

Numerical Modelling of Dam Breaching

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Abstract

Understanding dam breaching is essential because of the catastrophic consequences that can follow a dam failure. Such events are rare; however, when they do occur the downstream area is usually not sufficiently prepared to manage the flood wave that is generated by a sudden dam breach. Engineers have realized that trying to prevent the flood from happening after the dam has breached is a very difficult issue. Therefore, the focus of engineers is often directed more towards the flood management aspect where the purpose is to protect the riverine population. This is performed by identifying the different ways the structure can fail, by calculating the consequences of each type of failure, and by evaluating the risk associated with that failure type. This risk is then reduced by employing effective emergency evacuation planning or by relocating the population that would be possibly affected by the dam failure. Therefore, it is evident that an accurate breach analysis must be conducted to predict how the flood will propagate downstream.

Until recently, research has been scarce in the field of physical modelling of dam breaching. Over the past few years, teams from the University of Ottawa, Canada, Delft University of Technology, Netherlands, and HR Wallingford, United Kingdom have worked on several physical models to help determine how various dam breaching characteristics vary due to changes in dam geometry and geotechnical properties. The purpose of this project is to use these new experimental data sets to compare and validate the applicability range of two existing pieces of software, MIKE11-DB and BREACH developed by the Danish Hydraulic Institute and National Weather Service, respectively. Several breaching characteristics such as the outflow hydrograph, peak flow, lag time, breaching time, breach width, and water level are considered in the present study. A sensitivity analysis is also performed on the model's main input parameters and their sensitivity and performance is ranked accordingly.

From the simulations run in both MIKE11 and BREACH the following conclusions have been made:

- I. BREACH performed poorly in modelling tests that have been conducted at laboratory scale compared to in the field.
- II. MIKE11 was able to reproduce the results of laboratory tests with better accuracy than BREACH. Several conclusions were drawn:
 - a. The general shape of the outflow hydrograph produced by MIKE11 was better simulated for cohesive materials than non-cohesive materials.
 - b. MIKE11 had a tendency to make the non-cohesive outflow hydrographs more gradual than what was recorded in the laboratory which resulted in a higher total breaching time.
 - c. Breach width simulations by MIKE11 were more accurate when using non-cohesive materials than cohesive materials.
- III. Both BREACH and MIKE11 were able to simulate dam breaching for the studied field tests.

- a. It was found that BREACH performed slightly better than MIKE11 when reproducing the peak discharge for the homogenous overtopping tests.
 - b. MIKE11 approximated a better peak discharge than BREACH when a moraine material was used and when failure occurred due to piping
 - c. MIKE11 was more accurate than BREACH when simulating other dam breaching characteristics such as time to peak discharge, lag time, and total breach time.
- IV. MIKE11 correctly reproduced the breach depths during simulation for overtopping failures, but had issues when a piping failure occurred. To initiate piping failure when using MIKE11, the initial pipe diameter had to be significantly increased compared to what was recorded in the field. BREACH initiated the dam breach with the recorded pipe diameter.
- V. A sensitivity analysis was completed for MIKE11.
 - a. The most sensitive parameter found in MIKE11 was the sediment grain size (D_{50}).
 - b. The upstream and downstream side slopes were found to be equally sensitive and the second most sensitive parameters out of all the parameters tested for MIKE11.
 - c. The porosity, crest width, and non-dimensional critical shear stress proved to be the least sensitive parameters in MIKE11
- VI. Problems arose with BREACH during the sensitivity analysis. Mainly, the simulation occasionally ended before an accurate breach of the embankment was complete. This caused large variations in the sensitivity analysis results and therefore the parameters may appear more sensitive than they actually are.
 - a. The most sensitive parameter in BREACH was determined to be the downstream side slope when using the predetermined test cases.
 - b. Both the gradation and internal angle of friction were found to be more sensitive than the upstream side slope but less sensitive than the downstream side slope.
 - c. The least sensitive parameter in BREACH was determined to be the unit weight for the predetermined test cases.

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Table of Contents

Acknowledgments	iv
List of Figures.....	viii
List of Tables	xiv
List of Symbols	xvi
1.0 Introduction.....	1
1.1 Objective	2
1.2 Importance and Novelty of the Study.....	5
1.3 Outline of Thesis.....	6
2.0 Dam Breaching – Theoretical Background	7
2.1 Dam Breaching Mechanisms.....	7
2.1.1 Overtopping	7
2.1.2 Piping.....	8
2.1.3 Foundation Defects.....	8
2.2 Five Stages of Breach Process in Dikes/Embankments	9
2.3 Typical Dam Breach Hydrograph and Characteristics.....	11
3.0 Literature Review	13
3.1 Physically-Based Numerical Models.....	13
3.2 Numerical Models based on Historical Data	32
3.3 Physical Experiments.....	37
3.4 Examples of Numerical Modelling.....	41
3.5 Model Validation and Comparative Studies.....	43
3.6 Discussion.....	44
4.0 Numerical Models	46
4.1 MIKE by DHI.....	46
4.1.1 Bed Resistance.....	46
4.1.2 Hydraulic Radius	47
4.1.3 Additional Resistance due to Vegetation.....	47
4.1.4 Boundary Conditions	48
4.1.5 HD Coefficients.....	48
4.1.6 Computational Grid	49
4.1.7 Cross Sections	49
4.1.8 Dambreach Structure.....	50
4.1.9 Types of Flow Used in Model	55

4.1.10	<i>Initial Conditions</i>	55
4.1.11	<i>Kinematic Routing Method</i>	56
4.1.12	<i>Saint Venant Equations</i>	56
4.1.13	<i>Steady State Energy Equation</i>	56
4.2	BREACH	59
4.2.1	<i>Introduction</i>	59
4.2.2	<i>Using Two Types of Sediments</i>	59
4.2.3	<i>Additional Resistance due to Vegetation</i>	59
4.2.4	<i>Overtopping</i>	60
4.2.5	<i>Grass Cover:</i>	60
4.2.6	<i>Piping</i>	61
4.2.7	<i>Calculating the Breach Width</i>	62
4.2.8	<i>Determining the Reservoir Water Level</i>	64
4.2.9	<i>Breach Channel Hydraulics</i>	65
4.2.10	<i>Sediment Transport</i>	66
4.2.11	<i>Sudden Collapse Breach Enlargement</i>	67
4.2.12	<i>BREACH Iteration</i>	69
4.2.13	<i>Model Testing</i>	69
5.0	Physical Model Experiments	70
5.1	University of Ottawa	70
5.2	Delft University of Technology Experiments	76
5.3	IMPACT-Project Experiments	87
5.3.1	<i>Field Modelling of Embankment Failure</i>	87
5.3.2	<i>Laboratory Modelling of Embankment Failure</i>	99
6.0	Methodology	110
6.1	Estimating Non-Dimensional Critical Shear Stress	110
6.2	Typical Values for Density and Specific Weight	112
6.3	Typical Values for Porosity	112
6.4	Typical Values for D_{50}	112
6.5	Typical Values for Cohesive Strength	113
6.6	Typical Values for Internal Angle of Friction	113
6.7	Calculating Void Ratio/Porosity	114
6.8	Comparing Results	115

7.0	Results	116
7.1	University of Ottawa Test Comparison.....	116
7.2	IMPACT Field Test Comparison	120
7.3	IMPACT Laboratory Tests Comparison	122
7.4	Delft University of Technology Test Comparison	127
7.5	Sensitivity Analysis.....	130
8.0	Discussion.....	133
8.1	University of Ottawa Laboratory Tests	133
8.2	IMPACT-Project Field Tests	135
8.3	IMPACT-Project Laboratory Tests	139
8.3	Delft University of Technology Laboratory Tests	144
8.4	Sensitivity Analysis.....	146
9.0	Conclusions	150
10.0	Recommendations for Future Work	152
11.0	References	154
12.0	Appendix.....	161
12.1	IMPACT Field Test Comparisons	161
12.2	IMPACT Non-Cohesive Lab Test Comparisons	169
12.3	IMPACT Cohesive Lab Test Comparisons.....	179
12.4	Delft University of Technology Test Comparisons.....	191
12.5	Sensitivity Analysis.....	195

List of Figures

Figure 2.1: Breach shapes due to overtopping – Singh, V. (1996)	8
Figure 2.2: Piping failure cross-section (from DHI Water & Environment, 2009)	8
Figure 2.3: Illustration of the 5 stage breaching process (by Visser, 1998)	10
Figure 2.4: Typical hydrograph and hydrograph characteristics.....	12
Figure 3.1: Definition sketch of breach section provided by Macchione (2008)	14
Figure 3.2: Comparison of dam breaches simulated by MPS and CIP with experimental results (by Hu and Sueyoshi, 2009).....	19
Figure 3.3: Comparison of breach development between the new procedure (total concentration and near-bottom concentration) and the experimental data (by Belikov et al, 2010).....	21
Figure 3.4: Plan views of the scouring estimated from the numerical model for a 0.3m dam at different times (by Belikov et al., 2010)	22
Figure 3.5: Longitudinal section through site of breach in dam (by Belikov et al., 2010)	22
Figure 3.6: Flooded area depths downstream of the Krasnodar Reservoir, Russia at various times (by Belikov et al., 2010).....	23
Figure 3.7: Evolution of free surface profiles (a) experimental results and (b) RANS model computed (by Ozmen-Cagatay and Kocaman, 2010)	25
Figure 3.8: Comparison of numerical and experimental results on a dry bed with a height ratio of 0 (by Ozmen-Cagatay and Kocaman, 2010).....	26
Figure 3.9: Comparison of numerical and experimental results on a wet bed with a height ratio of 0.1 (by Ozmen-Cagatay and Kocaman, 2010).....	26
Figure 3.10: Comparison of numerical and experimental results on a wet bed with a height ratio of 0.4 (by Ozmen-Cagatay and Kocaman, 2010).....	27
Figure 3.11: Comparisons between Dennema and Chaundry (1990) and the 2-D and 3-D models presented by Biscarini et al. (2010).....	29
Figure 3.12: Comparisons between Soares, Frazao and Zech (2002) and the 2-D and 3-D models presented by Biscarini et al. (2010).....	30
Figure 3.13: Comparisons between Soares (2002) and the 2-D and 3-D models presented by Biscarini et al. (2010) showing water elevation over triangular sill.....	31
Figure 3.14: Comparison of measured and predicted average breach widths (by Froehlich, 2008)	35
Figure 3.15: Comparison of measured and predicted average breach formation times (by Froehlich, 2008)	35
Figure 3.16: Glass flume layout from Schmocker and Hager (2009)	40
Figure 3.17: Typical erosion process of dike from Schmocker and Hagen (2009).....	41
Figure 4.1: Interaction location between flood plain and main channel	48
Figure 4.2: Area above maximum height (from DHI Water & Environment, 2009).....	50
Figure 4.3: Combined flow over the Dam (from DHI Water & Environment, 2009)	51
Figure 4.4: Breach interpolation (from DHI Water & Environment, 2009)	51
Figure 4.5: Adjusted pipe cross-section (from DHI Water & Environment, 2009)	53
Figure 4.6: Collapse ratio (from DHI Water & Environment, 2009)	53
Figure 4.7: Breach shape after collapse (from DHI Water & Environment, 2009).....	54
Figure 4.8: Energy equation set up (from DHI Water & Environment, 2009)	57
Figure 4.9: Dam failure simulation (from Fread, 1991)	60
Figure 4.10: Breach formation (from Fread, 1991)	62
Figure 4.11: Breach summary (from Fread, 1991)	64

Figure 4.12: Forces that may cause a sudden dam collapse (from Fread, 1991).....	68
Figure 5.1: Tilting flume and large flume in the hydraulics lab at the University of Ottawa (from Al-Riffai and Nistor, 2010)	71
Figure 5.2: Experimental setup for series 2 (from Al-Riffai, M., and Nistor, I., 2011).....	72
Figure 5.3: Geometry of embankment (from Al-Riffai, M., and Nistor, I., 2011).....	72
Figure 5.4: Outflow hydrograph from University of Ottawa tests	74
Figure 5.5: Water levels from University of Ottawa tests	74
Figure 5.6: Flumes used in TU Delft tests (from Zhu, 2006)	77
Figure 5.7: Top view of flume 1 and 2 used in TU Delft tests (from Zhu, 2006).....	78
Figure 5.8: A-A longitudinal section of flume 1 used in TU Delft tests (from Zhu, 2006)	78
Figure 5.9: B-B longitudinal section of flume 2 used in TU Delft tests (from Zhu, 2006)	78
Figure 5.10: TU Delft dike dimensions (from Zhu, 2006)	79
Figure 5.11: TU Delft flume set-up (from Zhu, 2006)	79
Figure 5.12: Photo of TU Delft dike (from Zhu, 2006)	80
Figure 5.13: TU Delft particle size distribution (from Zhu, 2006).....	82
Figure 5.14: Plan view of TU Delft wave gage set up (from Zhu, 2006)	83
Figure 5.15: Profile view of TU Delft wave gage set up (from Zhu, 2006).....	83
Figure 5.16: TU Delft formal test 1- discharge	84
Figure 5.17: TU Delft formal test 1- dike Height	84
Figure 5.18: TU Delft formal test 1- water level	84
Figure 5.19: TU Delft formal test 2 – discharge	85
Figure 5.20: TU Delft formal test 2 - water level	85
Figure 5.21: TU Delft formal test 2 - dike height	85
Figure 5.22: TU Delft formal test 3 – discharge	86
Figure 5.23: TU Delft formal test 3 - dike height	86
Figure 5.24: TU Delft formal test 3 - water level	86
Figure 5.25: Location of IMPACT field tests	88
Figure 5.26: Capacity curve for reservoir used in IMPACT field tests.....	88
Figure 5.27: Grading curve for Field Test #1(from Morris, 2005).....	90
Figure 5.28: Embankment geometry of Field Test #1(from Morris, 2005)	90
Figure 5.29: Initial breach notch for Field Test #1	91
Figure 5.30: Inflow and outflow for Field Test #1	91
Figure 5.31: Water level and breach width for Field Test #1	91
Figure 5.32: Grading curve for Field Test #2 (from Morris, 2005).....	92
Figure 5.33: Embankment geometry of Field Test #2 (from Morris, 2005)	93
Figure 5.34: Inflow, outflow, and water level of Field Test #2	93
Figure 5.35: Grading curve for Field Test #3 (from Morris, 2005).....	94
Figure 5.36: Embankment geometry of Field Test #3 (from Morris, 2005)	95
Figure 5.37: Inflow, outflow, and water level of Field Test #3	95
Figure 5.38: Grading curve for Field Test #4 (from Morris, 2005).....	96
Figure 5.39: Embankment geometry of Field Test #4 (from Morris, 2005)	97
Figure 5.40: Inflow, outflow, and water level of Field Test #4	97
Figure 5.41: Grading curve for Field Test #5 (from Morris, 2005).....	98
Figure 5.42: Embankment geometry of Field Test #5 (from Morris, 2005)	99
Figure 5.43: Inflow, outflow, and water level of Field Test #5	99

Figure 5.44: Grading curves for different sediments used in IMPACT lab tests - series 1(from Morris, 2005)	101
Figure 5.45: Grading curves for different sediments used in IMPACT lab tests - series 2(from Morris, 2005)	102
Figure 5.46: Grading curves for different sediments used in IMPACT lab tests - series 3(from Morris, 2005)	103
Figure 5.47: Inflow, outflow, water level and top breach width for Lab Test #2.....	104
Figure 5.48: Inflow, outflow, water level and top breach width for Lab Test #4.....	104
Figure 5.49: Inflow, outflow, water level and top breach width for Lab Test #5.....	105
Figure 5.50: Inflow, outflow, water level and top breach width for Lab Test #6.....	105
Figure 5.51: Inflow, outflow, water level and top breach width for Lab Test #7.....	106
Figure 5.52: Inflow, outflow, water level and top breach width for Lab Test #10.....	106
Figure 5.53: Inflow, outflow, water level and top breach width for Lab Test #12.....	107
Figure 5.54: Inflow, outflow, water level and top breach width for Lab Test #13.....	107
Figure 5.55: Inflow, outflow, water level and top breach width for Lab Test #15.....	108
Figure 5.56: Inflow, outflow, water level and top breach width for Lab Test #16.....	108
Figure 5.57: Inflow, outflow, water level and top breach width for Lab Test #17.....	109
Figure 6.1: Shields curve from Prasuhn (1992)	111
Figure 7.1: University of Ottawa test – discharge – recorded vs. MIKE11 with varying compaction before adjustment	116
Figure 7.2: University of Ottawa test – discharge – recorded vs. MIKE11 with varying Side Erosion Index.....	117
Figure 7.3: University of Ottawa test – discharge – recorded vs. MIKE11 with varying initial breach notch width.....	117
Figure 7.4: University of Ottawa test – discharge – Recorded vs. MIKE11 with varying initial breach notch depth.....	118
Figure 7.5: University of Ottawa test – discharge – Recorded vs. MIKE11 with varying compaction after adjustment.....	118
Figure 7.6: University of Ottawa Test – water level – recorded vs. MIKE11 with varying compaction after adjustment.....	119
Figure 7.7: IMPACT Field Test #1- discharge - recorded vs. BREACH vs. MIKE11	120
Figure 7.8: IMPACT Field Test #1 - water level - recorded vs. BREACH vs. MIKE11	120
Figure 7.9: IMPACT Field Test #1 - breach width - recorded vs. BREACH vs. MIKE11	121
Figure 7.10: IMPACT Lab Test #2- discharge - recorded vs. MIKE11	122
Figure 7.11: IMPACT Lab Test #2- water level - recorded vs. MIKE11	122
Figure 7.12: IMPACT Lab Test #2- top breach width - recorded vs. MIKE11.....	123
Figure 7.13: IMPACT Lab Test #2- Bottom breach width - recorded vs. MIKE11	123
Figure 7.14: IMPACT Lab Test #10- discharge - recorded vs. MIKE11	124
Figure 7.15: IMPACT Lab Test #10- Water level - recorded vs. MIKE11	125
Figure 7.16: IMPACT Lab Test #10- top breach width - recorded vs. MIKE11.....	125
Figure 7.17: IMPACT Lab Test #10- Bottom breach width - recorded vs. MIKE11	126
Figure 7.18: Delft University of Technology Lab Test #1 discharge results.....	127
Figure 7.19: Delft University of Technology Lab Test #1 water level results.....	127
Figure 7.20: Delft University of Technology Lab Test #1 dike height results.....	128
Figure 7.21: MIKE11 sensitivity analysis – non-dimensional critical shear stress.....	130
Figure 7.22: MIKE11 sensitivity analysis - non-dimensional shear stress - peak discharge.....	131

Figure 7.23: MIKE11 sensitivity analysis - non-dimensional shear stress - time to peak discharge	131
Figure 12.1: IMPACT Field Test #2 - discharge - recorded vs. BREACH vs. MIKE11	161
Figure 12.2: IMPACT Field Test #2 - discharge - recorded vs. MIKE11 with varying initial breach notch depth.....	162
Figure 12.3: IMPACT Field Test #2- water level - recorded vs. BREACH vs. MIKE11	162
Figure 12.4: IMPACT Field Test #3- discharge - recorded vs. BREACH vs. MIKE11	163
Figure 12.5: IMPACT Field Test #3 - discharge - recorded vs. MIKE11 with varying side erosion index.....	164
Figure 12.6: IMPACT Field Test #3- water level - recorded vs. BREACH vs. MIKE11	164
Figure 12.7: IMPACT Field Test #4- discharge - recorded vs. BREACH vs. MIKE11	165
Figure 12.8: IMPACT Field Test #4- water level - recorded vs. BREACH vs. MIKE11	166
Figure 12.9: IMPACT Field Test #5- discharge - recorded vs. BREACH vs. MIKE11	167
Figure 12.10: IMPACT Field Test #5- water level - recorded vs. BREACH vs. MIKE11	167
Figure 12.11: IMPACT Lab Test #4- discharge - recorded vs. MIKE11	169
Figure 12.12: IMPACT Lab Test #4- water level - recorded vs. MIKE11	169
Figure 12.13: IMPACT Lab Test #4- top breach width - recorded vs. MIKE11.....	170
Figure 12.14: IMPACT Lab Test #4- bottom breach width - recorded vs. MIKE11.....	170
Figure 12.15: IMPACT Lab Test #5- discharge - recorded vs. MIKE11	171
Figure 12.16: IMPACT Lab Test #5- water level - recorded vs. MIKE11	172
Figure 12.17: IMPACT Lab Test #5- top breach width - recorded vs. MIKE11.....	172
Figure 12.18: IMPACT Lab Test #5- bottom breach width - recorded vs. MIKE11.....	173
Figure 12.19: IMPACT Lab Test #6- discharge - recorded vs. MIKE11	174
Figure 12.20: IMPACT Lab Test #6- water level - recorded vs. MIKE11	174
Figure 12.21: IMPACT Lab Test #6- top breach width - recorded vs. MIKE11.....	175
Figure 12.22: IMPACT Lab Test #6- bottom breach width - recorded vs. MIKE11.....	175
Figure 12.23: IMPACT Lab Test #7- discharge - recorded vs. MIKE11	176
Figure 12.24: IMPACT Lab Test #7- water level - recorded vs. MIKE11	177
Figure 12.25: IMPACT Lab Test #7- top breach width - recorded vs. MIKE11.....	177
Figure 12.26: IMPACT Lab Test #7- bottom breach width - recorded vs. MIKE11.....	178
Figure 12.27: IMPACT Lab Test #12- discharge - recorded vs. MIKE11	179
Figure 12.28: IMPACT Lab Test #12- water level - recorded vs. MIKE11	179
Figure 12.29: IMPACT Lab Test #12- breach width - recorded vs. MIKE11	180
Figure 12.30: IMPACT Lab Test #13- discharge - recorded vs. MIKE11	181
Figure 12.31: IMPACT Lab Test #13- water level - recorded vs. MIKE11	181
Figure 12.32: IMPACT Lab Test #13- top breach width - recorded vs. MIKE11.....	182
Figure 12.33: IMPACT Lab Test #13- bottom breach width - recorded vs. MIKE11.....	182
Figure 12.34: IMPACT Lab Test #15- discharge - recorded vs. MIKE11	183
Figure 12.35: IMPACT Lab Test #15- water level - recorded vs. MIKE11	184
Figure 12.36: IMPACT Lab Test #15- top breach width - recorded vs. MIKE11.....	184
Figure 12.37: IMPACT Lab Test #15- bottom breach width - recorded vs. MIKE11.....	185
Figure 12.38: IMPACT Lab Test #16- discharge - recorded vs. MIKE11	186
Figure 12.39: IMPACT Lab Test #16- water level - recorded vs. MIKE11	186
Figure 12.40: IMPACT Lab Test #16- top breach width - recorded vs. MIKE11.....	187
Figure 12.41: IMPACT Lab Test #16- bottom breach width - recorded vs. MIKE11.....	187
Figure 12.42: IMPACT Lab Test #17- discharge - recorded vs. MIKE11	188

Figure 12.43: IMPACT Lab Test #17- water level - recorded vs. MIKE11	189
Figure 12.44: IMPACT Lab Test #17- top breach width - recorded vs. MIKE11.....	189
Figure 12.45: IMPACT Lab Test #17- bottom breach width - recorded vs. MIKE11.....	190
Figure 12.46: Delft University of Technology Lab Test #2 discharge results.....	191
Figure 12.47: Delft University of Technology Lab Test #2 water level results.....	191
Figure 12.48: Delft University of Technology Lab Test #2 dike height results.....	192
Figure 12.49: Delft University of Technology Lab Test #3 discharge results.....	193
Figure 12.50: Delft University of Technology Lab Test #3 water level results.....	193
Figure 12.51: Delft University of Technology Lab Test #3 dike height results.....	194
Figure 12.52: MIKE11 sensitivity analysis – crest width.....	195
Figure 12.53: MIKE11 sensitivity analysis – crest width - peak discharge	195
Figure 12.54: MIKE11 sensitivity analysis – crest width - time to peak discharge	196
Figure 12.55: MIKE11 sensitivity analysis – D50 (median grain size)	196
Figure 12.56: MIKE11 sensitivity analysis – D50 (median grain size) - peak discharge.....	197
Figure 12.57: MIKE11 sensitivity analysis – D50 (median grain size) - time to peak discharge	197
Figure 12.58: MIKE11 sensitivity analysis - porosity.....	198
Figure 12.59: MIKE11 sensitivity analysis - porosity - peak discharge	198
Figure 12.60: MIKE11 sensitivity analysis - porosity - time to peak discharge	199
Figure 12.61: MIKE11 sensitivity analysis - side slopes ($ss=h:v$)	199
Figure 12.62: MIKE11 sensitivity analysis - side slopes ($ss=h:v$) - peak discharge	200
Figure 12.63: MIKE11 sensitivity analysis - side slopes ($ss=h:v$) - time to peak discharge	200
Figure 12.64: MIKE11 sensitivity analysis - upstream side slopes ($uss=h:v$)	201
Figure 12.65: MIKE11 sensitivity analysis - upstream side slopes ($uss=h:v$) - peak discharge	201
Figure 12.66: MIKE11 sensitivity analysis - upstream side slopes ($uss=h:v$) - time to peak discharge.....	202
Figure 12.67: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$)	202
Figure 12.68: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$) - peak discharge	203
Figure 12.69: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$) - time to peak discharge.....	203
Figure 12.70: BREACH sensitivity analysis – crest width	204
Figure 12.71: BREACH sensitivity analysis – crest width - first peak discharge	204
Figure 12.72: BREACH sensitivity analysis – crest width –second peak discharge.....	205
Figure 12.73: BREACH sensitivity analysis – crest width - time to peak second peak discharge	205
Figure 12.74: BREACH sensitivity analysis – upstream side slope ($uss=h:v$).....	206
Figure 12.75: BREACH sensitivity analysis – upstream side slope ($uss=h:v$) - peak discharge.....	206
Figure 12.76: BREACH sensitivity analysis – upstream side slope ($uss=h:v$) - time to peak discharge.....	207
Figure 12.77: BREACH sensitivity analysis – downstream side slope ($dss=h:v$).....	207
Figure 12.78: BREACH sensitivity analysis – downstream side slope ($dss=h:v$) - peak discharge	208
Figure 12.79: BREACH sensitivity analysis – downstream side slope ($dss=h:v$) - time to peak discharge.....	208
Figure 12.80: BREACH sensitivity analysis – internal angle of friction	209

