-situ data but it is not clear whether it was at or post construction.

Conclusions & Recommendations

1. Best estimate: 11.2 %
2. Recommended range: 9-13%.

It worth noting that the moisture content is not directly used by any models according to modelling group previous data.

Reservoir stage-volume curve

Data Sources

Source	Data
Blind data	See blind data excel sheet "Dam Levee Storage Description"
Aware data	See aware data excel sheet "Dam Levee Storage Description"
Ali et al. (2021)	No data but stated that "Reservoir stage-storage relationships were developed using a topographic survey of the reservoir storage volume for verification"
Hanson et al. (2010)	NA

Data Review

Based on USDA ARS records, traditional optical survey equipment was used to survey the reservoir basin to determine the storage volume. Stage was recorded using a data logger as well as by a traditional gauge well and point gauge. This redundancy of measurement was in case the data logger battery life gave out during testing.

As can be seen in Figure I.1, blind data goes up to a level 31.7 m with a corresponding volume of 1332 m^3, while, aware data stops at a level 31.39 m with a corresponding volume = 942 m^3. Curves are slightly different and not identical. It worth noting that the aware data does not cover the full range of water levels for the test since water level reached 31.44 m for this case.

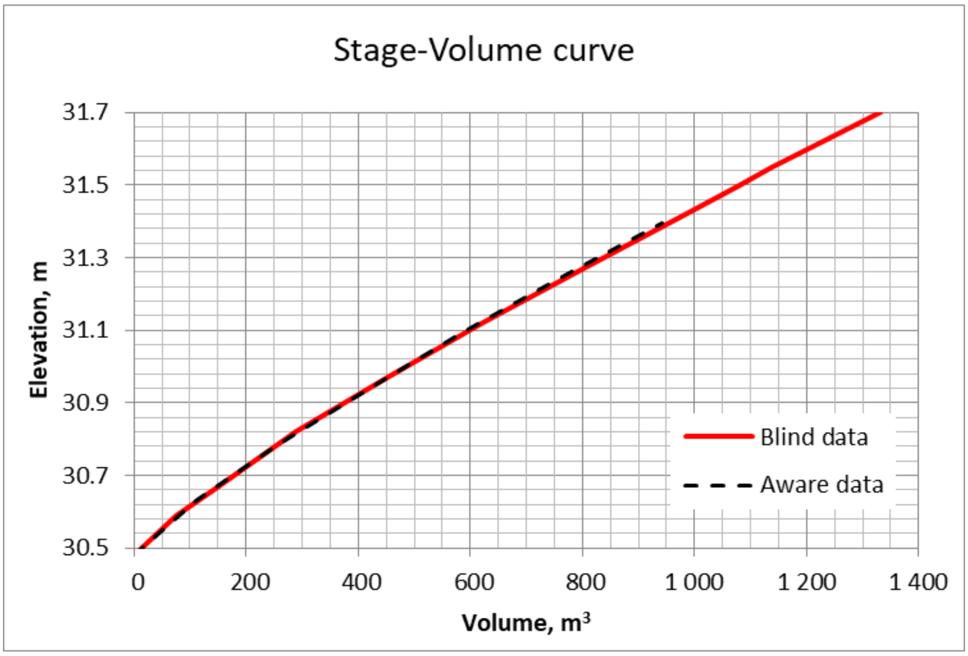


Figure I.1: Reservoir stage-volume curve for blind and aware data

Conclusions & Recommendations

1. Best estimate: use blind data
2. Recommended range: No range since potential uncertainty is less than 1%.

I.5 Inflow

Data Sources

Source	Data
Blind data	See blind data excel sheet “Initiating Conditions”
Aware data	See aware data excel sheet “Initiating Conditions”
Ali et al. (2021)	Figure 10a shows data for the first hour
Hanson et al. (2010)	Figure 8a shows data for the first hour

Data Review

As can be seen in Figure I.2, aware data differs from the other 3 sources for approx. 20 mins at the beginning of the hydrograph, then, it becomes identical to them. This was explained by USDA ARS staff as follows:

- The blind data set is the measured inflow at the sharp-crested weir
- The aware data set is the measured inflow at the sharp-crested weir along with taking into account flow over the long weir that was used to keep a constant head on the dam

- The aware data inflow would therefore be appropriate to use if the spillway (flow over wall) was ignored (no spillway)
- The blind data is consistent with the runs that were undertaken when developing Ali et al. (2021) paper.

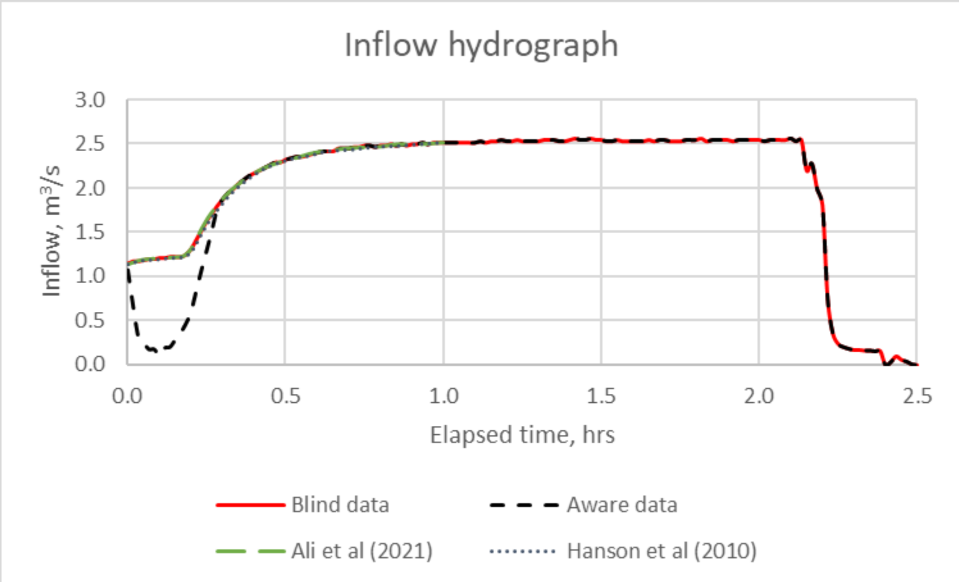


Figure I.2: Inflow data for the four data sources

Conclusions & Recommendations

1. Best estimate: use blind data
2. Recommended range: ±3% based on equipment accuracy and sharp crested weir formula standard error.

Initiating failure conditions

Data Sources

Source	Data
Blind data	40 mm diameter steel pipe at approx. 0.3 m above base
Aware data	40 mm diameter steel pipe at approx. 0.3 m above base
Ali et al. (2021)	40 mm diameter steel pipe at 0.4 m above base
Hanson et al. (2010)	NA

Data Review

Based upon USDA ARS review of data, it appears that there was a typo in the Ali et al. (2021) paper. Data appears to show that the pipe was at 0.32 m from base to pipe centre line (i.e. 0.3 m from base).

Conclusions & Recommendations

1. Best estimate: 40 mm diameter steel pipe at approx. 0.3 m above base
2. Recommended range: vary level by ±0.1 m.

Manning's n:

Data Sources

Source	Data
Blind data	NA
Aware data	

Source	Data
Ali et al. (2021)	
Hanson et al. (2010)	

Data Review

The modelling group previous estimates (based upon the case description) ranged from 0.009 to 0.033 but with all except one within the range 0.016-0.033. The latter range fits broadly with the range of values suggested by Chow (1959) in table 5-6 for excavated or dredged earth channels with no vegetation which is 0.016 to 0.030.

Conclusions & Recommendations

1. Best estimate: n= 0.025 (i.e. average of the recommended range)
2. Recommended range: 0.016-0.033.

Phase 3 - Big Bay Case Study Uncertainty Data Specification

Introduction

This note provides an approach for quantifying the uncertainties associated with the Big Bay test input and validation data. The data in this note comes mainly from the following sources:

1. Extracts from Burge, T. R.,2004. Big Bay Dam: Evaluation of failure, Land Partners Limited Partnership, Hattiesburg, Miss
2. Yochum, S.E., Goertz, L.A., and Jones, P.H., 2008. The Big Bay Dam Failure: Accuracy and Comparison of Breach Pre-dictions. ASCE Journal of Hydraulic Engineering, Vol. 134, No. 9, 1285-1293
3. Altinakar, M.S., McGrath, M.Z., Ramalingam, V.P. and Omari, H., 2010. 2D modeling of Big Bay dam failure in Mississippi: Comparison with field data and 1D model results. River Flow 2010 - Dittrich, Koll, Aberle & Geisenhainer (eds) - ISBN 978-3-939230-00-7
4. Ferguson, K.A., Anderson, S., & Sossenkina, E. (2014). Re-examination of the 2004 Failure of Big Bay Dam, Mississippi. USSD Annual Conference. San Francisco, CA: United States Society on Dams
5. Wahl, T., 2014. Evaluation of Erodibility-Based Embankment Dam Breach Equations, Hydraulic Laboratory Report HL-2014-02, U.S. Dept. of the Interior, Bureau of Reclamation
6. Macchione, F., Costabile, P., Costanzo, C., De Lorenzo, G., Razda, B., 2016. Dam breach modelling: influence on downstream water levels and a proposal of a physically based module for flood propagation software. Journal of Hydroinformatics, Volume 18, Issue 4.

The Modellers Edition of this note contains only the information required to simulate the Big Bay breaching process, without reference to the uncertainties that may exist in estimation of the 'observed' failure data. This is to encourage the modelling process to be as close as possible to a 'blind' simulation (whilst recognising that modellers have already analysed the test case deterministically).

I.6 Phase 3 – Big Bay Case Study Uncertainty Data Specification

Test case data as previously provided in modelling Phase 2. See file: Big_Bay_Aware_v2.xlsx.

I.7 Quantification of uncertainty in test case data:

To quantify the uncertainties associated with the test case data, the following was undertaken:

1. Relevant data was collected from the above-mentioned sources
2. An analysis was undertaken to assess the validity of the collected data amongst the above data sources
3. Based on the above, a best estimate of data uncertainties was made and a recommended range for each input/output parameter suggested (see details below).

Breach outflow.

Information withheld from modellers edition.

Peak reservoir water levels.

Information withheld from modellers edition.

Final breach depth.

Information withheld from modellers edition.

Failure time.

Information withheld from modellers edition.

Volume of released water.

Information withheld from modellers edition.

I.8 Uncertainty in Modelling Input data

Various input data was provided for this test case, namely:

I.8.1 Embankment geometry data

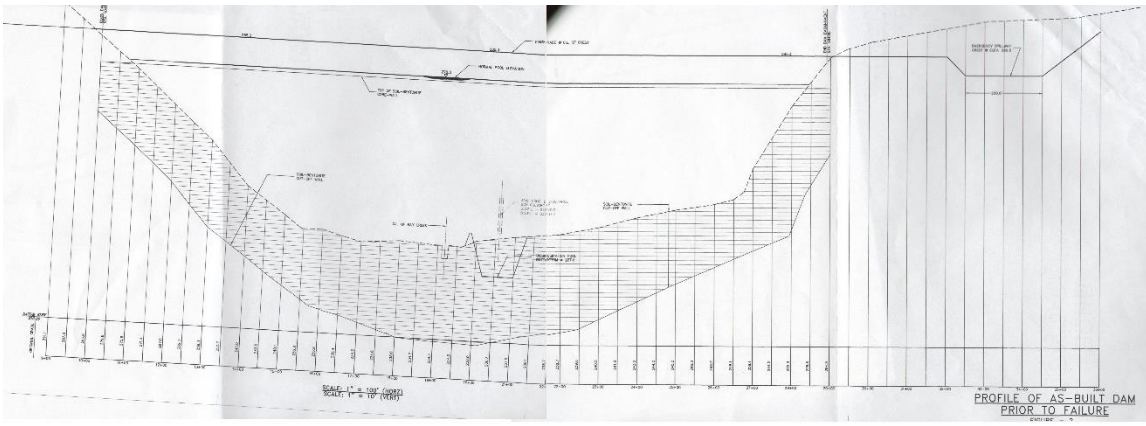
Dam height

Data Sources

Source	Data
Burge (2004)	Maximum height of the dam from the lowest natural ground elevation in the creek basin to the high crown point at the crest is 15.85 m (52 ft)
Yochum et al. (2008)	15.6 m (51.3 ft) high
Altinakar et al. (2010)	17.4 m (57.0 ft)
Ferguson et al. (2014)	18.3-21.3 m (60-70 ft)
Wahl (2014)	15.6 m
Macchione et al. (2016)	15.6 m (excluding the foundations)

Data Review

To establish the dam height, both dam crest and ground levels need to be also established. Based upon the below as built drawing of the dam those levels are 86.96 m (285.3 feet) and 71.4 m (234.3 feet) respectively. This gives a dam height of 15.56 m which is broadly consistent with what was mentioned in the Burge (2004), Yochum et al. (2008) Wahl (2014) and Macchione et al. (2016).



Conclusions & Recommendations

1. Best estimate: Dam height is 15.56 at the breach location
2. Recommended range: Allowing for undulations in crest assume 15.56 ±0.05 m.

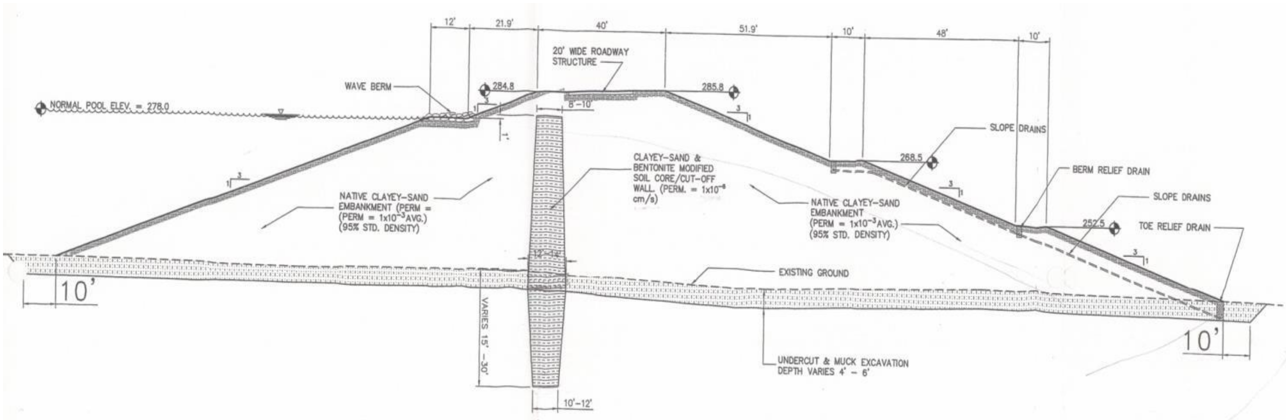
Dam crest width

Data Sources

Source	Data
Burge (2004)	The dam has a crest width of 12.2 m (40 ft)
Other sources	No data

Data Review

The crest width reported in Burge (2004) is consistent with the typical section through the dam shown below.



<u>Conclusions & Recommendations</u>

1. Best estimate: Dam crest width is 12.2 m
2. Recommended range: 12.2 ± 5%.

Dam crest length

<u>Data Sources</u>

Source	Data
Burge (2004)	The dam has an embankment length of 576 m (1890 feet) from the west abutment to the east abutment but appeared longer due to extension of the roadway crest by excavation into the east and west abutments
Yochum et al. (2008)	576 m (1890 feet)
Altinakar et al. (2010)	609.6 m (2000 feet)
Ferguson et al. (2014)	No data
Wahl (2014)	
Macchione et al. (2016)	576 m

<u>Data Review</u>

The length reported in Burge (2004) seems to be the right embankment length. Burge (2004) also explained why other authors reported a longer length.

<u>Conclusions & Recommendations</u>

1. Best estimate: Dam crest length is 576 m
2. Recommended range: No range is considered since breach does not reach full embankment length and there was no flow over the crest.

Dam upstream and downstream slopes

<u>Data Sources</u>

Source	Data
Burge (2004)	Upstream and downstream slopes of the dam are three horizontal to one vertical (3:1). The upstream slope contains a 12 foot wide rip-rap armoured wave berm at the normal pool elevation, and the back slope contains two 10 foot wide safety berms
Other sources	No data

Data Review

The dam upstream and downstream slopes in Burge (2004) is consistent with the above typical section through the dam.

Conclusions & Recommendations

1. Best estimate: Dam upstream and downstream slopes are three horizontal to one vertical (3:1)
2. Recommended range: Keep the crest width as defined – adjust d/s and u/s slopes to allow for berms. Maintain the soil volume. No range.

I.8.2 Embankment soil data

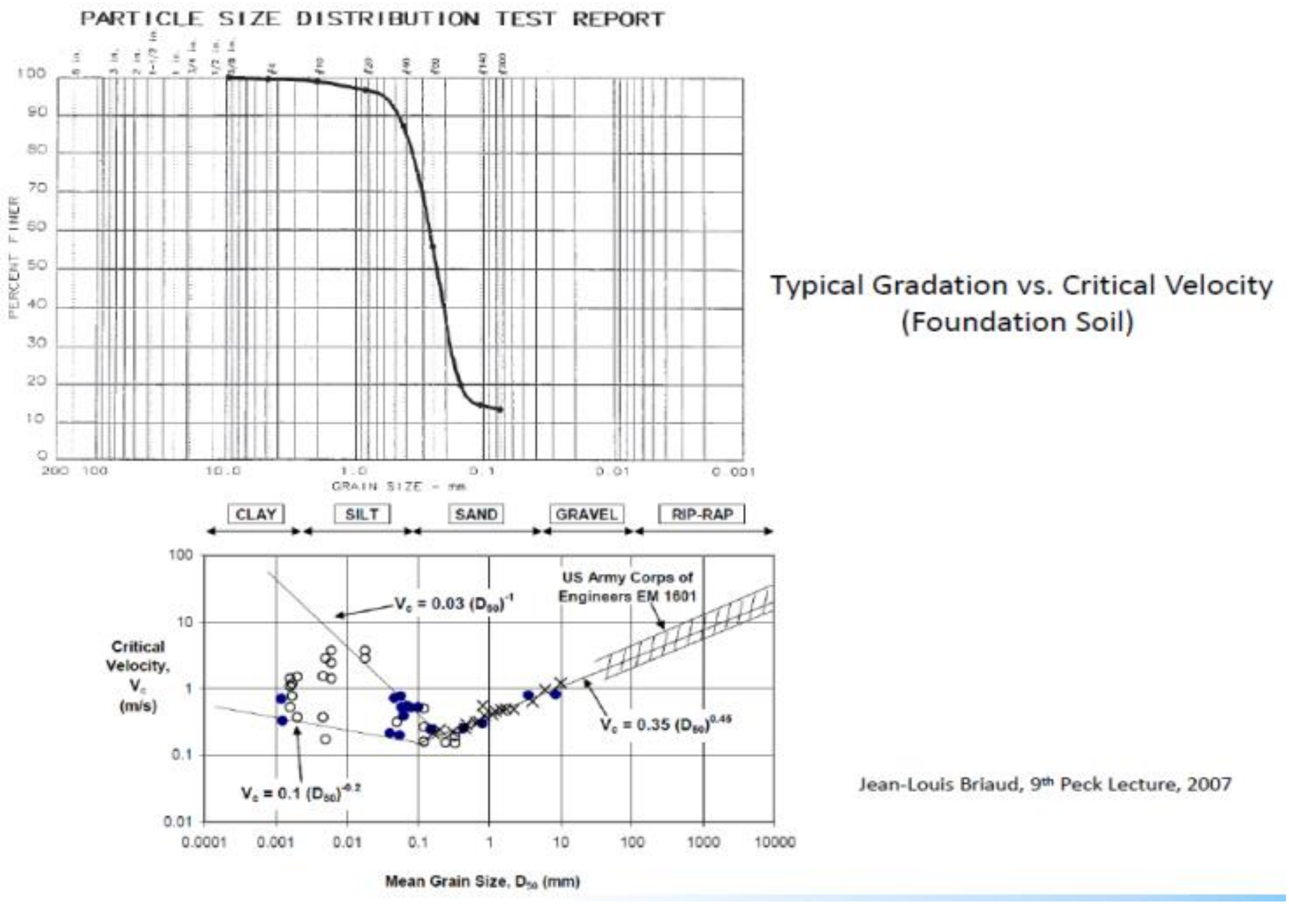
Soil type

Data Sources

Source	Data
Ferguson et al. (2014)	Both embankment and foundation soils at the site are comprised of primarily highly erodible silty sand and sandy silt materials
Wahl (2014)	Medium erodibility based on circumstantial evidence (i.e. available soil information), although if clay content was low and sand high, this could shift to high
Other sources	No data

Data Review

Not much information was mentioned on the soil type in the above papers/reports except by Ferguson et al. (2014) since their work focused on identifying the potential phases that the dam went through until full failure, and this requires knowledge on the soil type and properties. The designation of medium erodibility by Wahl (2014) was based mainly on the available soil permeability and compaction which are less than the information that was available to Ferguson et al. (2014). The comparison that was made by Ferguson et al. (2014) against the critical velocity required to initiate erosion along the bottom of canals and waterways as estimated by Jean-Luis Briaud shows that the materials at the site would classify as highly erodible as shown in the below figure.



Conclusions & Recommendations

1. Best estimate: Highly erodible silty sand and sandy silt materials, although Wahl suggests slightly less erodible than that
2. Refining the estimation: Assume K_d value in highly erodible Group 1, perhaps also tending towards Group 2. Also consider some uncertainty around the Group 1 K_d value – say x2 – leading to an overall estimated uncertainty range for K_d of 3 to 66 cm³/N.s. Closer consideration of Ferguson's paper shows some inconsistencies that may indicate samples with a larger clay content than the reference to 'silty' soils implies. Based upon this, apply a factor of 2 at the lower end, **hence a recommended range for K_d of 1.5 to 66 cm³/N.s**
3. Note that this suggested range also fits broadly with the modelling group previous estimates (based upon the soil description) which ranged from 5 to 84 but with all bar one within the range 5-50).

Table 1. Relationship of k_d, C_e, I, group number, and qualitative erosion resistance (after Hanson et al., 2010a).

k_d (cm³ N⁻¹ s⁻¹)	C_e (s m⁻¹)[a]	I (-log[C_e])[a]	Group Number[b]	Qualitative Description[b]
33	0.05 to 0.07	1.2 to 1.3	1	Extremely rapid
3.3	0.005 to 0.007	2.2 to 2.3	2	Very rapid
0.33	0.0005 to 0.0007	3.2 to 3.3	3	Moderately rapid
0.033	0.00005 to 0.00007	4.2 to 4.3	4	Moderately slow
0.0033	0.000005 to 0.000007	5.2 to 5.3	5	Very slow
0.00033	0.0000005 to 0.0000007	6.2 to 6.3	6	Extremely slow

[a] Based on a range of dry density (ρ) values from 1500 to 2000 kg m⁻³.
[b] Groupings and qualitative descriptions are based on Wan and Fell (2004).

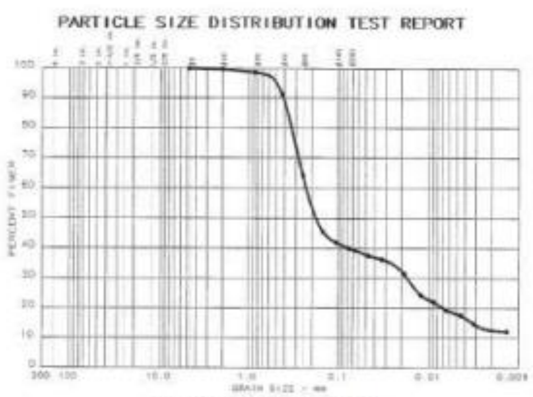
Grading size distribution (or simply D50 if data not available)

Data Sources

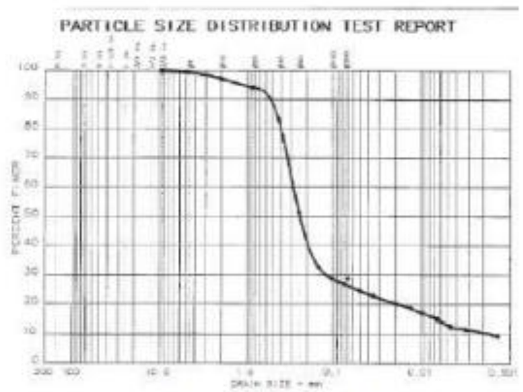
Source	Data
Ferguson et al. (2014)	The gradation in the above figure represents the foundation soils and may also represent some if not a large portion of the soils used to construct the embankment dam upstream and downstream of the cutoff wall and may also represent materials used for the cutoff wall prior to the addition of bentonite. In general, the soils at the site are relatively poorly graded fine sand with some silt and clay fraction along with minor amounts of fine gravel. Over 75 to 80 percent of the material consists of sand between 0.1 and 0.6 mm in size
Other sources	No data

Data Review

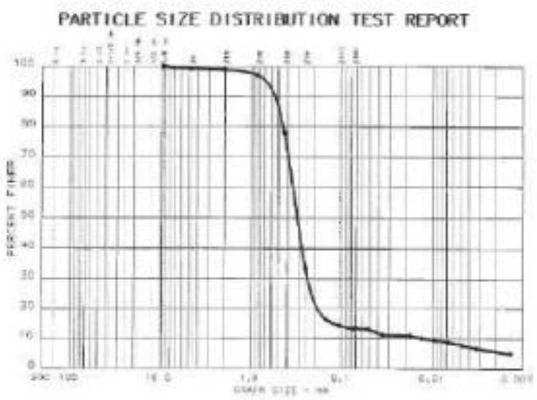
In addition to the above typical gradation curve, Ferguson et al. (2014) included in their paper gradings from post failure forensic borings as shown below.



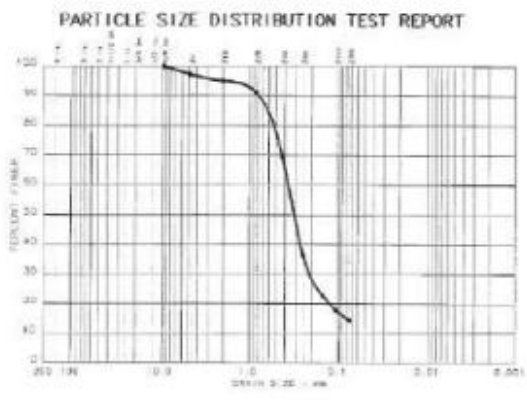
B-1 43–44.5' Sample 23,
PI=20, LL=35



B-2 49-50' Sample 25,
PI=16, LL=27

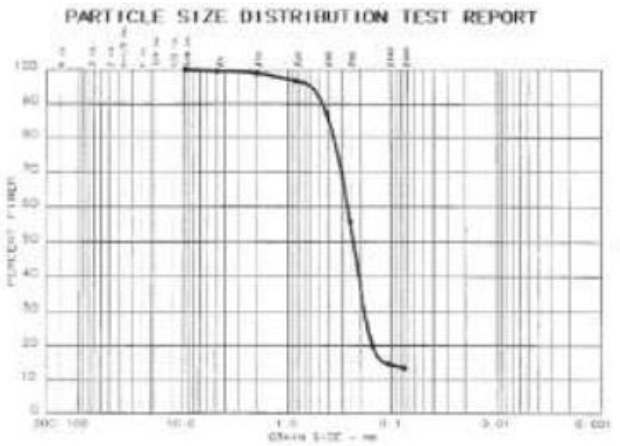


B-1 50-52' Sample 27, NP



B-2 65–66.5' Sample 33, NP

The top two gradings are for the cutoff wall and the two below are for the cohesive soils that are at a depth below the cutoff wall. The foundation and cutoff gradings show a D_{50} between 0.2-0.3 mm. They also contain clay content that ranges between 0-10 %.



In addition to representing the foundation soils, it is suspected that this gradation may also represent some if not a large portion of the soils used to construct the embankment dam upstream and downstream of the cutoff wall and may also represent materials used for the cutoff wall prior to the addition of bentonite.

Conclusions & Recommendations

1. Best estimate: D_{50} = 0.2-0.3 mm
2. Recommended range: D_{50} = 0.1-0.6 mm.

Assume soil is homogeneous

Potential Uses of this Data

This parameter can be used in breach model in various ways, namely:

- Estimate critical shear stress for coarse grained material (eg Shields number)
- Estimate erosion and/or sedimentation if an equilibrium sediment transport equation is used.

Plasticity index

Data Sources

Source	Data
Ferguson et al. (2014)	The gradation in the above figures show a plasticity index that ranges from 16-20 and a liquid limit that ranges from 27-35 for the cutoff material
Other sources	No data

Data Review

The plasticity index is probably on the high side for a soil that is predominantly coarse grained. One reason for this could be the addition of bentonite to the cut-off wall. For embankment soils and since it has more than 70-80 % coarse soils, it can be considered of low or no plasticity.

Conclusions & Recommendations

1. Cutoff wall
   a. Best estimate: cutoff wall plasticity index = 18
   b. Recommended range: cutoff wall plasticity index = 16-20.
2. Embankment Plasticity
   a. Best estimate: Embankment plasticity index < 7
   b. Recommended range: Embankment plasticity index = 0-7.

Potential Uses of this Data

This parameter can be used in breach model in various ways, namely:

- Estimate critical shear stress for fine grained material
- Estimate other soil properties as a proxy parameter.

Void ratio

Data Sources

Source	Data
Ferguson et al. (2014)	Based on table 1, initial void ratio was estimated to be 0.34 - 0.448 for the cutoff wall, 0.307- 0.537 for the foundation immediately below the cutoff wall and 0.628-0.682 for the cohesive soils that are further below the cutoff wall
Other sources	No data

Data Review

Values of void ratio seems reasonable for a predominantly coarse-grained soil.