<i>Figure 12.81: BREACH sensitivity analysis – internal angle of friction - peak discharge.....</i>	<i>209</i>
<i>Figure 12.82: BREACH sensitivity analysis – internal angle of friction - time to peak discharge</i>	<i>210</i>
<i>Figure 12.83: BREACH sensitivity analysis – gradation (D90/D30).....</i>	<i>210</i>
<i>Figure 12.84: BREACH sensitivity analysis – gradation (D90/D30) - peak discharge.....</i>	<i>211</i>
<i>Figure 12.85: BREACH sensitivity analysis – Gradation (D90/D30) - time to peak discharge</i>	<i>211</i>
<i>Figure 12.86: BREACH sensitivity analysis – unit weight</i>	<i>212</i>
<i>Figure 12.87: BREACH sensitivity analysis – unit weight - peak discharge</i>	<i>212</i>
<i>Figure 12.88: BREACH sensitivity analysis – unit weight - time to peak discharge</i>	<i>213</i>
<i>Figure 12.89: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay</i>	<i>214</i>
<i>Figure 12.90: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - bottom of breach elevation.....</i>	<i>214</i>
<i>Figure 12.91: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - top breach width</i>	<i>215</i>
<i>Figure 12.92: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - bottom breach width.....</i>	<i>215</i>

List of Tables

Table 1.1: List of physically based numerical models (from Morris and Hassan, 2002).....	3
Table 1.2: Teton dam failure data summary (from Zhu, 2006)	5
Table 3.1: Expressions for variables provided by Macchione (2008).....	14
Table 3.2: Measured embankment geometry from Coleman et al. (2002)	33
Table 3.3: Prediction of breaching parameters for Banqiao Dam (by Xu and Zhang, 2009).....	36
Table 3.4: Predictions of breaching parameters for Teton Dam (by Xu and Zhang, 2009)	36
Table 3.5: Means of initiation of erosion based on location of breach from Fall et al. (2003) ...	37
Table 3.6: Breach time based on material from Fall et al. (2003).....	38
Table 3.7: Breach time based on location of internal erosion from Fall et al. (2003).....	38
Table 3.8: The main parameters in the tests by Zhang et al. (2009)	39
Table 3.9: Comparison of breach parameters by Zhang et al. (2009)	39
Table 3.10: Prediction equations used by BC Hydro	42
Table 5.1: Grain size distributions and dry density for embankment materials for Al-Riffai, M., 2009.....	71
Table 5.2: Void ratio for different soil compactions.....	72
Table 5.3: Soil characteristics summary	73
Table 5.4: Breaching characteristics for University of Ottawa tests	75
Table 5.5: TU Delft flume dimensions	77
Table 5.6: TU Delft test dimensions	79
Table 5.7: Summary of TU Delft soil properties.....	81
Table 5.8: Soil properties of Field Test #1	89
Table 5.9: Soil properties of Field Test #2	92
Table 5.10: Soil properties of Field Test #3	94
Table 5.11: Soil properties of Field Test #4	96
Table 5.12: Soil properties of Field Test #5	98
Table 5.13: Impact lab test series 1 summary	101
Table 5.14: Impact lab test series 2 summary	102
Table 5.15: Impact lab test series 3 summary	103
Table 6.1: Typical critical shear stress values from Rennie (2011)	111
Table 6.2: Typical density and specific weight values from Zhu (2010)	112
Table 6.3: Typical porosity values from Zhu (2010)	112
Table 6.4: Typical D50 values from Zhu (2010).....	113
Table 6.5: Typical cohesive strength values from Zhu (2010)	113
Table 6.6: Typical internal angle of friction values from Peck et al. (1974) and Meyerhof (1956)	114
Table 6.7: Typical internal angle of friction values from Zhu (2010).....	114
Table 7.1: University of Ottawa comparison.....	119
Table 7.2: IMPACT Field Test #1 comparison.....	121
Table 7.3: IMPACT Lab Test #2 comparison.....	124
Table 7.4: IMPACT Lab Test #10 comparison.....	126
Table 7.5: Delft University of Technology Lab Test #1 comparison.....	128
Table 7.6: Comparison of IMPACT field test's percent errors.....	129
Table 7.7: Comparison of IMPACT laboratory test's percent errors.....	129
Table 7.8: Comparison of TU Delft laboratory test's percent errors	129
Table 7.9: Parameter sensitivity by rank - MIKE11	132

<i>Table 7.10: Parameter sensitivity by rank - BREACH</i>	<i>132</i>
<i>Table 12.1: IMPACT Field Test #2 comparison.....</i>	<i>163</i>
<i>Table 12.2: IMPACT Field Test #3 comparison.....</i>	<i>165</i>
<i>Table 12.3: IMPACT Field Test #4 comparison.....</i>	<i>166</i>
<i>Table 12.4: IMPACT Field Test #5 comparison.....</i>	<i>168</i>
<i>Table 12.5: IMPACT Lab Test #4 comparison.....</i>	<i>171</i>
<i>Table 12.6: IMPACT Lab Test #5 comparison.....</i>	<i>173</i>
<i>Table 12.7: IMPACT Lab Test #6 comparison.....</i>	<i>176</i>
<i>Table 12.8: IMPACT Lab Test #7 comparison.....</i>	<i>178</i>
<i>Table 12.9: IMPACT Lab Test #12 comparison.....</i>	<i>180</i>
<i>Table 12.10: IMPACT Lab Test #13 comparison.....</i>	<i>183</i>
<i>Table 12.11: IMPACT Lab Test #15 comparison.....</i>	<i>185</i>
<i>Table 12.12: IMPACT Lab Test #16 comparison.....</i>	<i>188</i>
<i>Table 12.13: IMPACT Lab Test #17 comparison.....</i>	<i>190</i>
<i>Table 12.14: Delft University of Technology Lab Test #2 comparison.....</i>	<i>192</i>
<i>Table 12.15: Delft University of Technology Lab Test #3 comparison.....</i>	<i>194</i>
<i>Table 12.16: Estimated parameter values for different sediment types.....</i>	<i>213</i>

List of Symbols

A	cross-sectional area [m^2]
B	breach width under the water line [m]
B_o	initial width of the rectangular shape channel [m]
c	erodible wetter perimeter [m]
C	Chezy coefficient [$\text{m}^{1/2}/\text{s}$]
c'	cohesion [kPa]
Cc	coefficient of curvature [ND]
$Ccal$	coefficient of calibration [ND]
Cu	coefficient of Uniformity [ND]
D	hydraulic depth of flow [m]
D_{30}	grain size representing the 30 th percentile of finer material [mm]
D_{50}	grain size representing the 50 th percentile of finer material [mm]
D_{90}	grain size representing the 60 th percentile of finer material [mm]
e	void ratio [ND]
f_D	Moody friction factor [ND]
fi	Reynolds stress [kPa]
Fr	Froude number [ND]
fx	viscous stress [kPa]
g	acceleration due to gravity [m/s^2]
g_x	gravity in the x-direction [m/s^2]
g_i	gravity in the i-direction [m/s^2]
h	water depth [m]
h_B	height of the bottom contour [m]
H_b	centreline breach crest elevation [m]
h_b	head on the breach-crest centerline [m]
H_b	breach level [m]
H_c	breach bottom elevation [m]
k_s	resistance number [ND]
K	coefficient of vertical detritus exchange between the bottom and the flow [ND]
l	mean length of the breach [m]
L_b	breach crest length[m]
n	Manning's bed resistance term [ND]
N_R	Reynolds number [ND]
p	pressure [kPa]
P	wetted perimeter [m]
ρ	density [kg/m^3]
ρ_w	density of water [kg/m^3]
PI	plasticity index for clay or silty soils [ND]

Q	discharge [m^3/s]
Q_b	breach discharge [m^3/s]
Q_p	flow through the pipe [m^3/s]
q_s	volumetric sediment transport rate per unit width [m^2/s]
q_t	total bed material transport per unit width [m^2/s]
R	hydraulic radius [m]
S	downstream slope of the dam face [ND]
SG	Specific Gravity [ND]
t	time [s]
τ	shear stress [Pa]
τ'_c	shields dimensionless critical shear stress [Pa]
u^*	shear velocity [m/s]
u	horizontal velocity [m/s]
ν	kinematic viscosity [m^2/s]
VF	volume fraction of fluid in each cell [ND]
W	fall velocity of the soil in the water [m/s]
x	space coordinate [m]
z_{obv}	obvert elevation of the pipe [m]
β	slope angle [$^\circ$]
γ	unit weight [kN/m^3]
ε	sediment porosity [ND]
θ	angle the side slopes make with the vertical [$^\circ$]
λ	Darcy-Weisbach resistance factor [ND]
Φ	angle of internal friction [$^\circ$]
ω	water content [%]
ω_{opt}	optimum water content [%]

ND refers to non-dimensional units

1.0 Introduction

Dams have been a useful tool for mankind for thousands of years by storing water during times of surplus, and by making it available for use during times of drought. There are currently over 45,000 large dams being used throughout the world today (DHI Water & Environment, 2009) and 800,000 dams have been constructed to date (Zagonjoli, M., 2007). These dams ensure a water supply to adjacent areas, mitigate flooding, or generate hydro-electric power (DHI Water & Environment).

Dams can usually be classified into two different groups: concrete and earthen/rock. Concrete dams can usually be classified into gravity, arch, or buttress resistant. Almost 80% of the world's major dams are constructed from natural erodible earthen materials (US Committee on Large Dams, 1975).

Dam failures are very rare, but they do occur. When dams do fail, there are usually catastrophic consequences. This is often because local communities are not sufficiently prepared. The amount of life or property loss that can occur from a dam breach has increased a large amount over the past few decades. This is because there has been a lot of development in areas that would be affected if a dam breach occurred (DHI Water & Environment).

Some examples of catastrophic dam breaches include the Teton Dam failure of 1976, and the Banqiao & Shimantan Dam breach of 1975. The Teton Dam failure occurred in Idaho in June of 1976 and was classified as a piping failure. Cracks in the abutments led to water forming a pipe in a weak soil layer. Within a couple of hours of the pipe forming, the dam had failed. The city of Rexburg, located downstream of the Teton Dam was evacuated and only 11 lives were lost. However, over 13,000 cattle died and almost 80% of the city was destroyed, which resulted in over \$2 billion in damages. The Banqiao & Shimantan Dam breach of 1975 was much more drastic. This failure was caused by overtopping because the area received a 1-in-2000 year storm while the dam was only designed for a 1-in-1000 year storm. The area actually received over 1000 mm of rain in a 24 hour period. The failure caused a flood wave that was 10 km wide and 7m high, which caused over 85,000 deaths due to flooding and another 145,000 due to subsequent starvation and epidemics.

With global warming on the rise, there are going to be more severe flow conditions within the life span of structures that have already been constructed. This has led to increased safety concerns. More torrential rainfall and the melting of ice-caps and glaciers may lead to more water entering the dam reservoir. This increased water flow will likely increase the number of dam failures and will therefore be a danger to anything downstream of the dam. The amount of life loss will depend on a number of different things including: the water depth, flow velocity, geographical distribution of the population, warning time to reach the population, and how easy it is to warn them. Therefore, if advanced warning messages are delivered to the population, lives can be saved.

For better risk assessment, it is important for the development and improvement of dam breach modelling software. Dam breach modelling and the associated flood risk is required to

save life as well as property located downstream of the dam (Zagonjolti, M., 2007). Wahl (1998) states that many different approaches have been taken to predict the breaching process and he classifies them as non-physically based, semi-physically based, and physically based.

A non-physically based model is one that fits equations to past dam failures. The user can select parameters such as the reservoir volume, dam height, or crest width and the breaching process is predicted from similar structures. An example of a non-physically based model was presented by Froehlich (1995). Wahl (2001) completed a qualitative analysis on how accurate these processes are. More information of Froehlich's non-physically based model and Wahl's analysis can be found in the literature review section of this thesis.

A semi-physically based model is similar to a non-physically based model but it uses simplified assumptions for some of the breaching process. These models usually provide a full outflow hydrograph instead of just one characteristic such as the peak outflow. However, the user is usually forced to make an assumption where there is limited guidance. Examples of assumptions a user would have to make are estimating how quickly the breach geometry develops or the final breach geometry. The model would then predict an outflow hydrograph. This type of model is good to look at different scenarios but does not give an accurate prediction of the breaching process. An example of a semi-physically based model is presented by Singh *et al.* (1989) and more details can be found in the literature review section of this thesis.

Physically based models attempt to simulate the breaching mechanisms. Such models use detailed computations combining hydraulics, sediment transport processes, and soil mechanics to try and simulate observed physical processes. These models estimate both the breaching process and the outflow hydrograph. NWS BREACH is by far the most popular physically based model (Morris and Hassan, 2002) but a number of models have been developed before and after its release. To see a list of physically based models compiled by Morris and Hassan (2002) the reader is referred to Table 1.1. For more information on many of the models please refer to the literature review section of this thesis.

1.1 Objective

The objective of this study is to help validate and confirm the range of usability of two of the most widely used dam breach numerical models. The two numerical models being reviewed are BREACH developed by the National Weather Service (NWS) and MIKE11 with the Dam Breach Module (DB) developed by the Danish Hydraulic Institute (DHI). Results of these models are compared to results of physical models completed in both the laboratory (conducted by the University of Ottawa, Delft University of Technology, and as part of the Impact-project) and field (conducted as part of the IMPACT-project). The feasibility of each model for different situations will also be determined and areas requiring improvement are discussed. Finally, the main parameters of each model are analysed and their sensitivity is ranked.

Table 1.1: List of physically based numerical models (from Morris and Hassan, 2002)

Model	Breach Morphology	Flow Calculation	Sediment Transport	Geo-mechanics of Breach Side Slopes	Limitations and Deficiencies
Christofano Reported by Hughes (1981) &Singh (1996)	Trapezoidal, with constant bottom width	Broad crested weir formula	Christofano's empirical formula	None	- Constant breach bottom width and shape - No lateral erosion mechanism - No slope stability mechanism - Unrealistic erosion relation
Harris-Wagner (1967) Reported by Singh (1996)	Parabolic - top width = 3.75 depth	Broad crested weir formula	Schoklitsch formula	None	- Constant sediment concentration - No slope stability mechanism - User input breach slope
BRDAM Brown & Rogers (1997, 1981)	Parabolic, with 45° side slopes	Broad crested weir formula	Schoklitsch formula	Failure of top wedge above pipe specified by user	- Constant sediment concentration - No slope stability mechanism - No lateral erosion mechanism - User input time for pipe failure
Ponce- Tsivoglu (1981)	Top width flow rate relation	Full Saint Venant equations	Exner equation with Meyer-Peter-Muller	None	- No slope stability mechanism - Use of the regime concept - No lateral erosion after peak flow
Lou Reported by Singh (1996)	Most effective stable section (Cosine curve shape)	Full Saint Venant equations	1.Christofano's empirical formula 2. Duboy and Einstein formula 3. Lou's formula	None	- No slope stability mechanisms - No lateral erosion after peak flow - Empirical formula to compute the erosion - Inappropriate method to model the breach growth
Nogueira Reported by Singh (1996)	Effective shear stress section (cosine curve shape)	Full Saint Venant equations	Exner equation with Meyer-Peter-Muller	None	- Same limitations as Lou's model
NWS BREACH (Fread, 1988)	Rectangular and trapezoidal	Broad crested weir formula for overtopping. Orifice Flow for piping	Meyer-Peter-Muller modified by Smart	Breach side slope stability and top wedge failure during piping or overtopping	- Uniform erosion of the breach - Incompatible computation method - Inaccurate slope stability analysis - Simplified modeling of the failure of composite embankments
BEED Singh & Quiroga (1987), Singh (1996)	Trapezoidal	Broad crested weir formula	Einstein-Brown	Breach side slope stability	- Uniform erosion of the breach - Incompatible computation method - Inaccurate slope stability analysis

Table 1.1 Continued: List of physically based numerical models (from Morris and Hassan, 2002)

Model	Breach Morphology	Flow Calculation	Sediment Transport	Geo-mechanics of Breach Side Slopes	Limitations and Deficiencies
SITES (1998)	3 stage failure: cover failure, headcut formation, and headcut erosion	Principles of hydrology and hydraulics to produce spillway flow-stage curve	For stages 1 and 2, a detachment model was used. For stage 3, an energy dissipater equation was used	Spillway exit channel stability	- Does not model complete embankment failure process - Empirical coefficient to compute erosion
NCP BREACH (Andrews and Coleman, 1998)	Parabolic	Empirical formula	Empirical formula	None	- Empirical formula based on small scale models - No slope stability mechanism
EDBREACH (Loukola and Houkuna, 1998)	Trapezoidal	Broad crested weir formula	Meyer-Peter-Muller	Top wedge failure during piping	- Uniform erosion of the breach - Inaccurate slope stability analysis
BRES (Visser, 1998)	5 stages of failure	Broad crested weir formula	Several transport formulae	None	- No slope stability mechanism - Incomplete computation method
DEICH N1/N2 (Broich, 1998)	Diffusion approach	Shallow water equations	Several transport formulae	None	- Parabolic breach shape - No slope stability mechanism - Unrealistic modeling of the vertical and lateral erosion
Renard and Rupro (Reported by Paquier, 1998)	Uniform erosion of the pipe	Orifice equation	Meyer-Peter-Muller	Failure of material above the pipe	- Unrealistic modeling of failure of material above the pipe - No slope stability mechanism
Flood Levee Breaches (Fujita and Tamura, 1987)	Rectangular breach shape above water level - trapezoidal below	Critical flow equation	Sediment transport rate estimated assuming energy slope consumed only in sediment transport	None	- Uniform erosion of the breach - No slope stability mechanism

1.2 Importance and Novelty of the Study

Until recently, there has been a lack of physical data on dam breaching (Zhu, Y. *et al.*, 2006 and Singh, V., 1996). An important issue when it comes to recording data on dam breaching is the absence of field (real cases) information. When a dam breach occurs in the field, engineers are usually not present until after the breach and subsequent flood have occurred. In most cases, there is not enough warning time for the researchers to travel to the breaching dam and set up equipment to record the process. The area surrounding large dams can also be very dangerous while a breach is occurring. This is because of the large amount of water being released and the subsequent flooding. Therefore, even if researchers could get to the location, they may choose to collect the data after the breach has occurred. This means that researchers often try to collect data from satellite imagery, photos, and witnesses of the breach phenomenon. A lot of approximations are made and therefore accurate conclusions are very difficult to draw. Table 1.2 shows approximations for the dam breach characteristics of the Teton Dam Failure. After referring to the table it is quite evident how much the approximated characteristics used by researchers can change.

Over the past few years, experiments in both the laboratory (conducted by the University of Ottawa, Canada, Delft University of Technology, Netherlands, and as part of the Impact-project, United Kingdom) and in the field (conducted as part of the IMPACT-project, Norway) have been completed. This new information is used in this study to help validate or rate the performance of two widely used dam breach models: BREACH and MIKE11.

Table 1.2: Teton dam failure data summary (from Zhu, 2006)

Breaching Characteristics		Wahl (1998)	Fread (1988b)	Havno <i>et al.</i> (1998)	Singh (1998)
Dam Slope	Upstream (H:V)	3:1	2.5:1	2.5:1	3:1
	Downstream (H:V)	2.5:1	2:1	2:1	2.5:1
Reservoir Capacity (10^6 m^3)		356	308	-	308
Peak Outflow ($10^4 \text{ m}^3/\text{s}$)		6.5	4.7-4.9	4.5-8.0	4.67
Breach Height (m)		86.9	-	91.5	79.0
Breach Side Slope (H:V)		1:1	-	0.75:1	-
Breach Width	Bottom (m)	-	-	55.0	46.0
	Top (m)	-	-	192.0	-
	Averaged (m)	151.0	-	-	-

1.3 Outline of Thesis

The thesis starts by describing important aspects of dam breaching such as the dam breaching mechanisms, characteristics, and parameters used to describe a dam breach. It then discusses the state-of-the-art research in dam breaching. Further, the thesis will provide information on the numerical and physical models that will be compared. A comparison will then be completed and the results are compared using a method similar to the one presented in the IMPACT-Project. Finally, a sensitivity analysis will be completed on many of the parameters used in both BREACH and MIKE11. The feasibility of each model is then determined for certain criteria and future work in this area is suggested.

2.0 Dam Breaching – Theoretical Background

2.1 Dam Breaching Mechanisms

Before the leading research on dam breaching modelling is discussed in more detail, it is important to understand the main causes of dam breaching. This section will explain the main reasons dam failures occur and how they develop. There are three major types of earthen dam failures. They are overtopping, foundation defects, and piping. Costa (1985) states that 34% of all dam failures are due to overtopping, 30% are due to foundation defects, and 28% are due to piping. This leaves 8% of the dam failures which are caused by other miscellaneous acts or processes.

2.1.1 Overtopping

Overtopping is the most common type of failure. It occurs when there are water levels or waves that are higher than the crest of the dam. This type of failure usually occurs following storms with precipitation that exceeds the reservoir and spillway capacity of the dam. The failure is also due to inadequate design, construction and maintenance, debris blocking the spillway, settlement causing the dam crest to be lowered, or a section of the dam crest that was built lower than the rest of the crest (Task committee on Dam/Levee Break, 2010).

When the dam is constructed out of homogeneous non-cohesive sediment compared to non-homogeneous or cohesive sediment, the breaching process is significantly different.

The breaching process for a homogeneous, non-cohesive dam is usually described in four stages and the erosion mechanism is sediment transport. First, the downstream slope of the dam near the crest becomes steeper. The stage is described by the upstream erosion of the downstream slope. This results in a narrower crest width. The third stage is defined by the crest being lowered due to downcutting. Finally, the fourth stage is described by the breach widening due to lateral erosion and this is where the mass failure occurs (Task committee on Dam/Levee Break, 2010). Wahl (1998) describes the first two stages as one stage and calls it the “breach initiation”.

The breaching process for a dam constructed of homogeneous, cohesive sediment is significantly different. This is because the erosion mechanism is the head cut or vertical drop erosion. The Task committee on Dam/Levee Break (2010) still describes this breaching process as occurring in four stages. The first stage is when the initial overtopping occurs, which results in sheet and rill erosion. These rills develop into large overfalls which eventually cause large headcuts in the downstream crest. The second stage is described by the headcut also reaching the upstream part of the crest. The erosion channel also widens. The third stage lowers the crest of the dam by downcutting. Finally, the fourth stage widens the initial breach and again, this is where the mass failure occurs. The task committee believes that the third and fourth stages are very similar for cohesive and non cohesive sediments. However, the erosion modes and the mass failure that occurs are very different.

The Task Committee on Dam/Levee Break (2010) also states that the overtopping failure of dams made out of composite sediments is also very different than dams constructed out of homogeneous sediments. They believe that when overtopping occurs on a dam with a clay, steel, or concrete core that erosion starts on the downstream slope by either sediment transport or by a headcut that advances until it reaches the core. This erosion may affect the stability of the core and cause it to fail. Common failures of the core include sliding, overturning and bending. The core would then wash away downstream and the breach would increase until mass failure occurs. If the cover is less erosive than the core, the cover may erode first and the core would only erode at the areas where the cover has eroded.

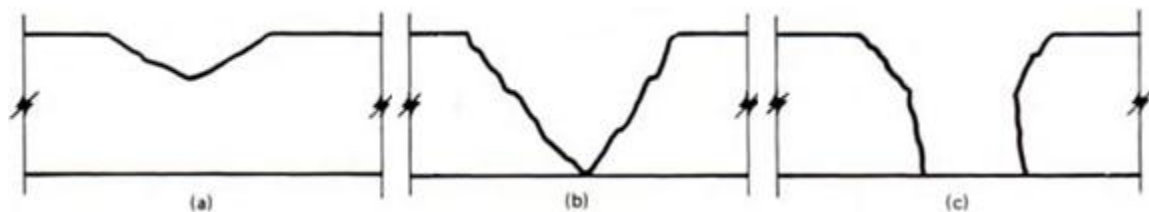


Figure 2.1: Breach shapes due to overtopping – Singh, V. (1996)

2.1.2 Piping

Another common type of dam failure is due to piping. Piping occurs from seepage or leakage through weak layers, structure joints, dead tree roots, and animal burrows in the embankment. For piping to occur, the water level does not need to reach the height of the dam crest. It is also possible for seepage to liquefy the material in the body of the dam and cause large volumes of the dam to slide as slurry. It is most common for a “pipe” to be formed from one end of the dam to another. The erosion within the pipe causes parts of the dam to slump and eventually collapse from the weight and water pressure. After the collapse, the breach acts very much like an overtopping breach. This includes both the downcutting and then widening. The piping failure takes much longer to occur than overtopping failure. Piping failure can take days as overtopping failure takes hours or less.

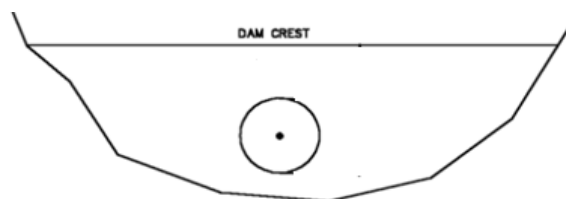


Figure 2.2: Piping failure cross-section (from DHI Water & Environment, 2009)

2.1.3 Foundation Defects

The last major type of dam failure is due to foundation defects. These defects include differential settlement, sliding and slope instability, high uplift pressure, and uncontrolled

foundation seepage. Where differential settlement occurs, often cracks and weak layers are found throughout the dam. These cracks and weak layers can lead to internal erosion which often results in piping failure. When there is a lot of seepage that passes through the foundation there might also be sand boils. High uplift pressure is another major foundation defect that can lead to failure. This adds to the slope instability and may cause the dam to slide. The sliding defect can form a large rupture almost instantaneously and is much faster than piping and overtopping. The sliding defect also has a much greater erosion potential. The size and position of the sliding failure is usually determined by the geological conditions (The Task Committee on Dam/Levee Break, 2010). The sliding breach is usually rectangular in shape and covers the whole height of the dam (Singh, 1996).

2.2 Five Stages of Breach Process in Dikes/Embankments

Visser (1998) and Zhu (2006) have both broken the breaching process down into five different stages. Visser (1998) studied dike breaching in sand dikes while Zhu (2006) studied the breaching process in clay dikes. Both processes are almost identical but Zhu (2006) stipulates that headcut erosion plays a more important role in clay embankments. The following outlines the five stages on the breach process outlined by Visser (1998) (see Figure 2.3 for an illustration of the breaching process). He assumes the breach erosion starts ($t = t_0$) with a flow that runs through a small initial breach at the top of the sand dike.

- I. The first stage starts with steepening of the slope angle β of the channel in the inner slope. The initial value is β_0 at $t = t_0$ and can reach a maximum critical value of β_1 at $t = t_1$.
- II. The second stage has erosion of the inner slope at a constant angle β_1 for $t_1 < t \leq t_2$. This yields a decrease in crest width in the breach width. This stage ends at $t = t_2$ when the crest totally disappears and the breach inflow starts to increase.
- III. The third stage is the lowering of the top of the dike in the breach with a constant angle of the breach side slopes. The side slopes are equal to the critical value γ_1 and this results in an increase of the width of the breach for $t_2 < t \leq t_3$. At $t = t_3$ the dike in the breach has completely washed away all the way to the original ground.
- IV. The fourth stage is the critical flow stage where the breach flow is virtually critical throughout the breach for $t_3 < t \leq t_4$. The breach still continues to grow laterally. The side slope angles still stay at the critical value. Whether the breach grows more in the vertical direction depends on the erodibility of the original ground. At $t = t_4$ the flow changes from critical to subcritical. Therefore, $Fr = 1$ for $t_3 < t \leq t_4$ and $Fr < 1$ for $t > t_4$.
- V. The fifth and final stage is the subcritical flow stage and the breach continues to grow, mainly laterally due to the subcritical flow for $t_4 < t \leq t_5$. At t_5 , the flow velocities in the breach become very small and any breach erosion stops. The side slope angles still remain at the critical value. The flow through the breach continues until $t = t_6$ and the water level downstream of the dike equals the water upstream.

During the first 3 stages, the initial breach cuts itself into the dike. Stage IV and V contain most of the flow that travels through the breach. At the end of stage IV, backwater starts to influence the flow through the breach. In theory, when a small downstream polder is present, the submergence of the dam can occur in any stage. However, in practice this usually occurs in stage III since the flow travelling through the breach is too small in the first two stages. Therefore, in the case when a small polder is present, the fourth stage can be skipped.

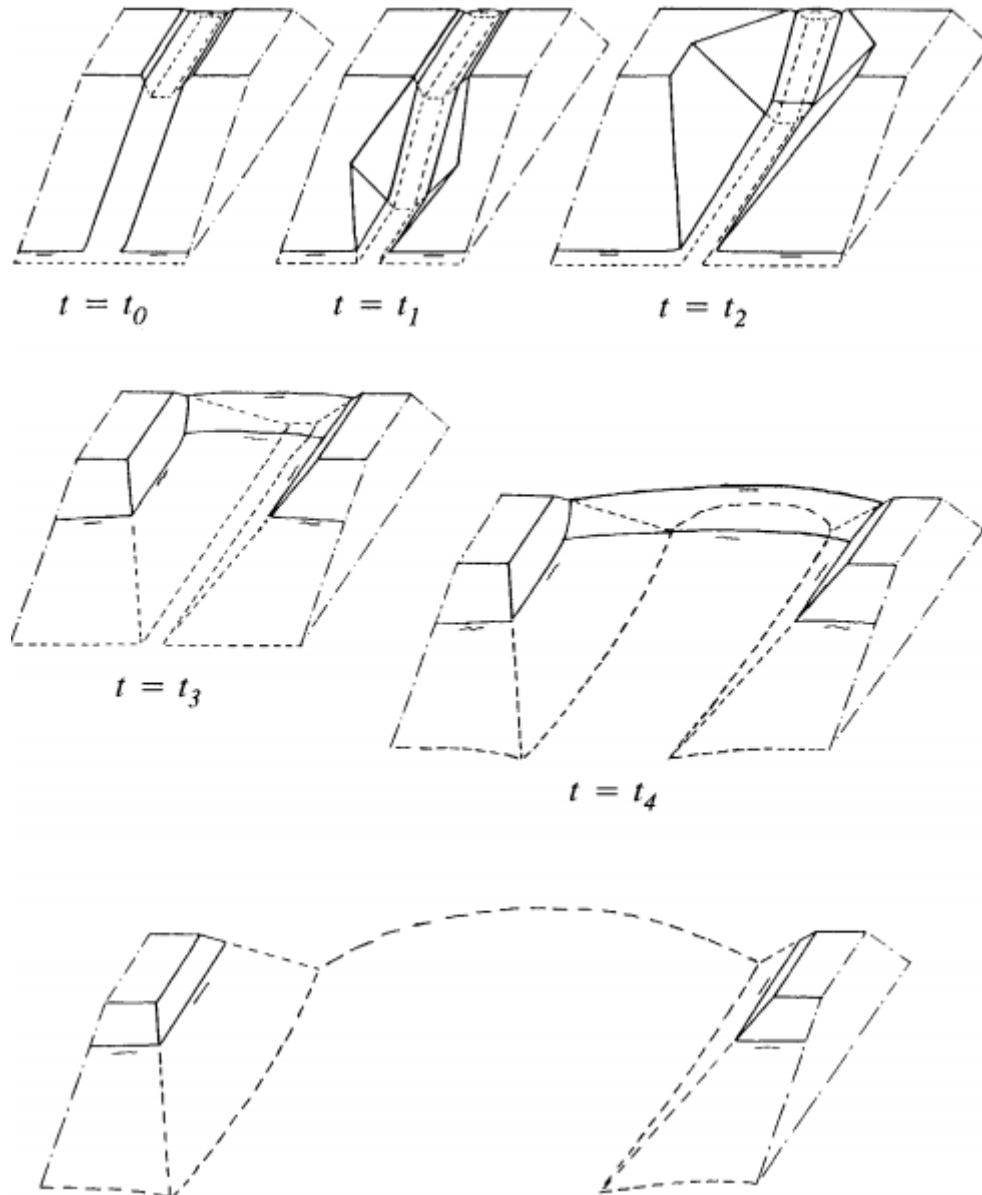


Figure 2.3: Illustration of the 5 stage breaching process (by Visser, 1998)

2.3 Typical Dam Breach Hydrograph and Characteristics

The main hydrograph characteristics that this study will be looking at are outlined in Figure 2.4. This figure is a typical hydrograph that can be expected from a dam breach. The main characteristics that this study will be looking at are: Peak Outflow, Time to Peak, Lag Time, and Breach Time.

Peak Outflow is one of the most important characteristics when it comes to looking at a dam breach hydrograph. It represents the maximum amount of flow that will be exiting through the dam breach at one instant. This is very important in calculating the inundated areas caused by a dam breach.

The Time to Peak represents the amount of time until the peak outflow is reached. The time to peak usually starts when the dam starts overtopping or when the test starts. When the time to peak begins depends on the author and can vary between studies. It is important to determine how an author calculates time to peak before making any comparisons.

Lag time is the amount of time it takes from the start of the test until there is a discharge through the breach. Again, the lag time can vary from author to author. Some record the lag time as soon as there is any flow through the breach as others wait until there is a significant amount of flow. It is therefore important to understand how the authors calculate the lag time before making any comparisons. In my opinion, lag time should begin at soon as any flow starts overtopping the breach.

The final characteristic that will be looked at is the Breach Time. This is the amount of time that it takes for the full breach to occur from start to finish. The timing starts at the end of the lag time and continues until the outflow of the hydrograph is constant and the inflow into the reservoir equals the discharge through the dam.

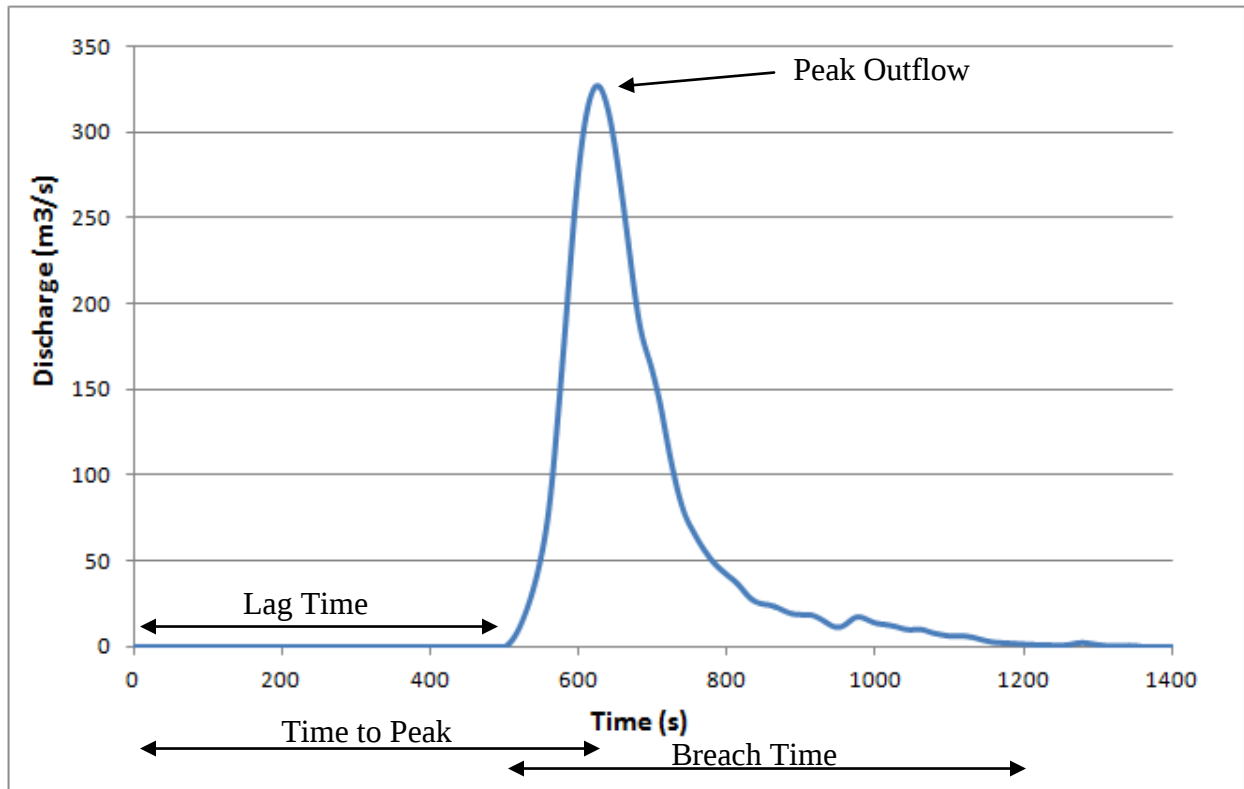


Figure 2.4: Typical hydrograph and hydrograph characteristics

3.0 Literature Review

A literature review has been concluded to examine the state-of-the-art research in the field. A number of different areas have been researched; including non-physically and physically based numerical models, field and laboratory experiments, numerical models in use, and validation studies that have occurred in the past.

3.1 Physically-Based Numerical Models

Franca and Almeida (2004) developed a computation model for rockfill dam breaching due to overtopping. This model has been named RoDaB. The authors have decided to develop this type of model because they say no specific type of model for a rockfill dam has been developed yet. The model is a physically based numerical model and has been based on the governing equations used in reservoir routing and breach erosion. Results from their laboratory experiments were used to fulfil the aspects of the dam breaching process which has yet to be recorded. The authors used two erosion parameters and the final breach geometry to model the dam breach. The authors finally used the model on a rockfill dam and compared the results to the BREACH model.

Macchione (2008) proposed a physically-based dam breach numerical model that predicts the peak discharge, outflow hydrograph, and breach development. The model takes into account the geometry of the embankment, the shape of the reservoir, the hydraulic characteristics of flow through the breach, the erosive capacity, and the shape of the breach. The model only needs one calibration parameter which was chosen based on historic dam failures.

Figure 3.1 shows a sketch of the breach section and defines the different variables used in the calculations. Several of the expressions based on the variables can be found in Table 3.1.

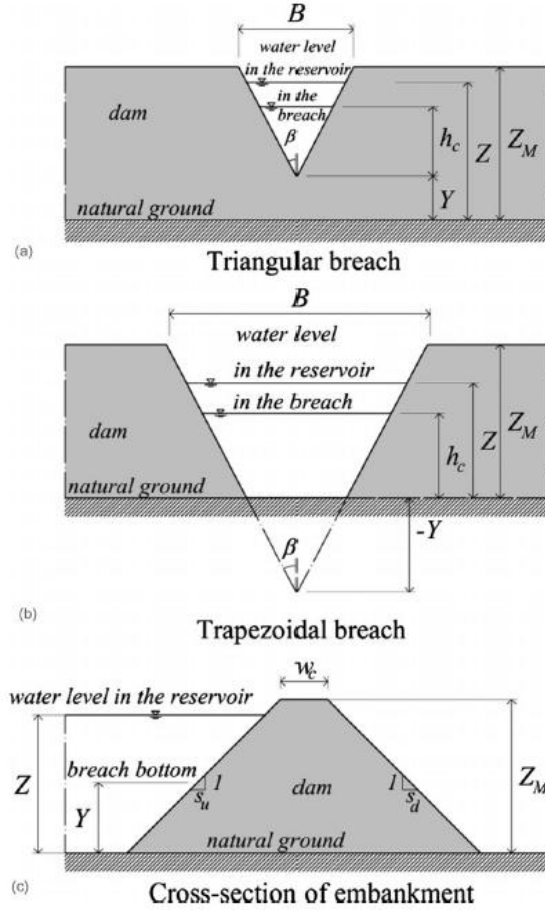


Figure 3.1: Definition sketch of breach section provided by Macchione (2008)

Table 3.1: Expressions for variables provided by Macchione (2008)

Breach	$R=$	$V=$	$h_c=$	$\frac{dA_b}{dY}=$	$l=$
Triangular	$\frac{1}{2}h_c \sin \beta$	$\left(\frac{1}{2}gh_c\right)^{1/2}$	$\frac{4}{5}(Z-Y)$	$-2(Z_M-Y)\tan \beta$	$(Z_M-Y)\frac{s_u+s_d}{2}+w_c$
Trapezoidal	$\frac{h_c(h_c-2Y)}{2\left(\frac{h_c}{\sin \beta}-Y\right)}$	$\left[\frac{h_c-2Y}{2(h_c-Y)}h_c\right]^{1/2}$	$\frac{1}{5}[2Z+3Y+(9Y^2+4Z^2-8YZ)^{1/2}]$	$-2Z_M \tan \beta$	$Z_M\frac{s_u+s_d}{2}+w_c$

The area of a triangular or trapezoidal breach section can then be found using:

Triangular:

$$A_b = (Z_M - Y)^2 \tan \beta \quad (3.1)$$

Trapezoidal:

$$A_b = (Z_M - 2Y)Z_M \tan \beta \quad (3.2)$$

The volumetric sediment transport rate per unit width (q_s) can be found using the shear stress (τ) and a coefficient that depends of the material properties. The shear stress can be found using the Meyer-Peter and Mueller equation:

$$q_s = k_0 \tau^{3/2} \quad (3.3)$$

The enlargement of the breach can then be found using the sediment transport rate, the mean length of the breach (l) and the erodible wetter perimeter (c). This is completed by assuming the eroded material below the water line is distributed evenly over the whole length of the breach.

$$dA_b \cdot l = c \cdot q_s \cdot dt \quad (3.4)$$

This can be re-written as:

$$\frac{dY}{dt} = c q_s \left(\frac{dA_b}{dY} \right)^{-1} l^{-1} \quad (3.5)$$

Using the equation for mean shear stress, Strickler's equation for the friction slope, and the proposed equation for the erodible wetted perimeter, the equation can be re-written again:

$$\tau = \gamma R S \quad (3.6)$$

$$S = \frac{v^2}{k_s^2 R^{4/3}} \quad (3.7)$$

$$c = \frac{2h_c}{\cos\beta} \quad (3.8)$$

$$\frac{dY}{dt} = v_e \frac{2h_c}{\cos\beta} \frac{v^3}{g^{3/2} R^{1/2}} \left(\frac{dA_b}{dY} \right)^{-1} l^{-1} \quad (3.9)$$

Using continuity equations, the discharge (Q) through the breach can then be found for a triangular breach and trapezoidal breach, respectively:

$$Q = \left(\frac{1}{2} g \right)^{1/2} \left[\frac{4}{5} (Z - Y) \right]^{5/2} \tan\beta \quad (3.10)$$

$$Q = \left(\frac{1}{2} g \right)^{1/2} [h_c (h_c - 2Y)]^{3/2} (h_c - Y)^{-1/2} \tan\beta \quad (3.11)$$

The change in the Z can then be described for a triangular breach and trapezoidal breach respectively:

$$\frac{dZ}{dt} = -\left(\frac{4}{5}\right)^{\frac{5}{2}} \left(\frac{g}{2}\right)^{\frac{1}{2}} \frac{Z^{1-\alpha_0}}{\alpha_0 W_0} (Z - Y)^{5/2} \tan\beta \quad (3.12)$$

$$\frac{dZ}{dt} = -\frac{Z^{1-\alpha_0}}{\alpha_0 W_0} (h_c - 2Y) h_c \left[\frac{g}{2} \frac{(h_c - 2Y)}{(h_c - Y)} h_c \right]^{1/2} \tan\beta \quad (Y < 0) \quad (3.13)$$

The parameter that must be calibrated was the characteristic velocity that affects the erosion velocity (v_e). Running an analysis based on historical data, the authors found that a mean parameter value of $v_e = 0.0698\text{m/s}$ describes the 12 historical cases of the dam breach that they have considered very well. There was a very low variability in this parameter and the authors stated that this was perhaps the most important result of the paper since it indicated the correctness of the physical layout described.

Using this parameter, the model gave excellent results when simulating historic dam failures that were presented by the Macchione (2008). The prediction error for peak discharges was $e = 0.008$ and average breach widths was $e = 0.013$. The authors stated that the calibrated parameter can range from 0.05 to 0.1 m/s but a mean value of 0.7m/s should be used for dams similar to the ones presented in this study. Overall, this paper shows a simple but accurate way to simulate a dam breach due to overtopping.

Hu and Sueyoshi (2009) have studied the numerical simulation of a dam breach. They used two novel numerical computation methods. The first was a CIP (Constrained Interpolation Profile) based on the Cartesian grid method. This method allows the free surface flow to travel as a multi-phase flow and it is solved using a Cartesian grid. The second method was the MPS (Moving Particle Semi-implicit) method. This is a Lagrangian-based method and no computational grid is used. Both computational methods have been developed by the authors over the past 10 years due to the increasing industrial needs for CFD techniques. To test the accuracy of the two models, they were validated against a newly conducted experiment. The two methods are described in more detail below.

The first method used by Hu and Sueyoshi (2009) was the CIP Cartesian grid method. In this method, the dam breach is described as a two-phase fluid problem. First, the CIP method is used as a flow solver. It is a fixed grid that covers the whole area that is being modelled and an incompressible and viscous fluid flow condition is assumed. The governing equations are given as:

$$\frac{\partial u_i}{\partial x_i} = 0 \quad (3.14)$$

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = -\frac{1}{\rho} \frac{\partial p}{\partial x_i} + \frac{1}{\rho} \frac{\partial}{\partial x_j} (2\mu S_{ij}) + f_i \quad (3.15)$$

where S is the force of gravity. Time evaluation of the second equation must be completed with a fractional step method. The equation is separated into an advection step, a step to calculate the diffusion, and a step to treat the velocity-pressure coupling. In the advection step, the CIP scheme is used. The advection phase uses the following two equations:

$$\frac{\partial q}{\partial t} + u_j \frac{\partial q}{\partial x_j} = 0 \quad (3.16)$$

$$\frac{\partial q_\xi}{\partial t} + u_j \frac{\partial q_\xi}{\partial x_j} = -q_\xi \frac{\partial u_j}{\partial x_j} \quad (3.17)$$

where:

$$q = u_i, q_\xi = \frac{\partial q}{\partial \xi} \text{ and } \xi = x_1, x_2, x_3 \quad (3.18)$$

The calculation of the right hand side term in the second equation can be used to calculate the diffusion. A function for interpolation can be constructed using a cubic polynomial to estimate the profile around a grid point x_m . Once the interpolation function is developed, a semi-Lagrangian approach is applied for time evolution:

$$\begin{cases} q_m^* = Q_m^n(x_m - u_m \Delta t) \\ q_{xm}^* = \partial Q_m^n(x_m - u_m \Delta t) / \partial x \end{cases} \quad (3.19)$$

A Poisson equation is then used to describe the velocity-pressure coupling:

$$\frac{\partial}{\partial x_i} \left(\frac{1}{\rho} \frac{\partial p^{n+1}}{\partial x_i} \right) = \frac{1}{\Delta t} \frac{\partial u_i^{**}}{\partial x_i} \quad (3.20)$$

In the dam breach simulation, the free surface can be determined by solving for ϕ_1 in:

$$\frac{\partial \phi_1}{\partial t} + u_j \frac{\partial \phi_1}{\partial x_j} = 0 \quad (3.21)$$

The second method provided by Hu and Sueyoshi (2009) for a dam breach simulation was the MPS (Moving Particle Semi-implicit) method. This method was originally used for ship hydrodynamic research and it is a gridless Lagrangian method. By numbering the particles with the subscript i , the velocity (u) and pressure (p) can be described as a function of time and positions (r).

$$u_i = u(t, r_i), p_i = p(t, r_i) \quad (3.22)$$

Within the MPS method, only water is considered, therefore the density is constant. The water is also assumed incompressible. The Navier-Stokes equations are then used to describe the fluid:

$$\frac{Du}{Dt} = -\frac{1}{\rho}\Delta p + \nu \nabla^2 u + f \quad (3.23)$$

To solve the Navier-Stokes equations, first and second order differential operators are required. The first order differential operator is given as:

$$\nabla \phi_i = \frac{d}{n^0} \sum_{j \neq i}^N w(r_{ij}) \frac{\phi_j - \phi_i}{r_{ij}} \frac{r_j - r_i}{r_{ij}} \quad (3.24)$$

where $r_{ij} = (r_i - r_j)$ is the distance from particle i to particle j , d is the number of dimensions, and $w(r_{ij})$ is the weight function in terms of the distance. The weight is found using:

$$w(r) = \begin{cases} \frac{r_0}{r} - 1, & r \leq r_0 \\ 0, & r > r_0 \end{cases} \quad (3.25)$$

The second order differential operator is given by:

$$\nabla^2 \phi_i = \frac{2d}{\lambda} \sum_{j \neq i}^N w(r_{ij}) (\phi_j - \phi_i) \quad (3.26)$$

$$\lambda = \sum_{j \neq i}^N r_{ij}^2 w(r_{ij}) \quad (3.27)$$

The pressure in the MPS method is then solved using Poisson's equation:

$$\nabla^2 p_i^{n+1} = -\frac{\rho}{\Delta t^2} \frac{(n_i^* - n^0)}{n^0} \quad (3.28)$$

To test the models, Hu and Sueyoshi (2009) also constructed a new experimental setup. They used a water tank with a removable plate. The plate can be removed at a high speed and the motion of flow was recorded using a digital camera. The experiment was repeated 10 times and they found that it produced similar results every run. For a sample of the results from the MPS model, CIP model, and the experiment runs the reader is referred to Figure 3.2. The authors concluded that the methods being studied are both robust and accurate. They stated that the methods were accurate even when there was a large deformation of fluid, which is often experienced with dam breaching.

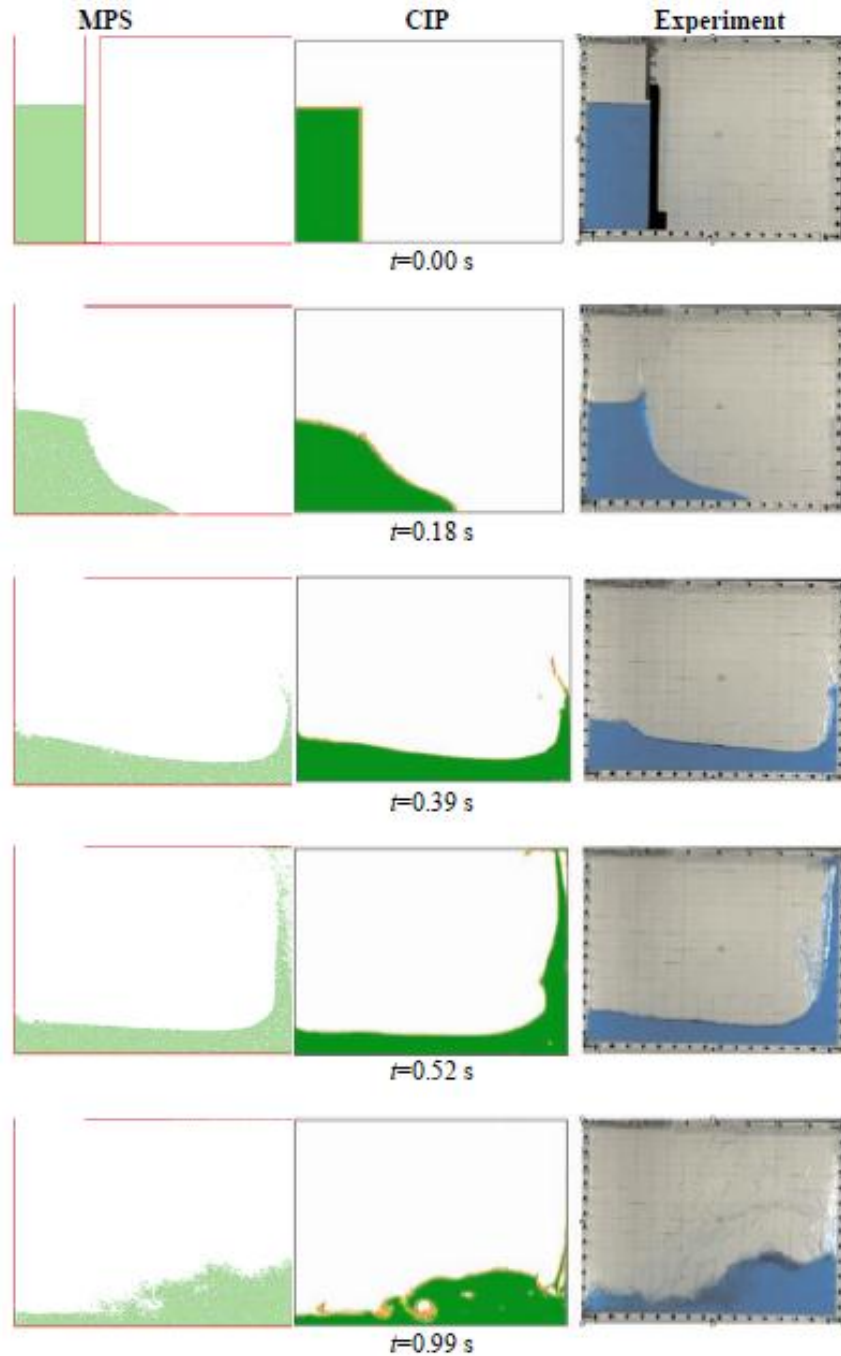


Figure 3.2: Comparison of dam breaches simulated by MPS and CIP with experimental results (by Hu and Sueyoshi, 2009)

Belikov *et al.* (2010) studied the numerical modelling of a breach wave through the dam at the Krasnodar reservoir off the Kuban River in Russia. The authors used a new method to model the breach of the dam. They compared this method with experimental results and finally showed the affected area from the dam breach as well as other characteristics. The authors stated that many software packages such as MIKE11 do not take into account characteristic features of the initial period of breach formation. These models therefore do not reproduce the physical

mechanism of breach formation and yield large deviations from what was actually happening in the breaching process. The authors concluded that a breach-development model that was more in tune with actual processes was needed.