Conclusions & Recommendations

1. Cutoff Wall
   a. Best estimate: cutoff wall void ratio = 0.394 (range average)
   b. Recommended range: cutoff wall void ratio = 0.34 - 0.448.
1. Embankment Void ratio
   a. Best estimate: Embankment void ratio = 0.422 (range average)
   b. Recommended range: Embankment void ratio = 0.307- 0.537.

Potential Uses of this Data

Void ratio and/or porosity can be used for:

- Calculating the volume of material transported if an equilibrium sediment transport equation
- Calculating the stability of soil from a block failure.

Permeability

Data Sources

Source	Data
Ferguson et al. (2014)	Based on table 1, permeability was found to be 5.8x10-4 - 1.5x10-7 cm/s for the cutoff wall and 3.2x10-3 - 3x10-5 cm/s for the foundation immediately below the cutoff wall
Burge (2004)	Typical dam section (see above) shows a permeability of 1x10-3 cm/s for the embankment
Other sources	No data

Data Review

Values of permeability seems reasonable for a predominantly coarse-grained soil.

Conclusions & Recommendations

1. Cutoff wall
   a. Best estimate: cutoff wall permeability = 5.8×10^{-4} cm/s (range maximum)
   b. Recommended range: cutoff wall permeability = 5.8×10^{-4} - 1.5×10^{-7} cm/s.
1. Embankment Permeability
   a. Best estimate: Embankment permeability = 1×10^{-3} cm/s

b. Recommended range: Embankment permeability = 3.2×10^{-3} – 3×10^{-5} cm/s.

Potential Uses of this Data

- Not directly used (maybe guides on choice of other parameters).

The following soil properties may also be needed to undertake the modelling, but no data were found for them in the above sources. Typical or estimated values will need to be established for them.

Grain density

Typical range 2600-2700 Kg/m^3.

Potential Uses of this Data:

- Not directly used (maybe guides on choice of other parameters such as the Sediment Specific Gravity).

Unconfined compressive strength

Based on modelling summary sheets for this case, it is not used by any models according to modelling group previous data.

Soil densities and/or unit weights (eg dry, unsaturated and saturated)

1. Typical values (from EMBREA guidance) are:

Table 2: Porosity, and unit weights of typical soils (Terzaghi et al 1996).

Description	Porosity (%)	Dry Unit Weight (kN/m3)	Saturated Unit Weight(kN/m3)
Uniform Sand, loose.	46	14	18.5
Uniform Sand, dense.	34	17.2	20.5
Mixed Sand, loose.	40	15.6	19.5
Mixed Sand, dense.	30	18.2	21.2
Glacial till, mixed.	20	20.8	22.7
Soft Glacial Clay.	55	12.0	17.4
Stiff Glacial Clay.	37	16.7	20.3
Soft slightly organic Clay.	66	9.1	15.5
Soft very organic Clay.	75	6.7	14.0
Soft bentonite.	84	4.2	12.5

2. Dry unit weight range: 18.2 – 20.8 → 18 – 21.

Note the difference between soil density or grain density. EMBREA model, for example, uses dry soil density, but other models may use grain density or other types of density. This needs to be considered when using different models.

Potential Uses of this Data

- Used if a model performs block and roof stability calculations.

Water content

Not used by any models according to modelling group previous data.

Friction Angle

For example values see table at: https://civilengineeringbible.com/subtopics.php?i=89.

Recommended Range = 30-34.

Potential Uses of this Data

- Used if a model performs block or roof stability calculations.

Cohesion

Recommended Range = 5-15 kPa (based on judgement and see table at: https://civilengineeringbible.com/subtopics.php?i=91)

Potential Uses of this Data:

- Used if a model performs block and roof stability calculations.

Jet Test Erodibility Coefficient

See earlier notes – conclusions are:

Recommended Range = **K_d values to range from 1.5 to 66 cm^3/N.s**

Potential Uses of this Data:

- Used to estimate the eroded material using an erodibility equation
- Can be used to estimate the headcut migration coefficient if a model performs headcut erosion.

HET Erodibility Index (if available)

As discussed above – Group 1 > I < 2.

Table 1. Relationship of k_d, C_e, I, group number, and qualitative erosion resistance (after Hanson et al., 2010a).

k_d ($cm^3 N^{-1} s^{-1}$)	C_e ($s\ m^{-1}$)[a]	I ($-\log[C_e]$)[a]	Group Number[b]	Qualitative Description[b]
33	0.05 to 0.07	1.2 to 1.3	1	Extremely rapid
3.3	0.005 to 0.007	2.2 to 2.3	2	Very rapid
0.33	0.0005 to 0.0007	3.2 to 3.3	3	Moderately rapid
0.033	0.00005 to 0.00007	4.2 to 4.3	4	Moderately slow
0.0033	0.000005 to 0.000007	5.2 to 5.3	5	Very slow
0.00033	0.0000005 to 0.0000007	6.2 to 6.3	6	Extremely slow

[a] Based on a range of dry density (ρ) values from 1500 to 2000 kg m^{-3}.
[b] Groupings and qualitative descriptions are based on Wan and Fell (2004).

Table 2: Representative Erosion Rate Index for Non-Dispersive Soils (ICOLD Bulletin 164)

Unified soil classification	Representative erosion rate index (IHET) Likely minimum	Best estimate	Likely maximum
SM with <30% fines	1	<2	2.5
SM with >30% fines	<2	2 to 3	3.5
SC with <30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with liquid limit <65%	3	4	5
CH with liquid limit <65%	4	5	6

Recommended range: SM < 30% fines = 1 – 2.5 range IHET.

Potential Uses of this Data:

- Can be used to estimate the critical shear stress
- Can be used to estimate the material erodibility.

Critical shear stress

Table 4: Limits and best estimates for a triangular distribution of critical shear stress for different soil types

I_{HET}	Critical shear stress limits for triangular distribution (Pa)		
	Low	Best	High
Up to 3	1	2	5
3.5	2	5	25
4	5	25	60
5	25	60	100

1. Recommended range: For IHET 1-2.5, then range = 1 – 5 Pa.

Potential Uses of this Data:

- Used to estimate the excess shear stress for a number of erodibility and equilibrium sediment transport equations.

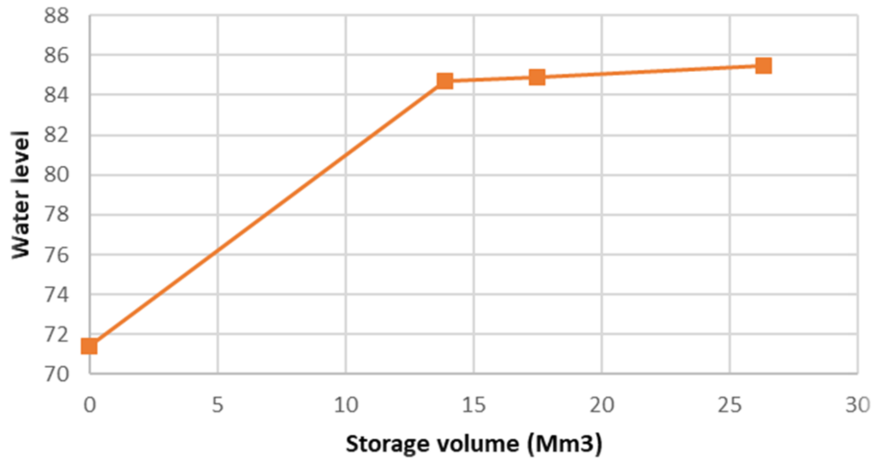
I.8.3 Reservoir stage-volume curve

Data Sources

Source	Data
Burge (2004)	No relevant data
Yochum et al. (2008)	17,500,000 m^3 (14200 acre-ft) - This seems to be the capacity at the water level at failure (See above for details)
Altinakar et al. (2010)	Maximum Storage 26,365,674 m^3 (21375 acre-ft) which can be assumed to be at the spillway level (i.e. 85.50 m.a.s.l.) Normal Storage 13,876,670.7 m^3 (11250 acre-ft) - This seems to be the capacity at the normal pool level (i.e. 84.73 m.a.s.l.))
Ferguson et al. (2014)	13876670 m^3 (11250 acre-ft) - This seems to be the capacity at the normal pool level (i.e. 84.73 m m.a.s.l.)
Wahl (2014)	17,500,000 m^3 (14200 acre-ft) - This seems to be the capacity at the water level at failure (See above for details)
Macchione et al. (2016)	17,500,000 m^3

Data Review

The above information provides a broad picture of the stage-volume curve of the reservoir as shown below.



Conclusions & Recommendations

1. Best estimate: Use Option 1 from the blind test data as adjusted in the below table:

Elevation (m)	Volume (Mm3)
71.40	0.00
84.73	13.88
84.89	17.50
85.50	26.37

2. Recommended range: Use best estimate data and consider +0% - -5%.

I.8.4 Initiating failure conditions

Data Sources

Source	Data
Yochum et al. (2008)	Elevation of 72.6 m.a.s.l. (238 ft) just above the toe slope elevation and similar in elevation to the bottom of the discharge box
Ferguson et al. (2014)	Defects above and below the conduit
Other sources	No relevant data

Data Review

The description of the failure provided by Ferguson et al. (2014) pointed to failure above and below the conduit. This means that the flow path may have been close to the base of the embankment (i.e. around a level of 71.4 m.a.s.l. as well as and above the conduit).

Conclusions & Recommendations

1. Best estimate: 71.4 m.a.s.l.
2. Recommended range: 71.4-73.7 m.a.s.l.

I.8.5 Manning's n

Data Sources

Source	Data
All sources	NA for the embankment dam material

Data Review

The modelling group previous estimates (based upon the case description) ranged from 0.016 to 0.035. The latter range fits broadly with the range of values suggested by Chow (1959) in table 5-6 for excavated or dredged earth channels with no vegetation which is 0.016 to 0.030.

Conclusions & Recommendations

1. Best estimate: n = 0.025 (i.e. approx. average of the recommended range)
2. Recommended range: 0.016-0.035.

J Phase 3 – HRW P1 and Big Bay Modelling Results

HRW undertook Monte Carlo (MC) analyses for a number of different parameter combinations:

- Geometric parameters
- Soil parameters
- K_d and Manning's n
- Pipe level
- All parameters excluding K_d and n
- All parameters.

For each set of MC runs:

- Results distributions for Q_p, B_w, T_{pc} were plotted
- Performance parameters PR1, PR2 and PR3 were calculated and best run parameters and results identified
- Range plots produced covering:
  - Q_p versus T_p
  - Q_p versus B_w
  - B_w ranges
  - T_{pc} ranges.

J.1 MC analyses for ARS P1

J.1.1 Varying geometric parameters

Table J.1: Phase 3: ARS P1 – HRW: MC analysis results for varying geometric parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2.707	1693	6.8	1000.9	0.74	1.30	0.93	1.30
Max	3.372	5338	7.8	4640.0	0.77	1.39	0.99	1.39
Min	2.309	1092	5.8	354.0	0.73	1.21	0.87	1.21
Best Run (Pr1) 157	3.019	1561	6.71	859	0.73	1.28	0.91	1.28
Best Run (Pr2) 108	2.381	1108	6.78	379	0.73	1.24	0.90	1.24
Best Run (Pr3) 300	2.995	1540	6.72	848	0.73	1.30	0.92	1.30
mode	2.401	1116	6.75	411	0.73	1.282	0.907	1.282
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**

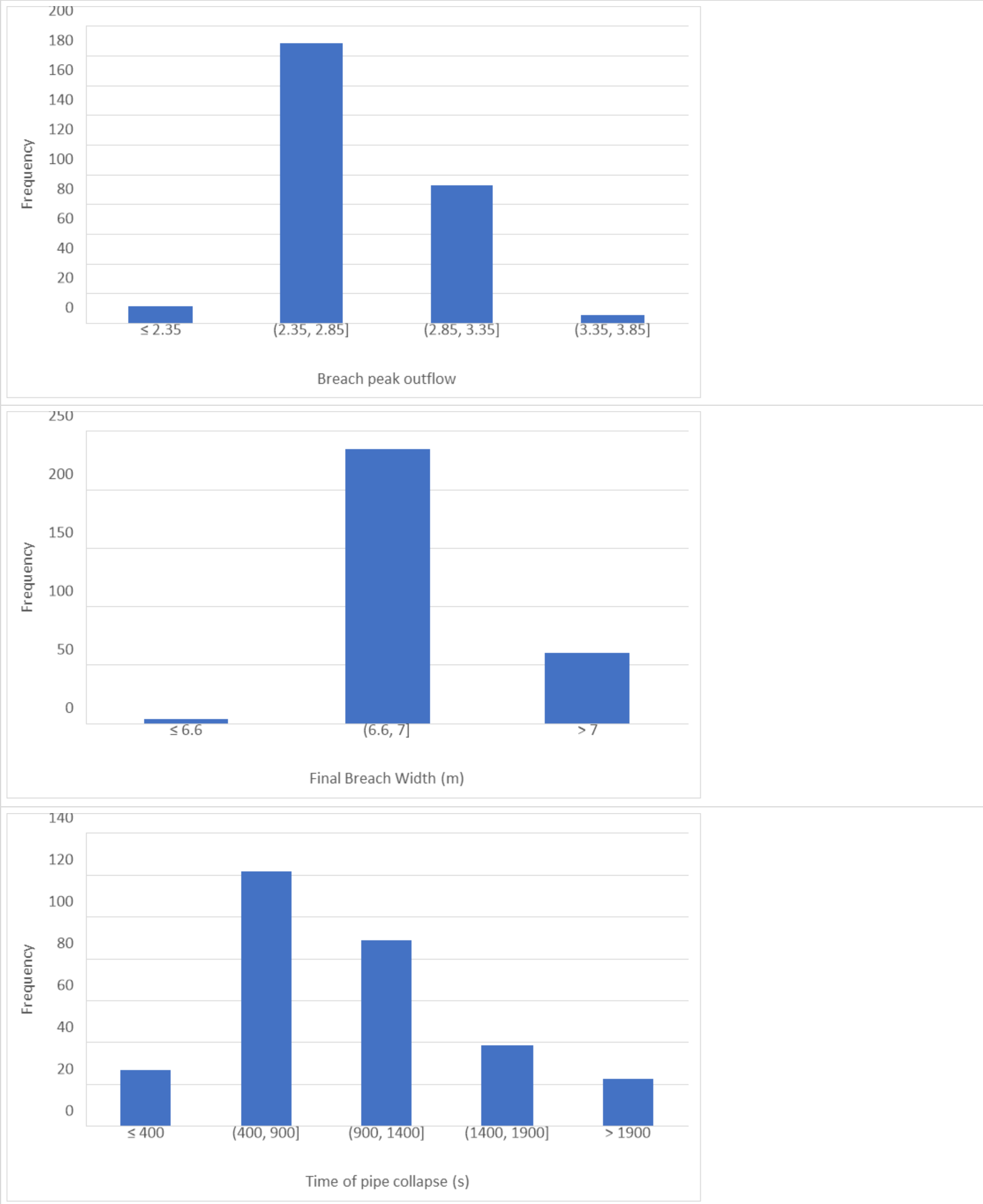
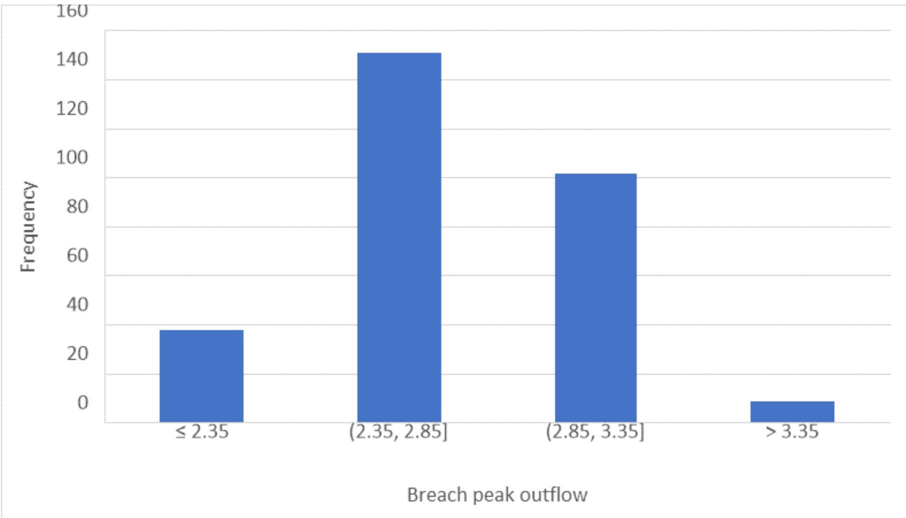


Figure J.1: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying geometric parameters

J.1.2 Varying soil parameters

Table J.2: Phase 3: ARS P1 – HRW: MC analysis results for varying soil parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2.723	1596	6.8	779.4	0.74	1.30	0.93	1.30
Max	3.516	4124	9.5	1915.0	0.76	1.31	0.94	1.31
Min	2.251	515	3.8	259.0	0.73	1.30	0.92	1.30
Best Run (Pr1) 37	3.083	1586	6.47	842	0.74	1.30	0.92	1.30
Best Run (Pr2) 287	2.491	1518	7.53	381	0.73	1.30	0.92	1.30
Best Run (Pr3) 180	2.989	1570	5.62	417	0.73	1.30	0.92	1.30
mode	3.007	1339	5.36	388	0.73	1.301	0.92	1.301
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**



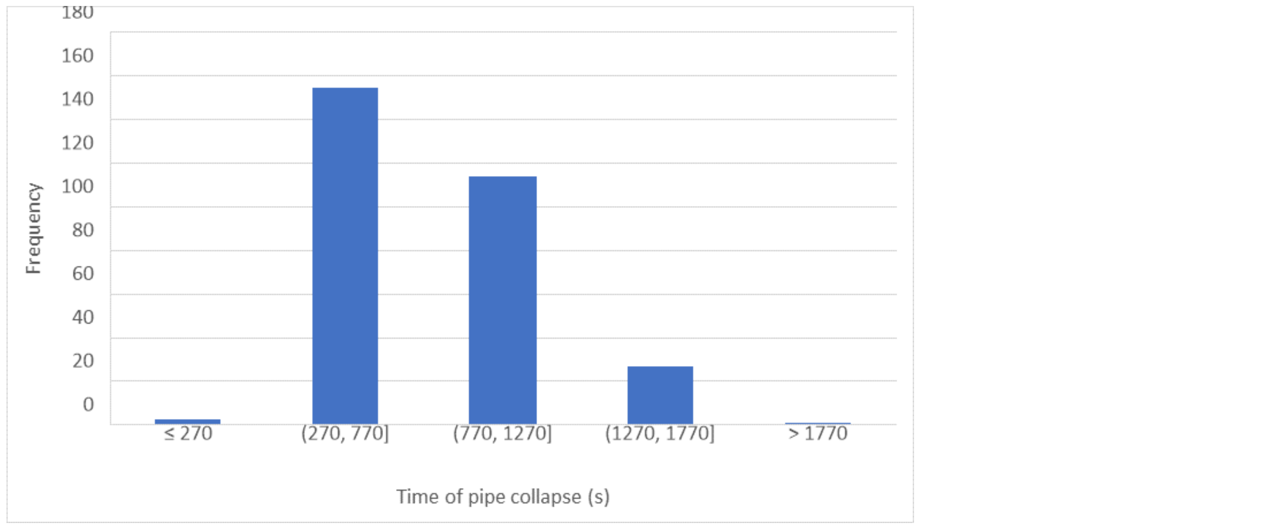
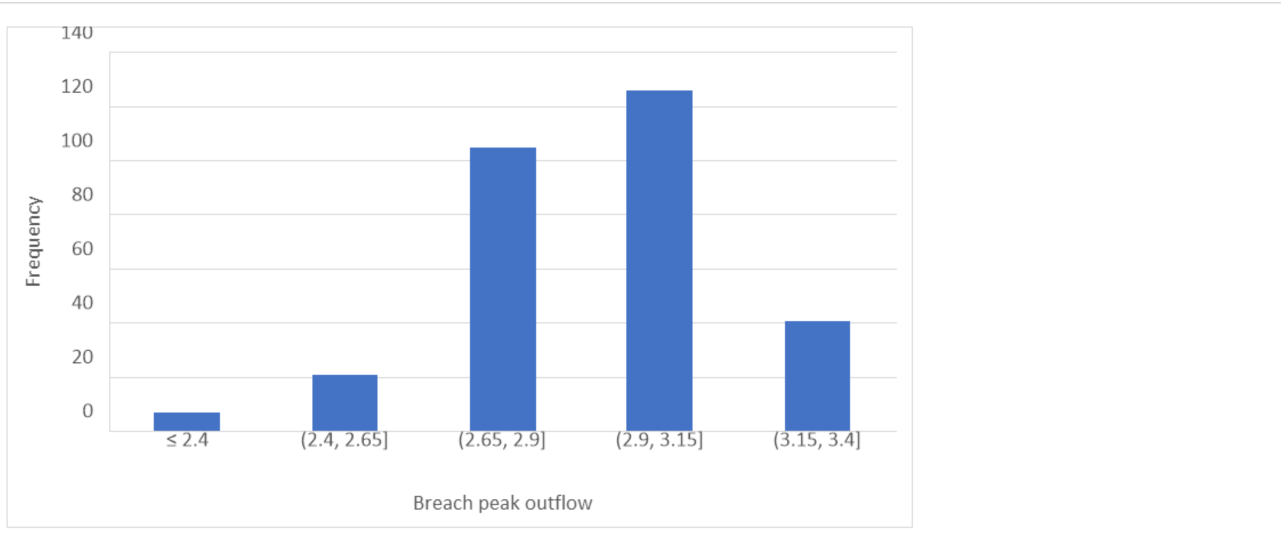


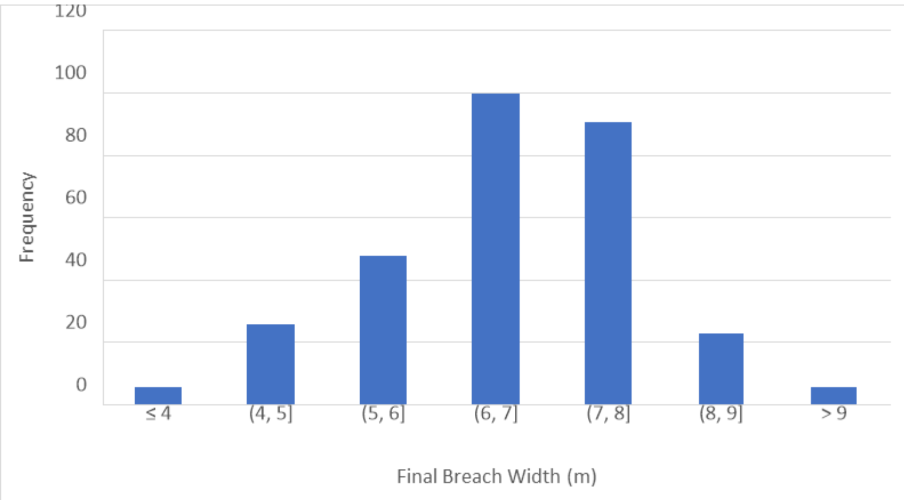
Figure J.2: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying soil parameters

J.1.3 Varying K_d and Manning's n parameters

Table J.3: Phase 3: ARS P1 – HRW: MC analysis results for varying K_d and Manning's n parameters

MC Runs	Flow	Time	Breach width	ime of Pip	BDBC	BWBC	BDAC	BWAC
Average	2.918	1621	6.7	892.8	0.74	1.30	0.92	1.30
Max	3.318	4013	9.2	1894.0	0.75	1.31	0.94	1.31
Min	2.251	914	3.8	669.0	0.73	1.30	0.92	1.30
Best Run (Pr1) 219	2.974	1536	6.64	843	0.73	1.30	0.92	1.30
Best Run (Pr2) 127	3.152	3598	4.09	1875	0.75	1.30	0.94	1.30
Best Run (Pr3) 22	3.062	1630	6.88	1010	0.74	1.30	0.93	1.30
mode	3.133	1242	7.76	837	0.735	1.301	0.924	1.301
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**





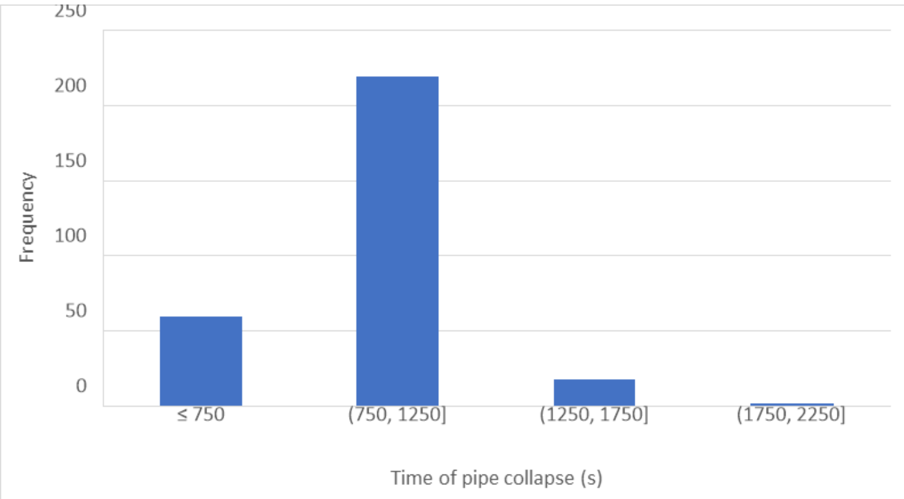


Figure J.3: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying K_d and Manning's n parameters

J.1.4 Varying pipe level parameter

Table J.4: Phase 3: ARS P1 – HRW: MC analysis results for varying pipe level parameter

MC Run2	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2.781	1678	6.6	996.7	0.74	1.30	0.93	1.30
Max	3.182	5331	6.8	4662.0	0.76	1.30	0.94	1.30
Min	2.293	1052	4.8	370.0	0.73	1.30	0.92	1.30
Best Run (Pr1) 225	3.028	1591	6.69	894	0.74	1.30	0.92	1.30
Best Run (Pr2) 229	2.325	1073	6.81	388	0.73	1.30	0.92	1.30
Best Run (Pr3) 225	3.028	1591	6.69	894	0.74	1.30	0.92	1.30
mode	3.112	1105	6.76	419	0.732	1.301	0.921	1.301
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**

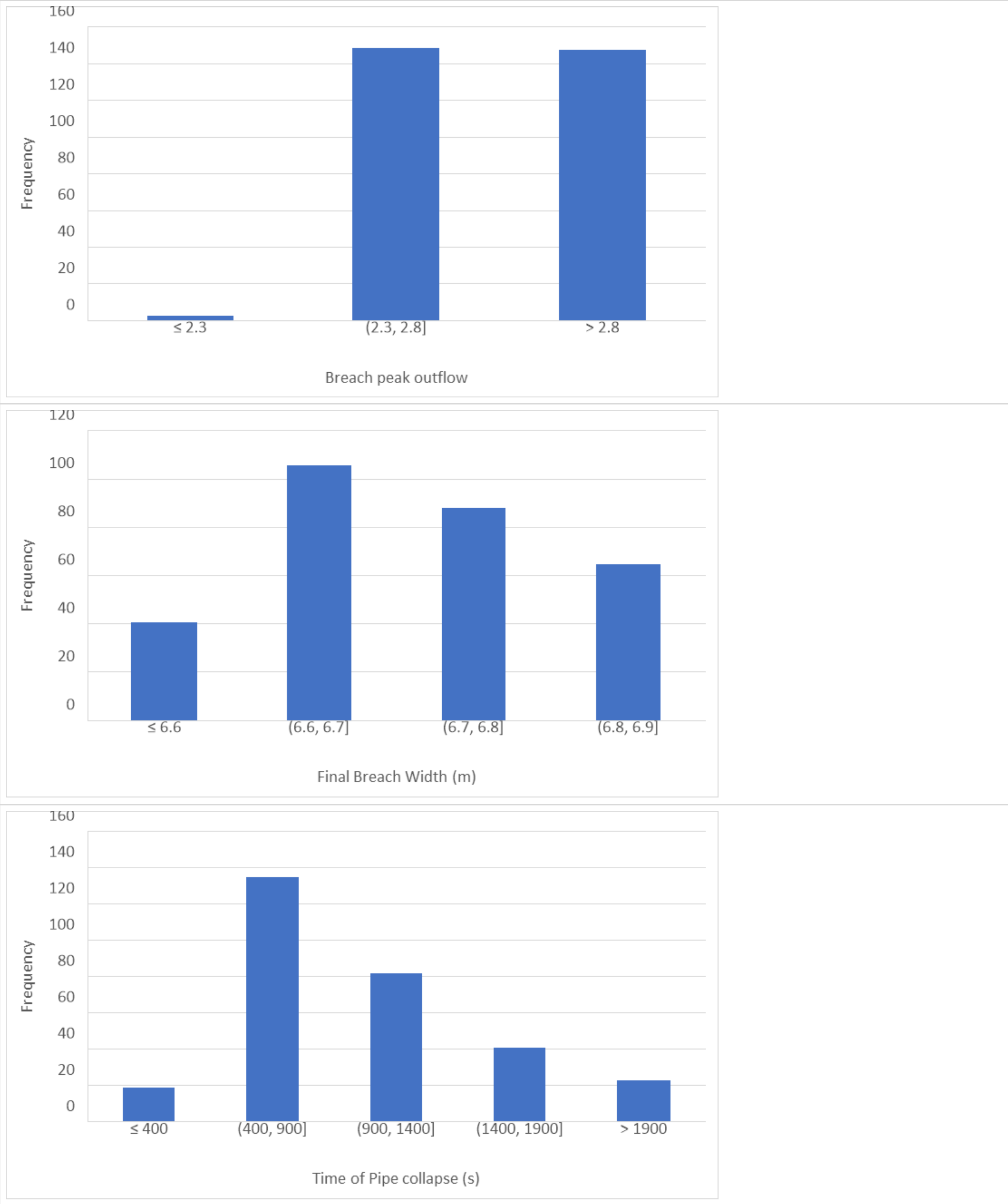
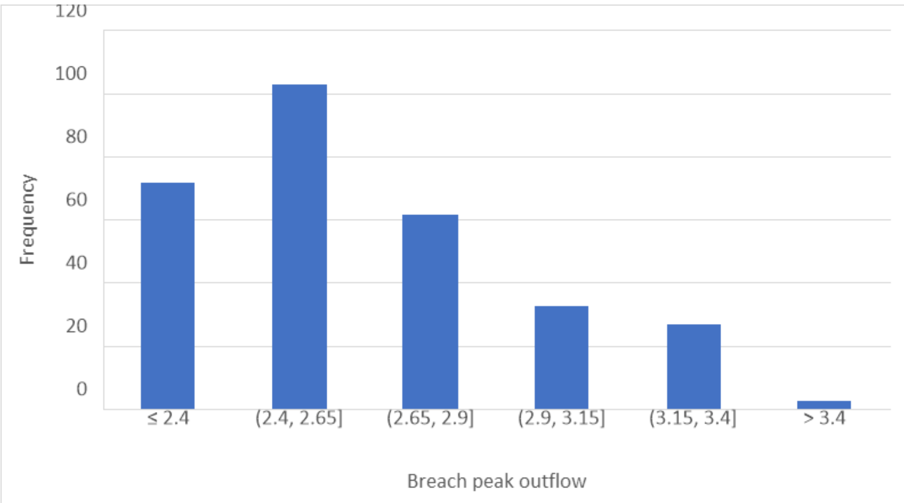


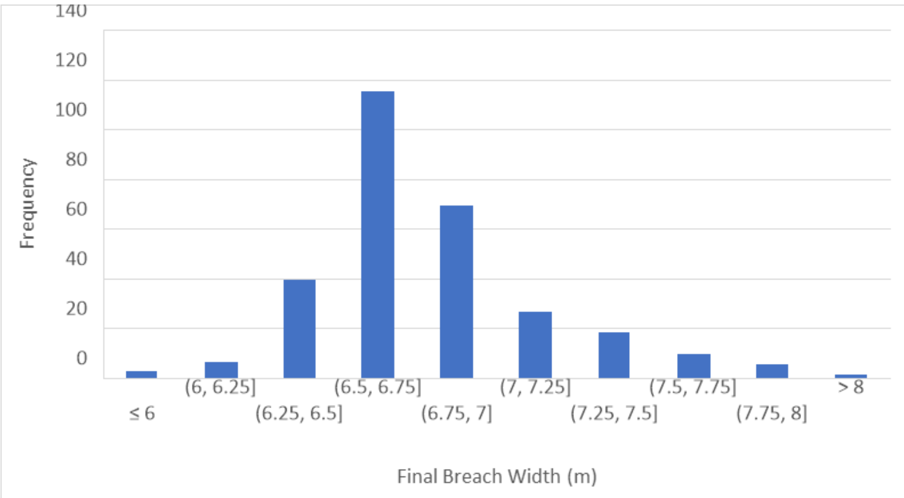
Figure J.4: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying pipe level parameter

J.1.5 Varying all parameters excluding K_d and n

Table J.5: Phase 3: ARS P1 – HRW: MC analysis results for varying all parameters excluding K_d and n

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2.647	1606	6.8	839.9	0.74	1.29	0.92	1.29
Max	3.461	5256	8.1	4666.0	0.77	1.38	0.97	1.38
Min	2.250	1000	5.5	323.0	0.72	1.21	0.87	1.21
Best Run (Pr1) 14	2.97	1628	6.49	983	0.74	1.23	0.89	1.23
Best Run (Pr2) 223	2.269	1059	6.61	372	0.73	1.28	0.92	1.28
Best Run (Pr3) 101	2.662	1615	6.76	384	0.73	1.24	0.90	1.24
mode	2.251	1109	6.61	473	0.73	1.284	0.941	1.284
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**





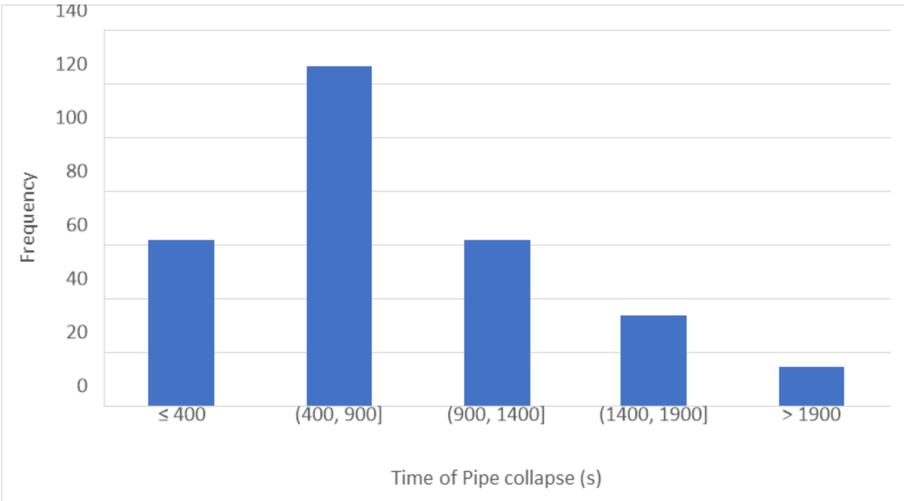
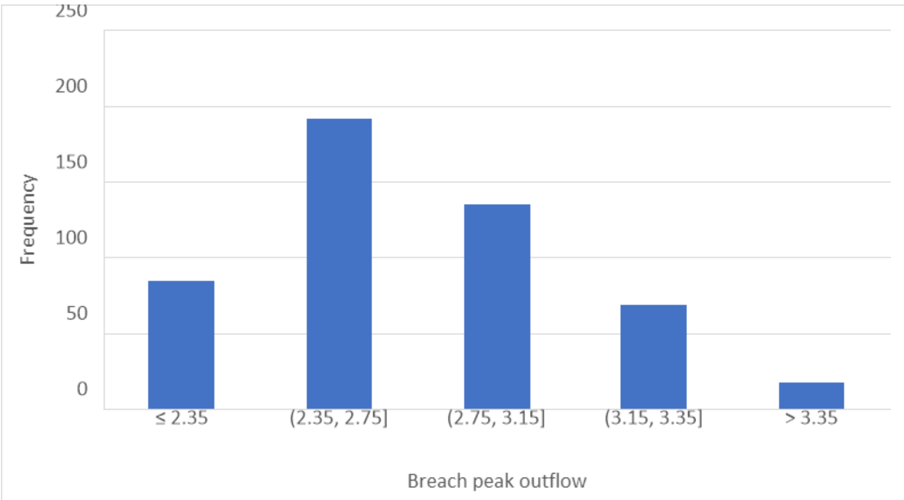


Figure J.5: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying all parameters excluding K_d and n

J.1.6 Varying all parameters

Table J.6: Phase 3: ARS P1 – HRW: MC analysis results for varying all parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2.736	1809	6.9	967.6	0.74	1.30	0.93	1.30
Max	3.653	6809	9.5	5882.0	0.77	1.39	0.99	1.39
Min	2.251	400	3.3	175.0	0.72	1.21	0.88	1.21
Best Run (Pr1) 4	2.983	1574	6.47	704	0.73	1.34	0.94	1.34
Best Run (Pr2) 264	2.45	1216	6.49	388	0.73	1.29	0.92	1.29
Best Run (Pr3) 157	2.774	1501	5.23	430	0.73	1.28	0.91	1.28
mode	2.326	1340	8.12	336	0.732	1.293	0.918	1.293
P1 Observed Data	**2.979**	**1560**	**6.5**	**840**	**0.61**	**1.28**	**1.2**	**1.28**



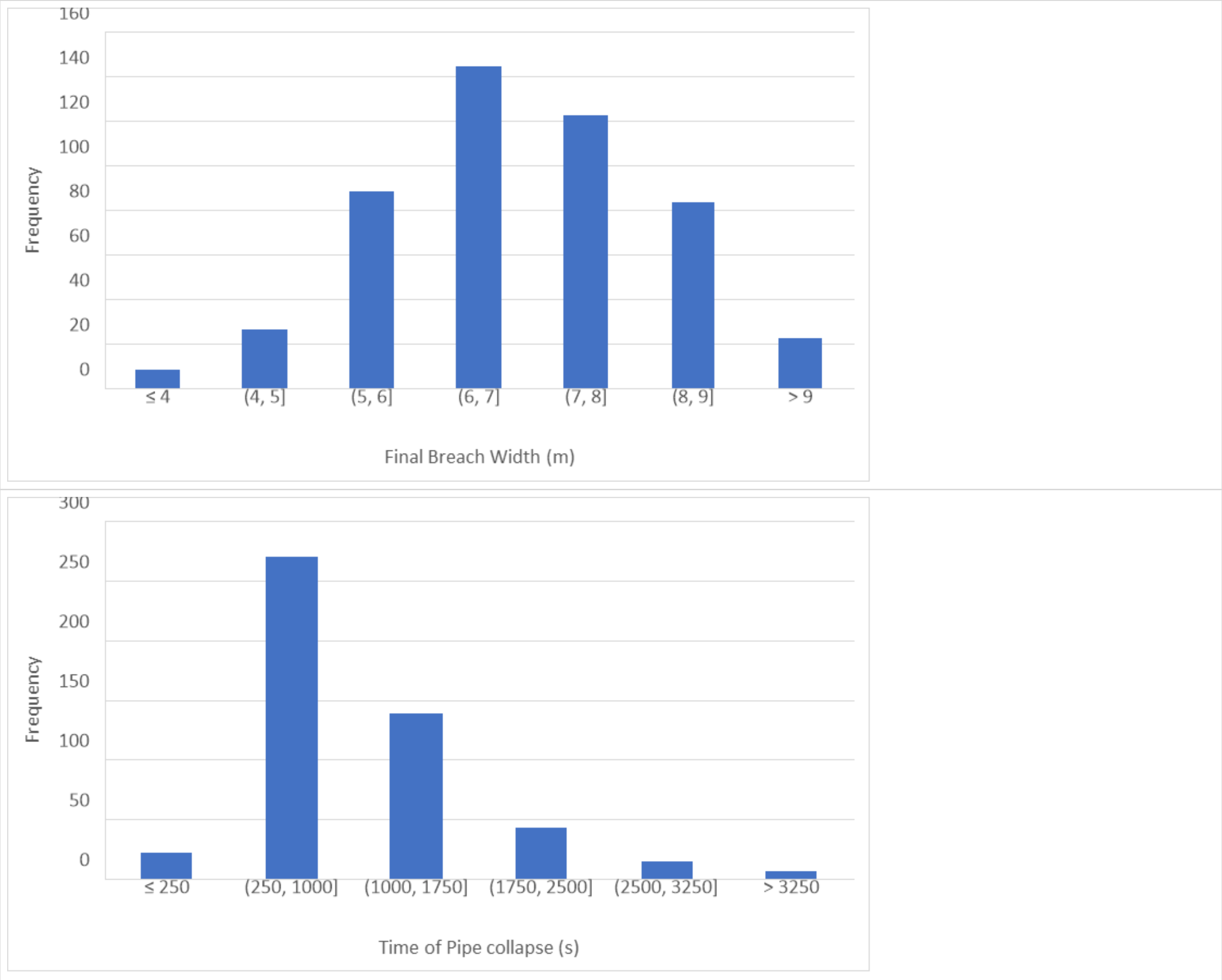
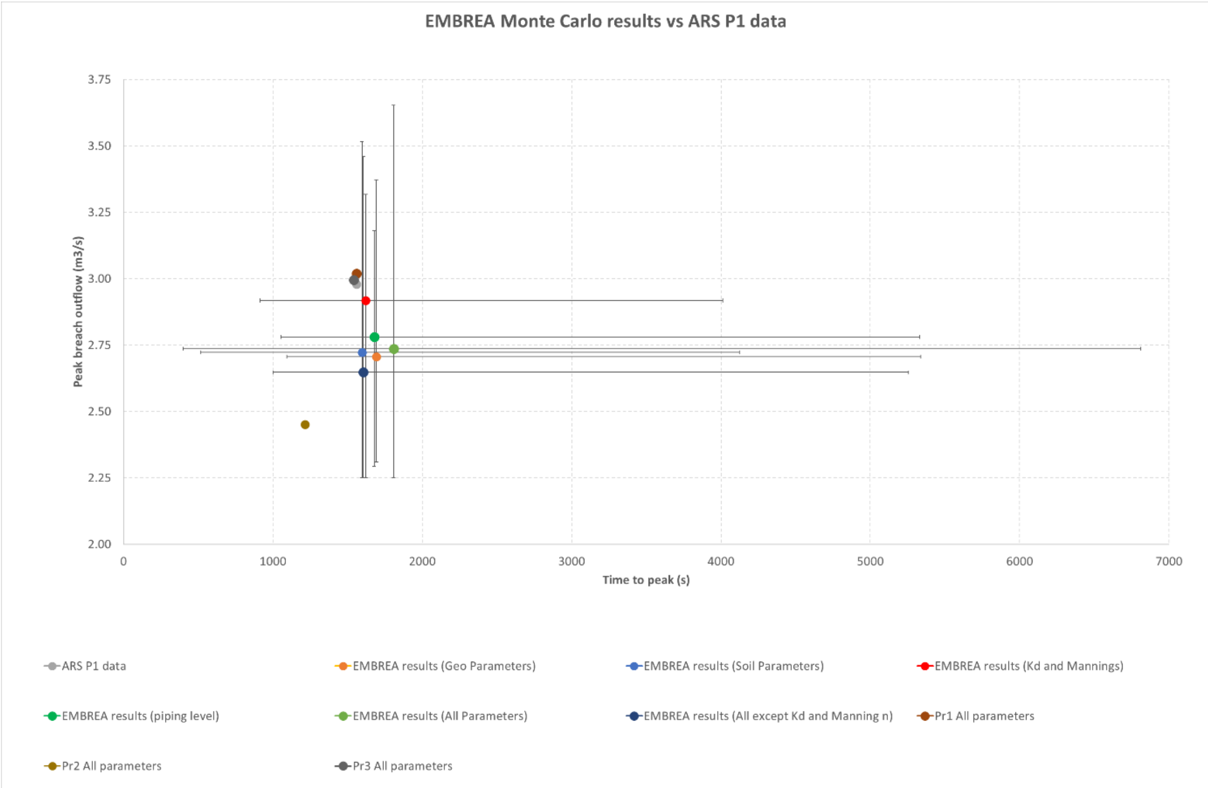


Figure J.6: Phase 3: ARS P1 – HRW: Frequency distribution for MC analysis results for varying all parameters

J.1.7 Prediction Range Plots



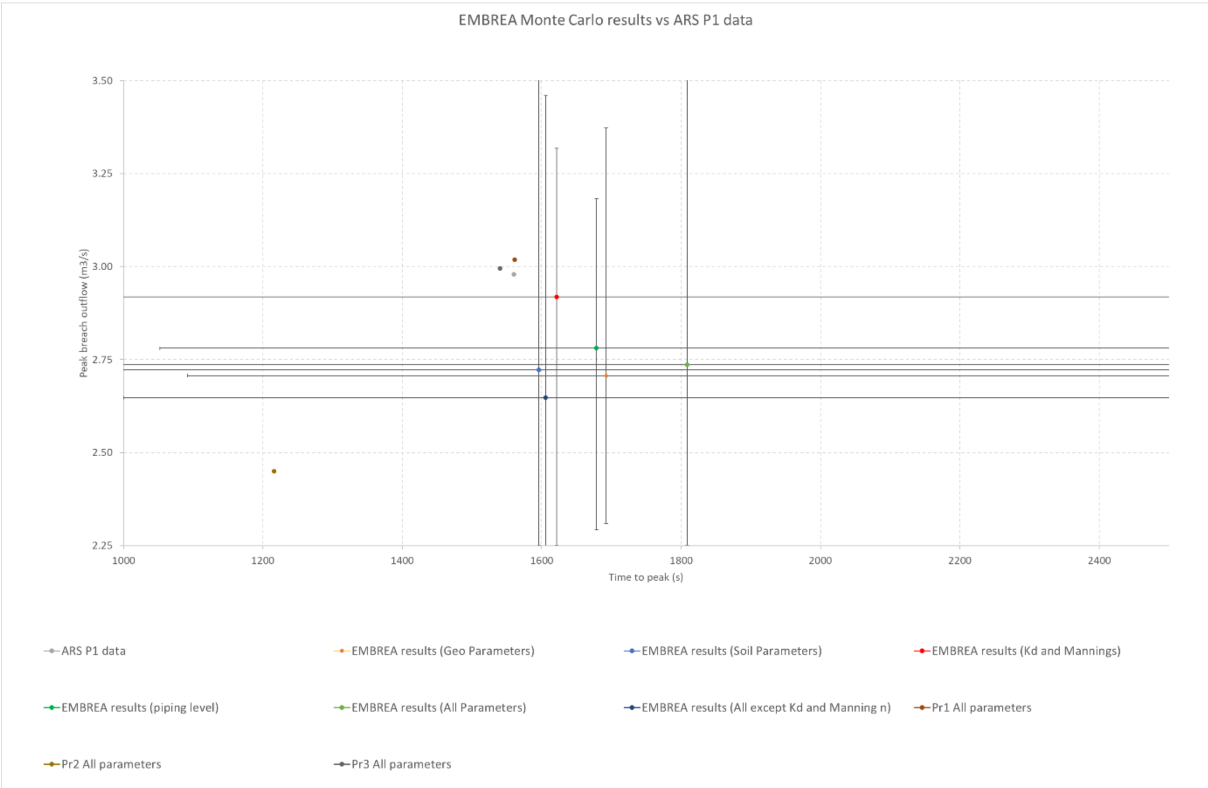
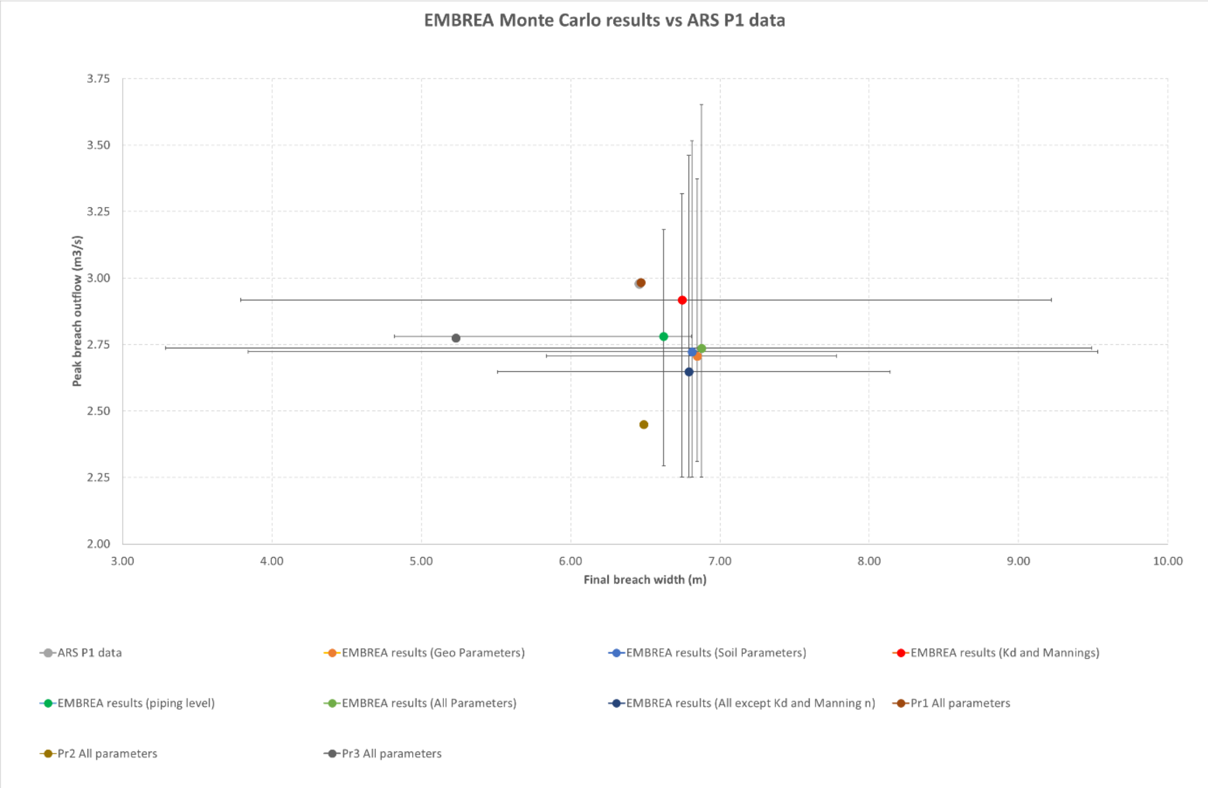


Figure J.7: Phase 3: ARS P1 – HRW: Qp versus Tp range plot (full range and zoom in)



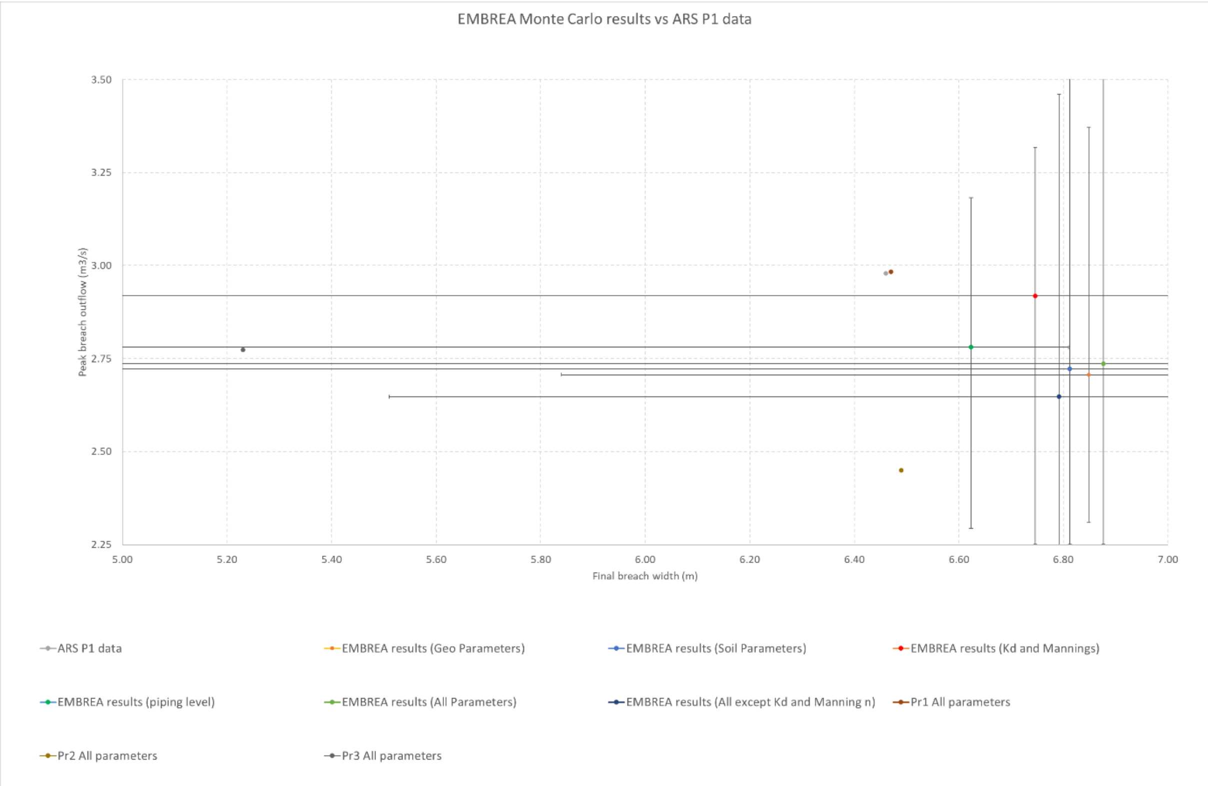
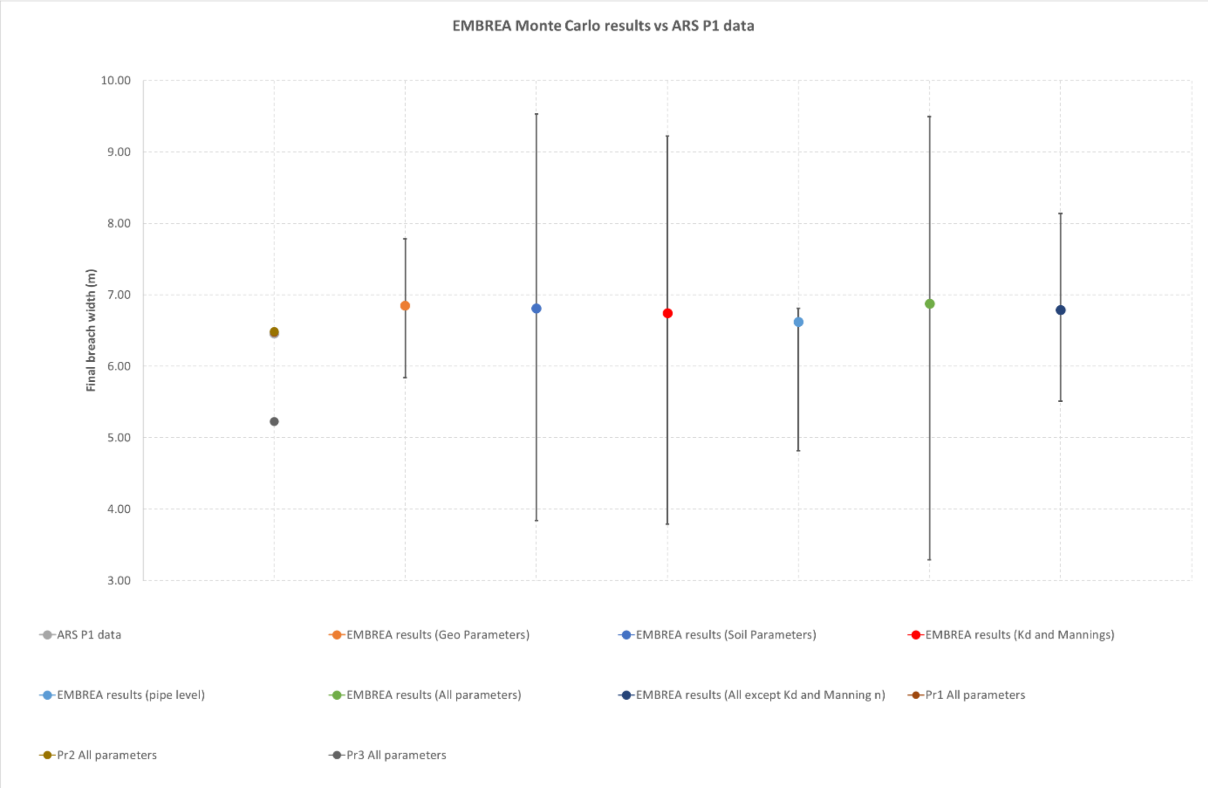


Figure J.8: Phase 3: ARS P1 – HRW: Qp versus Bw range plot (full range and zoom in)



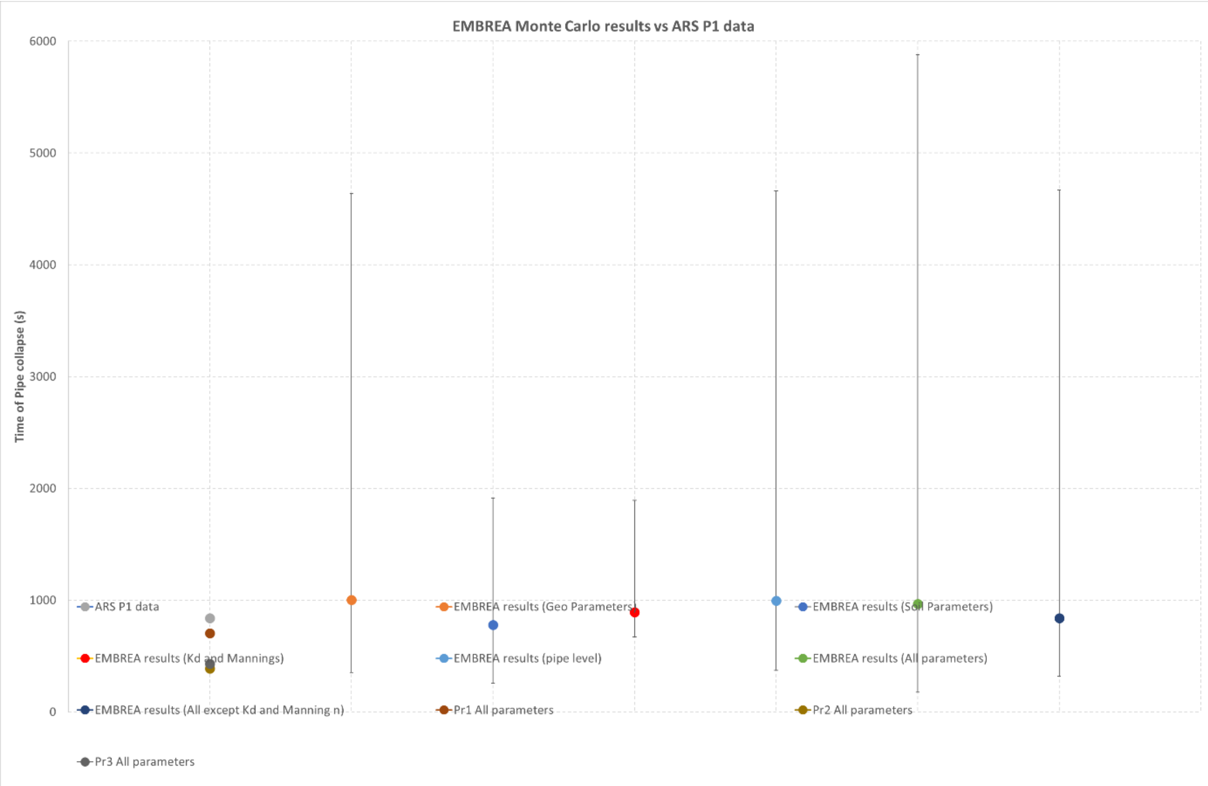


Figure J.9: Phase 3: ARS P1 – HRW: Bw range and Tpc range for different MC analyses