Belikov *et al.* (2010) developed a method which was based on the solution of a system of differential equations. This method describes the sediment transport and the change in elevation of the bottom of the breach. The model considers the effect of sliding of the underwater slopes, and caving of the slopes that are located above the water level when calculating the elevation of the breach bottom. The system of differential equations was given as:

$$\frac{\partial hS}{\partial t} + \frac{\partial USh}{\partial x} + \frac{\partial VSh}{\partial y} = -K(S - S_s) \quad (3.29)$$

$$(1 - p) \frac{\partial Z}{\partial t} = K(S - S_s) + \frac{\partial}{\partial x} D \frac{\partial Z}{\partial x} + \frac{\partial}{\partial y} D \frac{\partial Z}{\partial y}; \quad (3.30)$$

$$K = \begin{cases} \alpha U_* + (1 - \alpha)W, & U_* > W \\ W, & U_* \leq W \end{cases}, 0 \leq \alpha \leq 1. \quad (3.31)$$

$$D = D_1 + D_2 + D_3, D_1 = \beta_o ShW; \quad (3.32)$$

$$D_2 = \beta_1 \sqrt[4]{\left(\frac{tg\gamma}{tg\phi}\right)^2 - 1}, \gamma > \phi, h > 0; \quad (3.33)$$

$$D_3 = \beta_2 \sqrt[4]{\left(\frac{tg\gamma}{tg\phi_c}\right)^2 - 1}, \gamma > \phi_c, h = 0; \quad (3.34)$$

$$S_H = \alpha_1 \frac{(U_* - U_{*N})^2}{2gh} \left(\frac{0.13}{tg\phi} + 0.01 \frac{|U|}{W} \right), \quad (3.35)$$

where h is the water depth, U and V are the velocity components in the x and y direction, S is the bulk concentration of the detritus particles in the flow, S_s is the equilibrium concentration of the detritus (sediment) particles (determined using Bagnold's formula), K is the coefficient of vertical detritus exchange between the bottom and the flow, p is the porosity, γ is the angle of repose of the soil, ϕ and ϕ_c are the natural angles of repose of the sediment under and above the water surface, W is the fall velocity of the sediment in the water, U_* and U_{*N} are the dynamic and non-displacing velocities of the current and respective, and $\alpha, \alpha_1, \beta_o, \beta_1$, and β_2 are empirical coefficients as determined from experimental and field data.

The model proposed by Belikov *et al.* (2010) was confirmed by experimental data that was conducted in a rectangular flume that was 25.0m long and 3.48m high. The width of the breach opening along the crest for both the numerical and physical model can be observed in

Figure 3.3. From these plots it can be observed that the model works satisfactorily. The scouring prediction from the numerical model for a 0.3m dam can be seen in Figure 3.4. Finally, estimations of the dam breach at the Krasnodar Reservoir, Russia, are shown in Figure 3.5 and Figure 3.6.

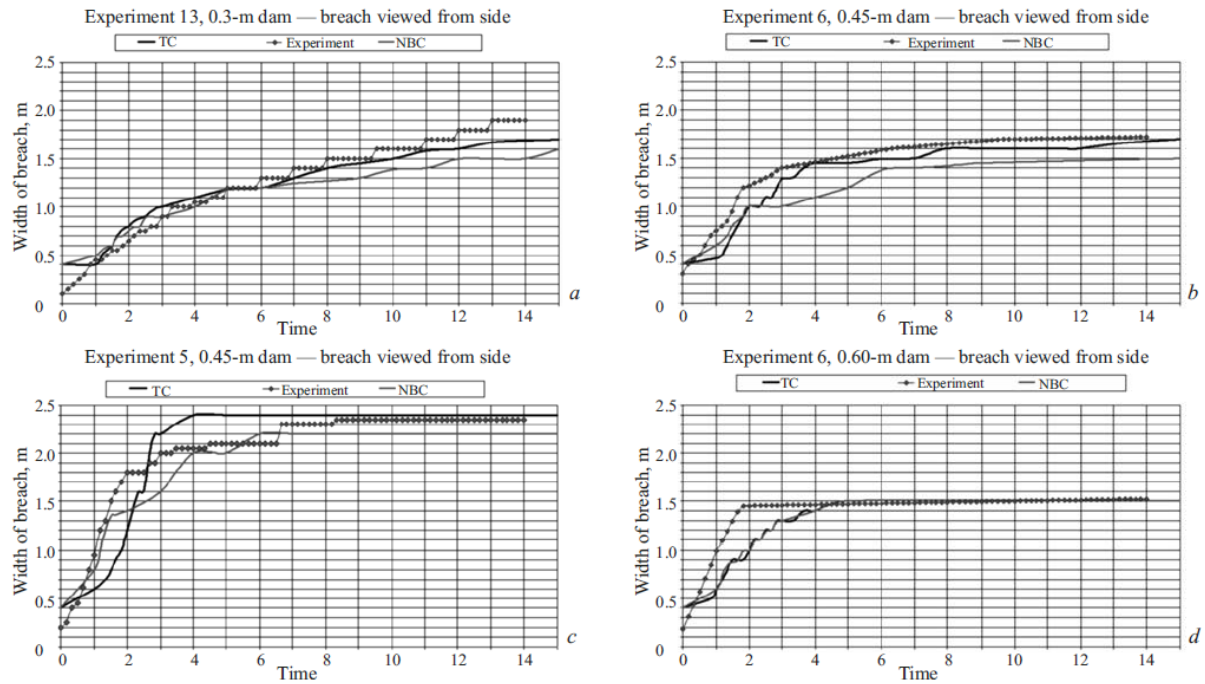


Figure 3.3: Comparison of breach development between the new procedure (total concentration and near-bottom concentration) and the experimental data (by Belikov et al, 2010)

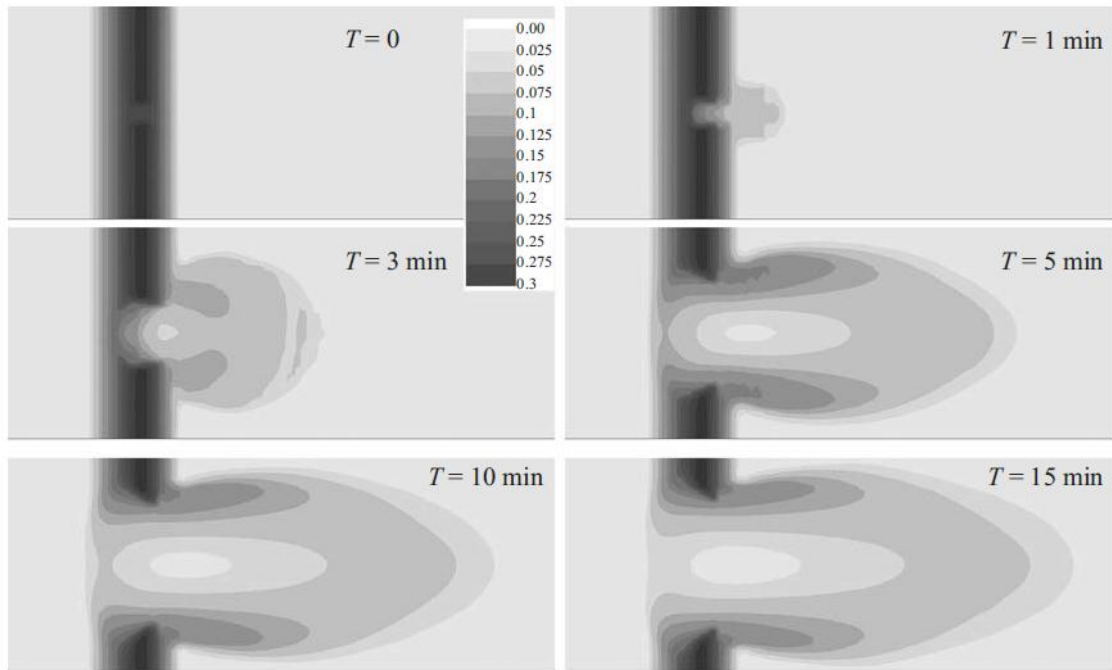


Figure 3.4: Plan views of the scouring estimated from the numerical model for a 0.3m dam at different times (by Belikov et al., 2010)

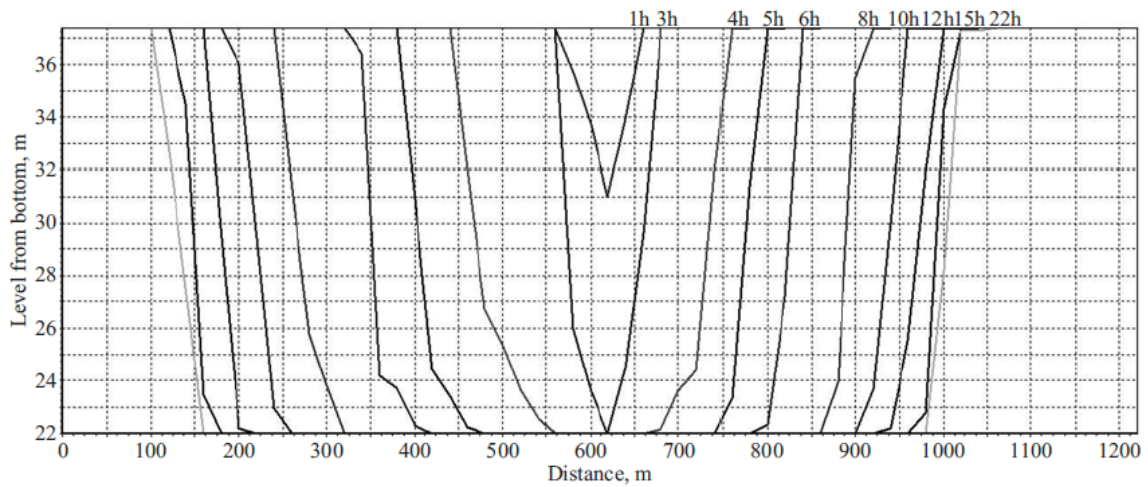


Figure 3.5: Longitudinal section through site of breach in dam (by Belikov et al., 2010)

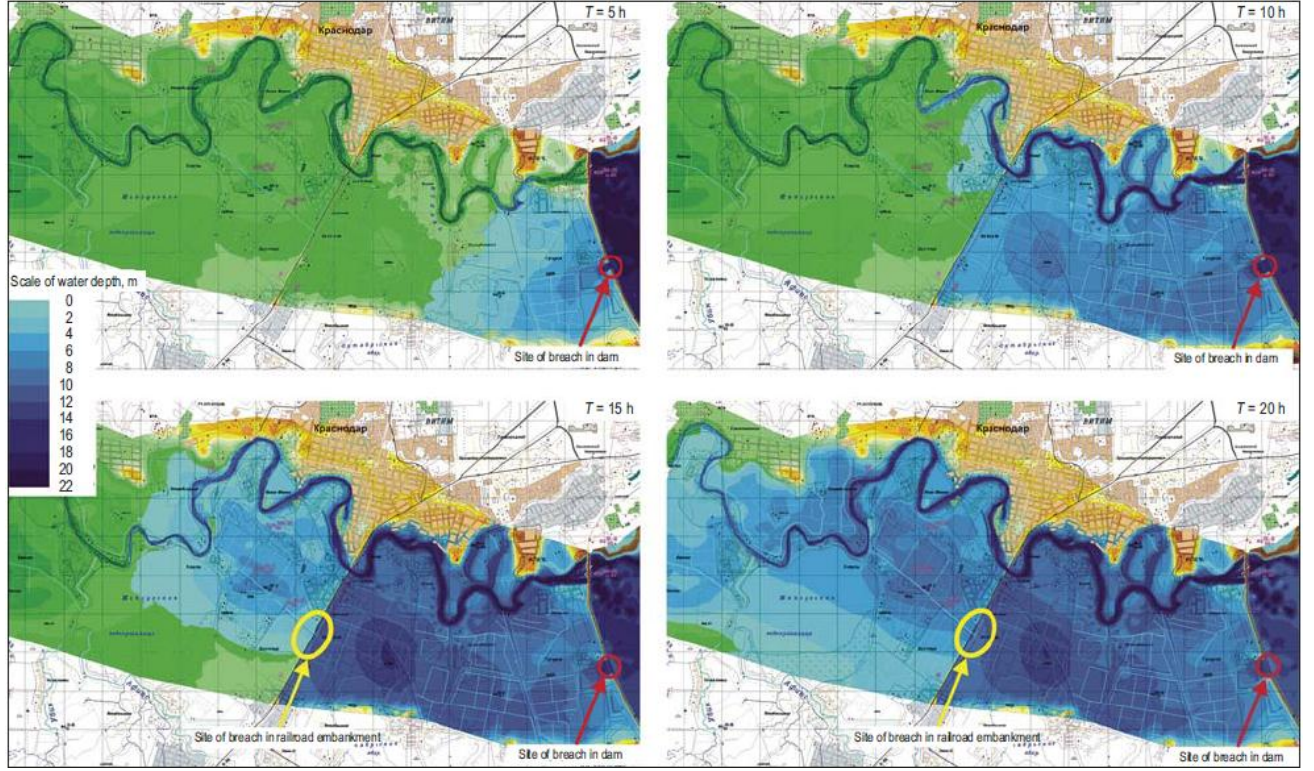


Figure 3.6: Flooded area depths downstream of the Krasnodar Reservoir, Russia at various times (by Belikov et al., 2010)

Ozmen-Cagatay and Kocaman (2010) conducted research on dam-breach flows during initial dam breaching using SWE and RANS approaches. They then compared these results to experimental results that they obtained in the laboratory.

The SWE approach was based on the shallow water equations using a finite volume, finite difference or finite element solver. The SWEs were the result of the vertical integration of the Navier-Stokes equations in which the fluid was considered incompressible and the pressure distribution was assumed hydrostatic. The standard 1-D SWEs were:

$$\frac{\partial(h-h_B)}{\partial t} + \frac{\partial[u(h-h_B)]}{\partial x} = 0 \quad (3.36)$$

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} = -\frac{1}{\rho} \frac{\partial p}{\partial x} + g_x + f_x \quad (3.37)$$

where x is the space coordinate, t is time, h is the height of fluid, h_B is the height of the bottom contour, u is the horizontal velocity, g_x is gravity in the x -direction, p is pressure, ρ is density, and f_x is the viscous stress included by defining a laminar velocity and bottom roughness. The volume of fluid method was then used to define a variable bottom contour and depth of fluid.

The RANS approach was developed using the Reynolds-averaged Navier Stokes equations. The governing and continuity equations for Newtonian, incompressible fluid flow were given as:

$$\frac{\partial}{\partial x_i} (u_i A_i) = 0 \quad (3.38)$$

$$\frac{\partial u_i}{\partial t} + \frac{1}{V_F} \left(u_j A_j \frac{\partial u_i}{\partial x_j} \right) = -\frac{1}{\rho} \frac{\partial p}{\partial x_i} + g_i + f_i \quad (3.39)$$

where u_i represents the velocity equation in the direction specified, V_F is the volume fraction of fluid in each cell, A is the area open to flow, p is pressure, ρ is density, t is time, g_i is gravity in the i -direction, and f_i is the Reynolds stresses requiring a turbulence model for closure. The standard k - ϵ turbulence closure model was used to determine turbulent viscosity.

The experiments were conducted by Ozmen-Cagatay and Kocaman (2010) at the hydraulics laboratory in Cukurova University, Turkey. A rectangular flume that was 9m long, 0.3m wide and 0.34m high was used in the experiments. For the dam breach modelling, a mechanism was set in place to allow for the instantaneous removal of a vertical plate. The free surface profile was determined during the experiment using digital images with dyes in the water and tailwater. A total of 3 cameras were used and 50 frames/second were shot.

Evolution of the free surface profiles between experimental results and the RANS numerical model are shown in Figure 3.7. Experimental and numerical comparisons of free surface profiles during the initial stages of dam-breaches at different times can be observed in Figure 3.8 to Figure 3.10.

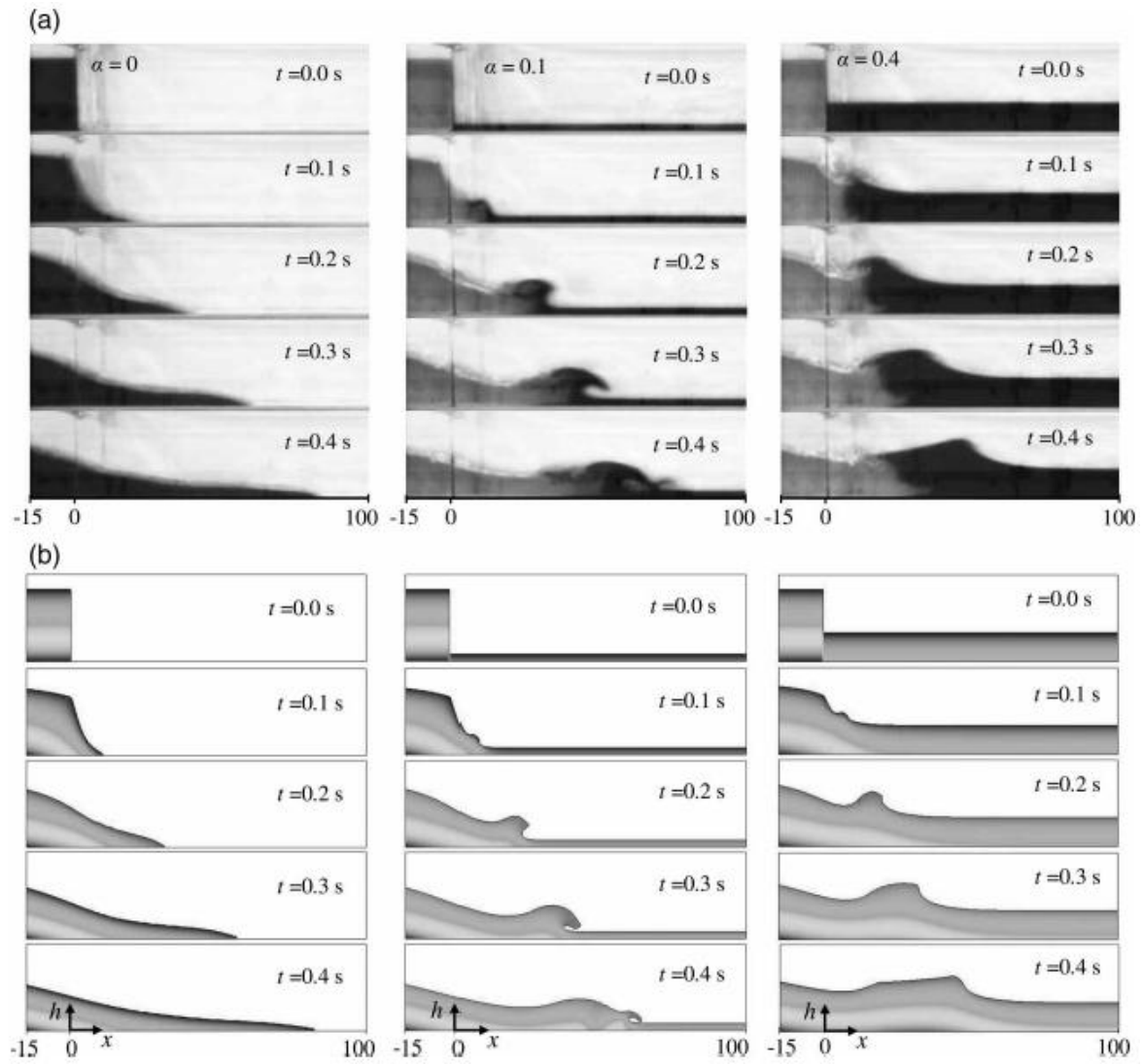


Figure 3.7: Evolution of free surface profiles (a) experimental results and (b) RANS model computed (by Ozmen-Cagatay and Kocaman, 2010)

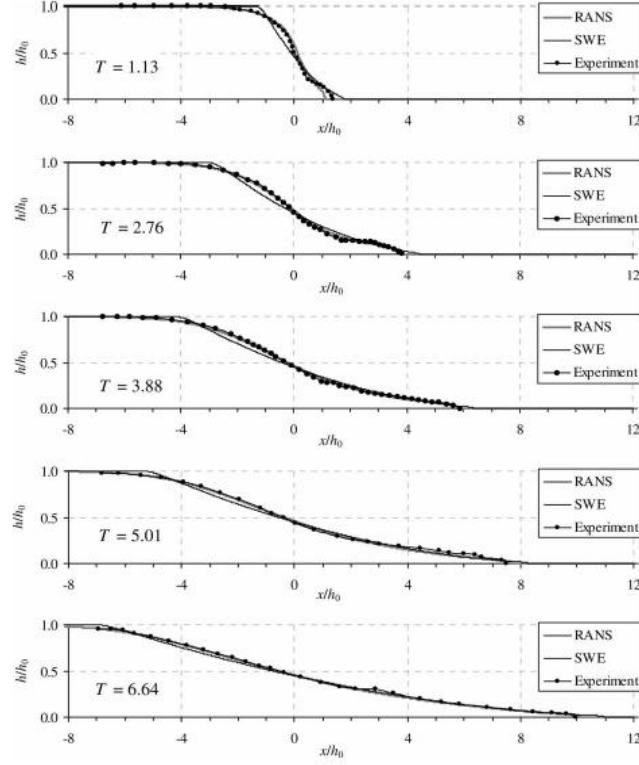


Figure 3.8: Comparison of numerical and experimental results on a dry bed with a height ratio of 0 (by Ozmen-Cagatay and Kocaman, 2010)

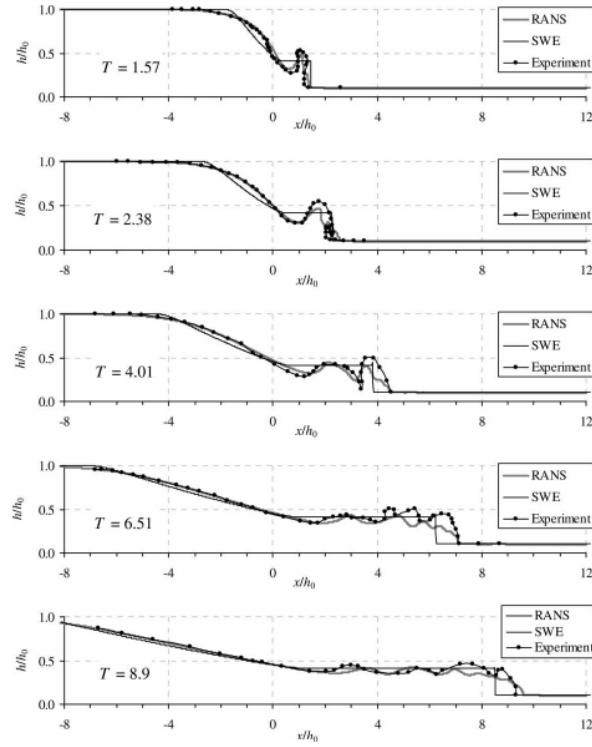


Figure 3.9: Comparison of numerical and experimental results on a wet bed with a height ratio of 0.1 (by Ozmen-Cagatay and Kocaman, 2010)

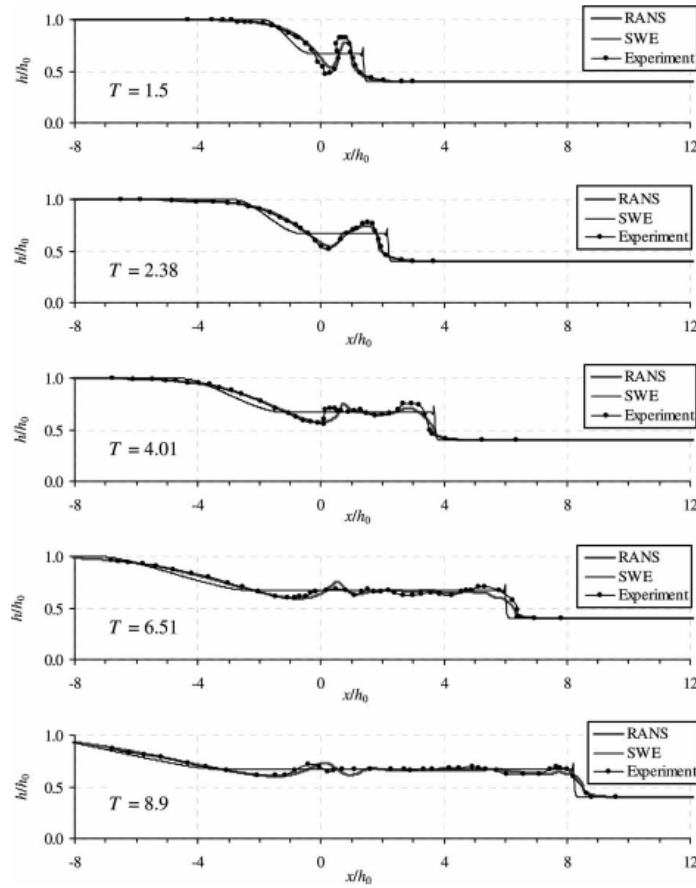


Figure 3.10: Comparison of numerical and experimental results on a wet bed with a height ratio of 0.4 (by Ozmen-Cagatay and Kocaman, 2010)

Ozmen-Cagatay and Kocaman (2010) concluded that for the dry-bed test, both models, except for the earliest stage resulted in good agreement with the observed data. During the initial stage, they concluded that the SWE approach is sufficient to represent dam breach flows for wet bed tests. As the depth ratio increases, the SWE approach provided reasonable accuracy. The RANS approach produced results that were closer to the measured data than the SWE approach. If vertical accelerations were ignored, the SWE approach produced similar results and takes less computation time.