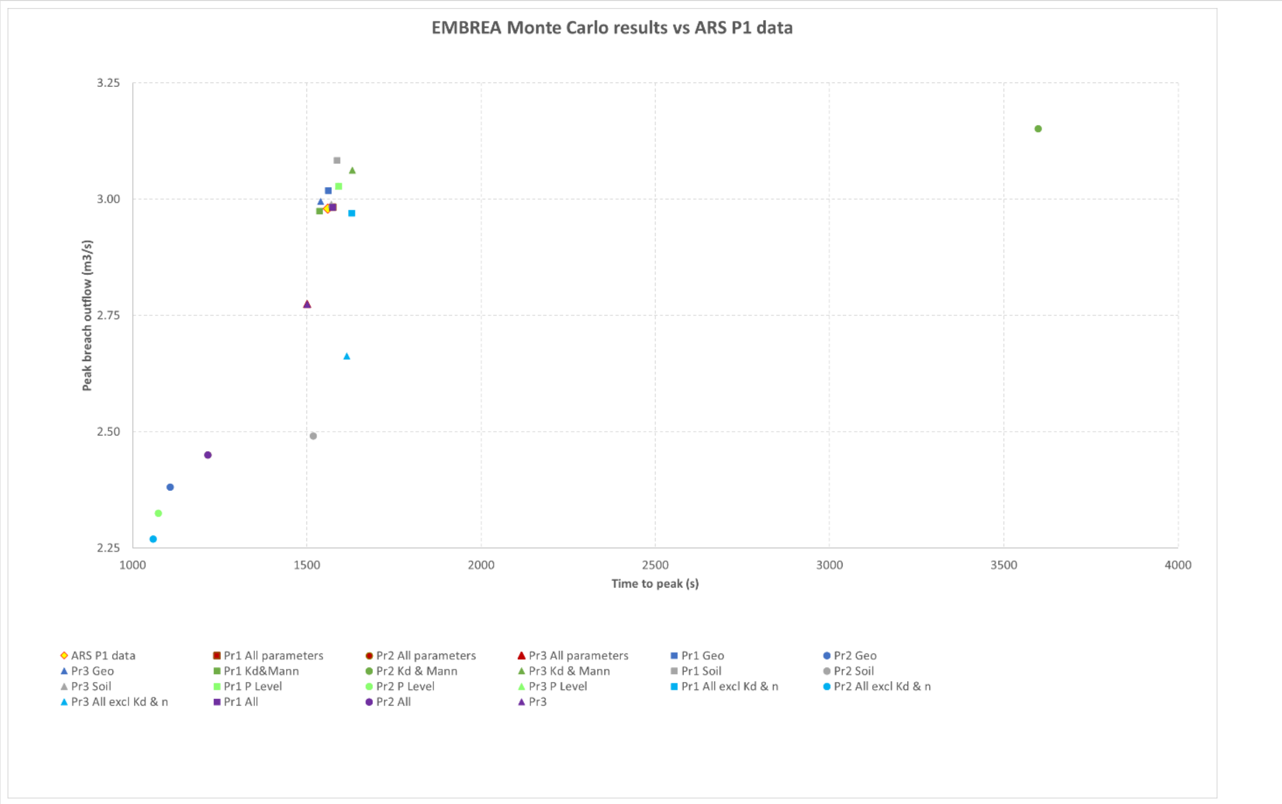


Figure J.10: Phase 3: ARS P1 – HRW: Distribution of PR best run values

J.2 MC analyses for Big Bay

HRW undertook Monte Carlo (MC) analyses for a number of different parameter combinations:

- Geometric parameters
- Soil parameters
- K_d and Manning's n
- Pipe level
- All parameters excluding K_d and n
- All parameters.

For each set of MC runs:

- Results distributions for Q_p, B_w, T_{pc} were plotted
- Performance parameters PR1, PR2 and PR3 were calculated and best run parameters and results identified
- Range plots produced covering:
  - Q_p versus T_p
  - Q_p versus B_w
  - B_w ranges
  - T_{pc} ranges.

J.2.1 Varying geometric parameters

Table J.7: Phase 3: Big Bay – HRW: MC analysis results for varying geometric parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2813	3077	109.2	272.5	4.42	6.81	8.48	6.81
Max	2851	3130	109.8	280.0	4.60	7.16	8.66	7.16
Min	2786	2710	108.8	270.0	4.35	6.67	8.35	6.67
mode	2838	3110	109.15	270	4.38	6.71	8.42	6.71
Best Run (Pr1) 18	2848	3080	109.24	280	4.58	7.14	8.60	7.14
Best Run (Pr2) 55	2798	3100	109.13	270	4.35	6.67	8.37	6.67
Best Run (Pr3) 55	2798	3100	109.13	270	4.35	6.67	8.37	6.67
Big Bay Observed Data	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10**	**10**

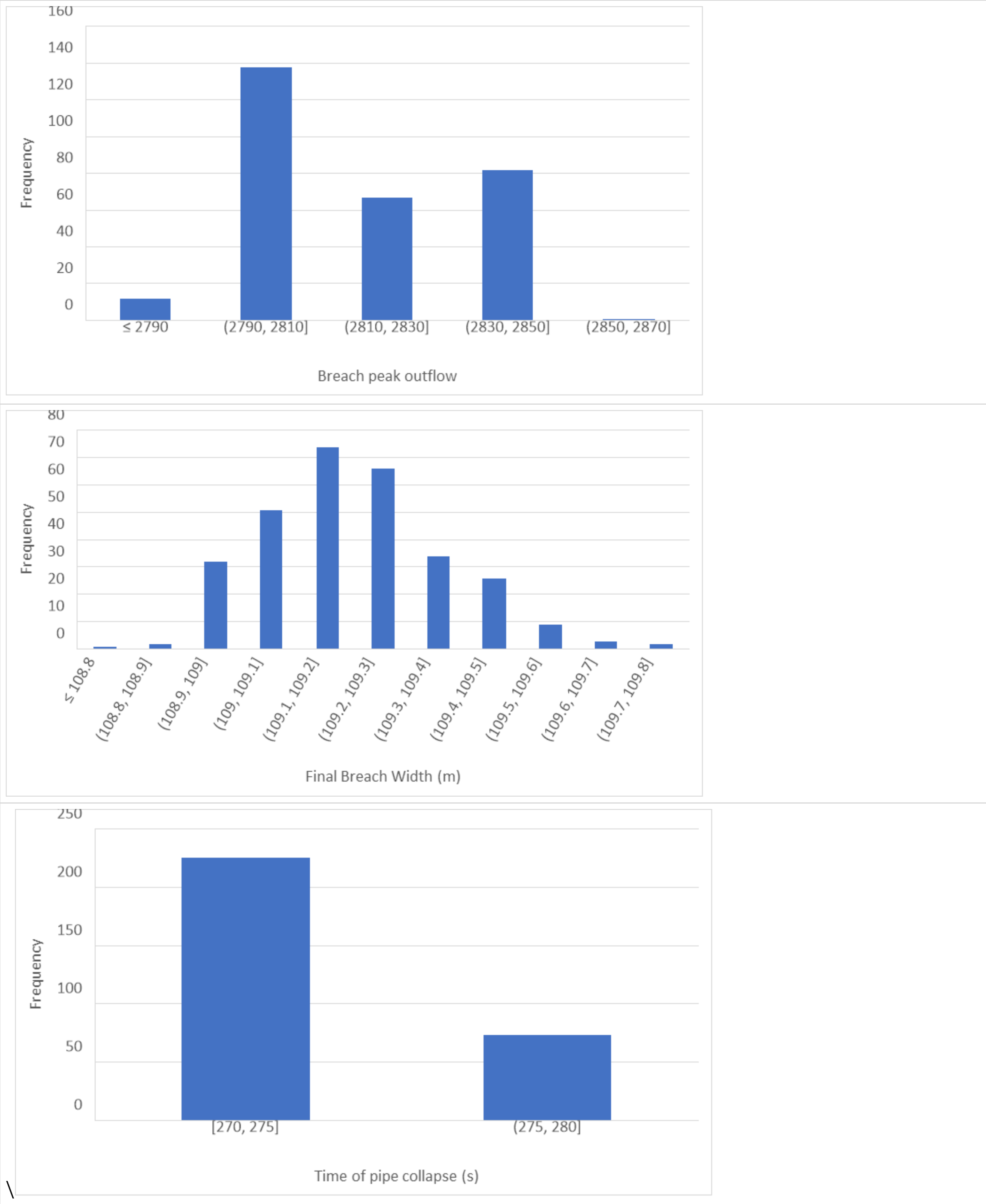
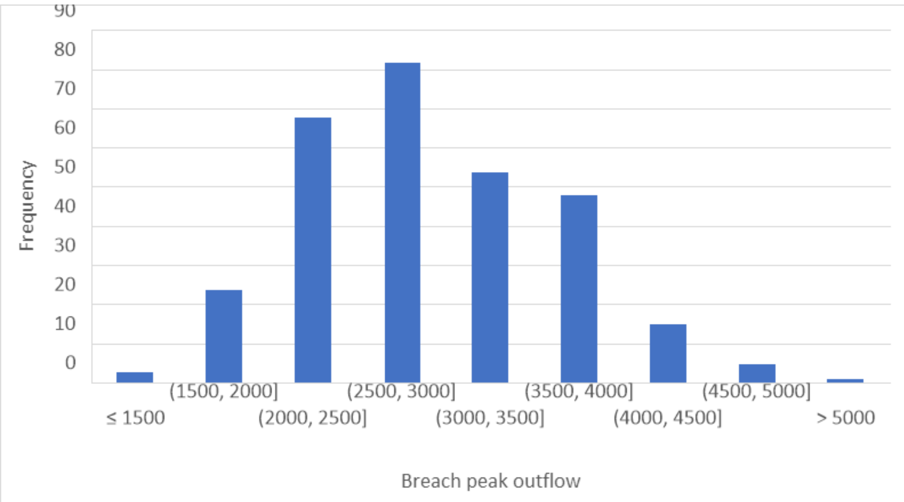


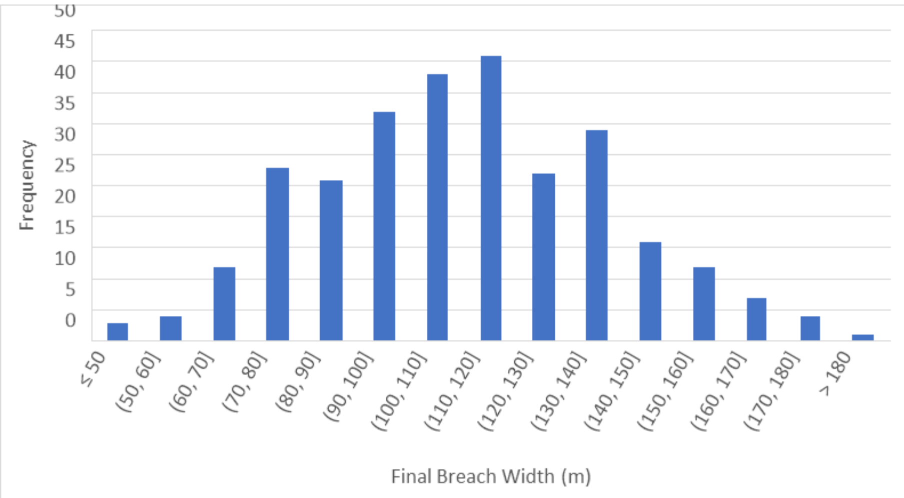
Figure J.11: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying geometric parameters

J.2.2 Varying soil parameters

Table J.8: Phase 3: Big Bay – HRW: MC analysis results for varying soil parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2890	2933	109.9	316.4	4.33	6.64	8.36	6.64
Max	5131	6000	188.7	1530.0	5.46	8.73	9.44	8.73
Min	1412	1780	45.6	110.0	2.51	3.28	6.18	3.28
mode	2719	2900	116.37	170	4.41	7.50	8.71	7.50
Best Run (Pr1) 269	2537	3320	94.18	590	4.51	6.88	8.58	6.88
Best Run (Pr2) 64	2747	2900	103.13	200	2.72	3.28	6.54	3.28
Best Run (Pr3) 64	2747	2900	103.13	200	2.72	3.28	6.54	3.28
Big Bay Observed Data	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10**	**10**





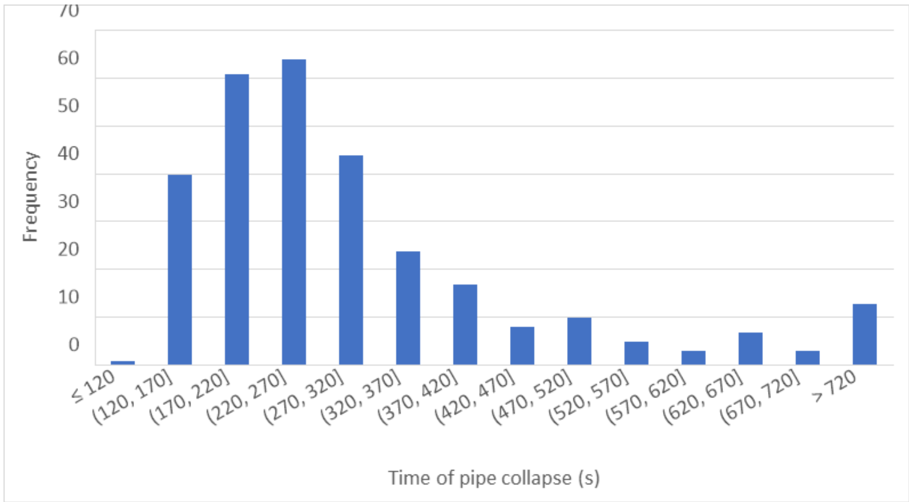
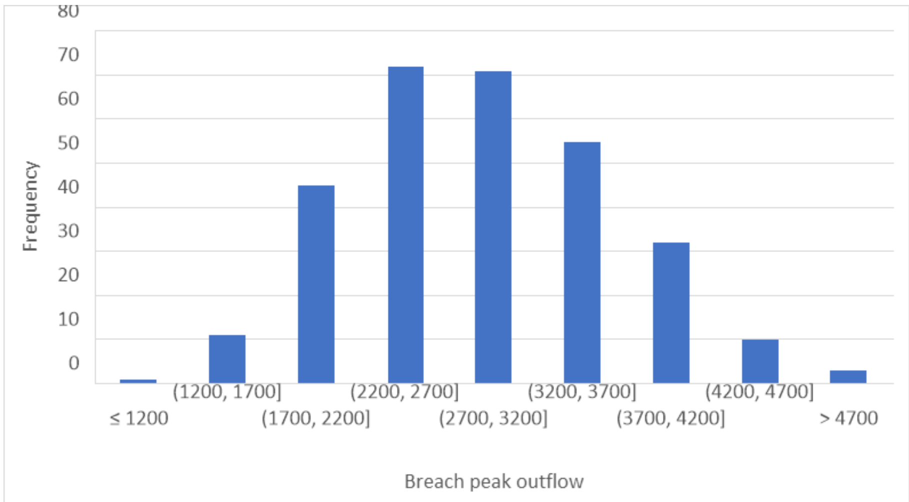


Figure J.12: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying soil parameters

J.2.3 Varying K_d and Manning's n parameters

Table J.9: Phase 3: Big Bay – HRW: MC analysis results for varying K_d and Manning's n parameters

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2877	2985	109.3	340.7	4.48	6.93	8.53	6.93
Max	4995	10760	182.1	3140.0	4.84	7.51	8.89	7.51
Min	1129	1780	30.2	150.0	4.24	6.68	8.24	6.68
mode	2280	3040	74.36	200	4.43	6.68	8.56	6.68
Best Run (Pr1) 299	2804	2940	104.47	480	4.38	6.71	8.43	6.71
Best Run (Pr2) 298	2245	3130	85.65	430	4.35	6.68	8.40	6.68
Best Run (Pr3) 92	2838	3110	108.41	330	4.34	6.73	8.38	6.73
Big Bay Observed Data	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10**	**10**



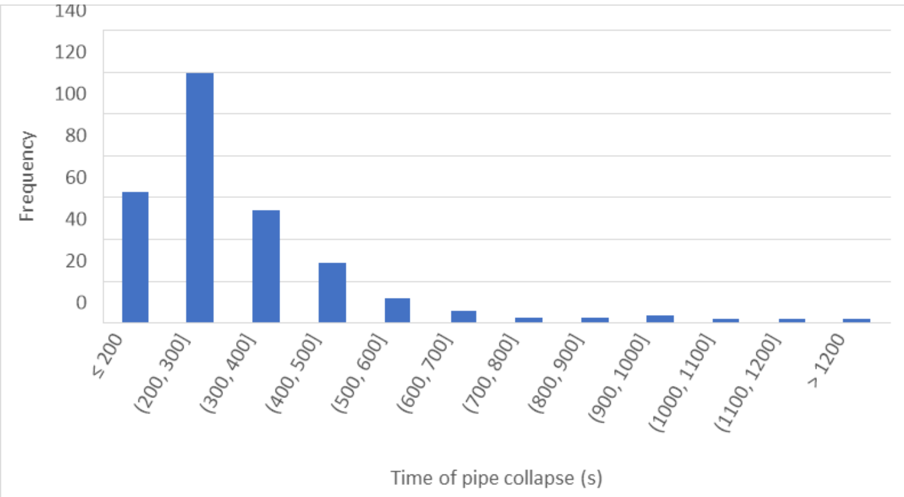


Figure J.13: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying K_d and Manning's n parameters

J.2.4 Varying pipe level parameter

Table J.10: Phase 3: Big Bay – HRW: MC analysis results for varying pipe level parameter

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2809	2923	108.6	280.6	4.27	6.99	8.31	6.99
Max	2854	3120	110.1	310.0	5.44	7.28	9.39	7.28
Min	2768	2490	106.9	270.0	3.56	6.61	7.59	6.61
mode	2803	2700	108.28	280	3.91	7.04	7.77	7.04
Best Run (Pr1) 275	2822	3110	107.84	280	4.18	7.16	8.24	7.16
Best Run (Pr2) 221	2802	2690	107.21	290	3.60	6.91	7.62	6.91
Best Run (Pr3) 221	2802	2690	107.21	290	3.60	6.91	7.62	6.91
Big Bay Observed Dat	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10**	**10**

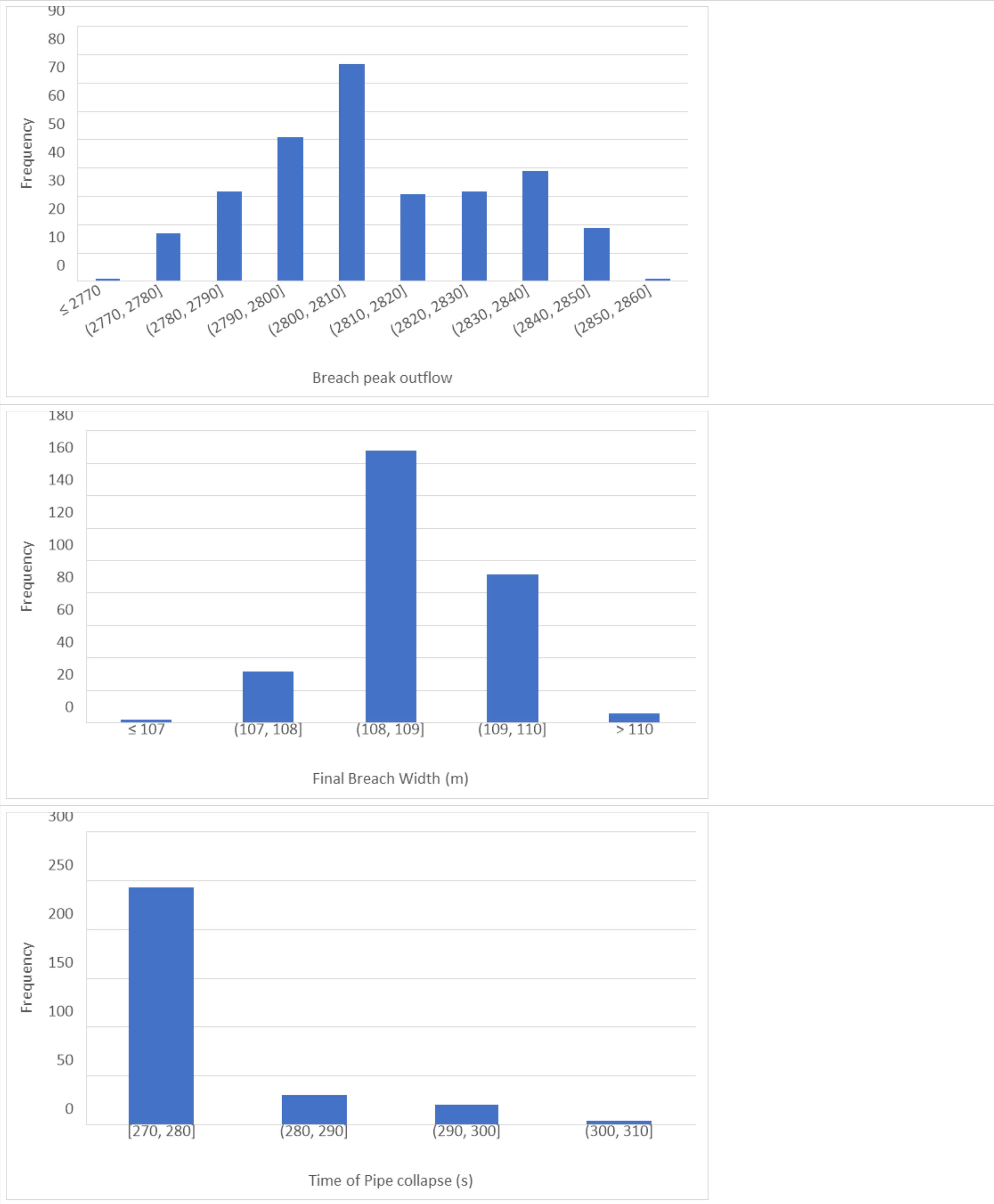
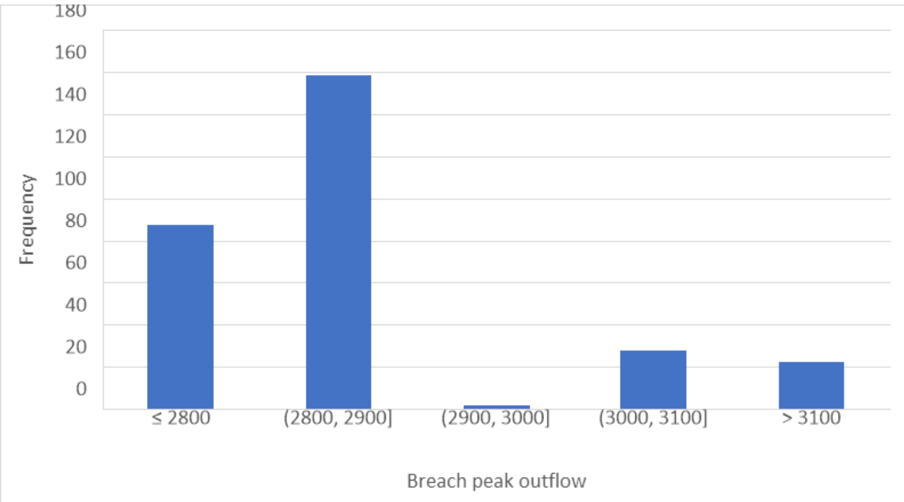


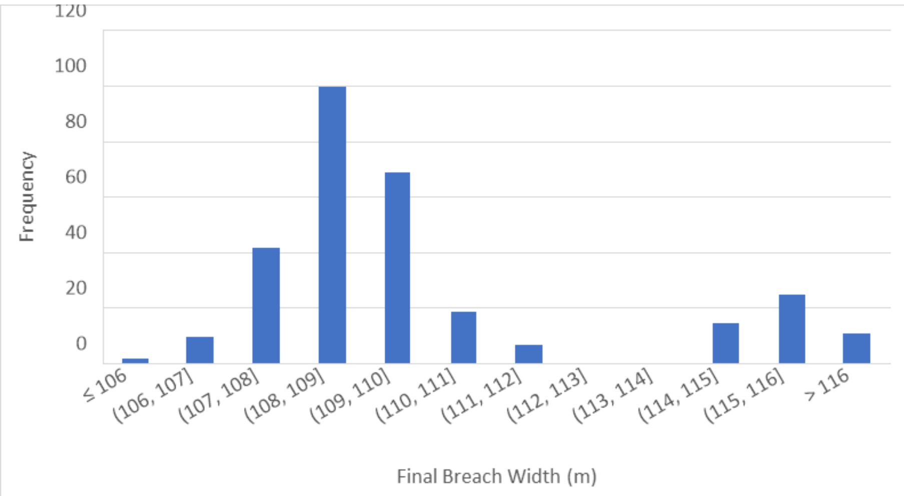
Figure J.14: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying pipe level parameter

J.2.5 Varying all parameters excluding K_d and n

Table J.11: Phase 3: Big Bay – HRW: MC analysis results for varying all parameters excluding K_d and n

MC Runs	Flow	Time	Breach width	Time of Pipe	BDBC	BWBC	BDAC	BWAC
Average	2861	2873	109.9	272.7	4.1	6.7	8.1	6.7
Max	3160	3300	116.8	330.0	6.2	8.7	10.0	8.7
Min	2701	2430	105.7	190.0	1.8	3.1	5.2	3.1
mode	2792	3020	107.9	280.0	4.0	7.5	8.2	7.5
Best Run (Pr1) 256	2821	3110	107.5	280.0	4.2	7.3	8.2	7.3
Best Run (Pr2) 40	3160	2650	116.2	210.0	1.8	3.4	5.2	3.4
Best Run (Pr3) 40	3160	2650	116.2	210.0	1.8	3.4	5.2	3.4
Big Bay Observed Data	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10**	**10**





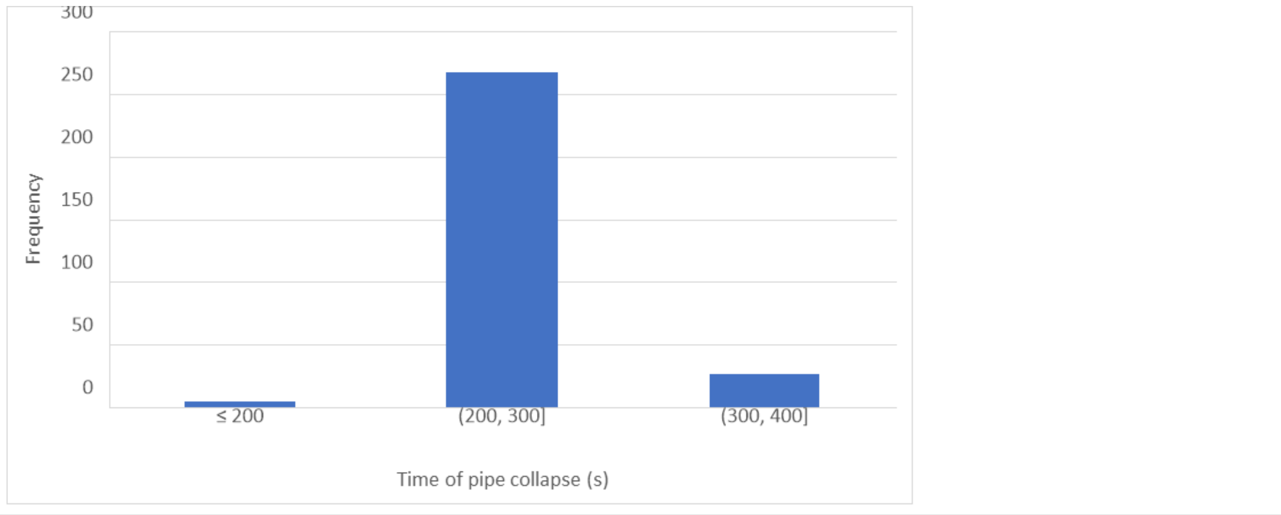
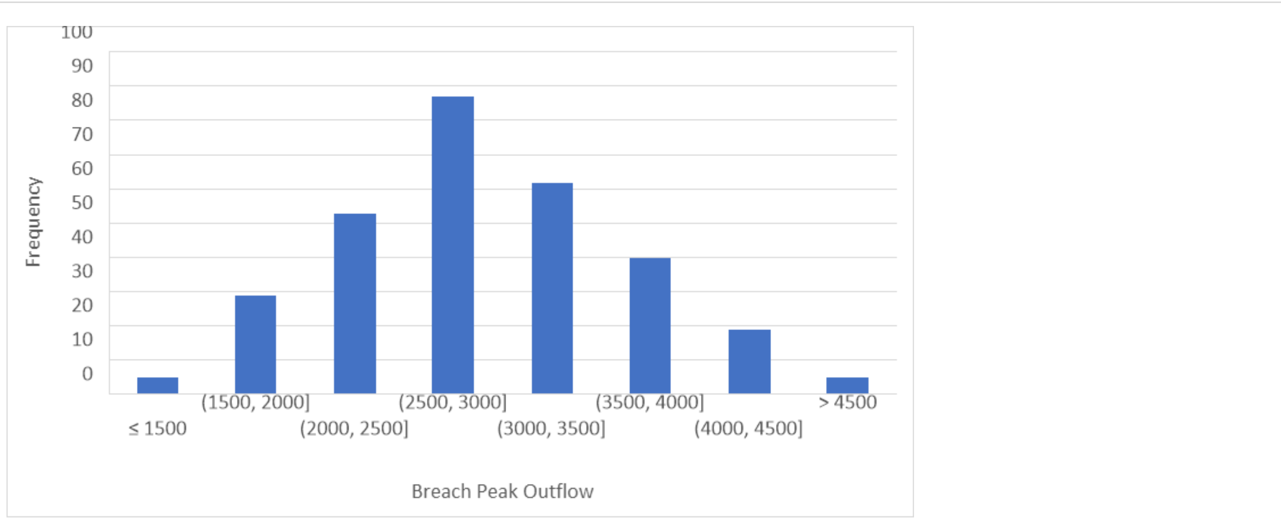


Figure J.15: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying all parameters excluding K_d and n

J.2.6 Varying all parameters

Table J.12: Table 16-18 Phase 3: Big Bay – HRW: MC analysis results for varying all parameters

MC Runs	**Flow - Qp**	**Time - Tp**	**Breach width**	**Time of Pipe**	**BDBC**	**BWBC**	**BDAC**	**BWAC**
Average	2910	2975	110.3	340	4.10	6.67	8.11	6.67
Max	5051	8650	189.2	2200	5.81	8.74	9.71	8.74
Min	1288	1730	36.1	130	1.88	3.23	5.24	3.23
mode	3448	3130	140.4	200	3.82	7.22	8.58	7.22
Best Run (Pr1) 95	2911	3150	106.8	410	4.14	7.12	8.22	7.12
Best Run (Pr2) 199	1774	5560	57.5	640	1.88	3.23	5.24	3.23
Best Run (Pr3) 14	2893	2970	104.4	310	1.97	3.54	5.44	3.54
Big Bay Observed Dat	**3313**	**3300**	**96.2**	**600**	**0.46**	**0.46**	**10.00**	**10.00**



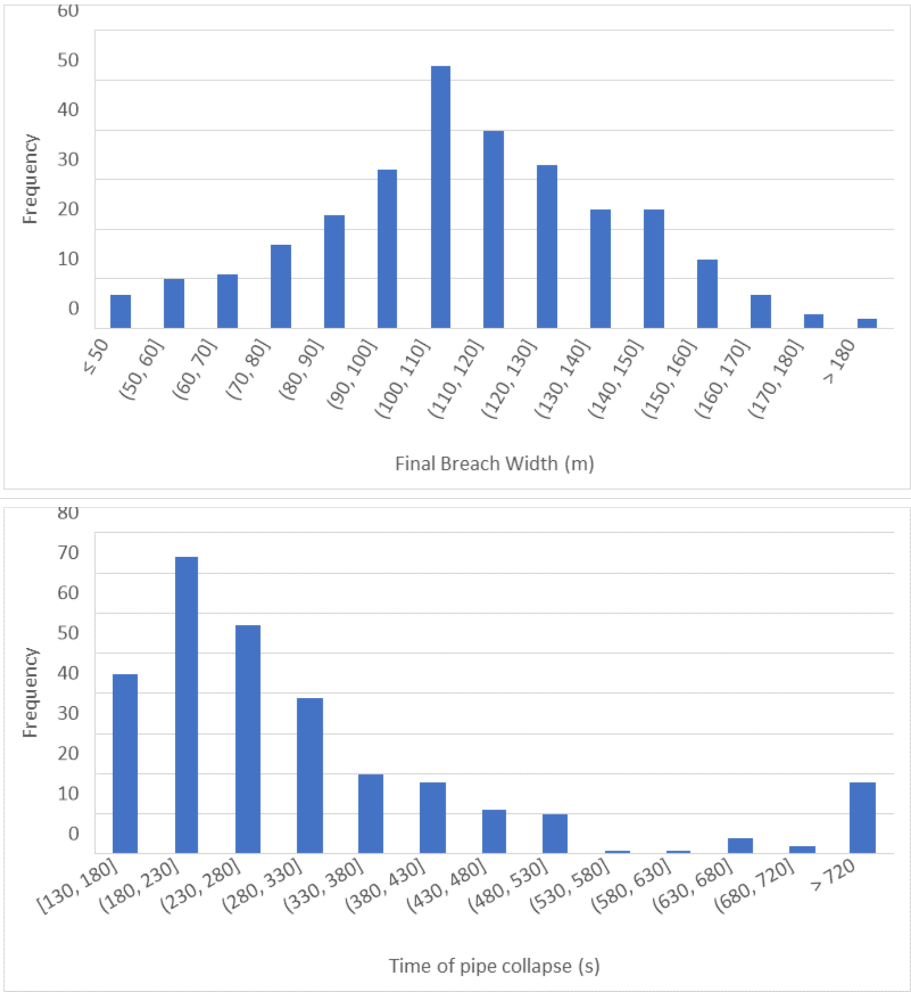


Figure J.16: Phase 3: Big Bay – HRW: Frequency distribution for MC analysis results for varying all parameters

J.2.7 Prediction Range Plots

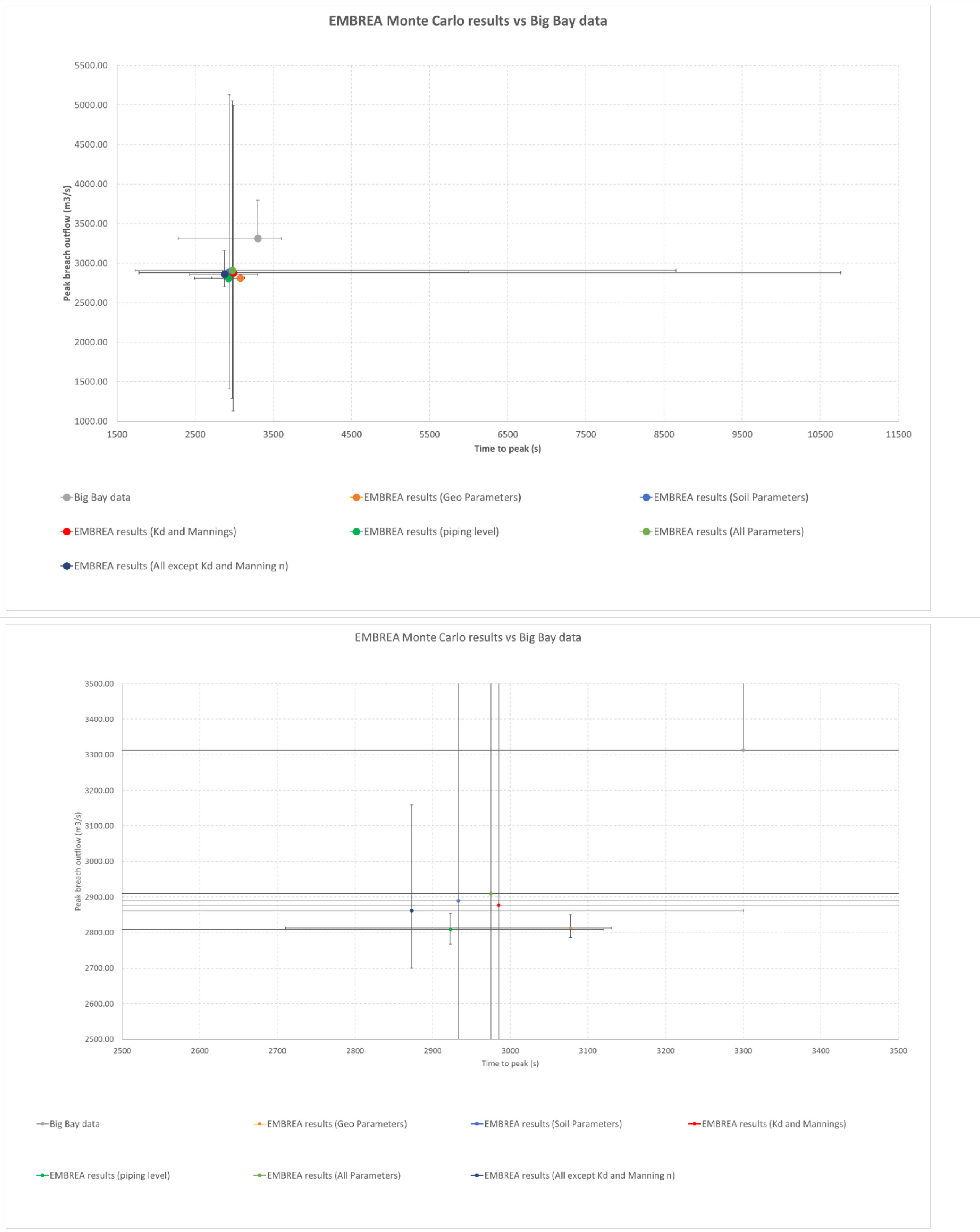
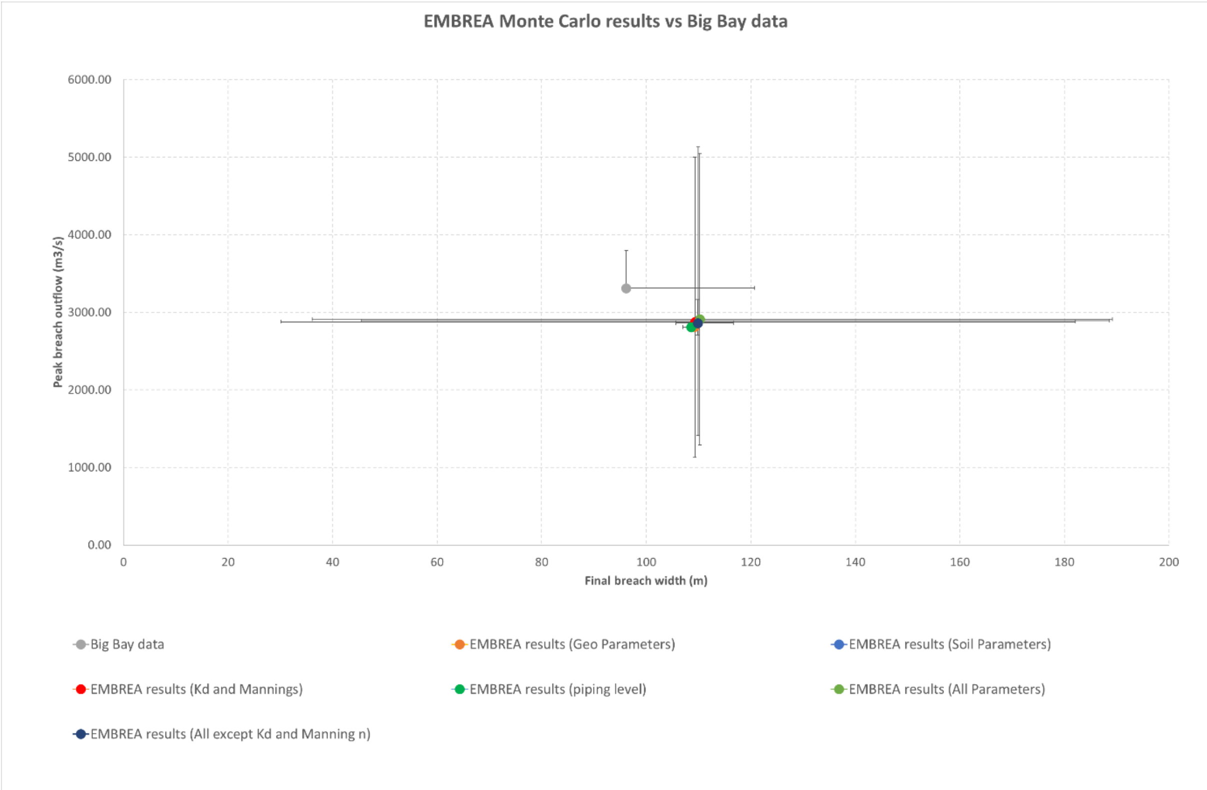


Figure J.17: Phase 3: Big Bay – HRW: Qp versus Tp range plot (full range and zoom in)



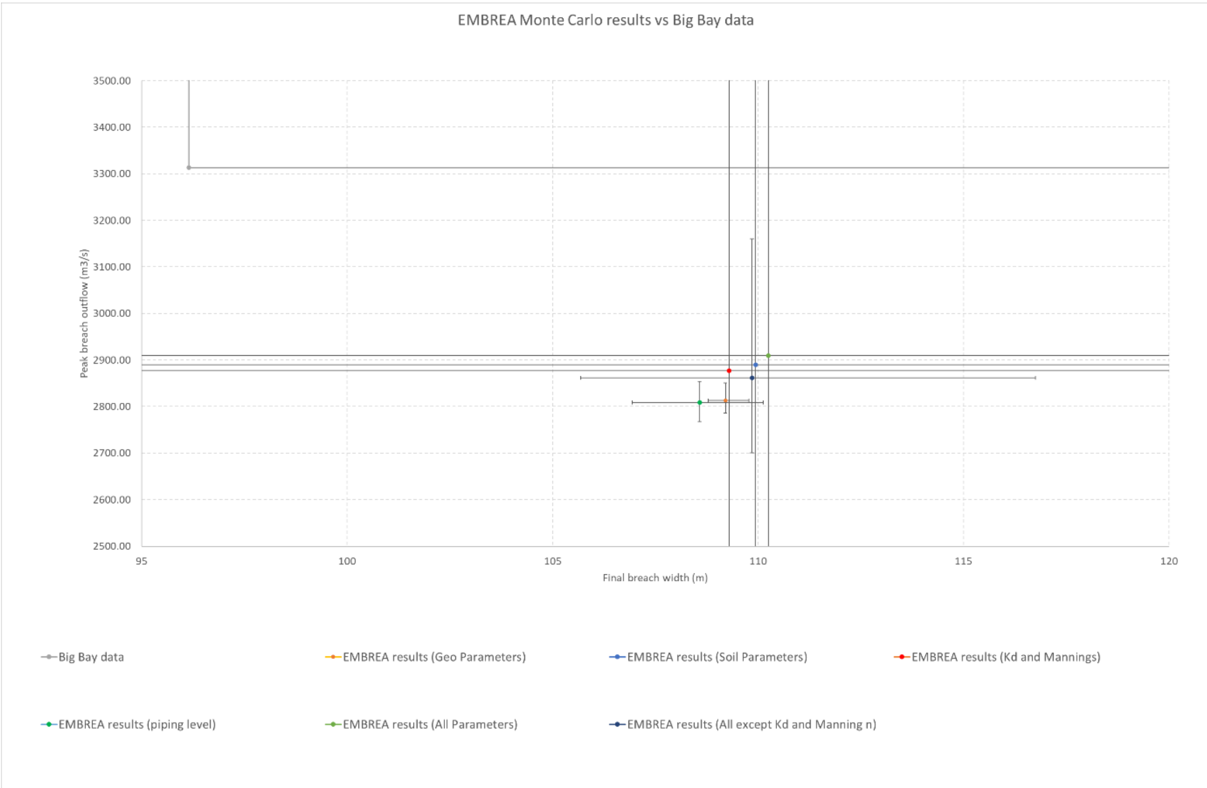


Figure J.18: Phase 3: Big Bay – HRW: Qp versus Bw range plot (full range and zoom in)

Figure J.19: Phase 3: Big Bay – HRW: Bw range and Tpc range for different MC analyses

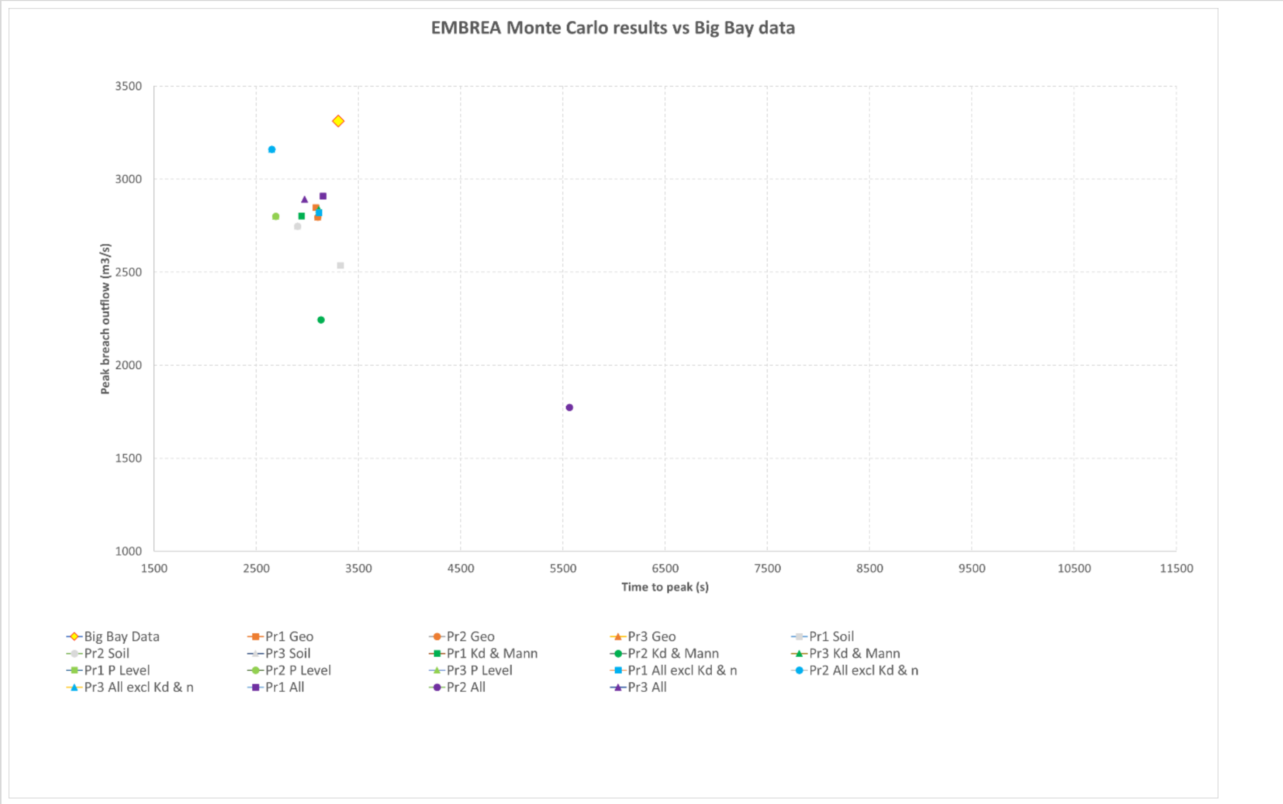
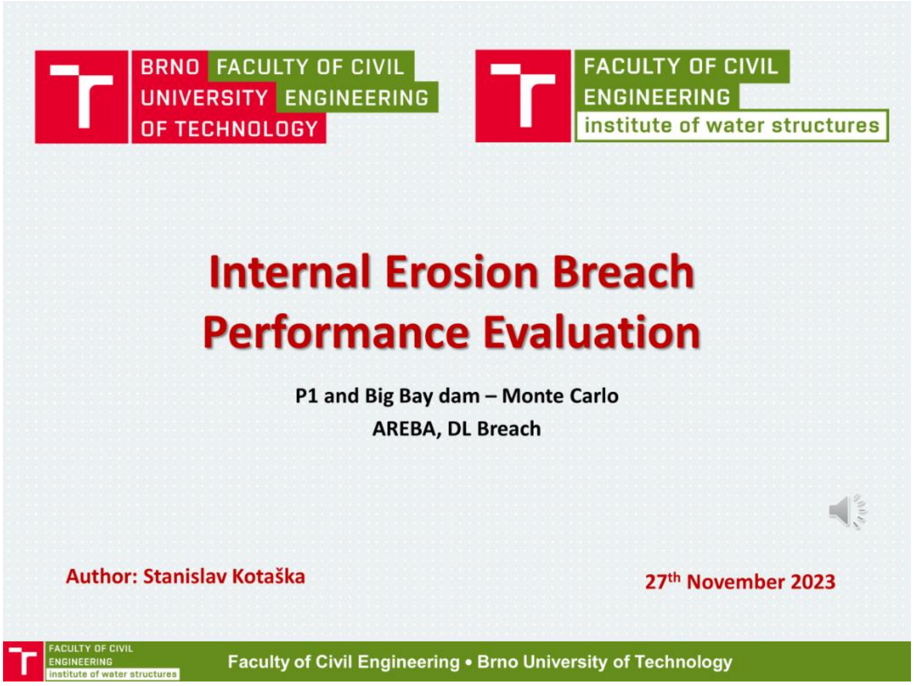


Figure J.20: Phase 3: Big Bay – HRW: Distribution of PR best run values

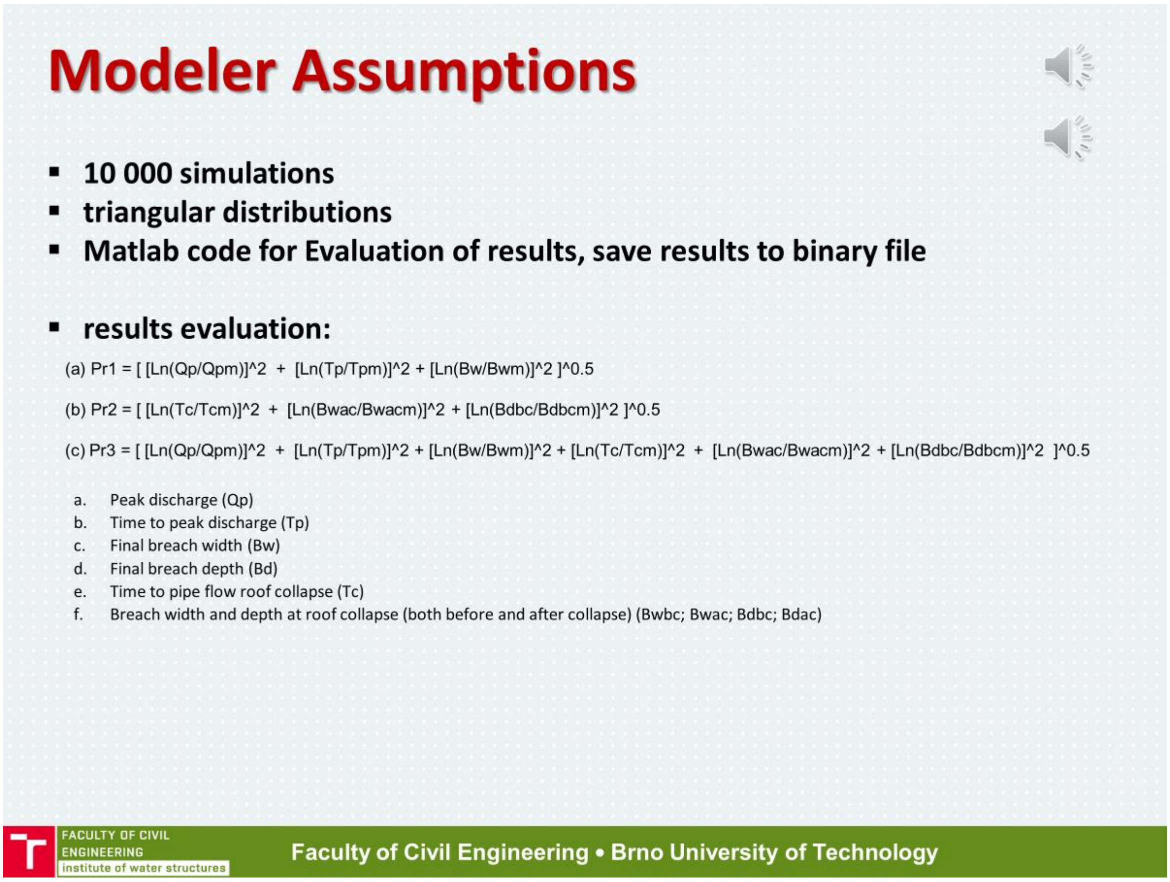
K Phase 3 – BUT – P1 & Big Bay Modelling Results

The following pages comprise the PPT slides with results plots from Stanislav Kotaška

File = IE_Performance_Evaluation_phase4_comments.pptx and includes commentary

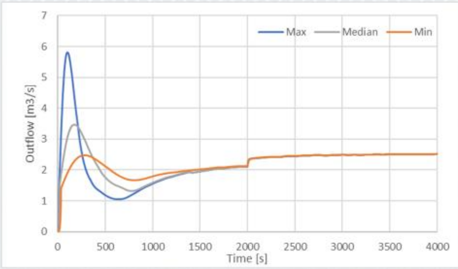


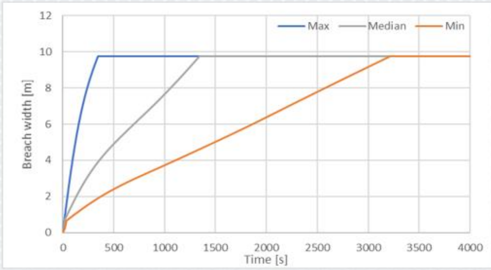
S1

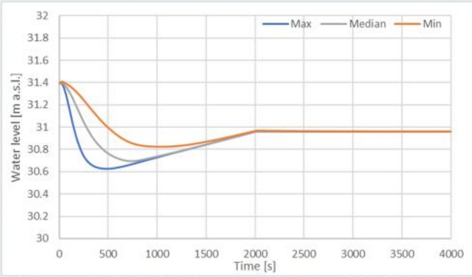


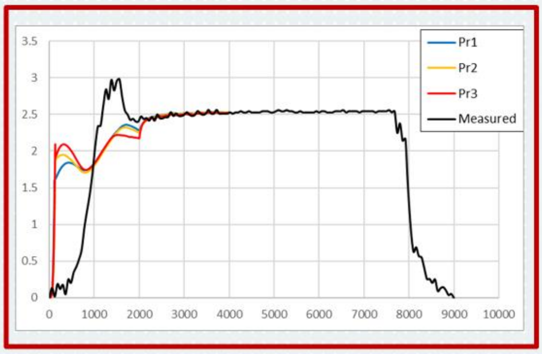
S2

AREBA – P1 – max, med, min









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S3

AREBA – P1 – max, med, min

Observed

Parameter	units	ARS P1
Peak discharge (Qp)	m3/s	2.979
Time to peak discharge (Tp)	secs	1560**
Average final breach width (Bw)	m	6.46
Final breach depth (Bd)	m	1.25
Time to pipe flow roof collapse (Tc)	secs	840**
Breach depth or vertical diameter before roof collapse (Bdbc)	m	0.61m (2ft)
Breach width or horizontal diameter before roof collapse (Bwbc)	m	1.28m (4.2ft)
Breach depth after roof collapse (Bdbc)	m	1.2m (3.95ft)
Breach width after roof collapse (Bdac)	m	1.28m (4.2ft)

* from time 12:15 on 12/3/2004

**from time 09:38 on 20/07/2006

Calculated

- Inputs

Parameter	c	rho	D50	Tc	p	kd	fi	Hdam	Cw	IF	Mann
max	4.66	1685.95	0.11	0.06	0.38	231.64	31.75	31.67	1.93	30.81	0.03
min	5.11	1708.49	0.13	0.04	0.38	77.44	31.99	31.62	1.86	30.73	0.02
mean	5.86	1695.47	0.11	0.05	0.35	153.29	32.31	31.65	1.99	30.75	0.02

- outputs

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr1	Pr2	Pr3	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
max	5.81	104	9.75	1.19	12	0.59	30.59	2.82	5.83	6.47	0.45	7.33	0.17	18.05	0.59	15.33
min	2.52	3999	9.75	1.14	34	0.66	30.48	1.04	5.10	5.21	0.03	0.89	0.17	10.29	0.44	15.30
mean	3.47	177	9.75	1.17	19	0.72	30.46	2.22	5.48	5.91	0.02	4.74	0.17	14.36	0.33	15.29

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr1	Pr2	Pr3	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
Pr1	2.53	3577	4.60	1.24	108	0.74	30.48	0.91	4.45	4.54	0.03	0.69	0.12	4.21	0.30	15.30
Pr2	2.53	3999	4.79	1.25	132	0.89	30.48	1.00	4.34	4.46	0.03	0.89	0.09	3.42	0.13	15.30
Pr3	2.53	3999	6.48	1.26	129	0.90	30.48	0.96	4.35	4.45	0.03	0.89	0.00	3.51	0.12	15.30