Biscarini *et al.* (2010) used a CFD (Computational Fluid Dynamics) approach for dam breach flow studies. This paper used similar methods presented in the Ozmen-Cagatay and Kocaman (2010) paper. Biscarini *et al.* (2010) used two different models and compared them to analytical, numerical, and experimental results provided in literature.

Biscarini *et al.* (2010) used a 2-D shallow water model similar to the SWE approach provided by Ozmen-Cagatay and Kocaman (2010). They also used a 3-D multiphase model that used RANS equations with the VOF method. These two models were compared to three pieces of literature. The first comparison was a dam breach over a dry bed without friction presented by Fennema and Chaundry (1990). The second comparison was presented by Soares (2002) and

was a dam breach over a triangular bottom. Finally, the third comparison was presented by Soares, Frazao, and Zech (2002) and was a dam breach flow over a 90° bend.

A comparison between the 2-D and 3-D models presented by Biscarini *et al.* (2010) with the results mentioned earlier can be observed on Figure 3.11, Figure 3.12 and Figure 3.13. It was concluded by the authors that the RANS-VOF model produced water surface levels that are lower immediately upstream of the gate and higher downstream of the gate with respect to the SWE method. The SWE model had a higher peak arrival time than the 3-D model. They stated that these differences are most likely related to the effects of gravity on the 3-D model. The authors also concluded that by looking at the comparison presented, the 3-D model can represent the unsteady flow correctly. The SWE model showed certain differences when the numerical and experimental results were compared. It can be seen that the SWE model underestimated the wave celerity when there was a triangular bottom. The SWE model was satisfactory when looking at the dam breach flow over a 90° bend, even though the wave celerity was underestimated. It was also noted that the simulation time for the RANS-VOF simulation was 2hrs compared to 15min for the SWE simulation.

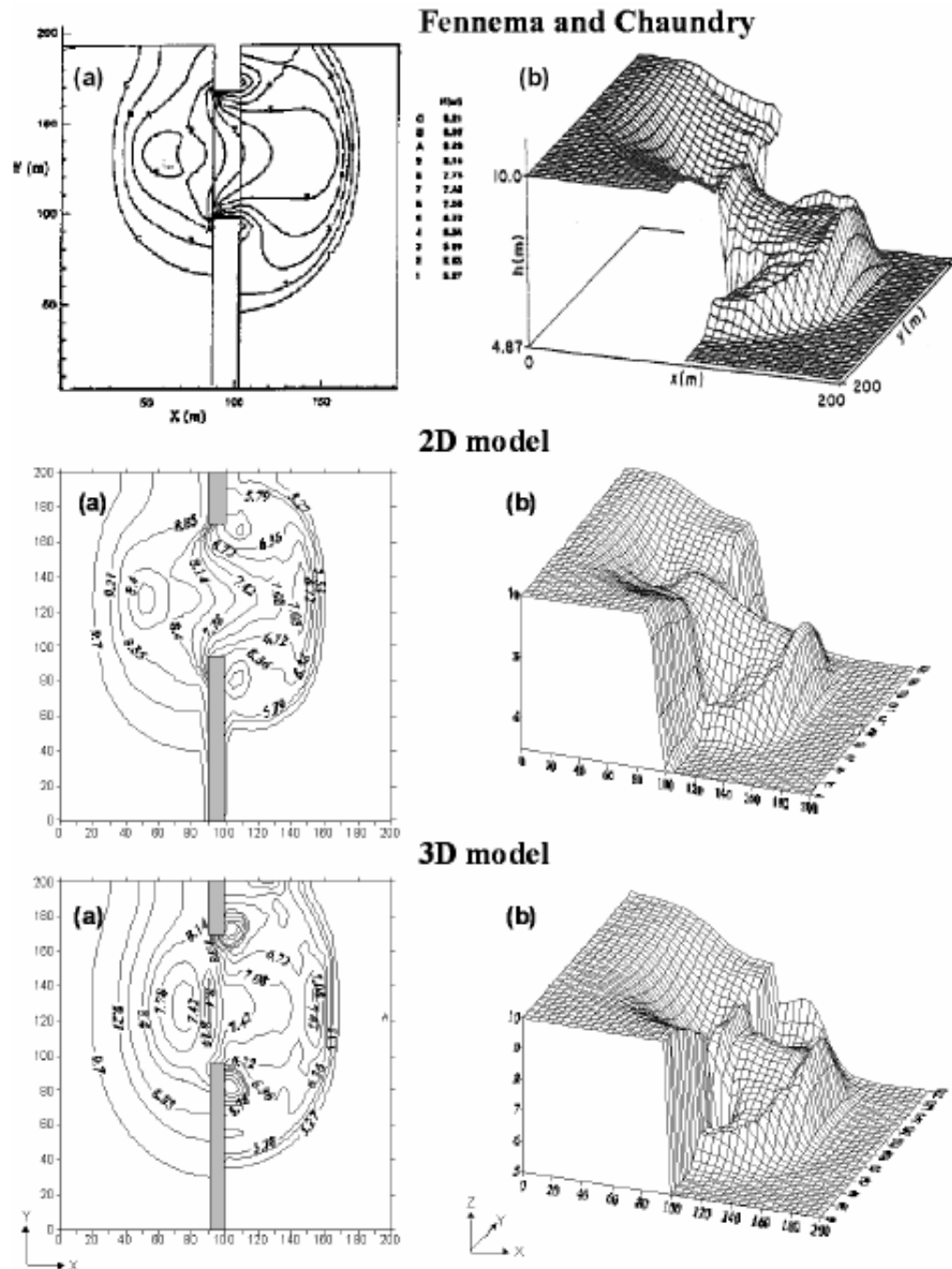


Figure 3.11: Comparisons between Dennema and Chaundry (1990) and the 2-D and 3-D models presented by Biscarini et al. (2010)

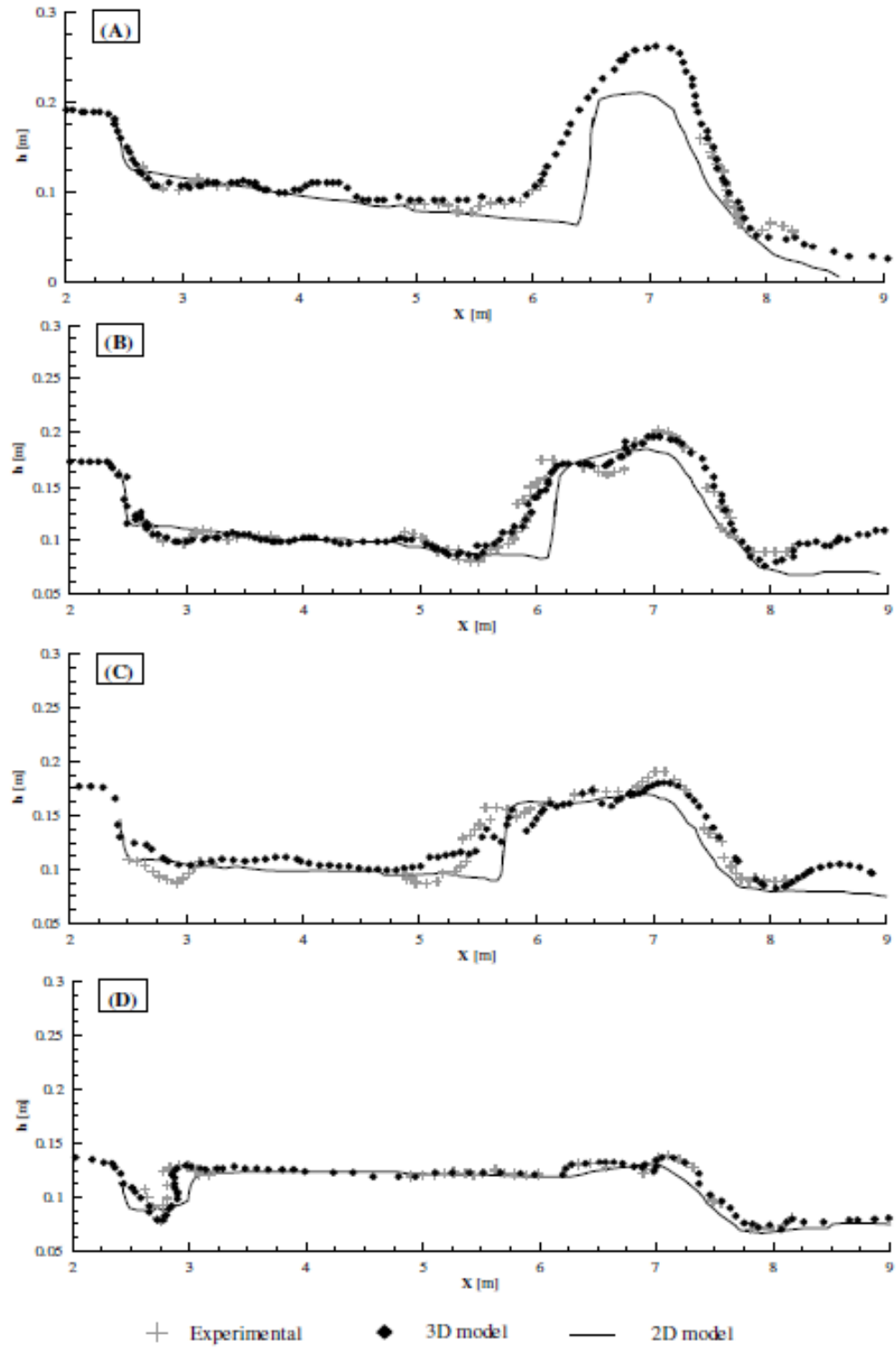


Figure 3.12: Comparisons between Soares, Frazao and Zech (2002) and the 2-D and 3-D models presented by Biscarini et al. (2010)

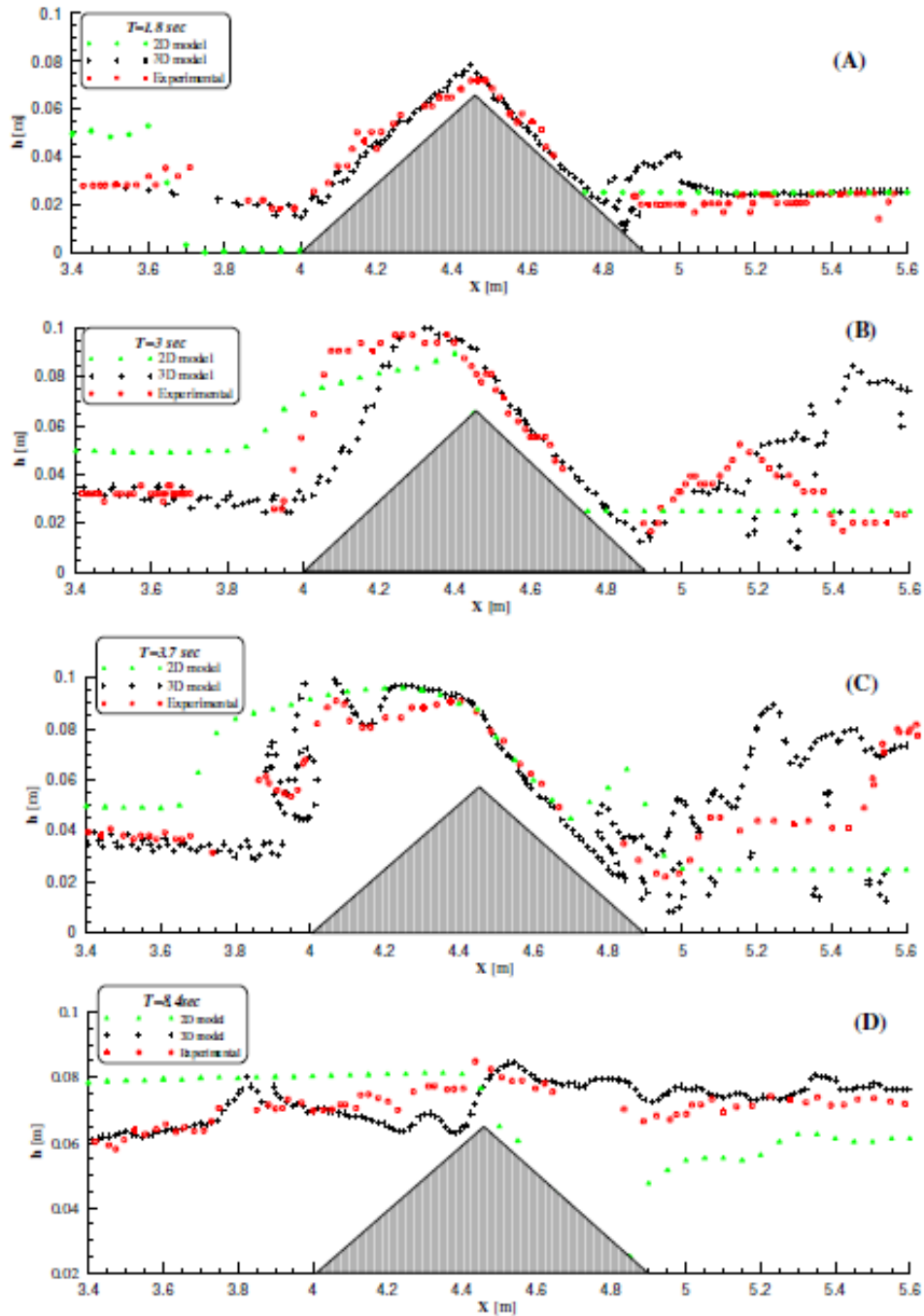


Figure 3.13: Comparisons between Soares (2002) and the 2-D and 3-D models presented by Biscarini et al. (2010) showing water elevation over triangular sill

Chang and Zhang (2010) also developed a physically-based dam breaching numerical model for a dam landslide caused by overtopping. There were 3 major components within this model and they were: breach evolution, erosion mechanics, and breach hydraulics. The authors also took into account the steepening process of the downstream slope so that they can have a

more accurate breach initiation time. The model can predict the breach shape and evolution, the erosion rate, and the outflow hydrograph.

The authors used the model to simulate the overtopping breaching of the Tanhjiashan Dam landslide in China and of the Xiaogangjian Dam landslide that were caused by the Wenchun earthquake in 2008. Due to the large number of soil deposits, the soil erodibility varied throughout the depth of the dam. When the soil erodibility was taken into account, the key breaching parameters estimated by the model were in good agreement with the observed data. The breach evolution also agreed well with the breaching process that was observed.

Chang and Zhang (2010) have also completed a sensitivity analysis on the importance of the soil erodibility variation in dam landslides. They concluded that the soil erodibility affects the breaching process significantly. When there was higher soil erodibility, there was a larger final breach geometry, shorter failure time, and larger peak outflow rate. Therefore, they concluded that all models should incorporate the soil erodibility variations to better simulate the overtopping breaching process.

3.2 Numerical Models Based on Historical Data

Foster *et al.* (2000a,b) developed a method for assessing the relative likelihood that an embankment would fail due to piping. It was called the University of New South Wales (UNSW) method. This method was based on historic failures and accidents that occurred in embankment dams. The probability of the dam failing due to piping was estimated by using the historical frequency of piping failures and comparing it to factors such as dam zoning, filters, dam age, core soil types, compaction, foundation, performance of the dam, and the amount of monitoring of the dam that occurs. The authors stated that this method should only be used for preliminary assessment and to prioritize in-depth dam studies. The authors also stated that piping failures happen very quickly and there is usually no time for remedial action after the process has begun.

Coleman *et al.* (2002) completed research on the breaching of non-cohesive homogenous embankments that have failed due to overtopping. The authors completed four small amplitude tests in a flume that have a width of 2.4m and a length of 12m. The dams were constructed to be approximately 0.3m high and were breached initially using a v-notch weir. Please refer to Table 3.2 to see the measured embankment geometry and flow data during testing.

Table 3.2: Measured embankment geometry from Coleman *et al.* (2002)

d_{50} (mm)	W (m)	ϕ_u^a (deg)	X_d^b (m)	t (s)	H (m)	Q_b^c (m ³ /s)	k' (m ^{0.5})	H_b^d (m)	X_p/X_d	S	L_e^e (m)	L_b^e (m)	A_b^e (m ²)
0.5	0.065	20.32	0.81	72	0.298	0.0018	0.56	0.238	0.42	0.459	0.113	0.328	0.0138
				100	0.300	0.0240	0.94	0.176	0.42	0.279	0.252	0.660	0.0558
				155	0.295	0.0579	1.50	0.114	0.42	0.155	0.595	1.002	0.1165
				187	0.297	0.0780	1.92	0.097	0.42	0.131	0.805	1.239	0.1664
				260	0.296	0.1420	2.56	0.059	0.42	0.075	1.198	1.526	0.2420
				302	0.297	0.1970	3.10	0.038	0.42	0.052	1.504	1.789	0.3198
0.9	0.065	20.32	0.81	44	0.300	0.0023	0.70	0.248	0.55	0.285	0.110	0.296	0.0088
				89	0.294	0.0197	1.00	0.181	0.55	0.246	0.329	0.633	0.0433
				113	0.294	0.0427	1.58	0.137	0.55	0.167	0.528	1.003	0.1007
				155	0.298	0.0747	2.00	0.101	0.55	0.126	0.834	1.267	0.1696
				217	0.296	0.1187	2.50	0.066	0.55	0.086	1.188	1.546	0.2423
				269	0.296	0.1954	3.06	0.047	0.55	0.078	1.522	1.890	0.3190
1.6	0.065	20.32	0.81	54	0.300	0.0027	0.72	0.253	0.57	0.254	0.133	0.288	0.0089
				87	0.300	0.0240	1.00	0.180	0.57	0.228	0.288	0.623	0.0437
				111	0.300	0.0430	1.50	0.139	0.57	0.138	0.586	0.992	0.0995
				147	0.299	0.0710	2.20	0.106	0.57	0.118	0.906	1.286	0.1608
				178	0.300	0.1083	2.60	0.081	0.57	0.110	1.192	1.546	0.2232
				235	0.298	0.1745	3.30	0.052	0.57	0.076	1.564	1.876	0.3061
2.4	0	21.80	0.75	25	0.290	0.0039	0.26	0.260	0.88	0.240	0.100	0.212	0.0048
				55	0.290	0.0110	0.42	0.209	0.88	0.190	0.220	0.356	0.0202
				75	0.290	0.0216	0.78	0.172	0.88	0.150	0.388	0.607	0.0516
				100	0.290	0.0394	1.25	0.142	0.88	0.110	0.517	0.897	0.0943
				125	0.290	0.0604	1.55	0.120	0.88	0.100	0.710	1.016	0.1283
				140	0.290	0.0737	n/a ^e	0.100	0.88	0.094	0.859	1.244	0.1668
				160	0.290	0.0913	1.75	0.093	0.88	0.079	1.000	1.341	0.1774
				180	0.290	0.1078	1.90	0.084	0.88	0.076	1.067	1.549	0.2317

^a $\phi_d = \phi_u$ (Fig. 4).

^b $H_s = 0.3$ m.

^cHalf-breach (Fig. 4) values.

^d $h_b = H - H_b$ (Fig. 4).

^en/a=not available.

Coleman *et al.* (2002) stated that erosion initially starts at the downstream face of the embankment and has an invert slope parallel to the face slope. The erosion of the invert slope eventually changes to flatten the breach by changing around a pivot point. They determined this pivot point to be a function of the embankment material size. They described the non-dimensional breach width under the water line (B) as:

$$B_* = 2k_* y_*^{0.5} \quad (3.40)$$

where y is the elevation above the breach bottom and where k the breach width at the embankment crest and is described as:

$$k_* = 2.82[\ln(H_{b*})] + 0.351 \quad (3.41)$$

where H_b is the centreline breach crest elevation. The breach discharge (Q_b) can then be determined using the breach crest length L_b and the amount of head on the breach-crest centerline (h_b).

$$Q_{b*} = 0.242L_{b*}(h_{b*})^{1.5} \quad (3.42)$$

The authors concluded that the findings can be used to calculate the erosion and flooding that would occur due to an embankment failure but the findings have yet to be tested against larger embankment failures.

Richards and Reddy (2007) have studied and compiled an in-depth review of published literature on the phenomenon of piping in soil. They indicated that tools to resist piping were seen as early as 1910-1935. They also stated that filter criteria was refined in the 1970's for more dispersive soils. They indicated that the amount of piping failures has also decreased recently, and it is believed that the design and construction of the dams play a large part in this. Richards and Reddy have also noted that standardized testing was available for cohesive dam breach tests that fail due to piping but no methods exist for non-cohesive dams. They believed that the numerical models that evaluate piping potential in earth dams was extremely limited and was an area that needed to be focused on in the future.

As mentioned earlier, Froehlich (2008) has also developed a non-physically based numerical model. He developed mathematical equations to predict the final breach width, side slopes, and development time using 74 embankment dam failures.

Froehlich (2008) developed his model based on a number of different model parameters. They include the reservoir water elevation at which the breach formation begins (Y_f), the critical overtopping depth (H_c), the height (H_b), the average width (B), average side slope ratio (z) on the final trapezoidal breach, and the breach formation time (t_f). Equations for B , z , and t_f were developed by completing a linear regression analysis of the 74 historic embankment failures:

$$\bar{B} = 0.27k_o V_w^{1/3} \quad (3.43)$$

where $k_o = 1.3$ for overtopping failure and 1.0 for other failure modes and V_w is the volume of the reservoir at the time of failure. The side-slope ratio can then be approximated using the following simple relation:

$$z = \begin{cases} 1.0, & \text{for overtopping failures} \\ 0.7, & \text{for other failure modes} \end{cases} \quad (3.44)$$

This simple relation was found by transforming the following equation:

$$\ln z = -0.416 + 0.389 X \text{ Mode} \quad (3.45)$$

The breach formation time can then be approximated using the following equation:

$$t_f = 63.2 \sqrt{\frac{V_w}{gH_b^2}} \quad (3.46)$$

A comparison of the measured and predicted values of breach width and breach formation times can be seen in Figure 3.14 and Figure 3.15.

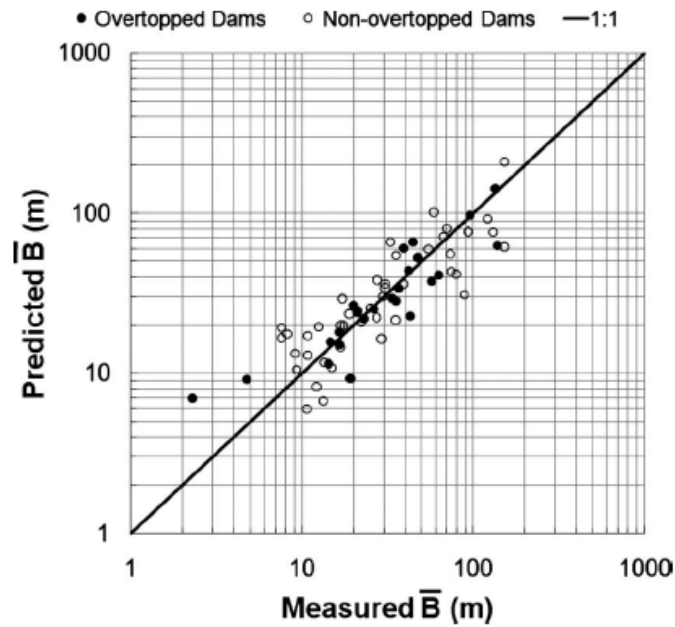


Figure 3.14: Comparison of measured and predicted average breach widths (by Froehlich, 2008)

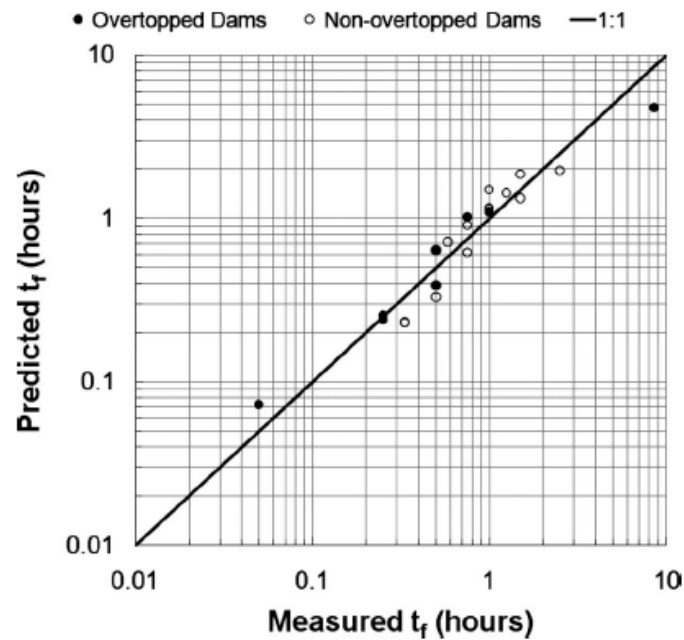


Figure 3.15: Comparison of measured and predicted average breach formation times (by Froehlich, 2008)

The author concluded that this model was believed to provide reasonable approximations for dam outflows and geometry. Froehlich stated that the geometry must be in the shape of a trapezoid and therefore there are limitations with using this model. The author also provided a probability distribution for each of the stochastic input variables. The simulations can then have probability distributions that cover all potential outcomes of the flood model. Due to the approximations made in Froehlich's (2008) model and the simplicity of it, the results will likely not be very accurate. Methods presented earlier in this study will likely produce more accurate results.

Xu and Zhang (2009) have also conducted research on the breaching parameters for earth and rockfill dams. They used a database of 182 rock and earthfill dam failures to develop two models: a multi-parameter nonlinear regression model and an additive linear model. The breaching parameters that were defined in the models were: breach depth, breach top width, average breach width, peak outflow rate, and the failure time. These parameters were defined by using five selected dam and reservoir control variables. These variables included dam height, reservoir shape coefficient, dam type, failure mode, and dam erodibility. They also looked at the relative importance of each of these control variables. They agreed with Chang and Zhang (2010), and stated that soil erodibility is the most important variable. They concluded that this variable will affect every one of the breaching parameters. They also concluded that the reservoir shape coefficient and failure mode are very important variables. The reservoir shape coefficient plays a role in all the parameters except the breach depth. The failure mode plays a large role in the breach width and the peak outflow.