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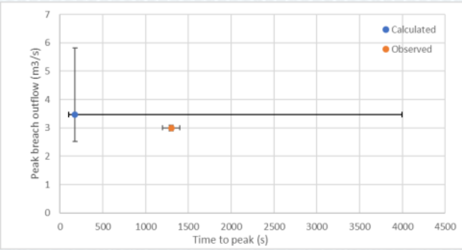
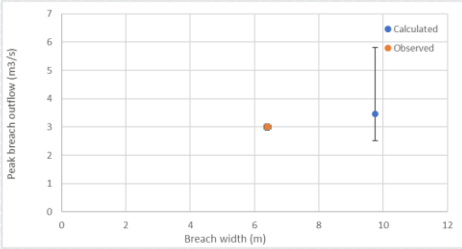
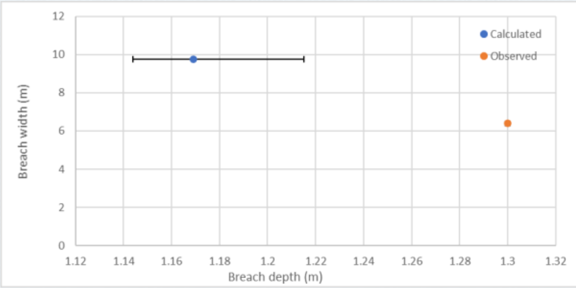
S4

AREBA – P1 – max, med, min







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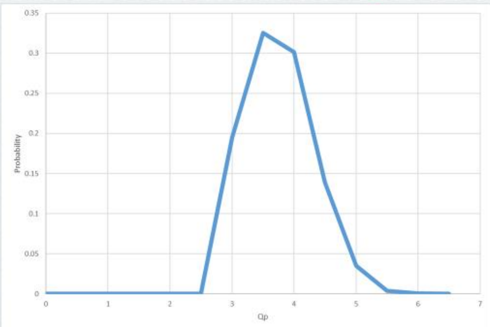
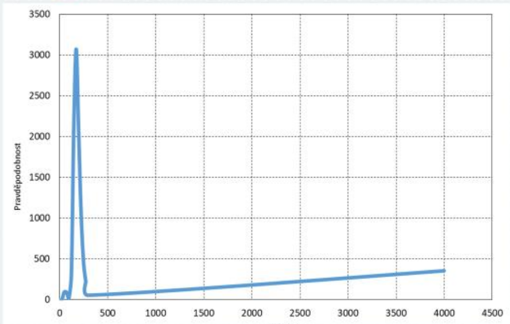
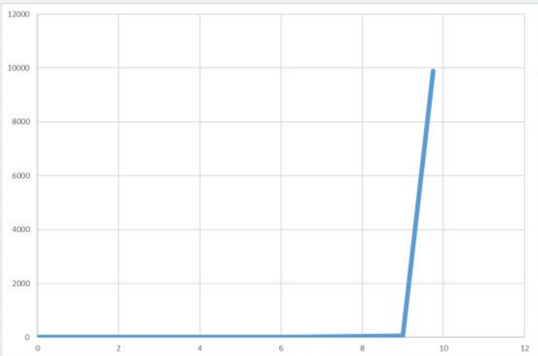
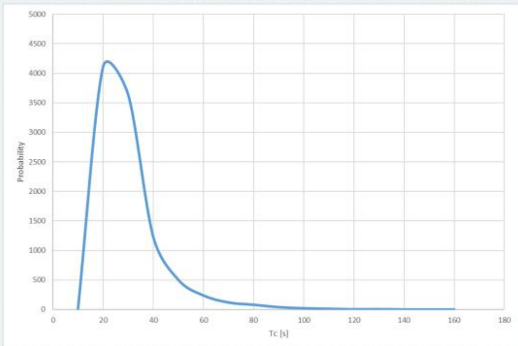
S5

AREBA – P1 – probability









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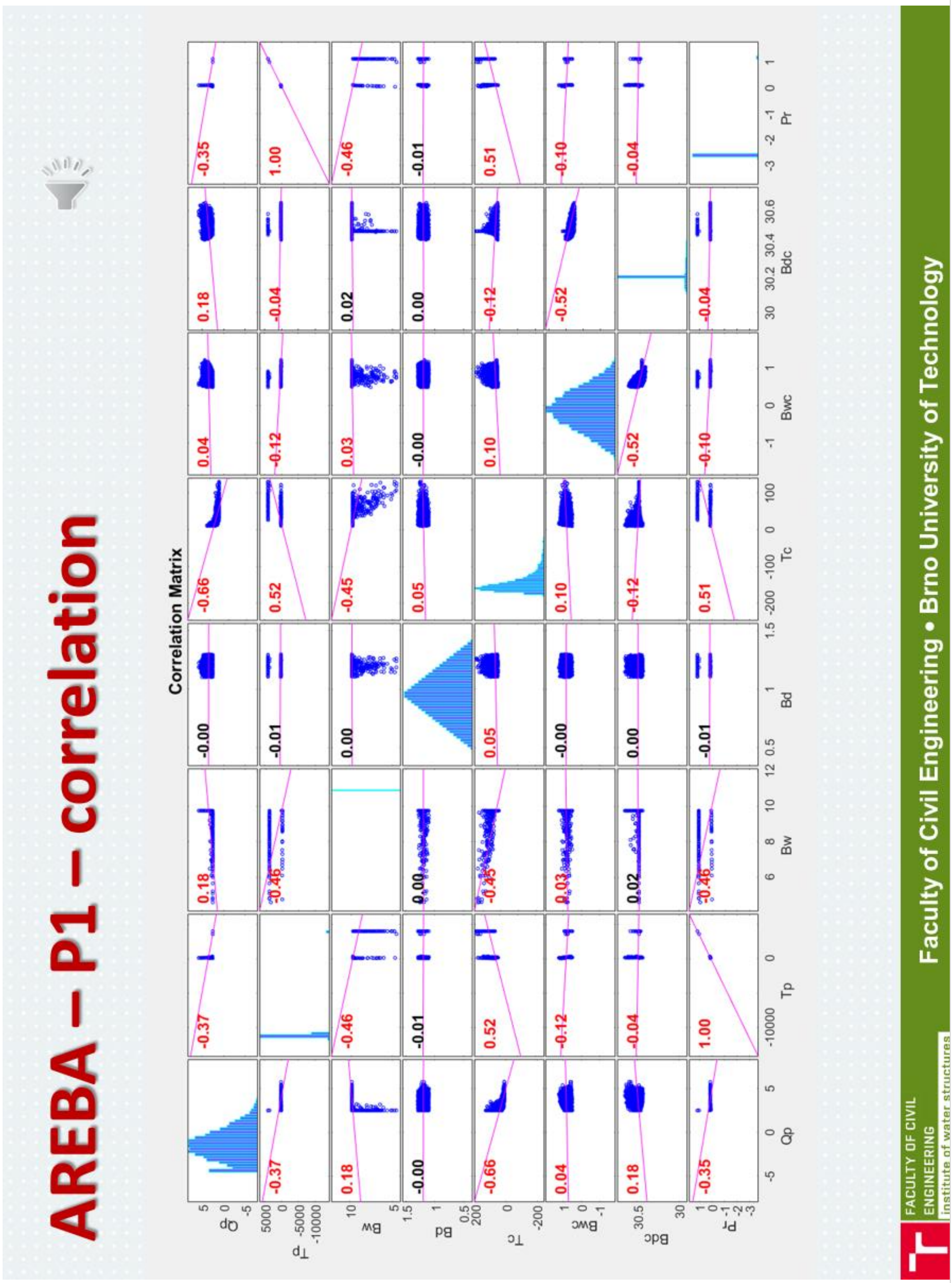
S6


AREBA – P1 – correlation
Correlation Matrix
Qp
Tp
Bw
Bd
Tc
Bwc
Bdc
Pr
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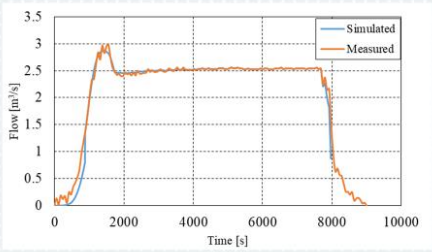
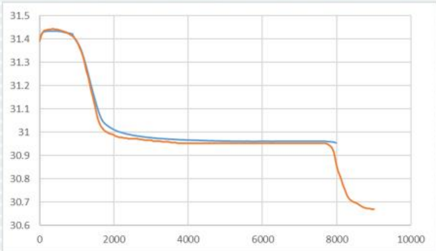
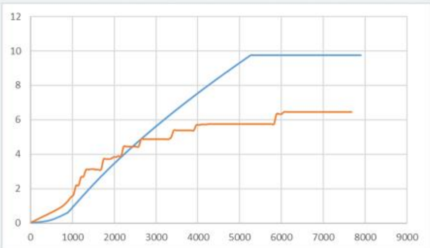
AREBA – P1 - Deterministic







Erodibility = 4.3
Tc = 5.7 Pa
Manning = 0.08

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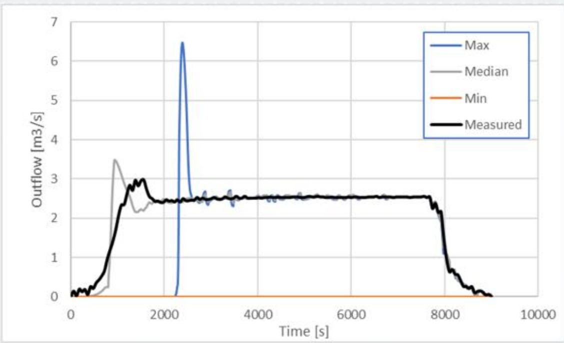
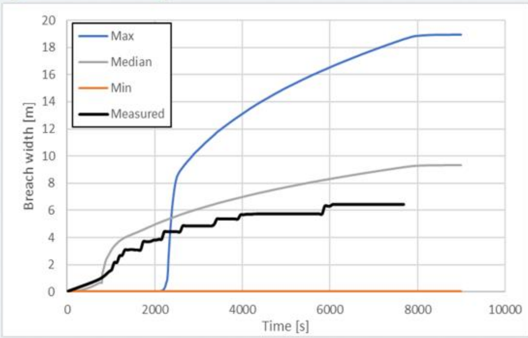
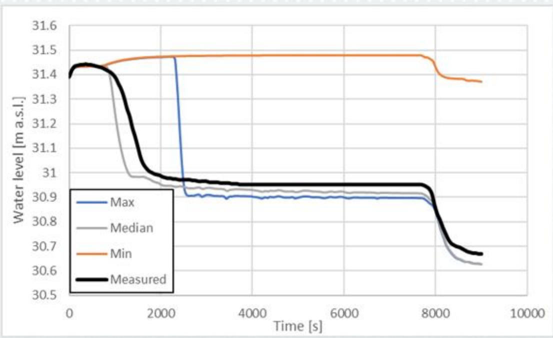
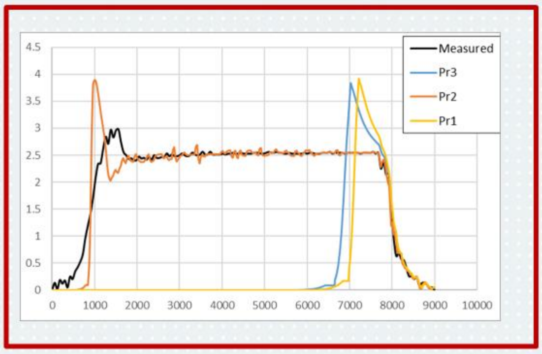
S8

DL Breach – P1 – max, med, min









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S9

DL Breach – P1 – max, med, min

Observed

Parameter	units	ARS P1
Peak discharge (Qp)	m3/s	2.979
Time to peak discharge (Tp)	secs	1560**
Average final breach width (Bw)	m	6.46
Final breach depth (Bd)	m	1.25
Time to pipe flow roof collapse (Tc)	secs	840**
Breach depth or vertical diameter before roof collapse (Bdbc)	m	0.61m (2ft)
Breach width or horizontal diameter before roof collapse (Bwbc)	m	1.28m (4.2ft)
Breach depth after roof collapse (Bdbc)	m	1.2m (3.95ft)
Breach width after roof collapse (Bdac)	m	1.28m (4.2ft)

* from time 12:15 on 12/3/2004
**from time 09:38 on 20/07/2006

Calculated

- Inputs

Parameter	'c'	'rho'	'D50'	'Tc'	'p'	'kd'	'fi'	'Hdam'	'Cw'	'IF'	'Mann'
Max	7378	2650.39	0.128	0.045	0.38	252.58	0.65	1.29	1.97	0.29	0.032
Min	5619	2618.63	0.124	0.079	0.36	125.22	0.62	1.25	2.13	0.22	0.030
Mean	6685	2655.23	0.124	0.093	0.35	86.37	0.63	1.22	1.92	0.31	0.018

- outputs

Parameter	'Qp'	'Tp'	'Bw'	'Bd'	'Tc'	'Bwc'	'Bdc'	'Pr1'	'Pr2'	'Pr3'	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
Max	6.48	2389.5	18.94	0	2369	4.89	0	7.87	Inf	Inf	0.60	60.25	1.16	1.08	1.80	Inf
Min	0	9.5	0.02	1.0294	10	0.04	1.0294	0.007	5.69	Inf	Inf	176.61	33.38	20.09	12.01	0.27381
Mean	3.49	939.5	9.30	0	799	0.66	0.60	8.705	0.66	8.73	0.025	75.62	0.13	0.0024	0.44	0.00054

Parameter	'Qp'	'Tp'	'Bw'	'Bd'	'Tc'	'Bwc'	'Bdc'	'Pr1'	'Pr2'	'Pr3'	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
Pr1	3.9245	2.0054	4.5827	0.0000	7239.4920	2.9149	0.0000	6.6711	65535.0000	65535.0000	0.0758	44.3102	0.1179	4.6393	0.6773	65535
Pr2	3.8892	0.2804	11.3992	0.0000	879.5160	1.3146	0.5995	8.6467	0.0559	8.6469	0.0709	74.3718	0.3225	0.0021	0.0007	0.0003
Pr3	3.8344	1.9526	4.7630	0.0000	6819.5160	1.8063	0.4532	6.6950	2.1430	7.0296	0.0635	44.6659	0.0929	4.3854	0.1186	0.0883

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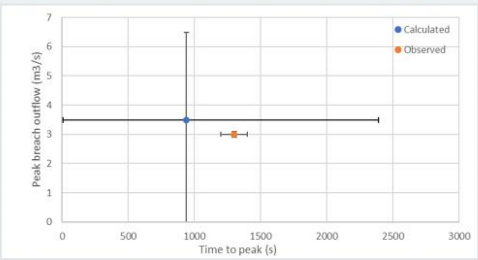
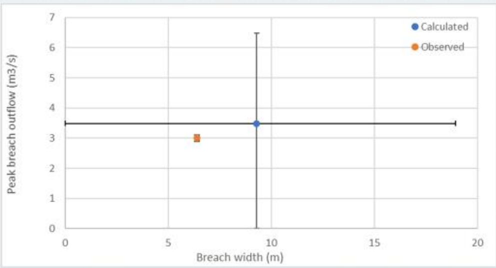
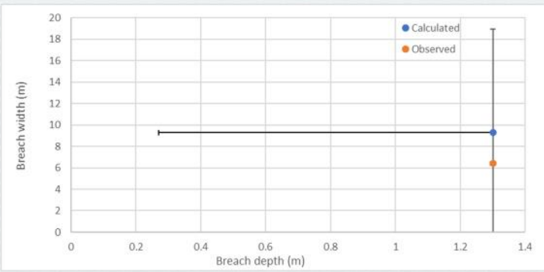
S10

DL Breach – P1 – max, med, min







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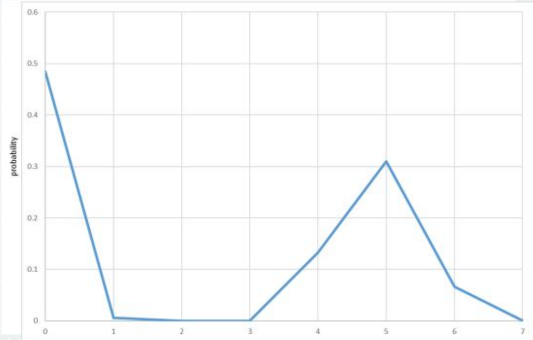
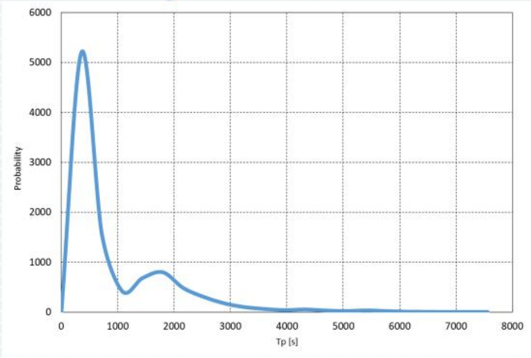
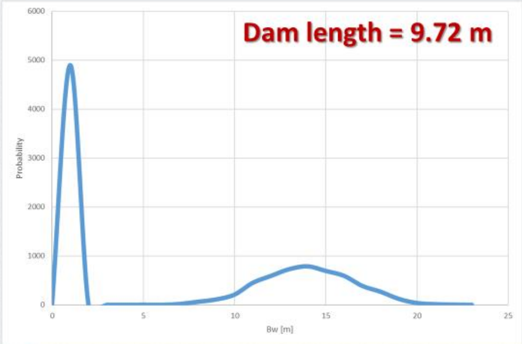
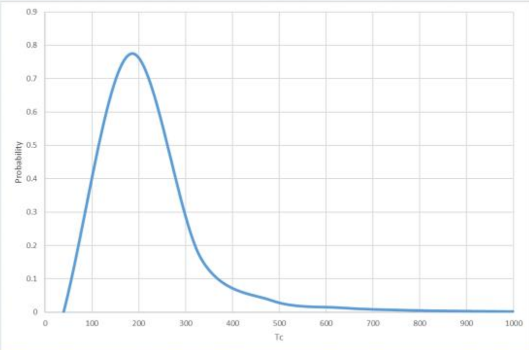
S11

DL Breach – P1 – probability









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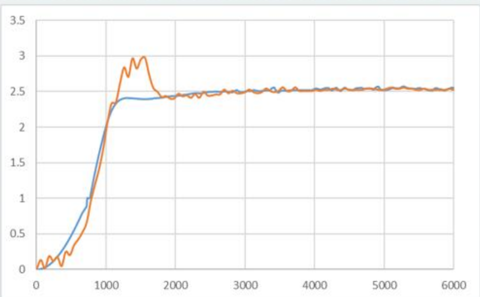
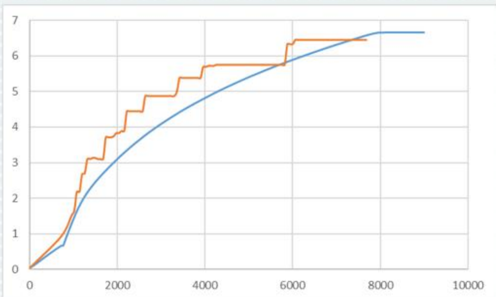
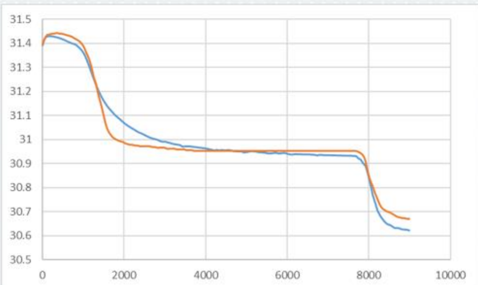
S12

DL Breach – P1 - Deterministic







Erodibility = 20
Tc = 0.6 Pa
Manning = 0.03

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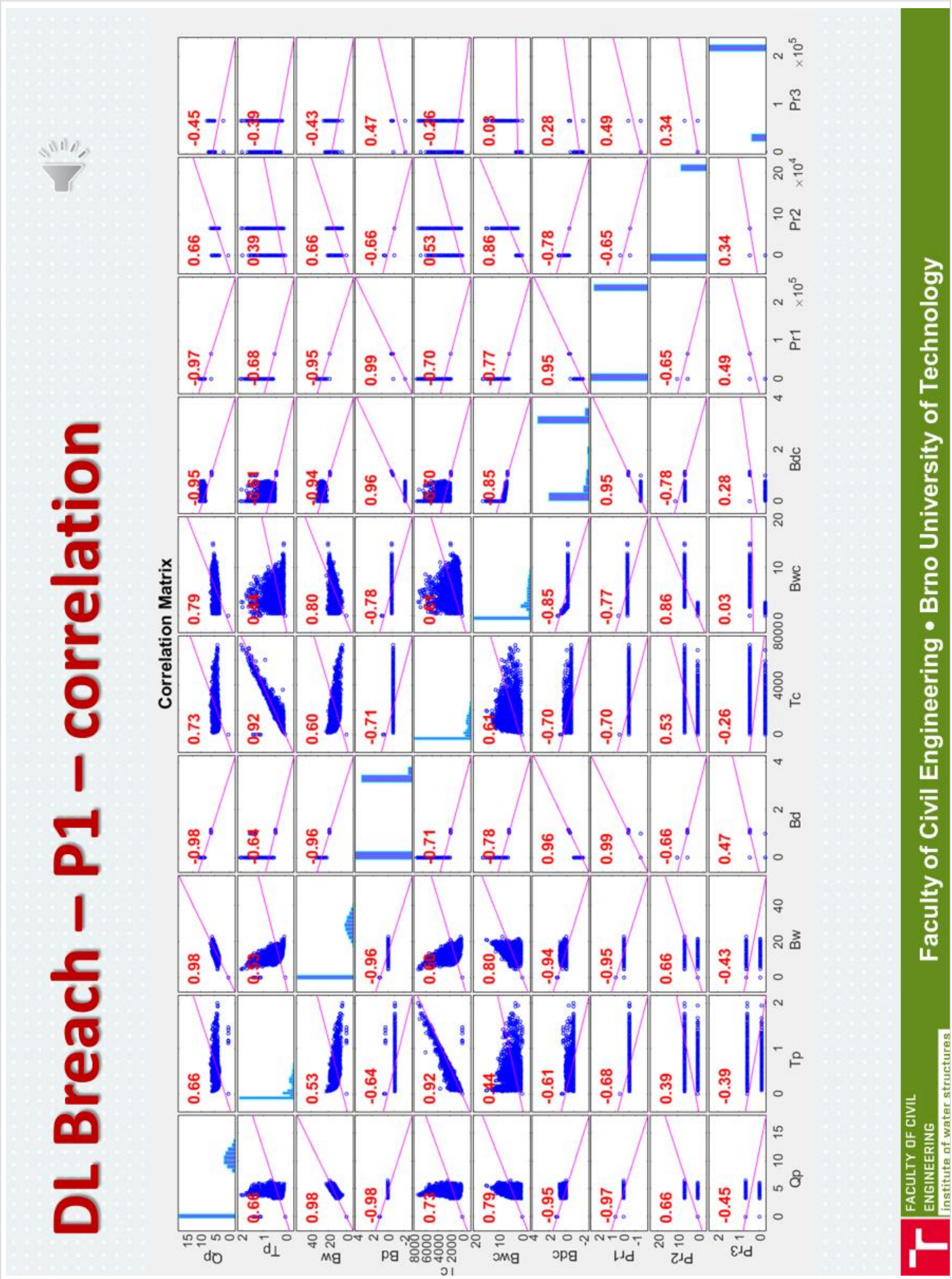
S13


DL Breach – P1 – correlation
Correlation Matrix
Qp
Tp
Bw
Bd
Tc
Bwc
Bdc
Pr1
Pr2
Pr3
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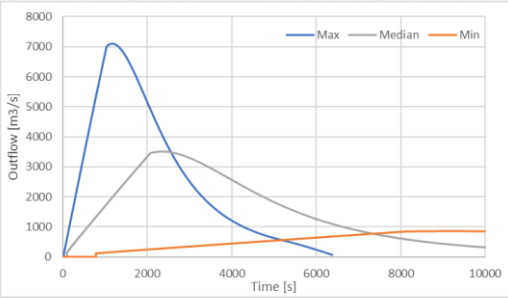
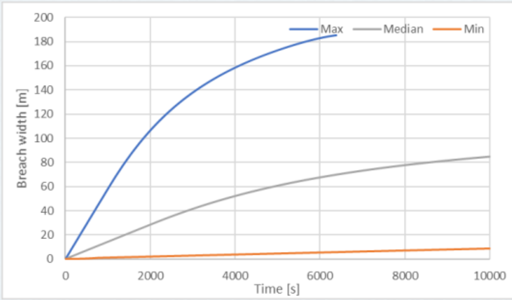
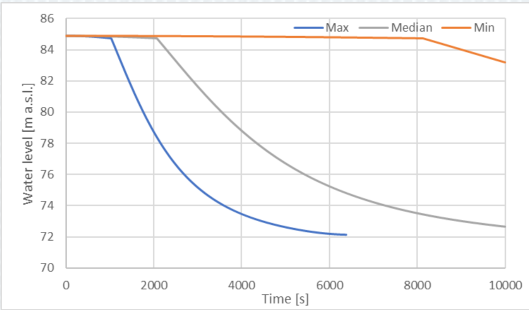
AREBA – Big Bay – max, med, min







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S15

AREBA – Big Bay – max, med, min

Observed

Parameter	units	Big Bay
Peak discharge (Qp)	m3/s	3313
Time to peak discharge (Tp)	secs	3300*
Average final breach width (Bw)	m	96.15
Final breach depth (Bd)	m	24.70
Time to pipe flow roof collapse (Tc)	secs	600*
Breach depth or vertical diameter before roof collapse (Bdbc)	m	0.46
Breach width or horizontal diameter before roof collapse (Bwbc)	m	0.46
Breach depth after roof collapse (Bdbc)	m	10m
Breach width after roof collapse (Bdac)	m	10m

* from time 12:15 on 12/3/2004

Calculated

- Inputs

Parameter	c	rho	D50	Tc	p	kd	fi	Hdam	Cw	IF	Mann
max	8.235	1913.515	0.157	1.879	0.251	63.612	31.988	86.848	12.100	72.833	0.029
min	9.462	1960.710	0.417	2.541	0.298	1.655	31.727	86.891	12.434	72.121	0.022
mean	12.879	2062.560	0.329	3.481	0.311	16.767	31.283	86.837	12.568	73.171	0.028

- outputs

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr1	Pr2	Pr3	Ln(Qp/Qpm)	Ln(Tp/Tpm)	Ln(Bw/Bwm	Ln(Tc/Tcm	Ln(Bwc/Bwcm)	Ln(Bdc/Bdcm)
max	7102.840	1169.000	185.232	15.548	19.000	0.881	72.474	1.445	4.663	4.882	0.582	1.077	0.430	11.920	5.899	3.923
min	861.230	9188.000	8.719	15.591	779.000	1.037	71.605	2.937	3.013	4.208	1.815	1.049	5.762	0.068	5.135	3.875
mean	3511.141	2348.000	84.769	15.537	93.000	1.451	72.481	0.368	3.335	3.355	0.003	0.116	0.016	3.476	3.725	3.923

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr1	Pr2	Pr3	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
Pr1	3328.77	2507	78.92	15.54	71	0.66	72.82	0.34	3.99	4.00	0.00002	0.08	0.04	4.56	7.41	3.94
Pr2	1484.89	5480	22.90	15.56	563	1.70	71.85	1.72	2.65	3.16	0.64	0.26	2.06	0.00	3.13	3.89
Pr3	2312.27	3524	46.68	15.55	216.00	1.70	72.38	0.81	2.85	2.96	0.13	0.00	0.52	1.04	3.14	3.92

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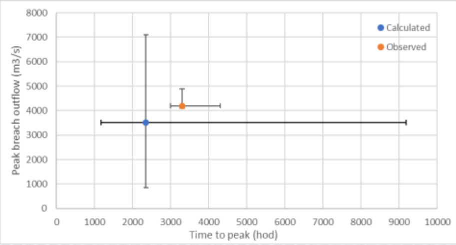
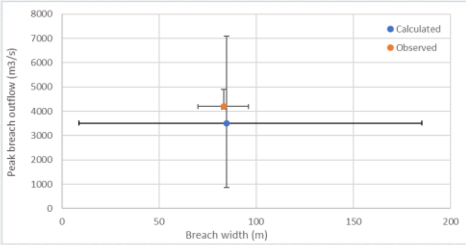
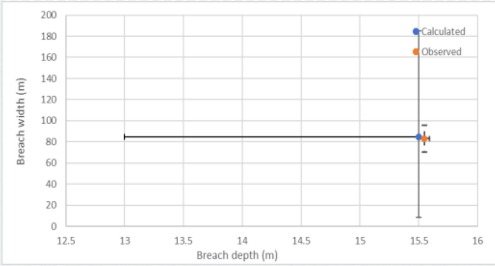
S16