After Xu and Zhang (2009) completed the two models, the authors wanted to test the accuracy on dam failures that have already occurred. They used the Banqiao and Teton dam failures to test their numerical models. The reader is directed to Table 3.3 and Table 3.4 to see the predictions of the breaching parameters for the Banqiao and Teton Dam failures. The authors concluded that the multi-parameter nonlinear regression model was a better model to predict all the parameters, except for the breach depth. The models appeared to predict the breach geometry parameters better than the peak outflow rate and failure time.

Table 3.3: Prediction of breaching parameters for Banqiao Dam (by Xu and Zhang, 2009)

Breaching parameter	Observed value	Best prediction models		Best-simplified prediction models	
		Mean prediction	95% prediction interval	Mean prediction	95% prediction interval
Breach depth (m)	29.5	25.2	19.9–30.6	25.3	20.2–30.3
Breach top width (m)	372	325	143–739	301	135–668
Breach average width (m)	291	273	116–643	262	114–601
Peak outflow (m ³ /s)	78,100	55,268	16,514–184,971	52,982	17,566–159,804
Failure time (h)	5.5	3.0	0.9–10.1	3.2	1.1–9.5

Table 3.4: Predictions of breaching parameters for Teton Dam (by Xu and Zhang, 2009)

Breaching parameter	Observed value	Best prediction models		Best-simplified prediction models	
		Mean prediction	95% prediction interval	Mean prediction	95% prediction interval
Breach depth (m)	87	78	57–98	77	58–96
Breach top width (m)	238	198	87–451	181	82–402
Breach average width (m)	151	150	64–354	127	55–291
Peak outflow (m ³ /s)	65,120	75,220	22,475–251,745	55,146	18,283–166,333
Failure time (h)	4.0	4.1	1.2–13.6	3.2	1.1–9.5

3.3 Physical Experiments

Fell *et al.* (2003) have used a logical framework to determine how quickly a dam will erode or have a pipe enlarged. Table 3.5 to Table 3.7 summarize their results. Please note that the equivalent time for a slow breach was weeks, months, or years. A medium breach had an equivalent time of days or weeks. A rapid breach occurred in hours to days. A very rapid breach occurred in less than three hours.

Table 3.5: Means of initiation of erosion based on location of breach from Fall *et al.* (2003)

Location	Means of Initiation of Erosion
Embankment	- Backward Erosion - Concentrated Leak • Transverse cracking or hydraulic fracturing due to horizontal or vertical differential settlement, desiccation, earthquake or slope instability • High permeability zone due to poor compaction, layers or coarse soil, ice lenses, desiccation during construction • High permeability zone or cracking associated with conduits and wall • Erosion into conduits or cracks in walls - Internal Instability
Foundation	- Backward erosion, including that following blowout or heave - Concentrated leak • Transverse cracking due to hydraulic fracture, differential settlement, earthquake and slope instability • High permeability zone due to coarse or structured soils, open jointing or solution features in rock, ice lenses
Embankment to Foundation	- Backward erosion, initiated by erosion of the embankment soils into open joints, coarse soils, or solution features in the foundation. This includes erosion (also called scour) at the contact of the embankment and the foundation caused by seepage flow along open joints.

Table 3.6: Breach time based on material from Fall et al. (2003)

Material Description	Likely Breach Time
Coarse Grained Rockfill	Slow-Medium
Soil of High Plasticity (LL>50%) and high clay size content including clayey gravels	Medium-Rapid
Soil of low plasticity (LL<35%) and low clay size content, all poorly compacted soils, silty sandy gravels	Rapid-Very Rapid
Sand, Silty sand, silt	Very Rapid

Table 3.7: Breach time based on location of internal erosion from Fall et al. (2003)

Location of Internal Erosion	Mechanism	Usual time for development
Embankment	Backward Erosion	Slow to Rapid/Very Rapid
	Crack/hydraulic fracture	Rapid or Very Rapid
	High permeability zone	Slow to Rapid
	Suffusion/internal instability	Slow
Adjacent or into a conduit or wall	High Permeability zone, crack, or hydraulic fracture	Rapid or Very Rapid
	Erosion into open joints or cracks	Slow
Foundation	Backward Erosion	Slow
	Backward erosion following blowout	Rapid to Very Rapid
	Backward erosion along a concentrated leak	Slow to Rapid
	Suffusion/internal instability	Slow
Embankment to foundation	Backward erosion initiating at the contact between embankment and foundation, and erosion (scours) at the embankment-foundation contact	Slow to Rapid/Very Rapid

Zhang *et al.* (2009) conducted research on breaching of cohesive homogeneous earth dams with different cohesive strengths that have failed due to overtopping. The authors completed their research in the field on dams that they constructed. They used a pumping station to pump water from the downstream river into their dam reservoir. The changing of the reservoir level was recorded by eight water level pressure sensors. The sensors also allowed for the discharge to be calculated. The erosion process was recorded using deformation sensors that had been buried into the dam body. 1m x 1m grids were also drawn onto the downstream slope so

the erosion process could be recorded by a high resolution camera. Four different experiments were completed and the main parameters of the test can be seen in Table 3.8.

A comparison of the important breach parameters by Zhang *et al.* can be seen in Table 3.9. The authors concluded that the final width and depth of the breach, and the peak outflow would become smaller and the speed of the vertical cutting and horizontal expansion would be lower, when the cohesion in the dam is larger. They believed the main reason this occurs was because of the anti-scouring ability of the dam when the cohesion was increased. They also mentioned that the horizontal expansion of the breach was higher than that of the vertical expansion. This was due to the combined process of erosion and unstable failure of the side slopes when the breach was eroded vertically. This was the main reason why the final breach width was larger than the final breach depth. The shape of the breach was also very dependent on the soil cohesion. When there was higher cohesion, the sidewalls of the breach remained at a steeper slope. With a higher cohesion, the bottom soil can be eroded by the water and the top of the side slope will stay stable. This was observed in tests F2 and F3. F3 had a higher cohesion and formed a trapezoidal shape. F2 had a lower cohesion and formed an inverse-trapezoidal shape. When the soil eroded the bottom part of the side slope in F2, the top part of the side slope collapsed almost instantly. In F3, the top part of the side slope was stable when the bottom part of the side slope eroded away.

Table 3.8: The main parameters in the tests by Zhang *et al.* (2009)

Test group	F1	F2	F3	F4
Clay content (%)	11.50	11.50	17.80	33
Compactness (%)	97	96	98	92
Water content (%)	19.42	15.74	17.60	28.55
C (kPa)	9.3	7.5	13	39.5
ϕ	28.25	27.8	16	14.4
Threshold friction velocity of undisturbed earth U_{*c} (m/s)	0.06	0.06	0.17	0.35

Table 3.9: Comparison of breach parameters by Zhang *et al.* (2009)

	C (kPa)	Depth of breach (m)	Width of breach on the top (m)	Width of breach on the bottom (m)	Peak outflow (m/s)	Speed of vertical cutting (cm/s)	Speed of horizontal expansion (cm/s)	Shape
F2	7.5	4.1	17	3.7	42.25	0.44	1.49	inverted trapezoid
F3	13	1.7	2.1	7.1	3.7	0.022	0.065	trapezoid
F4	39.5	1.1	1.3	0.9	1.07	0.011	0.024	big hole

In addition to Visser (1998) and Zhu (2006), Schmocker and Hager (2009) also completed research on dike breaching due to overtopping. They conducted a series of 39 plane dike tests that have breached due to overtopping. They stated that laboratory tests are related to scaling issues which are not currently understood. They believed the new series of tests will help examine model limitations. The authors varied the dike dimensions and sediment size so conclusions on test repeatability, side wall effect, and scaling effects could be concluded. The tests were completed in a glass flume which can be viewed in Figure 3.16.

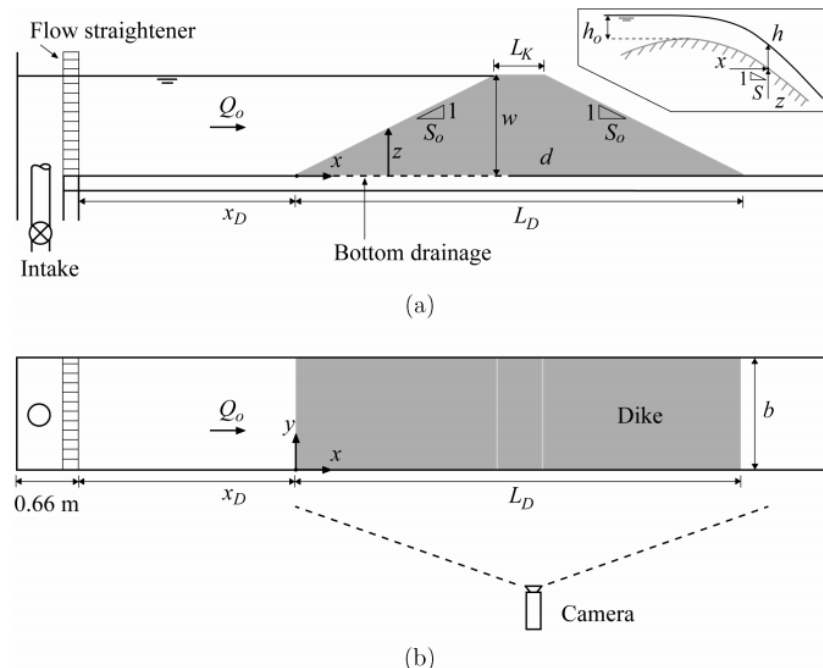


Figure 3.16: Glass flume layout from Schmocker and Hager (2009)

Schmocker and Hager (2009) concluded that when the sediment size reached 8mm in diameter, from the starting point of 1mm, there was a large issue with the amount of seepage. When the seepage was able to reach the downstream dike crest, a sliding failure would occur and a large portion of the dike would fail almost instantly.

Figure 3.17 shows the typical erosion process of a dike at six different stages throughout the experiment. Tests were also concluded during the experiment to ensure the sidewall effect would not affect the results. The authors used the Froude similitude for scaling. They concluded that it seemed that all of the scaling factor issues could be avoided if a sliding failure did not occur.

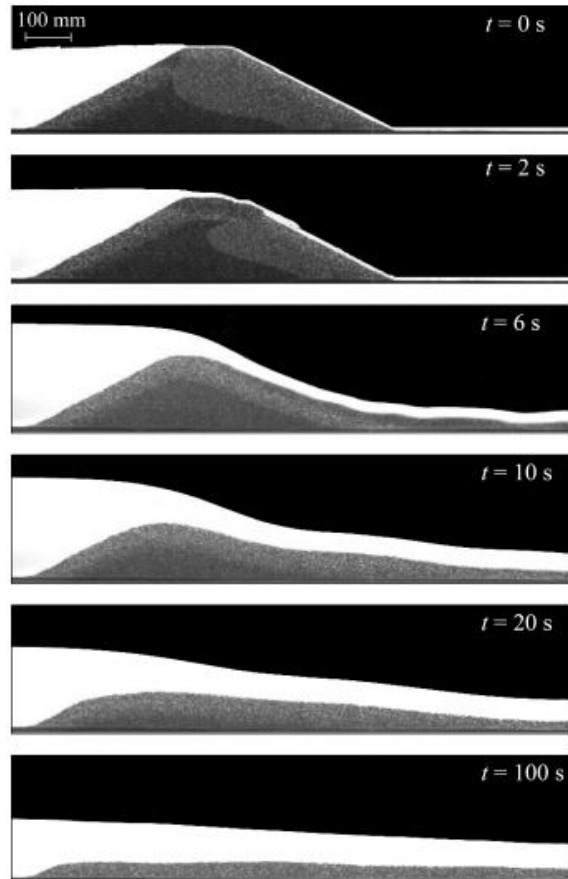


Figure 3.17: Typical erosion process of dike from Schmocker and Hagen (2009)

3.4 Examples of Numerical Modelling

Yusuf, Lang, and Garrett (2009) described a project initiated by BC Hydro to update the inundation maps for all their dams in the event of a failure or malfunction. BC Hydro believed that new maps were needed for better emergency planning and response. They completed updates for 41 dams across British Columbia and looked at several different breach scenarios and spillway discharges. This project was termed Flood Simulation and Mapping Model or FloodSiMM.

BC Hydro has also implemented an on-demand reservoir routing model and inundation mapping model for each dam. This will allow for a more accurate emergency response. For this to occur, a flexible model must be used to map areas with different project outflows and river systems. This will allow for BC Hydro to have updated inundation maps within 24hrs of the notice of emergency.

Past inundation maps created for BC Hydro's dams were completed using DAMBRK. The authors pointed out some limitations of DAMBRK including:

- Inability to simulate flood wave propagation through multiple channels
- Inability to simulate multiple breaches in the same reservoir

- Supercritical and subcritical reaches have to be run separately
- The 1-D approach is not sufficient for some applications where 2-D effects are present. Some of these include super-elevation around a bend, or the flow through a floodplain.
- DAMBRK does not provide visual representation.

The authors believed that the estimation of the dam breach parameters was the most difficult thing to undertake in dam breach flood inundation modelling because it contains one of the largest sources of uncertainty. They stated that numerical modelling that incorporates both soil mechanics and hydraulic principles was still being developed. For breach parameters, BC Hydro used prediction equations that are shown in Table 3.10.

Table 3.10: Prediction equations used by BC Hydro

Reference	Breach Width	Failure Time
1. USBR (1988)	$B_{avg} = 3h_w$	$t_f = 0.011B_{avg}$
2. MacDonald and Langridge-Monopolis (1984)	$V_{er} = 0.0261(V_w h_w)^{0.769}$	$t_f = 0.0179V_{er}^{0.364}$
3a. Von Thun and Gillette (1990) (highly erodible)	$B_{avg} = 2.5h_w + C_b$	$t_f = 0.015h_w$ $t_f = B_{avg}/(4h_w + 61)$
3b. Von Thun and Gillette (1990) (erosion resistant)	$B_{avg} = 2.5h_w + C_b$	$t_f = 0.02h_w + 0.25$ $t_f = B_{avg}/(4h_w)$
4. Froehlich (1995)	$B_{avg} = 0.1803K_o V_w^{0.32} h_b^{0.19}$	$t_f = 0.00254V_w^{0.53} h_b^{-0.9}$

B_{avg} = average breach width (m)
 C_b = offset factor, varies with reservoir storage (e.g. 54.9 for reservoir storage $> 1.23 \times 10^7 \text{ m}^3$)
 h_b = height of breach (m)
 h_w = depth of water above breach invert at time of failure (m)
 K_o = overtopping multiplier: 1.4 for overtopping, 1.0 for piping
 t_f = failure time (h)
 V_{er} = volume of embankment material eroded (m^3)
 V_w = volume of water stored above breach invert at time of failure (m^3)

The authors believed that the assessment of uncertainties was very important. They believed this because the historical dam failure data may or may not be representative of other dam failure scenarios. For example, Wahl (1998) stated that there is not very much information on large dam failures and that 75% of the data that does exist comes from dams having a height of less than 15m.

To compute the outflow hydrographs, BC Hydro used a 1-D hydrodynamic model, MIKE11. This technique treats the breach parametrically and does not try to simulate the complex processes that actually occur in a dam breaching. Chauhan *et al.* (2004) believed that this was a very conservative approach since the equations used are based on final breach geometry. This included the passing of the entire breach hydrograph, including the falling limb section, though the breach. In actual fact, the breach continues to grow while the limb falls.

This indicates that the peak flow in the hydrograph is expected to occur before the breach reaches its maximum size.

To route the floodwave and create the inundation maps a number of different hydraulic models were considered. These included Telemac2D, MIKE21, MIKEFLOOD, SOBEK, and InfoWorks RS. Telemac2D was chosen because it had the fastest computing times of all the models, it was very robust and had mesh generating features that were not offered in other models. MIKE11 was still used when only a 1-D model was needed.

The authors stated that to maintain a balance between computational efficiency and mesh detail, some floodplain elements such as elevated roads, culverts, bridge crossings, and ditches may not be fully represented in the hydraulic models. They also assumed that flood control measures such as dikes will remain intact.

3.5 Model Validation and Comparative Studies

Singh and Snorrason (1984) completed a study that evaluated breach parameters from a historic literature survey and then compared them to two dam breach models. The first model was the U.S. Army Corps of Engineers HEC-1 and the second model was a routing method developed by the National Weather Service in 1977 that was based on St. Venant Equations. A sensitivity analysis on important breaching parameters used in the models was also completed. This study is very similar to the one being completed in this thesis except newer state of the art models are being used in the present study. Singh and Snorrason used a total of eight dams in Illinois and compared the results of the dam breaching characteristics discussed earlier in Section 2.3.

Wahl (2004) analysed the amount of uncertainty that is present in the models that are based on historic dam failures. Most of these models often require the user to predict the basic geometry and parameters of a breach. This could include the peak breach outflows as well as other characteristics. While it is known that there is a large amount of uncertainty using these models, the amount has never been quantified. He concluded that the failure time ranged from ± 0.6 to ± 1 order of magnitude and the peak discharge ranged from ± 0.5 to ± 1 order of magnitude for the five methods of prediction he used.

Davies *et al.* (2007) completed a case study on modelling the magnitude of dam landslide breach floods. Dam landslides are usually constructed out of unconsolidated and poorly graded soil. The authors stated that dam landslide failures are very rapid and result in sudden floods downstream. These are very serious natural hazards and can lead to flash floods. The authors compared the peak outflow they recorded from the Poerua dam landslide in October 1999 to field estimates, laboratory tests, and computer modelling. The authors concluded that some of the empirical estimates were less reliable. They also stated that the physically-based Numerical Model, Boss BREACH, was reasonably accurate and very quick to use. This allowed for calculations to be completed in a much more efficient manner.

Vasquez and Roncal (2009) tested River2D and Flow-3D for simulating instantaneous dam-breach flows using high-quality experimental data. They looked at 3 different test cases. The first case was a simple straight channel with a change in elevation at the dam location. The second test case had an isolated obstacle downstream of the dam. This simulated structures such as powerhouses, bridges, or buildings, and this was very important when the dam was close to an urban area. The third test case considered the presence of a raised triangular sill downstream of the dam.

The authors concluded that both models showed good agreement with the first test case and they were unable to determine which model performed better. The second case is a much more complex dam breach than the straight flume. This resulted in the numerical results being less accurate. It appeared to the authors that Flow-3D produced more accurate results than River2D. They believed that this is most likely due to the vertical velocities that were created when the flood wave hit the structure. Since River2D cannot incorporate these vertical velocities the results were not as accurate. For the third test case, Flow-3D produced excellent results when compared to the experimental results. It was able to capture all the water level fluctuations at the different gauges. The results computed by River2D were not as precise. The model was able to simulate some of the fluctuations but over-predicted some of the water levels downstream of the sill.

3.6 Discussion

A thorough analysis of the various papers and documents found in the literature indicated that the most widespread method of research on dam breaching is using physically-based numerical models. Most of these physically-based numerical models are based on experiments performed in laboratory or in the field. Further, the results of such experimental programs are often compared to historical dam failures to validate. A main issue with many of the physically-based models presented in both the literature review and in Table 1.1 is that their results are not available for the industry to use. This makes it very difficult for others to validate them or personally use them in the industry. The results from the experiments completed for model calibration/validation are also usually not fully available. This makes it very difficult for other researchers to validate or use them and, therefore, the reader is left making small assumptions that may or may not be true.

There are a few researchers who have studied non-physically based numerical models. These researchers, such as Xu and Zhang (2009) and Froelich (2008), have used a large data base of historic dam failures and conducted regression analysis based on this data. The authors have used 182 embankment failures and 74 embankment failures, respectively. The literature presented for the non-physically based numerical models also does not include much of the actual data regarding the material and equipment used to construct the physical model. In some cases data was missing while in other cases the authors do not cite where from they received the data. This leads the reader to believe that approximations were once again used. This was very

evident when looking at the Teton Dam failure. As mentioned by Zhu (2006), many researchers have used the Teton Dam failure to calibrate or validate their models. However, the data provided by the researchers is usually limited and contradicting. Table 1.2, in the Introduction section, shows a summary of the data used by different authors. It can be concluded that there is a lot of missing and contradicting data. Since there was no data recorded during the dam failure, the authors are making their own predictions on characteristics of the dam breach. The accuracy of the models can therefore be biased during the validation stage because the modeling of the dam failure may be influenced by the physical model results.

Yusuf, Lang, and Garrett (2009) described the process of creating flood inundation maps for dams in British Columbia. This project was funded by BC Hydro and the authors looked at a number of different programs and scenarios when creating the flood maps. The authors analyzed numerous programs to route the flood downstream of the dam, subsequent to its breach, such as Telemac2D, MIKE21, MIKEFLOOD, SOBEK, and InfoWorks RS. In the end, the authors chose Telemac2D to route the outflow hydrograph downstream and this was due to its quick computation time. The authors have however used MIKE11 to create the breach outflow hydrograph due to the simplicity of the model.

Singh and Snorrason (1984), Wahl (2004), Davies *et al.* (2007), and Vasquez and Roncal (2009) have completed studies on dam breaching models validation. Another major study on numerical models validation was also completed through the IMPACT-Project and is presented by Morris (2005). The IMPACT-Project is described in more detail later in this thesis. All of the authors mentioned except Wahl (2004) compared the results of two or more models to laboratory or field experiments. Wahl, however, completed research on the uncertainty associated with a number of non-physically based models to see how accurate they were.

4.0 Numerical Models

4.1 MIKE by DHI

The MIKE by DHI software suite is a series of models developed by the Danish Hydraulic Institute. The focus of this research will be on the MIKE11 model but there are other models included in the software suite. MIKE11 is 1-D model that can simulate a dam breach using the Dambreak (DB) module. The outflow hydrograph from this dam breach can then be inputted into MIKE21 or any other flood routing software to route the floodwave downstream. The floodwave can be routed using MIKE11 but the river morphology cannot change due to the increased flow (as it should). MIKE21 can model the river morphology over time due to the great increase in flow. MIKEFLOOD can be used to integrate the two MIKE models together. The user can therefore use the dam break module in MIKE11 and route the flood wave using MIKE21 simultaneously.

MIKE11 is a software package that includes many different modules. These include the hydrodynamic module (HD), advection-dispersion (AD), and water quality (WQ). The focus of this research will be done using the hydrodynamic module (HD).

The hydrodynamic module computes unsteady flows in rivers and estuaries using an implicit, finite difference scheme. It can describe both supercritical and subcritical flows within the river or estuary depending on the local flow conditions. Within the HD module, other computational models can be included to describe dam breaks or flow around structures.

4.1.1 Bed Resistance

When calculating the bed resistance MIKE11 can use multiple methods including Chezy, Manning, and Darcy-Wiesbach. The method that is desired can be chosen within the Hydrodynamic Parameters Editor under the Bed Resistance tab.

When using Chezy, the bed resistance term within the momentum equation is:

$$\frac{gQ|Q|}{C^2AR} \quad (4.1)$$

where Q is the flow, A is the river cross-section area, and R is the hydraulic radius. When using the Manning equation, the bed resistance term is:

$$\frac{gQ|Q|}{M^2AR^{4/3}} \quad (4.2)$$

where M is the Manning number and it is equivalent to the Strickler number or the inverse of the more commonly used Manning n . Therefore, since n usually ranges between 0.01 and 0.10, M usually ranges between 100 and 10. The Chezy coefficient and Manning's number can be related by:

$$C = \frac{R^{1/6}}{n} = MR^{1/6} \quad (4.3)$$

4.1.2 Hydraulic Radius

The hydraulic radius can be found for each cross section. The hydraulic radius for a cross-section with a constant resistance is simply the cross-sectional area divided by the wetted perimeter. If there are multiple relative resistance factors, the hydraulic radius can be found using:

$$R_h = \left(\frac{\sum_{i=1}^N \left(\frac{A_i^{\frac{5}{3}}}{r_{r_i} P_i^{\frac{2}{3}}} \right)}{A} \right)^{3/2} \quad (4.4)$$

Within MIKE11, it is possible to split each cross-section into a main channel and one or two flood plains. This is done by setting markers indicating where the channel banks are located. This would split the cross-section into three parallel channels. The hydraulic parameters would then be carried out for each of the channels.

4.1.3 Additional Resistance due to Vegetation

Additional resistance due to vegetation can be expressed by using the dimensionless Darcy-Weisbach coefficient. The Darcy-Weisbach coefficient can be found using:

$$\frac{1}{\sqrt{\lambda}} = -2 \log_{10} \left(\frac{k_s}{14.84R} \right) \quad (4.5)$$

After rearrangement yields:

$$\lambda = \frac{1}{\left(2 \log_{10} \left(\frac{k_s}{14.84R} \right) \right)^2} \quad (4.6)$$

where λ is the Darcy-Weisbach resistance factor, k_s is the resistance number, and R is the hydraulic radius.

The dimensionless Darcy-Weisbach coefficient can also be calculated using:

$$\lambda = \frac{8g}{C^2} \quad (4.7)$$

where g is the acceleration due to gravity and C is the Chezy coefficient.

By using the Darcy-Weisbach coefficient, it is possible to estimate the amount of bottom shear created by the velocity distribution between the main channel and the flood plains. A coefficient is determined for both the interaction occurring between the left flood plain and the main channel and the interaction between the right flood plain and the main channel. These interactions occur at M4 and M5 in Figure 4.1.

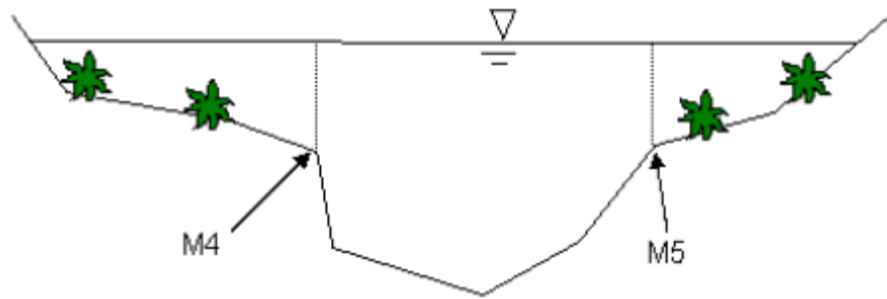


Figure 4.1: Interaction location between flood plain and main channel

4.1.4 Boundary Conditions

Boundary conditions are needed at each of the model boundaries and more specifically at the upstream and downstream ends of the river being modelled. The relationships that can be applied at the boundary conditions are: constant values of h or Q , varying values of h or Q , or a rating curve showing the relationship between h and Q (this can only be used at the downstream boundary).

The type of boundary condition that is chosen depends on the situation and the data available. Typical upstream boundary conditions include a constant inflow from a reservoir or an inflow hydrograph. Typical downstream boundary conditions include a constant water level, a time series of water level, or a reliable rating curve from a gauging station.

4.1.5 HD Coefficients

Within the HD module there are multiple HD parameters that have been set with a default value. These can be changed by the user if desired. These HD parameters include:

- **Alpha Coefficient:** This is used in the momentum equation as the velocity distribution coefficient. (default = 1.0)
- **DELH Coefficient:** The depth and top elevation of the slot is controlled by DELH during low flow conditions. The slot begins at a height of DELH off the bottom of the river and continues to a depth of $5 \times \text{DELH}$. (default = 0.1m)
- **DELHS Coefficient:** When the water surface gradient across a structure changes direction, DELHS helps prevent instabilities. DELHS specifies the difference in water level across a weir to obtain a stable solution. (default = 0.01m)
- **DELTA Coefficient:** has a dissipating influence of the forward centring of the dh/dx term in the numerical scheme. This can be demonstrated using a Taylor-series. A default value of 0.5 has no dissipative effect. A value of 1.0 has a maximum influence.
- **EPS Coefficient:** This is used in approximating the diffusive wave. If the slope of the water surface becomes larger than EPS, then the river becomes upstream centered. (default = 0.0001)

- **FroudeExp:** This is used in the suppression of the convective terms in the Momentum equation for supercritical flow. The default suppression is applied if there is a negative value for FroudeExp or FroudeMax.
- **FroudeMax:** This is where the suppression of the convective terms in the Momentum equation takes place. As noted in FroudeExp, the default suppression is applied if a negative value for FroudeMax is entered.
- **Inter1Max:** This is the maximum number of iterations that can be completed in a time step to find a solution around a structure. (default = 10)
- **MaxIterSteady:** When completing an autostart with initial conditions, this is the maximum number of iterations that can be completed to find a steady state water profile solution. (default = 100)
- **NODE Compatibility:** This is to determine if the calculations done at the different nodes should be completed with water level compatibility or with energy level compatibility. This default is to be completed with water level compatibility since energy level compatibility is not yet available.
- **NoITER:** This defines the max number of iterations in each time step to find a solution. (default = 1)
- **Theta:** This is used in the resistance term in the momentum equation. The default is 1.
- **ZetaMin:** This specifies the minimum sum of head loss factors around a structure. ZetaMin is an optional parameter the user can input.

4.1.6 Computational Grid

The most important part of modelling is choosing the computational grid. A good computational grid will avoid various problems. Some guidelines are:

- Topographical information should be available for the entire area being studied.
- The boundary conditions should be known at all boundaries.
- The boundary should be located far enough away from the study area to ensure that the study area doesn't affect the boundary.
- Limit the number of branches in the river if possible. Only include more branches if they are required.
- It is important to select the right grid size. However, for inexperienced users this must be done on a trial and error basis. Ideally, the model should have a small grid space and then increased grid spacing in subsequent runs.

4.1.7 Cross Sections

The cross-sections are inputted by a number of x-z coordinates and give the topographical information of the area. The flow should be approximately perpendicular to the cross-section. The x-coordinate is the transverse distance from the left bank top. The z-coordinate is the corresponding bed elevation for the x-coordinate. For each point, a resistance value (Manning, etc) can be applied.

The number of cross-sections and points within the cross-section are determined both physically and mathematically. Physically there should be enough cross-sections to show the variation in channel slope as well as any changes within the cross-section. Mathematically, the number of cross-sections is determined when choosing the computational grid. All of this information is entered into the cross-section editor.

If, during the simulation, the water level is higher than the inputted cross-section, the model will assume the banks extend directly upward. Please refer to Figure 4.2. The model will let the user know if the water level has extended four times higher than the cross-section (four times is the default and can be changed). When designing the cross-section, the user should try to avoid this by making the cross-sectional area large enough.

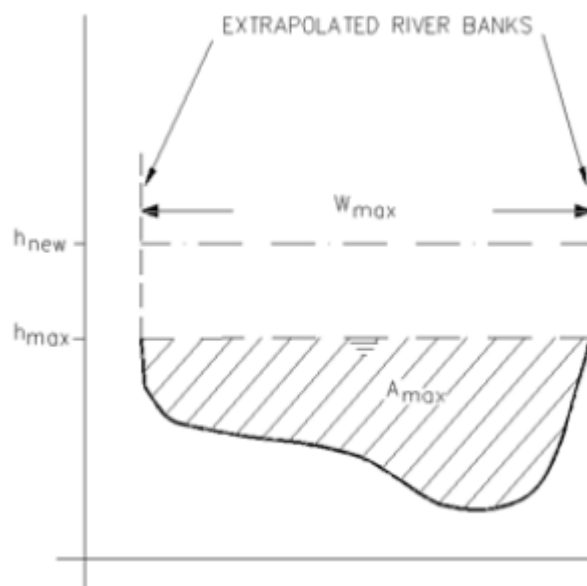


Figure 4.2: Area above maximum height (from DHI Water & Environment, 2009)

4.1.8 Dambreak Structure

A dam break structure can be used within MIKE11 to simulate a dam breach. This is a structure that consists of a structure that represents the dam overflow as well as a structure that represents the actual breach of the dam. The dam break structure can be breached either by overtopping or by piping.

The breach can develop within the dam break structure using two different methods. The first method is using the energy equation to describe the flow through the dam breach. The second method is to use the numerical method presented by the National Weather Service in DAMBRK. This project will only use the energy equation to calculate the dam breach since BREACH is an updated version of DAMBRK.

When using the energy equation, the flow over a dam is much like over a broad crested weir with a few exceptions. First, the shape of dam will change over time. The dam crest is decreased and the breach increases as time passes. Secondly, the flow-height relationships for

the crest and for the breach are different. Please refer to Figure 4.3 to see the combined flow. Therefore, the flow must be calculated separately for flow over the crest and for the flow through the breach.

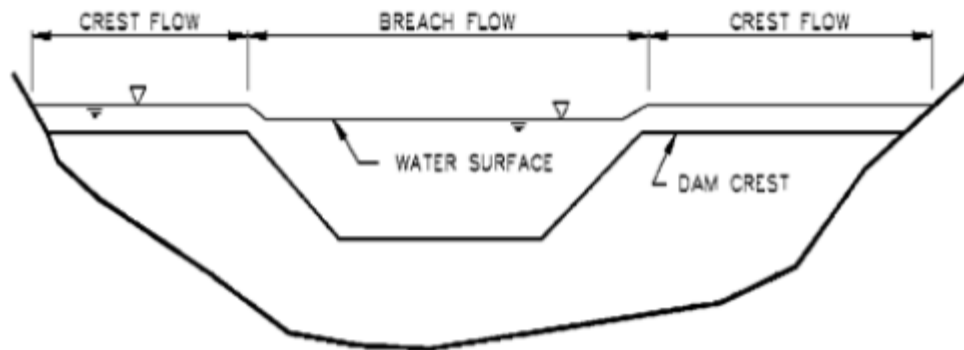


Figure 4.3: Combined flow over the Dam (from DHI Water & Environment, 2009)

The dam can breach either as a trapezoidal section or as a circular piping failure if the erosion based method is used. During the development of the trapezoidal breach, the breach increases in size and changes shape. The initial breach shape of the trapezoid is inputted by the user and includes the level of the breach bottom, the corresponding breach width, and the side slopes. The breach can then develop depending on the sediment transport capacity of the flow or as a known function of time. For this project, the breaches will develop based on the sediment transport capacity since it is assumed that the breach development is not known before.

When the geometry of the breach is known as a function of time, this can be inputted into the boundary editor. All inputs will be relative to the start of the breach. Any time that falls between the inputted times will result in the data will be interpolated. For an example, please refer to Figure 4.4. In this case, breach geometry is given at times 0hr and 1hr and is represented in a solid line. The 0.5hr breach geometry would then be interpolated and this is shown as the dashed line.

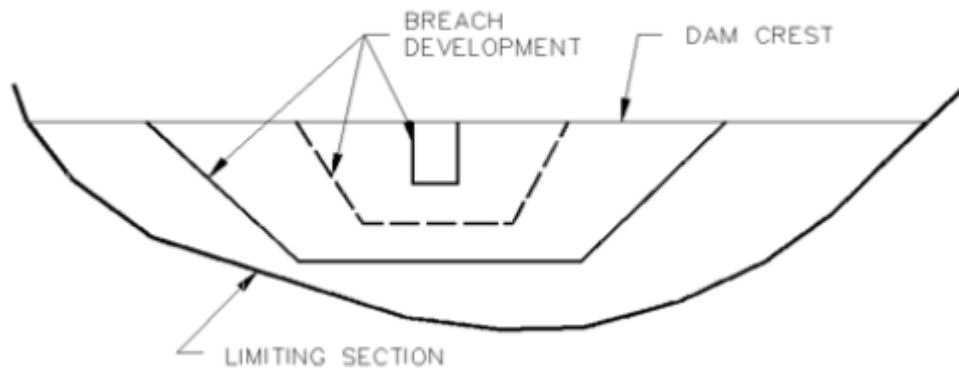


Figure 4.4: Breach interpolation (from DHI Water & Environment, 2009)

When erosion is used to develop the breach using the energy equation, the initial and final breach shape must be inputted into the model. The Engelund-Hansen equation is the used to calculate the sediment transport in the breach. The Engelund-Hansen equation is:

$$\Phi = 0.1 \frac{\theta^{5/2}}{f} \quad (4.8)$$

$$\Phi = \frac{q_t}{\sqrt{(s-1)gd^3}} \quad (4.9)$$

$$f = 2 \frac{u_f^2}{u^2} \quad (4.10)$$

where Φ is the sediment transport rate(dimensionless), θ is the total shear stress (dimensionless), q_t is the total bed material transport per unit width, f is the friction factor, and u_f and u are the friction and current velocities.

The first step that is taken in the model is calculating the flow resistance in terms of the total dimensionless shear stress. This can be calculated using the Engelund formulation. The user must enter a critical shear stress from Shield's curve. The shear stress that is calculated is compared to the critical shear stress. If the computed shear stress is greater than the critical, then sediment transport occurs and it must be calculated.

The sediment transport rate is again calculated from the Engelund-Hansen method. This is in terms of m^2/s per m width of pure sediment only. It must then be converted to how much it lowers the breach level. This can be done using:

$$\frac{dH_b}{dt} = \frac{q_t}{L_b(1-\epsilon)} \quad (4.11)$$

where H_b is the breach level, t is time, q_t is the sediment transport rate, ϵ is the sediment porosity, and L_b is the length of the breach.

Modelling the change of the width of the breach due to sediment transport was one of the more difficult things to do according to the Danish Hydraulic Institute. This is because all of the classical theories of sediment transport do not take into account the steep side slopes that would occur in a breach channel. Therefore, they have made the sediment transport rate on the side slopes proportional to the sediment transport occurring on the bottom of the breach channel. The equation below shows the relation, with the proportionality constant, x , being between 0.5-1.

$$\frac{dW_b}{dH_b} = 2x \quad (4.12)$$

where W_b is the breach width, H_b is the breach height, and x is the proportionality constant.

Pipe failure can also be modelled based on erosion. This starts off by a flow forming through the dam and the sediment transport enlarging the pipe until the dam collapses. Some assumptions that DHI made about piping failures are:

- The shape of the pipe is circular.
- The pipe is directly horizontal through the dam.
- The pipe is always running full. Therefore, it must always be below the waterline on the dam.
- The location of the centre of the pipe must be within the final breach area, and limiting section. If not, the model will change the dimensions of the pipe, as seen in Figure 4.5.

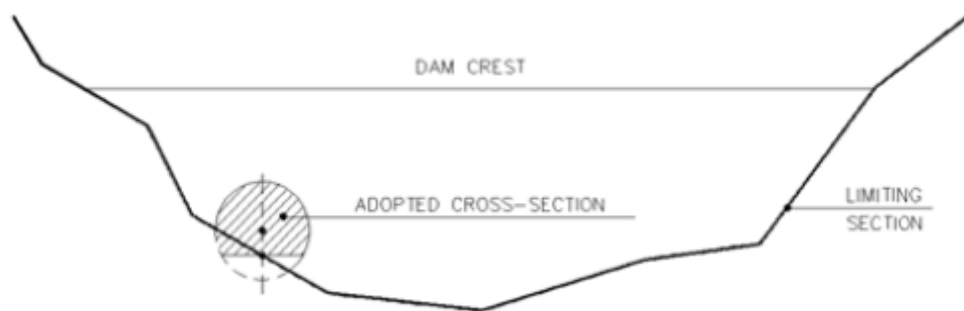


Figure 4.5: Adjusted pipe cross-section (from DHI Water & Environment, 2009)

The dam will collapse when a ratio of the pipe diameter and the distance between the crest of the dam and the pipe invert is met. This ratio is defined by user when the dam-breach structure is inputted. An illustration of this can be seen in Figure 4.6.

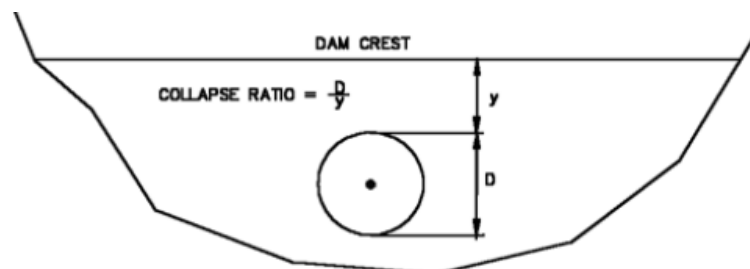


Figure 4.6: Collapse ratio (from DHI Water & Environment, 2009)

When the collapse happens, the shape of the breach can be seen in Figure 4.7. The bottom width of the breach is equal to the diameter of the pipe when the breach occurs. The bottom of the breach will be the same elevation as the pipe invert. Some of the material that has collapsed will settle on the breach bed. The amount of material that will settle on the breach bed will be calculated using f_{lost} . This is the fraction of the total material that has collapsed that is going to be washed away instantly. The material that is not washed away will be distributed evenly over the width the breach bed.

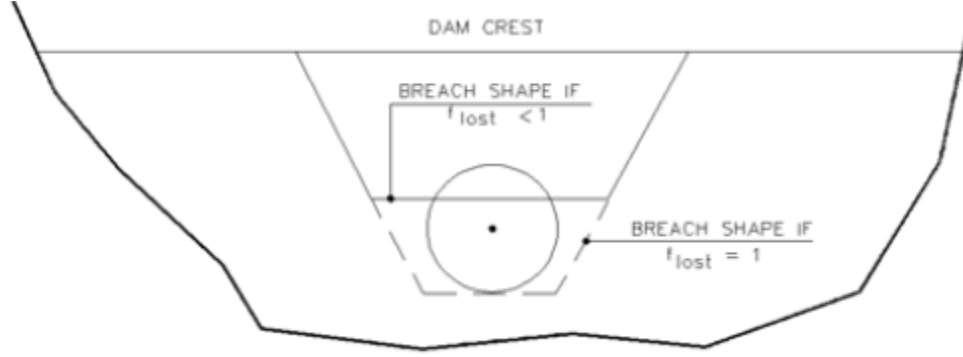


Figure 4.7: Breach shape after collapse (from DHI Water & Environment, 2009)

Another parameter that the model must calculate is the flow through the pipe. This is done using the following equations:

$$Q_p = A \sqrt{\frac{2g\Delta H}{\left(1.5 + \frac{fL}{4R}\right)}} \quad (4.13)$$

$$\frac{1}{\sqrt{f}} = 2 \log_{10} \left(\frac{12R}{k_s} \right) \quad (4.14)$$

$$\Delta H = h_1 - \max(h_s, z_{obv}) \quad (4.15)$$

where Q_p is the flow through the pipe, A is the cross-sectional area of the pipe, g is gravity, f is the Darcy friction factor, R is the hydraulic radius, h_1 is the upstream water level, h_2 is the downstream water level, and z_{obv} is the overt of the pipe.

The pipe will get larger and larger due to erosion. This erosion is due to the sediment transport relationship discussed earlier. The depth of water used in the equation to calculate the sediment transport is calculated by:

$$y_p = \frac{\Delta H}{2} + D \quad (4.16)$$

where y_p is the depth of water, ΔH is calculated above, and D is the diameter of the pipe.

The change in the pipe diameter can then be solved for by using the area of sediment lost and calculating the new effective transport area. The change in the pipe radius is calculated using:

$$\Delta R_p = C_{cal} \frac{q_t}{2L_p(1-\epsilon)} \Delta t \quad (4.17)$$

where C_{cal} is the coefficient of calibration and it is entered as one of the erosion parameters. DHI states that there is no recommendation as to what C_{cal} should be and, if desired, the user should play around with it to get the sought results. This allows for a calibration of the formation of the pipe if data becomes available.

A limiting cross-section can be inputted into MIKE11 to represent a boundary where the breach cannot exit. An example of this might be a hard rock or natural bottom of the valley. The full cross-section can be specified, but the model will only use the area within the limiting cross-section boundary.

4.1.9 Types of Flow Used in Model

The types of flow can vary in MIKE11. The user can choose between three different flow types depending on the problem. The user must choose the most appropriate flow description for the problem that is being simulated.

The first type of flow is called the dynamic wave approach. This approach uses the full momentum equation. This includes all the acceleration forces and therefore allows the model to simulate tidal flows, etc. and backwater profiles can be calculated. This should be used where the inertia change of the water body is important. Therefore, this should be used with tidal flow and in river systems where the water surface slope, bed slope, and bed resistance factors are small. The governing equations used in this flow type are the Saint Venant Equations which are covered later in this study.

The second type of flow is called the diffusive wave approach. This only uses certain terms in the momentum equation. More specifically, the model only uses the bed friction, gravity, and hydrostatic gradient terms in the momentum equation. Therefore, it ignores all the inertia terms. This should be used in cases where backwater analyses is needed and in cases where bed resistance is very large. Slowly propagating flood waves would have negligible inertia change anyway. This is not used for tidal flows.

The last type of flow is called the kinematic wave approach. This only calculates the flow based on the assumption of a balance between the friction and gravity forces. This flow type is only used in rivers that are very steep and that have no backwater effects.

The diffusive and kinematic wave approach should only be used when the omitted terms have an insignificant change on the model. These models do however provide a better computational efficiency. When there is doubt it is best to use the dynamic wave type.

4.1.10 Initial Conditions

When the user starts the simulation, a number of initial conditions can be selected. They can be either:

1. User Specified – This selection will allow the user to enter the initial conditions into the simulation editor under the simulation tab.
2. Auto Start, Quasi-Steady Solution – An automatic calculation is performed in which the water levels and discharges are determined using the boundary conditions given at the

start. This is done by using a steady state version of the Saint Venant Equations. The model iterates to find the correct discharge going into each of the branches.

3. Hot Start – A hot start uses the initial conditions defined by the user in a previous model run. It will take the conditions from an existing results file.
4. Combined Auto Start and User-Specified Initial Conditions – In some cases, the auto start cannot be completed without some user defined initial conditions. The user must input some of the initial conditions and the auto-start will complete the rest.

4.1.11 Kinematic Routing Method

MIKE11 can use two different routing methods to determine the spatial variation of the discharge at kinematic routing branches. These are the Muskingum method and the Muskingum-Cunge method. This study will not go into detail on how the different methods work. Briefly, these methods handle a discharge-storage relationship to solve for discharges at later time steps.

4.1.12 Saint Venant Equations

MIKE11 applied with the dynamic wave description solves the equations of conservation and momentum; also known as the Saint Venant Equations. This is done by the model making these assumptions:

- The water is incompressible and homogeneous
- The slope of the bottom is small. Therefore, the cosine of the angle the slope makes with the horizontal can be taken as 1.
- The wave lengths are very large compared to the water depth. Therefore, vertical accelerations in waves can be neglected and hydrostatic pressure can be used.
- The flow is subcritical.

The resulting equations used by MIKE11 are:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \quad (4.18)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(\alpha \frac{Q^2}{A} \right)}{\partial x} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0 \quad (4.19)$$

where Q is the discharge, A is the flow area, q is the lateral inflow, h is the stage above datum, C is the Chezy resistance coefficient, R is the hydraulic or resistance radius, and α is the momentum distribution coefficient.

4.1.13 Steady State Energy Equation

The steady state energy equation can be used when the energy equation is selected in the “Quasi Steady” HD parameter editor. A set up of the energy equation can be seen in Figure 4.8.

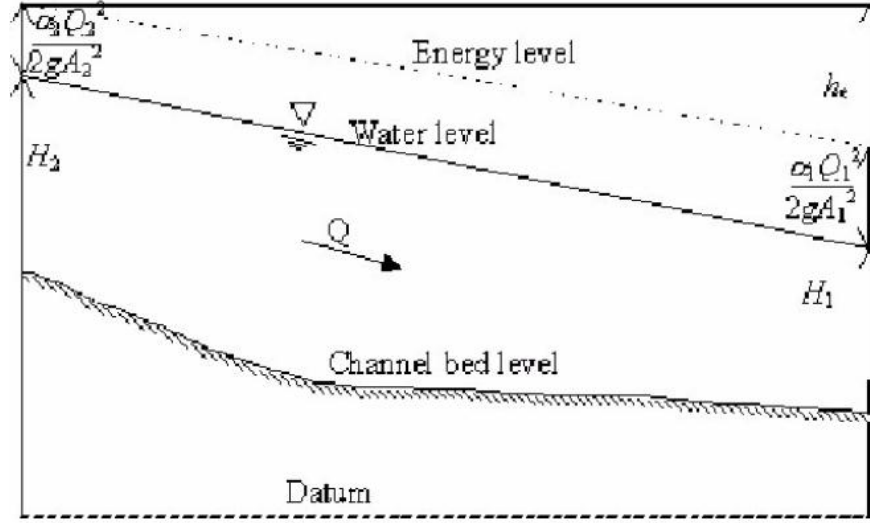


Figure 4.8: Energy equation set up (from DHI Water & Environment, 2009)

The steady state energy equation used in the model is given as:

$$H_1 + \alpha_{m11}\beta_{sup} \frac{\alpha_1 Q^2}{2gA_1^2} = H_2 + \alpha_{m11}\beta_{sup} \frac{\alpha_2 Q^2}{2gA_2^2} + h_e \quad (4.20)$$

where H_1 and H_2 are the water levels in the cross sections, Q is the discharge, α_1 and α_2 are the velocity distribution coefficients, g is the acceleration, A_1 and A_2 are the flow areas, h_e is the energy head loss, α_{m11} is the user defined velocity distribution coefficient and β_{sup} is the suppression factor.

The velocity distribution coefficients are calculated using:

$$\alpha = \frac{A_t^2 \left(\frac{K_{LF}^3}{A_{LF}^2} + \frac{K_{CH}^3}{A_{CH}^2} + \frac{K_{RF}^3}{A_{RF}^2} \right)}{K_{TOT}^3} \quad (4.21)$$

where K_{LF} , K_{CH} , and K_{RF} are the conveyance for the left floodplain, channel, and right flood plain, and A_{LF} , A_{CH} , A_{RF} are the flow area for the left floodplain, channel, and right flood plain. The conveyance for each section is found using:

$$K_i = \frac{A_i R_i^{2/3}}{n} \quad (4.22)$$

where n is the Manning's coefficient, A is the area, and R is the hydraulic radius. The suppression factor depends on the Froude number and is found by using:

$$\beta_{sup} = \begin{cases} 1 - Fr^2 & \text{for } Fr \leq 1 \\ 0 & \text{for } Fr > 1 \end{cases} \quad (4.23)$$

where the Froude number is found by using the equation:

$$Fr = \max\left(\frac{Q^2}{A_1^2 g R_1}, \frac{Q^2}{A_2^2 g R_2}\right) \quad (4.24)$$

4.2 BREACH

4.2.1 Introduction

BREACH was developed by D.L. Freud of the National Weather Service and has been widely used by the industry since its introduction in 1988. It is a physically-based mathematical model based on the principles of hydraulics, sediment transport, soil mechanics, the geometric and material properties of the dam, and the reservoir properties. The model predicts the outflow hydrograph through the dam breach and other breach characteristics such as time of formation and size of breach.

BREACH was developed by Freud after he released DAMBRK and BREACH is considered the upgraded model. DAMBRK is a parametric model that uses empirical observations of dam failures that have occurred in the past. These observations include the time of breach formation, width-depth relation, and the depth of breach. With these observations, the model can then develop the outflow hydrograph. The BREACH model uses critical properties that are measurable from the description of the dam materials and is therefore considered a more accurate model. It is important to note that the measured characteristics are usually within a range of values, and it is up to the engineer to determine the most critical combinations of values.

4.2.2 Using Two Types of Sediments

Within BREACH, the dam can consist of either one homogeneous material or two different materials. If the dam consists of two materials, then there must be an outer zone and an inner core. Each zone must have its own friction angle (Φ), cohesion (C), average grain size in mm (D_{50}), and unit weight (γ).

4.2.3 Additional Resistance due to Vegetation

The grass cover of the downstream face of the dam must also be specified. There are three options for grass cover within the model: 1) “a grass cover with specified length of either good or fair stand”, 2) “material identical to the outer portion of the dam”, or 3) “material of larger grain size than the outer portion”.

The downstream face geometry of the dam must also be provided within the model. This is done by including the elevation of the top of the dam (H_u), the elevation of the bottom of the dam (H_l), and its slope as a ratio of vertical to horizontal. Then, the only thing that is needed to describe the upstream slope geometry is the slope since the elevation of the top and bottom of the dam have already been specified. The crest width and the spillway flow vs. head table must also be provided if the dam is man-made. If the dam has naturally formed, it will obviously not have these features.

The reservoir characteristics are also needed in the model. First, the reservoir surface area (S_a) vs. water elevation must be specified. Then, the initial water level (H_i) and the inflow hydrograph specifying the inflow (Q_i) vs. the hour of its occurrence (T_i) must be provided.