AREBA – Big Bay – max, med, min







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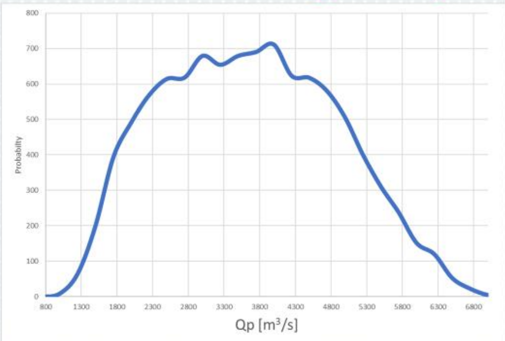
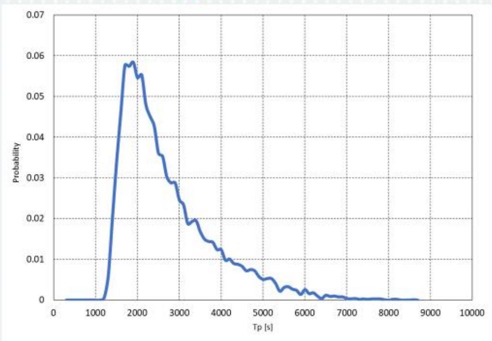
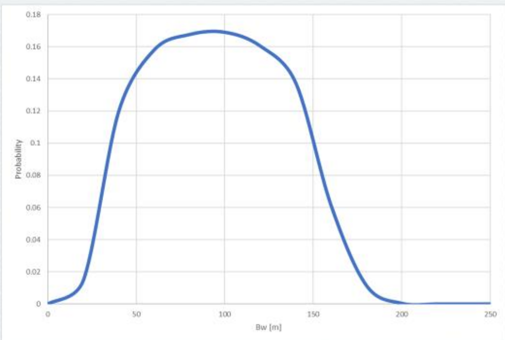
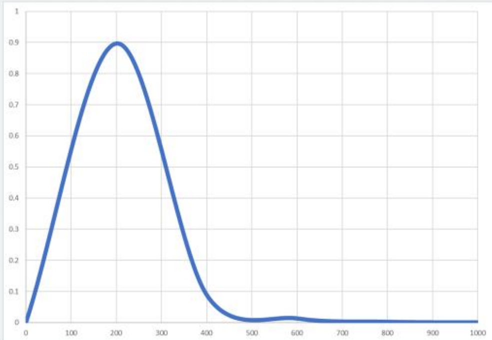
S17

AREBA – Big Bay – probability









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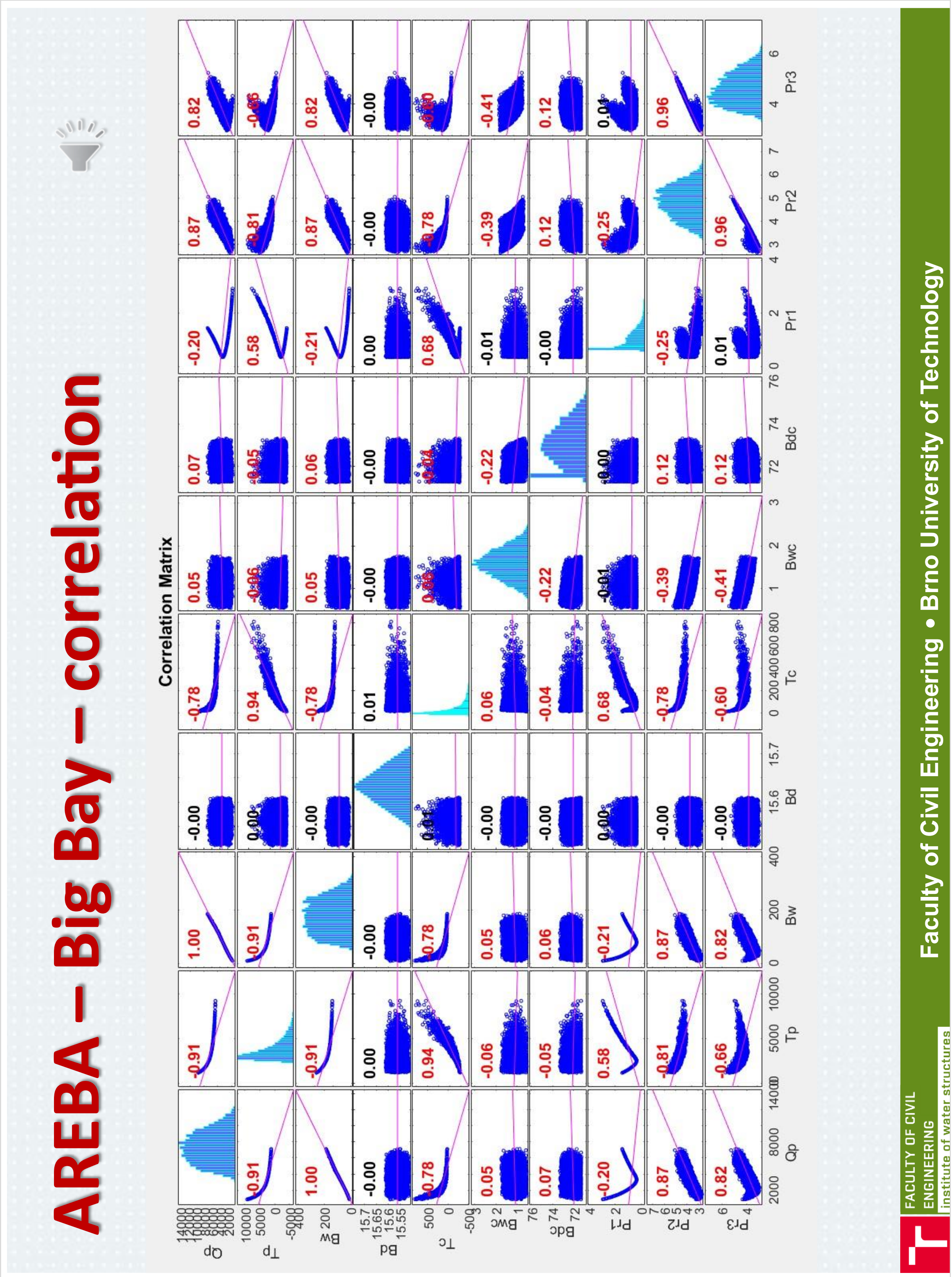
S18


AREBA – Big Bay – correlation
Correlation Matrix
Qp
Tp
Bw
Bd
Tc
Bwc
Bdc
Pr1
Pr2
Pr3
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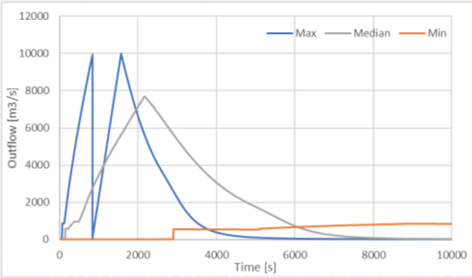
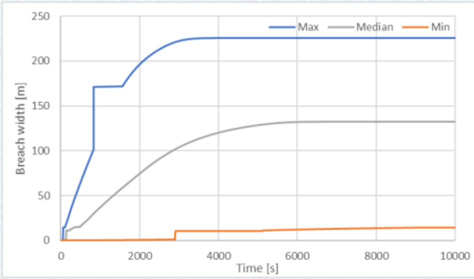
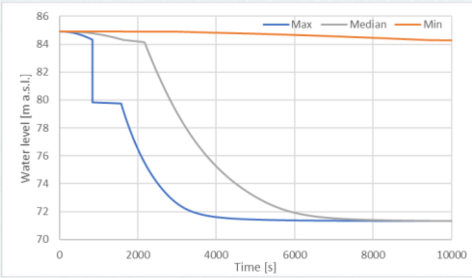
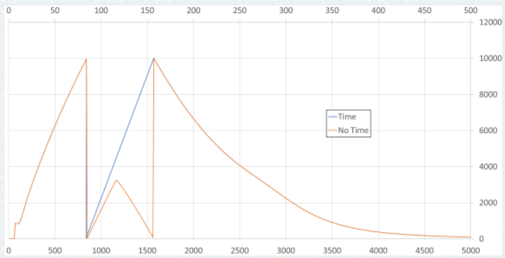
DL Breach – Big Bay – max, med, min









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S20

DL Breach – Big Bay – max, med, min

Observed

Parameter	units	Big Bay
Peak discharge (Qp)	m3/s	3313
Time to peak discharge (Tp)	secs	3300*
Average final breach width (Bw)	m	96.15
Final breach depth (Bd)	m	24.70
Time to pipe flow roof collapse (Tc)	secs	600*
Breach depth or vertical diameter before roof collapse (Bdbc)	m	0.46
Breach width or horizontal diameter before roof collapse (Bwbc)	m	0.46
Breach depth after roof collapse (Bdbc)	m	10m
Breach width after roof collapse (Bdac)	m	10m

* from time 12:15 on 12/3/2004

Calculated

- **Inputs**

Parameter	c	rho	D50	Tc	p	kd	fi	Hdam	Cw	if	Mann
max	8838.294	2510.013	0.479	2.867	0.277	64.263	0.592	15.536	12.373	13.950	0.028
min	13590.213	2596.280	0.331	2.808	0.306	1.823	0.658	15.570	11.925	13.893	0.018
mean	13302.207	2567.222	0.393	3.934	0.269	35.945	0.629	15.559	12.088	13.928	0.023

- **outputs**

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr	Log_Pr	Ln(Qp/Qpm)	Ln(Tp/Tpm)	Ln(Bw/Bwm)
max	9999.892	0.436	225.789	-2.500	59.508	0.683	1.295	3.638	1.547	0.753	0.643	0.997
min	847.509	2.486	14.356	0.000	2899.512	1.159	1.147	2.571	2.555	2.562	0.881	3.087
mean	7691.423	0.603	132.564	-2.500	139.500	1.039	1.161	2.505	0.901	0.366	0.229	0.217

Parameter	Qp	Tp	Bw	Bd	Tc	Bwc	Bdc	Pr1	Pr2	Pr3	Ln(Qp/Qpm)^2	Ln(Tp/Tpm)^2	Ln(Bw/Bwm)^2	Ln(Tc/Tcm)^2	Ln(Bwc/Bwcm)^2	Ln(Bdc/Bdcm)^2
Pr1	2120.13	9780	30.56	-2.50	679.50	0.50	1.39	7.21	1.12	7.29	0.20	50.44	1.31	0.02	0.01	1.23
Pr2	2909.53	5650	46.48	-2.50	719.50	1.11	-0.47	7.69	0.03	7.07	0.02	58.54	0.53	0.03	0.77	-9.87
Pr3	2257.32	7620	33.20	-2.50	1269.50	1.06	-0.12	7.44	1.43	6.99	0.15	54.05	1.13	0.56	0.70	-8.04

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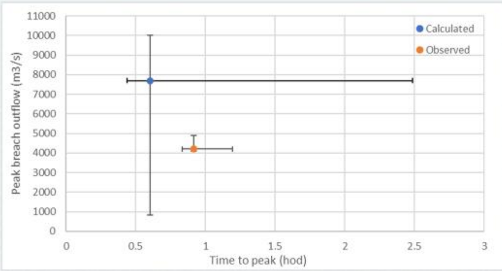
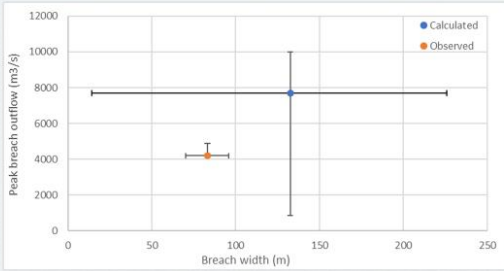
S21

DL Breach – Big Bay – max, med, min





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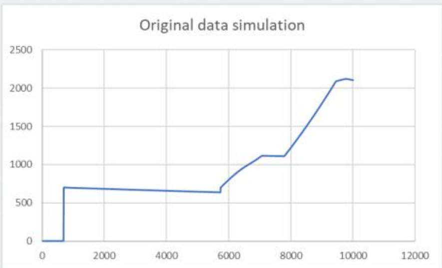
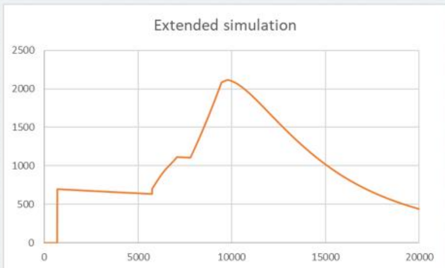
S22

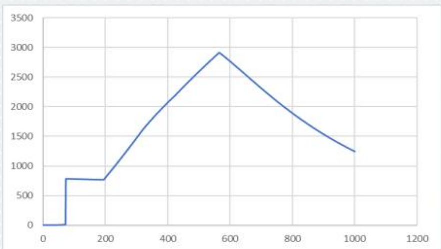
DL Breach – Big Bay – Pr1, Pr2, Pr3

Pr1

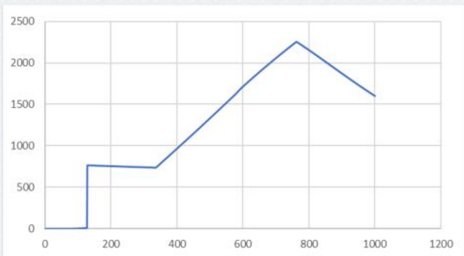




Pr2



Pr3



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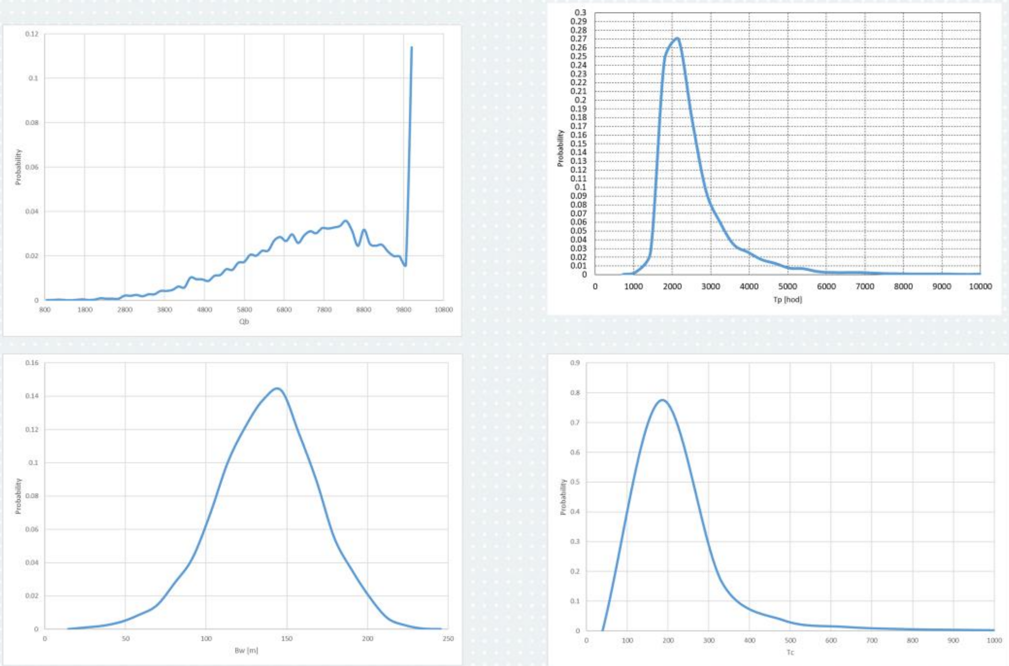
S23

DL Breach – Big Bay – probability



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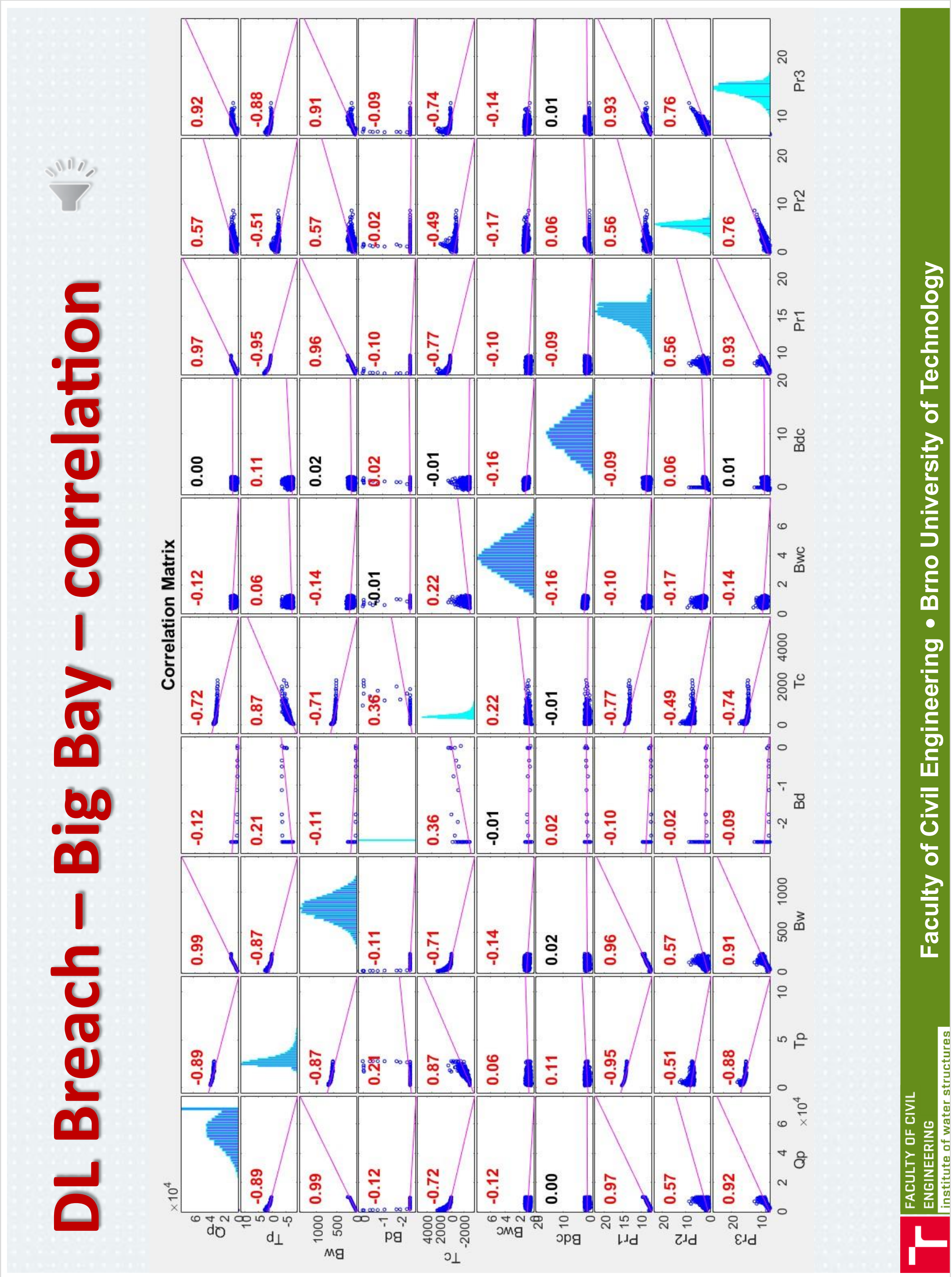
S24


DL Breach – Big Bay – correlation
Correlation Matrix
Qp
Tp
Bw
Bd
Tc
Bwc
Bdc
Pr1
Pr2
Pr3
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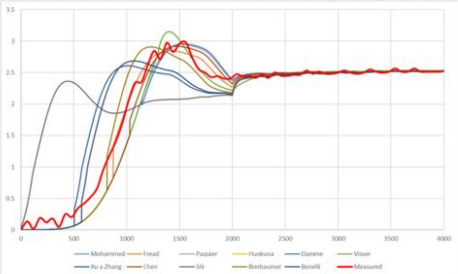
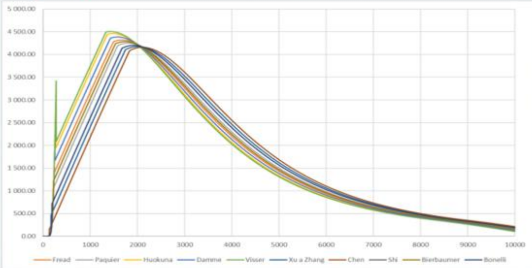
Conclusions – Roof collapse (prelinear results)

P1



Big Bay



Appendix 1 – Validation tests with dam and soil properties and piping breach characteristics

P1												
Models	**Measured**	**Fread**	**Paquier**	**Damme**	**This study**	**Visser**	**Xu and Zhang**	**Huokuna**	**Bierbaumer**	**Mohammed**	**Chen**	**Bonelli**
Equations	-	**Eq. 1**	**Eq. 2**	**Eq. 3**	**Eq. 5**	**Eq. 4**	**Eq. 26**	**Eq. 6**	**Eq. 8- 14**	**Eq. 15 - 19**	**Eq. 20-21**	**Eq. 7**
Time of collapse t_p [s]	840	872	1111	804	1143	7420	505	-	814	-	1029	573
Pipe diameter D [m]	1.28	0.61	0.92	0.52	0.96	1.84	0.21	-	0.54	-	0.8163	0.26
Diameter %	-	52.08	28.13	59.38	24.93	43.75	83.98	-	57.81	-	36.23	79.34
Time %	-	3.81	32.26	4.29	36.07	783.33	39.88	-	3.10	-	22.50	31.79
***P_r* [-]**	-	1.85	1.48	2.01	**1.45**	2.35	2.98	-	1.98	-	**1.58**	2.71
Big Bay												
Models	**Measured**	**Fread**	**Paquier**	**Damme**	**This study**	**Visser**	**Xu a Zhang**	**Huokuna**	**Bierbaumer**	**Mohammed**	**Chen**	**Bonelli**
Equations	-	**Eq. 1**	**Eq. 2**	**Eq. 3**	**Eq. 5**	**Eq. 4**		**Eq. 5**	**Eq. 7 - 13**	**Eq. 14 - 18**	**Eq. 19-20**	**Eq. 6**
Time of collapse t_p [s]	**600**	236	213	223	253	279	170	270	225	-	132	190
Pipe diameter D [m]	**10**	11.69	8.73	10	13.96	17.48	3.8824	16.26	10.26	-	1.223	5.97
Diameter %	-	16.90	12.70	0.00	39.60	74.80	61.18	62.60	2.60	-	87.77	40.30
Time %	-	60.67	64.50	62.83	57.83	53.50	71.67	55.00	62.50	-	78.00	68.33
***P_r* [-]**	-	0.95	1.04	**0.99**	**0.93**	0.95	1.58	0.93	**0.98**	-	2.59	1.26

Conclusions

- better representation of downstream condition – new method?

- better representation of the time of peak outflow and value of peak - new method for describing roof collapse

- better representation of the breach widening during overtopping - new method for description Manning´s n

L Phase 3 – KSU – Big Bay & P1 Modelling Results

The following pages comprise the PPT slides with results plots from Antony Atkinson & Mitchell Neilsen.

File = Big Bay Dam and ARS-P1 Uncertainty Analysis.pptx

Uncertainty Experiments in Dakota for DLBreach and WinDAM C

Anthony Atkinson

Mitchell Neilsen

KANSAS STATE UNIVERSITY

S1

Uncertainty Methodology

- Experiment
  - All applicable parameters are used resulting in different parameter sets for each scenario/model pair
  - Parameters tested in subsets to allow for greater resolution
  - Subsets arranged to group potentially related parameters
  - Use Sandia Lab's Dakota program to run the experiment
- Analysis
  - Comparison of results of three performance functions
  - Formulation of probability density curves
  - Post hoc analysis with R and python

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S2

Uncertainty Methodology (cont.)

Big Bay				USDA-ARS P1			
DLBreach		WinDAM C		DLBreach		WinDAM C	
Parameter (units)	Range (default)	Parameter (units)	Range (default)	Parameter (units)	Range (default)	Parameter (units)	Range (default)
Cohesion (Pa)	5,000-15,000 (10,000)			Cohesion (Pa)	4000-5250 (5000)	Shear Strength (lb/ft^2)	229-314 (272)
Critical Shear Stress (Pa)	1.0-5.0 (1.5)	Critical Shear Stress (Pa)	1.0-5.0 (2.5)	Critical Shear Stress (Pa)	0.0-0.16 (0.144)	Critical Shear Stress (Pa)	0.0-0.16 (0.144)
Soil Diameter (m)	0.0001-0.0006 (0.0003)	Total Unit Weight (kN/m^3)	18-20 (19)	Soil Diameter (m)	0.0001-0.00015 (0.00013)	Total Unit Weight (kN/m^3)	17.26-20.5 (18.73)
Erodibility (cm^3/(N·s))	1.5-66 (33)	Erodibility (cm^3/(N·s))	1.5-66 (33)	Erodibility (cm^3/(N·s))	23-270 (120)	Erodibility (cm^3/(N·s))	23-270 (120)
Friction Angle Tangent	0.577-0.675 (0.6)			Friction Angle Tangent	0.577-0.675 (0.625)		
Porosity	0.235-0.349 (0.297)			Porosity	0.33-0.45 (0.34)		
Manning's N	0.016-0.035 (0.025)	Manning's N	0.016-0.035 (0.025)	Manning's N	0.016-0.033 (0.025)	Manning's N	0.016-0.033 (0.025)
		Breach Height (m)	0.0-0.6 (0.3)	Breach Depth from Top (m)	0.9-1.1 (1.0)	Breach Height (m)	0.2-0.4 (0.3)
Breach Diameter (m)	0.01-0.05 (0.013)	Breach Diameter (m)	0.01-0.05 (0.013)	Clay Content (%)	0.6-0.8 (0.7)		
Dam Height (m)	15.51-15.61 (15.56)	Dam Height (m)	15.5-15.6 (15.56)	Dam Height (m)	1.2-1.4 (1.3)	Dam Height (m)	1.2-1.4 (1.3)
Dam Crest Width (m)	11.6-12.8 (12.2)	Dam Crest Width (m)	11.6-12.8 (12.2)	Dam Crest Width (m)	1.78-2.18 (1.98)	Dam Crest Width (m)	1.78-2.18 (1.98)
Reservoir Volume (m^3)	13870000.0-17284449.6 (15577224.8)	Resevoir Volume (m^3)	13870000.0-17284449.6 (15577224.8)	Surface Elevation (m)	0.821-1.003 (0.912)	Surface Elevation (ft)	2.694-3.291 (2.992)

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S3

Uncertainty Methodology (cont.)

Parameter groupings

Big Bay	DLBreach	•cohesion, erodibility, critical shear stress •soil diameter, porosity, friction angle tangent •dam height, crest width, reservoir volume •breach width, Manning's n, reservoir volume
	WinDAM C	•breach width, breach height, reservoir volume •Manning's n, dam height, crest width •total unit weight, erodibility, critical shear stress •all parameters
USDA-ARS P1	DLBreach	•cohesion, erodibility, critical shear stress •soil diameter, porosity, friction angle tangent •surface elevation, breach elevation from crest, Manning's n •clay content, dam height, crest width
	WinDAM C	•total unit weight, erodibility, critical shear stress •surface elevation, breach height, Manning's n •shear strength, dam height, crest width •all parameters

Real-world results

	Peak flow rate (m^3/s)	Time to peak flow (s)	Max breach width (m)	Max breach depth (m)	Time to collapse (s)	Breach width before collapse (m)	Breach width after collapse (m)	Breach depth before collapse (m)	Breach depth after collapse (m)
Big Bay	3313	3300	96.15	24.7	600	0.46	10.0	0.46	10.0
P1	2.979	1560	6.46	1.25	840	1.28	1.28	0.61	1.2

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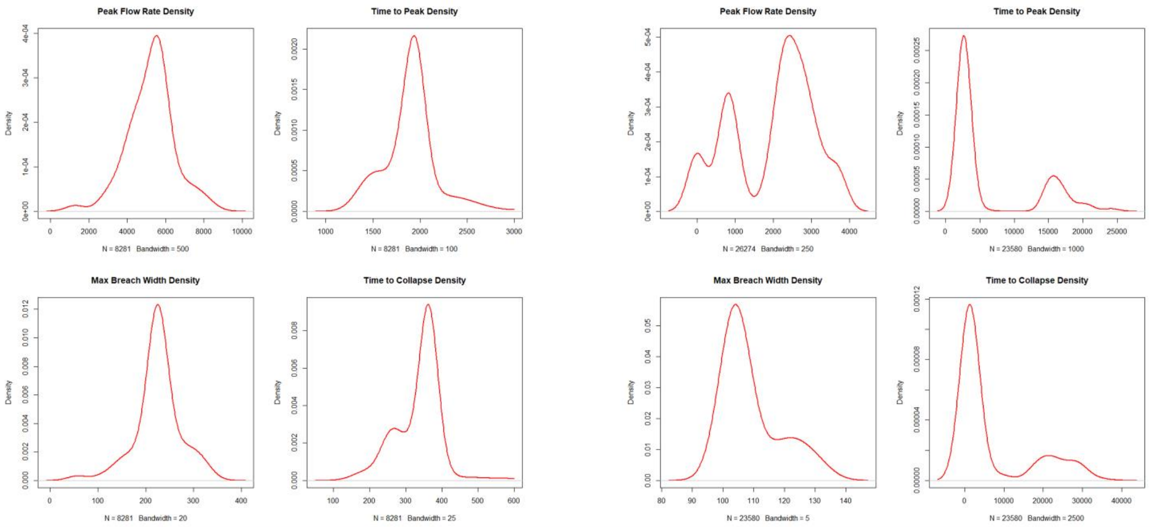
S4

Probability Densities

Big Bay DLBreach

Big Bay WinDAM C



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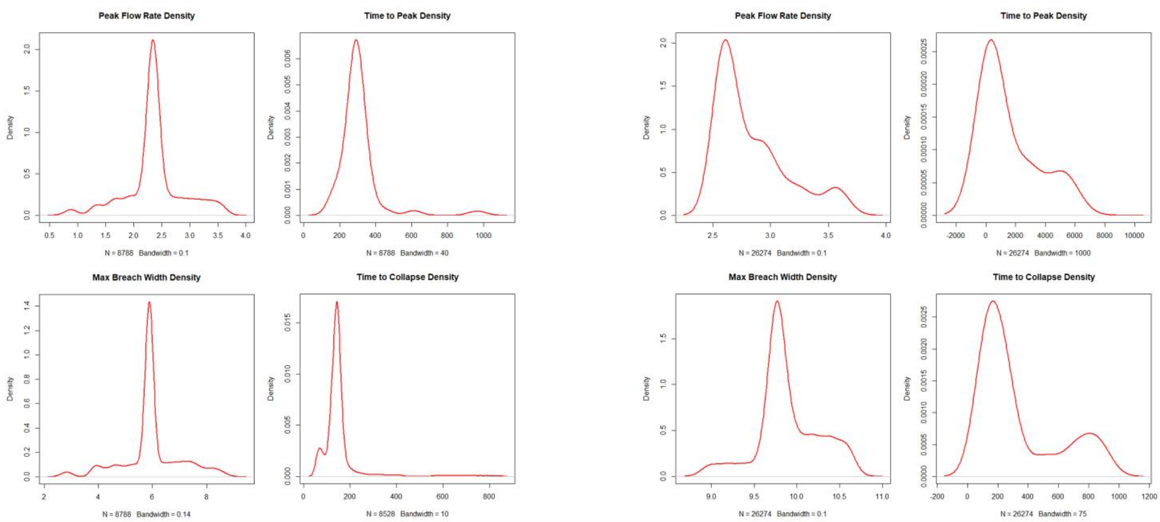
S5

Probability Densities (cont.)

ARS-P1 DLBreach

ARS-P1 WinDAM C



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S6

Probability Statistics

Uncertainty quantization

Big Bay	DLBreach				WinDAM C			
	Mean	Median	Mode	SD	Mean	Median	Mode	SD
Peak flow rate (m³/s)	5240	5553	5667	1198	1973	2359	2348	1091
Time to peak (s)	2150	1945	1962	1556	6278	2880	2662	6293
Max breach width (m)	226	224	222	46	109	105	104	9
Time to collapse (s)	498	360	354	1012	6715	1440	1251	9753
ARS-P1	**DLBreach**				**WinDAM C**			
	Mean	Median	Mode	SD	Mean	Median	Mode	SD
Peak flow rate (m³/s)	2.37	2.35	2.35	0.5	2.84	2.69	2.59	0.31
Time to peak (s)	305	290	291	115	1554	396	327	1874
Max breach width (m)	5.96	5.91	5.90	1.04	10.96	9.78	9.77	3.8
Time to collapse (s)	152	140	145	90	326	216	147	261

Percent error of real-world values

Big Bay	DLBreach				WinDAM C			
	Mean	Median	Mode	Average	Mean	Median	Mode	Average
Peak flow rate	58.16	67.61	71.05	65.61	40.45	28.80	29.13	32.79
Time to peak	34.85	41.06	40.55	38.82	90.24	12.73	19.33	40.77
Max breach width	135.05	132.97	130.89	132.97	13.36	9.20	8.16	10.24
Time to collapse	17.00	40.00	41.00	32.67	1019.2	140.00	108.50	422.56
Average	61.27	70.41	70.87	67.52	290.81	47.68	41.28	126.59
ARS-P1	**DLBreach**				**WinDAM C**			
	Mean	Median	Mode	Average	Mean	Median	Mode	Average
Peak flow rate	20.44	21.11	21.11	20.89	4.67	9.70	13.06	9.14
Time to peak	80.45	81.41	81.35	81.07	0.38	74.62	79.04	51.35
Max breach width	7.74	8.51	8.69	8.31	69.66	51.39	51.24	57.43
Time to collapse	81.90	83.33	82.74	82.66	61.19	74.29	82.50	72.66
Average	47.63	48.59	48.47	48.23	33.98	52.50	56.46	47.65

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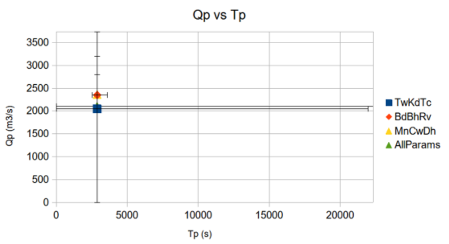
S7

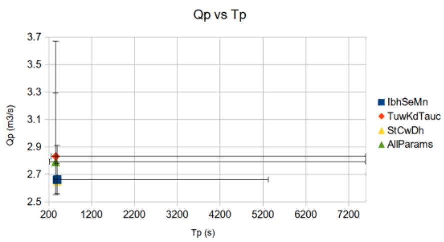
Peak Flow vs Time to Peak Flow

Big Bay

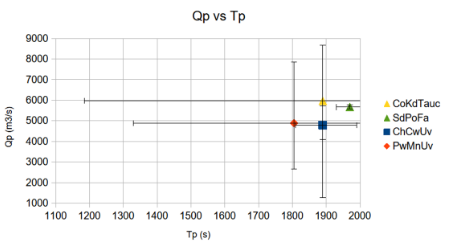
ARS-P1

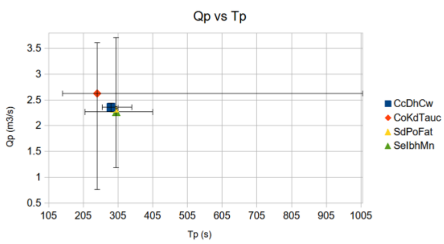
WinDAM C





DLBreach





	Qp	Tp
Big Bay	3313	3300
P1	2.979	1560

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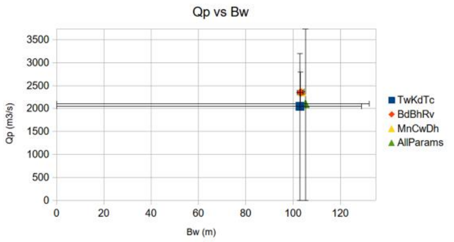
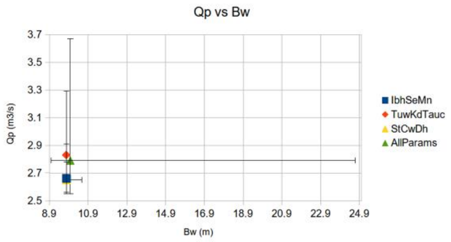
Peak Flow vs Max Breach Width

Big Bay

ARS-P1

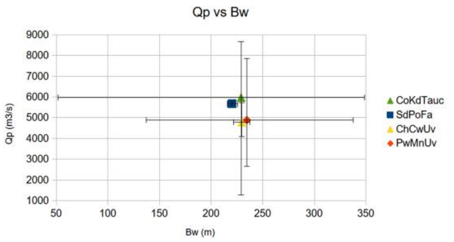
WinDAM C

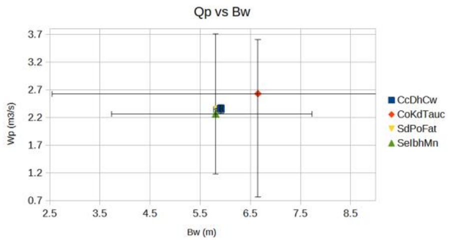




	Qp	Bw
Big Bay	3313	96.15
P1	2.979	6.46

DLBreach





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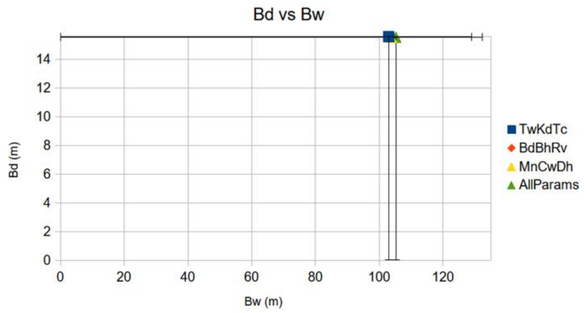
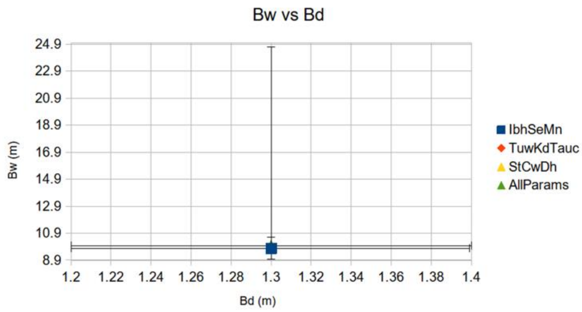
S9

Max Breach Depth vs Max Breach Width

Big Bay

ARS-P1





	Bw	Bd
Big Bay	96.15	24.7
P1	6.46	1.25

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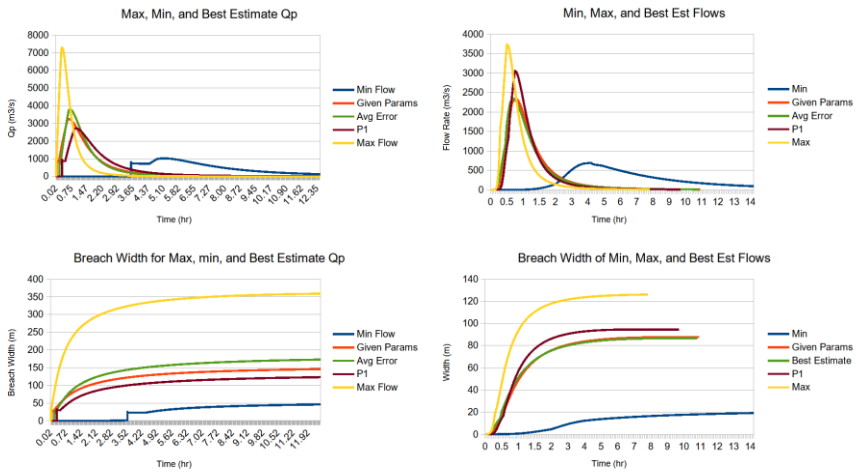
S10

Over-time Progression

Big Bay



DLBreach WinDAM C

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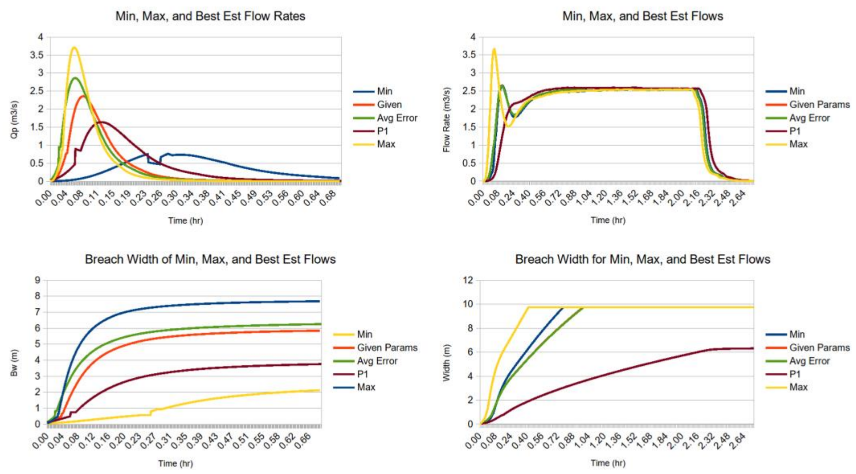
S11

Over-time Progression (cont.)

ARS-P1



DLBreach WinDAM C

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S12

Optimization Methodology

- Results from the uncertainty analysis can be post processed for optimization using Python
  - Scikit-learn XGBoost regression model to approximate model behavior
  - SciPy dual-annealing technique to find global optimum
  - Full parameter runs give better results than many runs of subsets of parameters
- Dakota also has options for performing optimization experiments

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S13

Optimization Results

Big Bay WinDAM C

Inputs			Outputs			
	Average error	P1		Given	Average error	P1
Manning's N	0.025	0.025	Peak flow rate	2345.48	2336.14	3058.33
Crest width	11.6	11.6	Time to peak	2880	2880	3240
Dam height	15.56	15.5	Breach width	103.068	101.553	103.062
Breach Diameter	0.028	0.01	Breach depth	15.56	15.56	15.499
Breach height	0.6	0.1	Time to collapse	2160	180	2520
Reservoir volume	15577224.206	13880000.0	Width before	23.287	20.876	23.909
Unit Weight	19.0	19.0	Width after	27.066	24.594	28.612
K_d	32.0	41.125	Depth before	11.948	11.052	12.061
τ_c	2.5	3.125	Depth after	15.56	15.56	15.499
			Average error	8.92	8.03	914.96
			P1 metrics error	16.38	15.94	5.55
			P1 value	0.3777	0.3789	0.1075

Big Bay DLBreach

Inputs			Outputs			
	Average error	P1		Given	Average error	P1
Cohesion	10015.5	5000	Peak flow	4901.434	3785.274	2719.279
K_d	19.625	19.625	Time to peak	1769.76	2474.748	3494.736
τ_c	2.0	2.0	Breach width	237.0434	178.1826	127.5885
Reservoir volume	15577224.21	15577224.21	Time to collapse	319.752	634.752	1084.752
Breach diameter	0.01	0.013	Width before	0.7976	0.8366	0.4226
Manning's N	0.025	0.025	Width after	27.3358	24.0642	29.6755
Dam height	15.56	15.61	Average error	89.05	58.81	57.03
Crest width	11.6	11.6	P1 metrics error	80.82	41.53	18.82
Sediment diameter	0.0003	0.0003	P1	1.1644	0.6940	0.3497
Porosity	0.349	0.349				
Angle tangent	0.675	0.675				

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Optimization Results (cont.)

ARS-P1 WinDAM C

Inputs			Outputs			
	Average error	P1		Given	Average error	P1
Shear strength	28354.244	26000	Peak flow rate	2.65	2.61	2.59
Crest width	1.58	1.78	Time to peak	432	2700	2736
Dam height	1.224	1.2	Breach width	9.781	8.925	8.979
Unit weight	19.025103	17.26	Breach depth	1.3	1.224	1.2
K_d	23.4	23	Time to collapse	324	828	504
τ_c	0.0931648	0.0	Width before	1.57	1.088	0.826
Breach height	0.33	0.4	Width after	1.771	1.134	0.881
Surface elevation	0.775	1.003	Depth before	0.594	0.689	0.786
Manning's N	0.02637102	0.033	Depth after	1.3	1.224	1.2
			Average error	30.24	18.72	29.66
			P1 metrics error	44.92	41.21	42.48
			P1 value	1.354	0.650	0.666

ARS-P1 DLBreach

Inputs			Outputs			
	Average error	P1		Given	Average error	P1
Surface elevation	1.003	1.003	Peak flow	2.3569	2.86134	1.64018
Breach height	1.0	1.0	Time to peak	284.76	219.744	444.744
Dam height	1.4	1.4	Breach width	5.9142	6.4595	4.0816
Crest width	1.78	2.18	Time to collapse	139.752	79.74	219.744
Manning's N	0.025	0.025	Width before	0.4943	0.4787	0.4871
Clay Content	0.07	0.07	Width after	0.6552	0.8183	0.7453
Sediment diameter	0.00013	0.00013	Average error	50.77	46.51	55.13
Porosity	0.33	0.45	P1 metrics error	37.03	29.96	51.08
Angle tangent	0.577	0.577	P1	1.719	1.960	1.464
Cohesion	5250	4835				
K_d	121.4	44.265				
τ_c	0.016	0.0				

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S15

Future Work

- Run sensitivity analysis first to filter out parameters with little influence
- Use same set of parameters for all scenario/model pairs for better comparisons
- Do more full parameter runs

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M Phase 3 – Geosyntec – Big Bay Modelling Results

Geosyntec undertook modelling analysis of the Big Bay case study using WinDAM C – adapted for MC analyses using Python.

In setting up the runs:

- Made relatively minor changes to geometry from Phase 2, based on additional information
- Held dam geometry fixed:
  - Anticipate only minimal impacts due to relative certainty.
- Held headwater elevation fixed:
  - Important parameter, but relatively known.
- Varied k_d, T_c, and initial conduit elevation
- Ran 10,000 realizations
- Tracked Q_p, T_p, B_w:
  - Will need additional scripting/post-processing to track other variables.
- Processed and saved 10,000 output files.

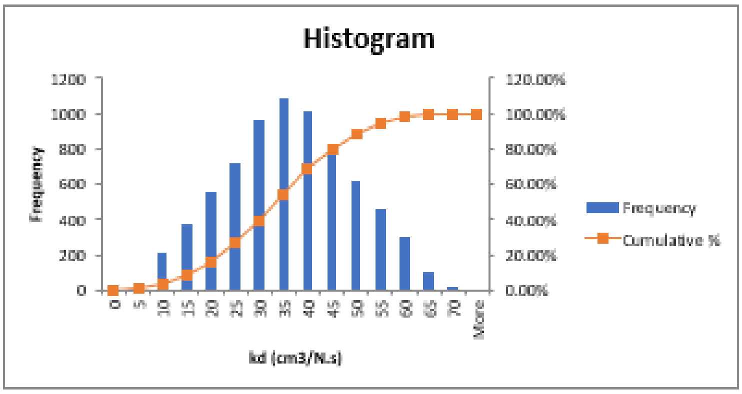
The input distributions - as defined in the specification - are shown in Figure M.1.

In analysing the top 20 runs (based upon lowest PR1 values) it was noted that:

- Best runs all have k_d between ~41 and ~46 cm^3/N.s
- Most have Tc close to ~3 Pa (i.e., on the high end of the distribution)
- Most have initial conduit elevation on the low end of the distribution.

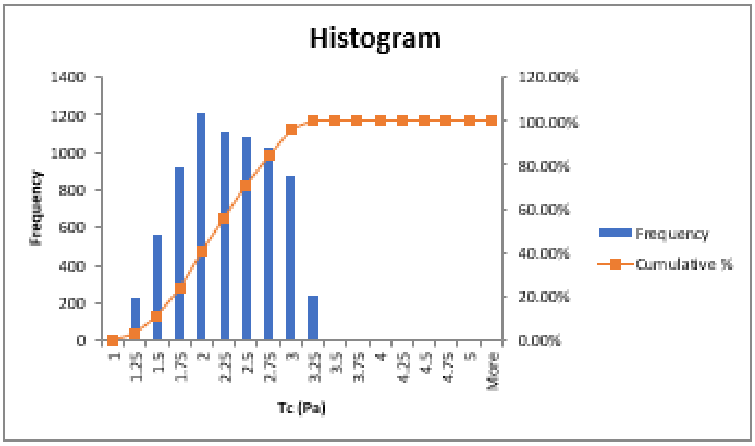
Histograms of outputs can be seen in Figure M.2 and plots shown outflow and breach width in Figure M.3.

An analysis of parameter correlations was also undertaken – Figure M.4. The main focus here is on the correlations with the first 3 column parameters (since others are outputs). Dependence of results on K_d is shown to be very strong; Tc also affects T_p, albeit to a 'secondary' level.



Triangular (symmetric)

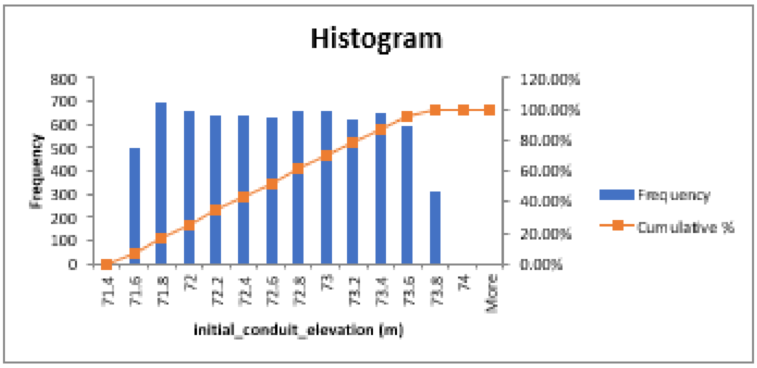
min	1.5	cm3/N.s
max	66	cm3/N.s
peak	33.75	cm3/N.s



Triangular (skewed)

min	1	Pa
max	5	Pa
peak	2	Pa

Note: ~25% of simulations with Tc > ~3.3 Pa did not result in erosion.
These were removed from the results



Uniform

min	71.4	m
max	73.7	m

Figure M.1: Phase 3: Big Bay – Geosyntec: Input distributions

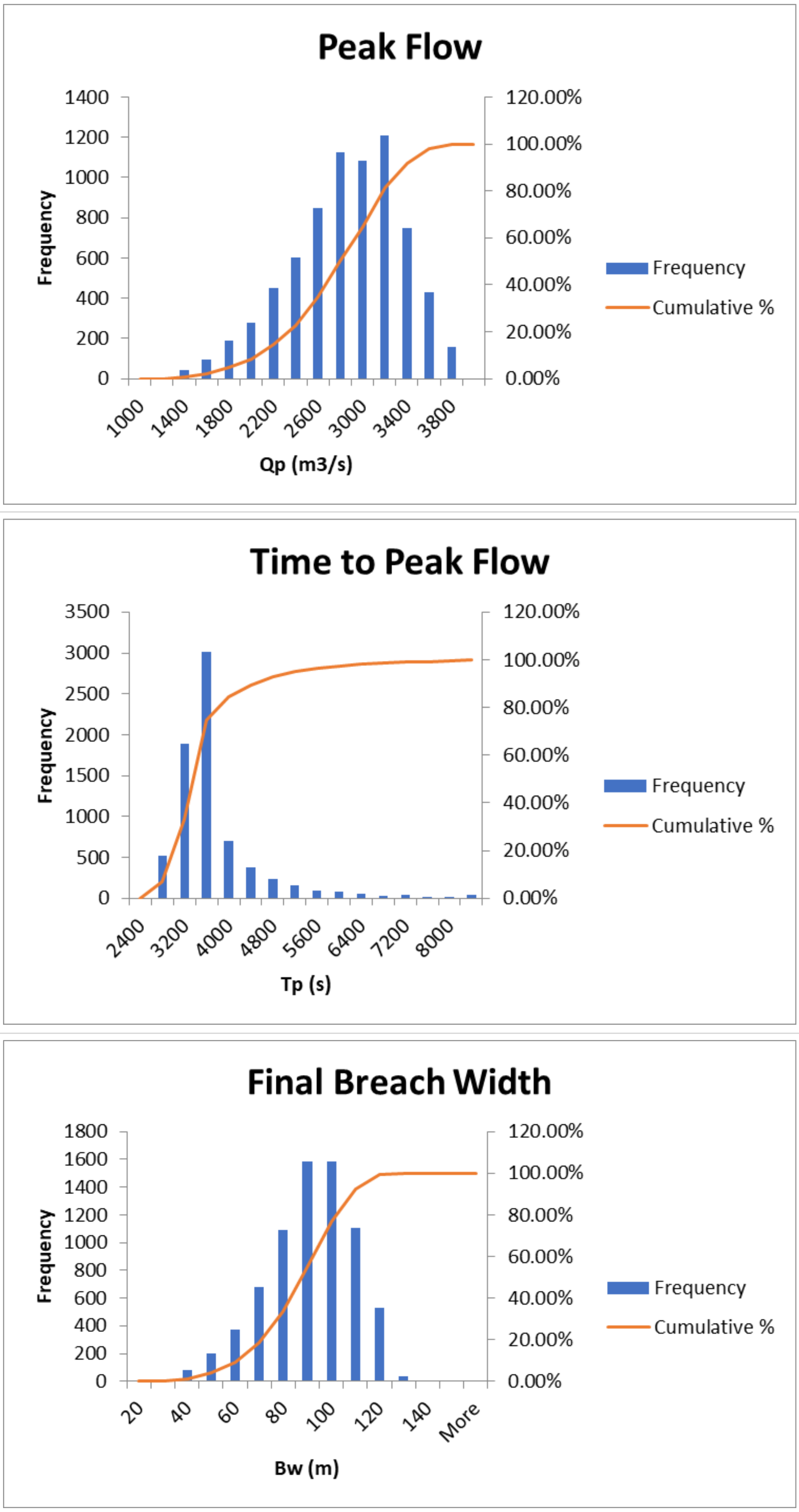


Figure M.2: Phase 3: Big Bay – Geosyntec: Histograms of outputs

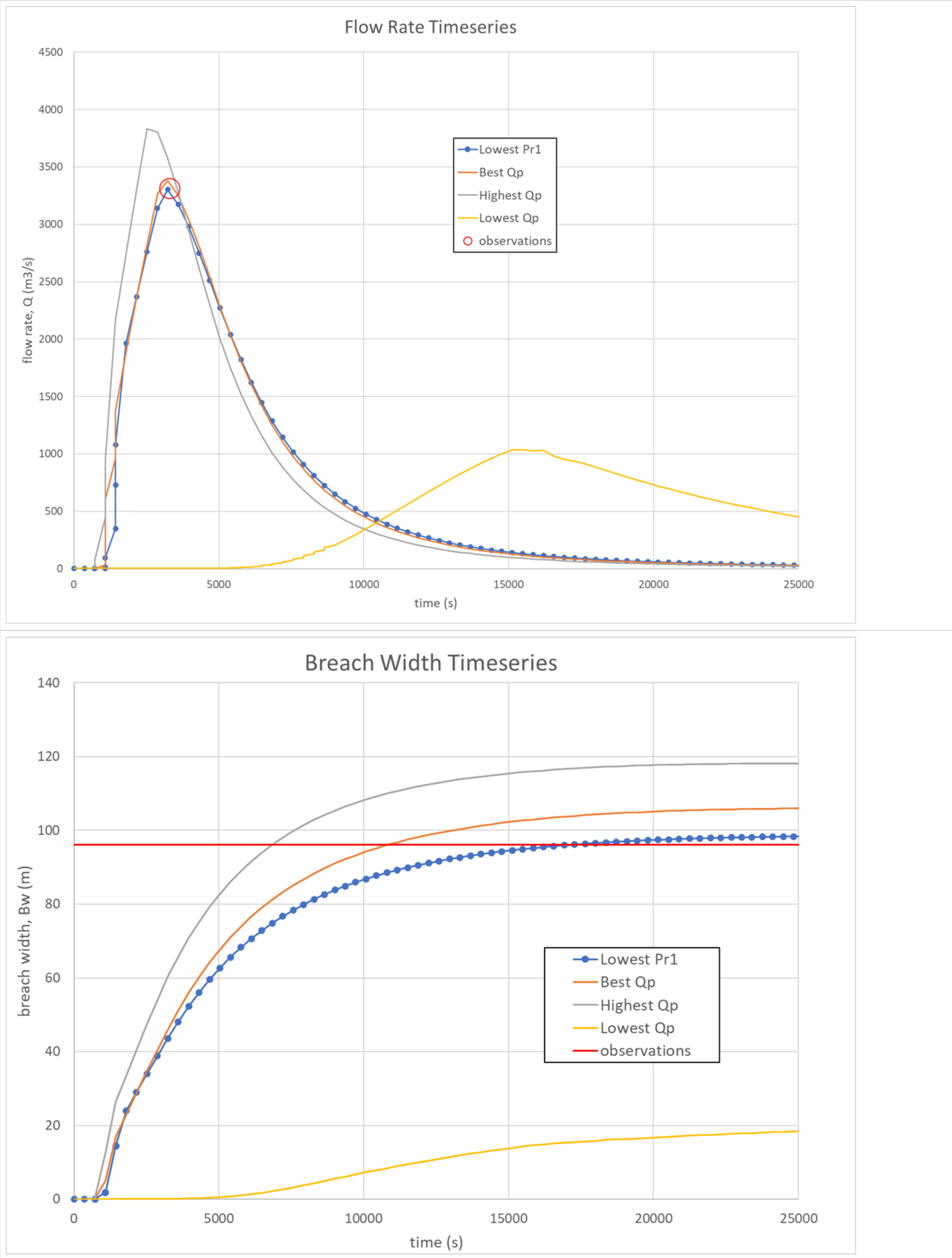


Figure M.3: Phase 3: Big Bay – Geosyntec: outflow and breach width predictions

	kd (cm3/N.s)	*Tc (Pa)*	*initial_conduit_elevation (m)*	*Qp (m3/s)*	*Tp (s)*	*Bw (m)*	*Qp/Qpm*	*Tp/Tpm*	*Bw/Bwm*	*ln(Qp/Qm)*	*ln(Tp/Tm)*	*ln(Bw/Bwm)*	*Pr1*
kd (cm3/N.s)	1												
Tc (Pa)	-0.00519	1											
initial_conduit_elevation (m)	0.00401	-0.02242	1										
Qp (m3/s)	0.987629	-0.00956	-0.05007	1									
Tp (s)	-0.80828	0.164798	0.037523	-0.87027	1								
Bw (m)	0.988387	-0.06044	0.005758	0.996023	-0.87627	1							
Qp/Qpm	0.987629	-0.00956	-0.05007	1	-0.87027	0.996023	1						
Tp/Tpm	-0.80828	0.164798	0.037523	-0.87027	1	-0.87627	-0.87027	1					
Bw/Bwm	0.988387	-0.06044	0.005758	0.996023	-0.87627	1	0.996023	-0.87627	1				
ln(Qp/Qm)	0.960526	-0.00685	-0.0483	0.990687	-0.91646	0.987425	0.990687	-0.91646	0.987425	1			
ln(Tp/Tm)	-0.89057	0.201943	0.050718	-0.93441	0.972899	-0.93993	-0.93441	0.972899	-0.93993	-0.95628	1		
ln(Bw/Bwm)	0.951716	-0.05084	0.007066	0.981243	-0.93256	0.985583	0.981243	-0.93256	0.985583	0.995514	-0.96419	1	
Pr1	-0.75083	-0.00681	0.010914	-0.83137	0.913998	-0.8308	-0.83137	0.913998	-0.8308	-0.89338	0.88287	-0.90412	1

Figure M.4: Phase 3: Big Bay – Geosyntec: Parameter correlation analysis hr wallingford

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