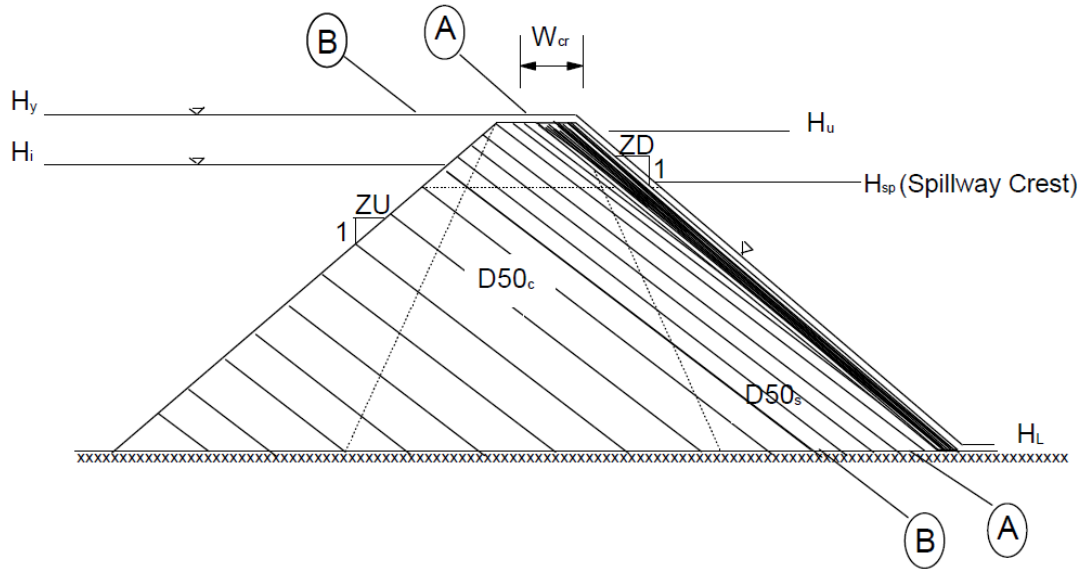


Figure 4.9: Dam failure simulation (from Fread, 1991)

4.2.4 Overtopping

If the reason for failure is overtopping, the water level must be higher than the top crest of the dam. It is obvious that the water must “overtop” the dam before a failure will occur. The erosion first starts on the downstream face of the dam. The first stages are shown as A-A in Figure 4.9 where a small rectangular stream is assumed to be present. When there is no grass present on the downstream this stream turns into an erosive channel that is cut into the dam. The flow into the channel is found using the broad-crested weir relationship:

$$Q_b = 3 * B_o (H - H_c)^{1.5} \quad (4.25)$$

where Q_b is the flow going over the dam into the breach channel, B_o is the initial width of the rectangular shape channel, and H_c is the breach bottom elevation. H_c remains at the elevation of the top of the dam (H_u) until the channel erodes all the way through the width of the crest. This point is signified by B-B in Figure 4.9. When this point is reached, H_c will start to erode vertically downward following the upstream face of the dam. H_c will continue to lower vertically until it reaches the bottom of the dam.

4.2.5 Grass Cover:

When there is grass cover on the downstream face of the dam, the velocity (v) is computed at each time step using Manning’s equation. This continues until the velocity passes the maximum permissible velocity (VMP) for grass (from Chow, 1959). When the velocity is greater than the VMP, failure of the downstream face will occur and a small channel (depth = 1ft, width 2ft) will form. The channel will then erode much like if grass cover didn’t exist. The steps to compute the velocity are:

$$q = 3 (H - H_c)^{1.5} \quad (4.26)$$

$$y = \left[\frac{qn'}{1.49 \left(\frac{1}{zD} \right)^{0.5}} \right]^{0.6} \quad (4.27)$$

$$n' = aq^b \quad (4.28)$$

$$v = q/y \quad (4.29)$$

where q is flow overtopping the dam in ft³/s per foot of crest length, $(H - H_c)$ is the head above the crest as described earlier, n' is the Manning Coefficient (from Chow, 1959), and a and b are fitting coefficients to form the graphical curves illustrated in Chow, 1959.

4.2.6 Piping

If the type of failure is caused by piping, the water level must be higher than the midpoint elevation (H_p). After the water level is higher than H_p , the size of the pipe starts to increase due to erosion. The bottom of the pipe will erode downward and the top of the pipe will erode upwards. Both the top and bottom will erode at the same rate. The flow into the pipe is found using the orifice flow equation:

$$Q_b = A \left[\frac{2g(H-H_p)}{1 + \frac{fL}{D}} \right]^{0.5} \quad (4.30)$$

where Q_b is the flow going through the pipe, g is gravitational acceleration, A is the cross-sectional area of the pipe, $(H - H_p)$ is the difference in head, L is the length of the pipe, D is the diameter of the pipe, and f is the Darcy friction factor. To find the Darcy Friction factor the Moody diagram must be used. A mathematical representation of the moody curves from Morris and Wiggert (1972) was used in the model and is presented below.

$$f = \frac{64}{N_R} \text{ when } N_R < 2000 \quad (4.31)$$

$$f = 0.105 (D_{50}/D)^{0.167} \text{ when } N_R \geq 2000 \quad (4.32)$$

$$N_R = 83333 Q_b D/A \quad (4.33)$$

Where f is the Darcy friction factor and NR is the Reynolds number.

When the head on the pipe is less than the pipe diameter, the flow changes from orifice controlled to weir controlled. The equation shown in the overtopping section for weir controlled should then be used. The transition is assumed to occur when the following equation is satisfied:

$$H < H_p + 2(H_{pu} - H_p) \quad (4.34)$$

When the flow changes from orifice to weir, it is assumed that the material that is above the pipe collapses and is transported down the river. The erosion then continues to cut a channel through the dam. This erosion continues much like the overtopping erosion that was discussed earlier.

The above process of dam breaching was for a man-made dam. The process is almost identical for a dam landslide except it is assumed there is no crest width (W_{cr}). Therefore, the erosion starts at B-B in Figure 4.9. Both overtopping and piping failure can still be modelled for a dam landslide.

4.2.7 Calculating the Breach Width

In BREACH, the width is controlled by two mechanisms. The first mechanism is that the breach has an initial rectangular shape. Please refer to Figure 4.10 to see the width progression of the breach. The width of the breach is found using the following equation:

$$B_o = B_r y \quad (4.35)$$

B_o is the width of the breach, B_r is a factor based on optimum channel hydraulic efficiency, and y is the depth of flow within the breach channel. B_r has a value of 2.0 for overtopping failures and 1.0 for piping failures. BREACH also assumes that depth of flow within the breach channel is the same as the critical depth at the entrance. This can be calculated using:

$$y = \frac{2}{3(H - H_c)} \quad (4.36)$$

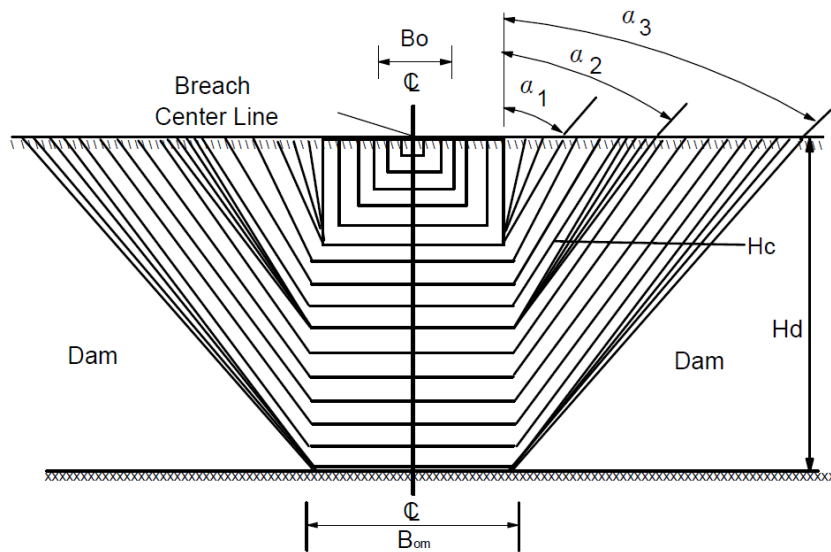


Figure 4.10: Breach formation (from Fread, 1991)

The second mechanism comes from the stability of the slopes. As seen in Figure 4.10, the initial rectangular channel turns into a trapezoidal channel. The channel turns into a trapezoid because the sides of the breach channel collapse. This forms a side slope with an angle (α) to the vertical. The collapse occurs when the depth of breach (H_c') reaches the critical depth (H') which can be found below:

$$H'_k = \frac{4 * C * \cos \Phi * \sin \theta'_{k-1}}{\gamma [1 - \cos(\theta'_{k-1} - \Phi)]} \quad (4.37)$$

where C is the cohesion, Φ is the angle of internal friction, θ is the angle the side slopes make with the vertical as seen in Figure 4.11, γ is the unit weight, and k represents the three different collapse conditions. The angle of the side slopes with either the vertical or horizontal can then be found using:

$$\theta = \theta'_{k-1} \text{ when } H_k \leq H'_k \quad (4.38)$$

$$\theta = \theta'_k \text{ when } H_k > H'_k \quad (4.39)$$

$$B_o = B_r \gamma \text{ when } k = 1 \quad (4.40)$$

$$B_o = B_{om} \text{ when } k > 1 \quad (4.41)$$

$$B_{om} = B_r \gamma \text{ when } H_l = H'_l \quad (4.42)$$

$$\alpha = 0.5\pi - \theta \quad (4.43)$$

where:

$$\theta'_o = 0.5\pi \quad (4.45)$$

$$\theta'_k = \frac{\theta'_{k-1} + \Phi}{2} \text{ for } k = 1, 2, 3 \dots \quad (4.46)$$

$$H_k = H'_c - y/3 \quad (4.47)$$

When $H_k > H'_k$ the subscript, k , is increased by 1. The term $y/3$ is subtracted from H'_c to take into account the stability of the breach cut after the influence of water is taken into account. The bottom and sides of the breach channel are assumed to erode equally except when there is slope instability and the side slope collapses. When the side slope collapses, the bottom erosion will stop until the collapse material is transported away using the sediment transport theorem presented.

The breach stops eroding in the downward direction when the breach bottom reaches the original valley floor. After this, the breach will continue to erode horizontally. The breach can erode to a maximum of the original valley width which is inputted by the user. The peak

discharge through the breach can occur when the erosion reaches the original floor or any time after when the breach continues to grow.

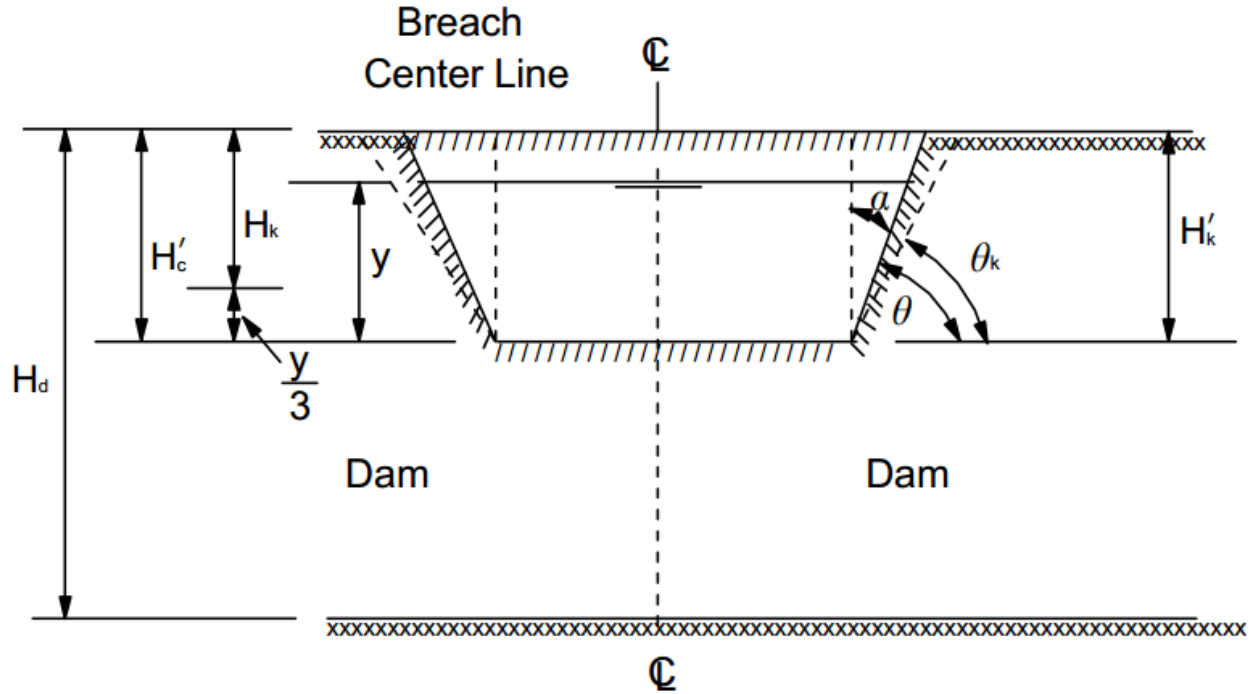


Figure 4.11: Breach summary (from Fread, 1991)

4.2.8 Determining the Reservoir Water Level

The water surface level (H) in the reservoir is computed by using the conservation of mass. The reservoir has an inflow (Q_i), spillway outflow (Q_{so}), crest overflow (Q_o), and breach outflow (Q_b). Using a time step of Δt (in hrs), the conservation of mass can be calculated using:

$$\overline{Q_i} - (\overline{Q_b} + \overline{Q_{sp}} + \overline{Q_o}) = S_a = \frac{\Delta H * 43560}{\Delta t * 3600} \quad (4.48)$$

where ΔH is the change in the water level during the time step and S_a is the surface area of the reservoir in acres at the initial elevation. The bar over the flows represent that they are an average flow over the time step. All flows are in cubic feet per second. The equation can then be re-arranged to solve for ΔH :

$$\Delta H = \frac{0.0826 * \Delta t}{S_a} (\overline{Q_i} - \overline{Q_b} - \overline{Q_{sp}} - \overline{Q_o}) \quad (4.49)$$

The reservoir water level after a time step can then easily be calculated by adding the difference in water elevation over a time step to the original water level.

The reservoir inflow is inputted by the user as a table vs. time. The flow over the spillway is also inputted by the user showing the relation of the discharge through the spillway

vs. the elevation in the reservoir. The flow through the breach was found using the relations previously shown for both piping and overtopping (weir) failures. However, if $H_c < H_u$ the following broad-crested weir equation must be used instead of the one presented earlier:

$$Q_b = 3 * B_o (H - H_c)^{1.5} + 2 \tan(\alpha)(H - H_c)^{2.5} \quad (4.50)$$

where B_o and α are found using the equations presented earlier. The crest overflow is found by computing the broad-crested weir equation and B_o is replaced by the crest length, and H_c is replaced by H_u .

4.2.9 Breach Channel Hydraulics

The model assumes that the flow in the breach can be adequately described using a quasi-steady flow. This is done by applying Manning's open channel flow equation at each time step. The equation is given below:

$$Q_b = \frac{1.49 S^{0.5} A^{1.67}}{n P^{0.67}} \quad (4.51)$$

where S is channel slope and is equal to $1/ZD$ (vertical to horizontal), A is the cross-sectional area, P is the wetted perimeter, and n is the Manning's number. The Strickler equation is used in BREACH to calculate n . Strickler's equation calculates n based on the average grain size of the material and can be seen below:

$$n = 0.013 D_{50}^{0.67} \quad (4.52)$$

The model assumes that a quasi-steady uniform flow is acceptable because of the short reach of the breach channel, very steep channel slopes, and small variations in flow along the breach channel. By using a quasi-steady uniform flow instead of an unsteady flow the computations are simplified and many numerical problems that might be encountered are eliminated.

When a rectangular breach channel is present, the depth of flow (y_n) can be related to the discharge through the breach using:

$$y_n = \left(\frac{Q_b n}{1.49 B_o S^{0.5}} \right)^{0.6} \quad (4.53)$$

When a trapezoidal breach is present, the depth of flow is found using an algorithm based on a Newton-Raphson iteration shown below:

$$y_n^{k+1} = y_n^k - \frac{f(y_n^k)}{f'(y_n^k)} \quad (4.54)$$

$$f(y_n^k) = Q_b P^{0.67} - 1.49 S^{0.5} A^{1.67} \quad (4.55)$$

in which

$$A = 0.5 (B_o + B) y_n^k \quad (4.56)$$

$$B = B_{om} + y_n \tan \alpha \quad (4.57)$$

$$P = B_{om} + y_n / \cos \alpha \quad (4.58)$$

$$f'(y_n^k) = 0.67 Q_b \frac{P'}{P^{1/3}} - 1.67 \frac{1.49}{n} S^{0.5} B A^{0.67} \quad (4.59)$$

where

$$P' = 1/\cos \alpha \quad (4.60)$$

The subscript, k , is an iteration counter and continues until:

$$|y_n^{k+1} - y_n^k| < \epsilon \quad \text{where } \epsilon \leq 0.01 \quad (4.61)$$

A first estimate of y_n must be computed using:

$$y_n^1 = \left(\frac{Q_b n}{1.49 B S^{0.5}} \right)^{0.6} \quad (4.62)$$

where

$$\bar{B} = 0.5 (B_{om} + B') \quad (4.63)$$

where B' is the top width of the channel breach at a water depth of (y_n) at time $(t - \Delta t)$.

4.2.10 Sediment Transport

The amount of sediment transport in BREACH is calculated using the Meyer-Peter and Muller sediment transport relation that was modified by Smart (1984) for use in steep channels. The relation is:

$$Q_s = 3.64 (D_{90}/D_{30})^{0.2} P \frac{D^{2/3}}{n} S^{1.1} (DS - \Omega) \quad (4.64)$$

where

$$\Omega = 0.0054 \tau_c D_{50} \text{ (noncohesive)} \quad (4.65)$$

$$\Omega = \frac{b'}{62.4} (PI)^{c'} \text{ (cohesive)} \quad (4.66)$$

$$\tau_c = a' t'_c \quad (4.67)$$

$$a' = \cos \theta (1 - 1.54 \tan \theta) \quad (4.68)$$

$$\theta = \tan^{-1} S \quad (4.69)$$

$$t'_c = \frac{0.122}{R^{0.970}} \text{ when } R < 3 \quad (4.70)$$

$$t'_c = \frac{0.056}{R^{0.266}} \text{ when } 3 \leq R \leq 10 \quad (4.71)$$

$$t'_c = \frac{0.0205}{R^{0.173}} \text{ when } R > 10 \quad (4.72)$$

$$S = 1/ZD \quad (4.73)$$

$$R = 1524 D_{50} (DS)^{0.5} \quad (4.74)$$

where Q_s is the sediment transport rate in cfs, D_{30} , D_{50} , and D_{90} are grain sizes representing the 30th, 50th, and 60th percentile of finer material, D is the hydraulic depth of flow, S is the downstream slope of the dam face, τ'_c is the shields dimensionless critical shear stress, PI is the plasticity index for clay or silty soils, and b' and c' are empirical coefficients with the ranges $0.003 \leq b' \leq 0.019$ and $0.58 \leq c' \leq 0.84$ from Clapper and Chen (1987).

4.2.11 Sudden Collapse Breach Enlargement

Another important process that the model looks at is a sudden collapse of the upper portions of the dam that will enlarge the breach channel. This collapse usually consists of a wedge portion and is caused by the pressure of the water on the upstream face which exceeds the resistive forces of shear and cohesion. This wedge has a vertical component, Y_n , and is shown in Figure 4.12. The wedge is then pushed towards the right. Any further erosion of the breach stops until all of the collapsed materials is transported away. A collapse check happens at every time step. This check consists of setting Y_c to 10 and then summing all the forces acting on the wedge. A collapse will occur if:

$$F_w > F_{sb} + F_{ss} + F_{cb} + F_{cs} \quad (4.75)$$

where F_w is the force created by the water pressure, F_{sb} is the resisting shear force on the bottom of the wedge, F_{ss} is the resisting shear force on the sides of the wedge, F_{cb} is the resisting

cohesion for on the bottom, and F_{cs} is the resisting cohesion for on the sides of the wedge. All of these forces are calculated using the equations presented below:

$$F_w = 0.5 * 62.4 * \bar{B} (Y_c + 2 h_d) \quad (4.76)$$

$$F_{sb} = \tan\Phi [(\gamma - 62.4)0.5 * ZU * \bar{B} * Y_c^2 + \gamma * B * W_{cc} * Y_c + \gamma * 0.5 * ZD * \bar{B} * Y_c^2 + 0.67 * 62.4 * h_d * W_{cc} + 62.4 * ZD' * B * y_n * Y_c] \quad (4.77)$$

$$F_{ss} = \gamma * K * \tan\Phi Y_c^2 [W_{cc} + (ZU + ZD)Y_c] \quad (4.78)$$

$$F_{cb} = CB_o [W_{cc} + (ZU + ZD)Y_c] \quad (4.79)$$

$$F_{cs} = 2C \left[W_{cc} + (ZU + ZD)Y_c \left(B_o + \frac{2Y_c}{\cos\alpha} \right) \right] \quad (4.80)$$

where

$$K = (1 - \sin\Phi)/(1 + \sin\Phi) \quad (4.81)$$

$$B = B_o + H_c \sin\alpha \quad (4.82)$$

$$ZD' = (1 + ZD^2)^{0.5} \quad (4.83)$$

where the parameters used in these equations are defined in Figure 28. If a collapse does not occur, y_c is increased by 2ft and the process is evaluated until there is a collapse.

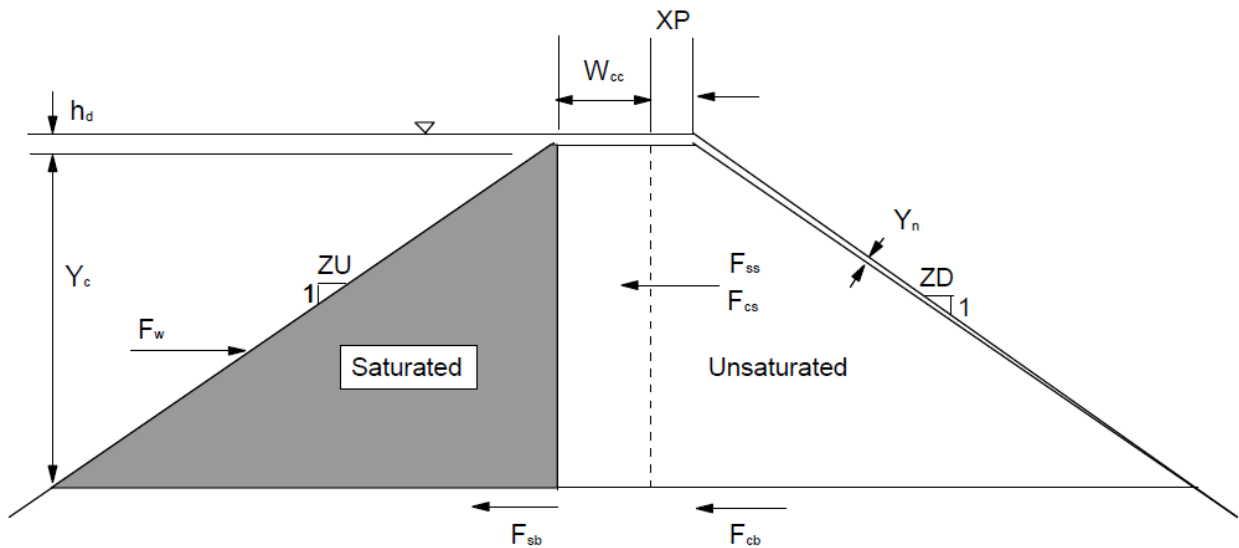


Figure 4.12: Forces that may cause a sudden dam collapse (from Fread, 1991)

4.2.12 BREACH Iteration

Iterations must be completed for the full simulation of BREACH to occur. This is because the flow into the breach depends on the size of the breach and the size of the breach depends on the sediment transport capacity. The sediment transport capacity subsequently depends on the flow into the breach. Therefore, an estimated incremental erosion depth ($\Delta H_c'$) is used at each time step to start the iteration. This can be extrapolated from the first two time steps. The steps to run through the model algorithm are shown below.

1. Compute H_c using $\Delta H_c'$
2. Compute the reservoir elevation
3. Compute Q_{sp} , Q_i , and Q_o associated with H
4. Compute ΔH
5. Compute the reservoir elevation
6. Compute the breach flow (Q_b)
7. Compute B_o , α , B , P , and R
8. Compute Q_s
9. Compute ΔH_c
10. Compare ΔH_c and $\Delta H_c'$ to find error. If error tolerance is unacceptable (inputted by user), a new estimation of $\Delta H_c'$ is made
11. Check for collapse
12. Extrapolate estimates for $\Delta H_c'$ and $\Delta H'$
13. Plot the outflow hydrograph consisting of the total flow at each time step

4.2.13 Model Testing

This model has been tested on four different dams including: Teton Dam, Lawn Lake Dam, Mantaro Dam landslide, and the Spirit Lake Blockage. Overall, these dam breaches were accurately modelled. However, further testing of this model is necessary to assess whether or not it can accurately predict man-made dam failures due to overtopping.

5.0 Physical Model Experiments

The physical modelling data that is being compared to the numerical models was collected from three different projects. All three of these projects have been completed in the last 10 years and add a significant amount of embankment breaches to the data base.

5.1 University of Ottawa

The first project was completed at the University of Ottawa and includes two different series of earthen embankment breaching tests (Orendorff, B., 2009 and Al-Riffai, M. and Nistor, I., 2011). Within these series, the tests varied compaction, soil saturation and breach geometry. Even though both series were available for use, only the second series was used in this study due to the increased success of its tests.

The first series (Orendorff, B., 2009) was completed in both the large flume (1.5m wide, 29.2m long, and 0.77m high) and tilting flume (0.38m wide, 12.2m long, and 0.61m high) in University of Ottawa's Hydraulics Laboratory. Please refer to Figure 5.1 for photos of the two flumes. There were a total of eight tests completed. Test A was completed on a 1:120 scale based on a 30m high embankment prototype with a D_{50} of approximately 0.5mm. Tests B and C were also based on a 30m high embankment prototype with a D_{50} of approximately 0.5mm but the scale was set at 1:100. The tilting flume was used to carry out breaching runs that varied the dry density. The large flume was used to carry out breaching runs to measure the effect of a toe drainage system that was installed downstream. Construction of the embankments was completed much like they were in the field. They were constructed in successive layers of 5cm using an 8.7kg tamper dropped from a height of 10cm for compaction. There was also form work on the sides of each lift. Controlled moisture content throughout construction was used to ensure that optimum moisture content was used and this was obtained from Standard Proctor tests. Like many experiments in hydraulics, exact scaling of the D_{50} was not possible due to the effects of cohesion. The actual D_{50} used should have been much smaller than 5 μ m. A median grain size of 220 μ m was used and therefore the number of blows applied to each layer was scaled down to achieve the desired dry density. The inverted horizontal toe drain had a 9mm thickness and the filter had a 6mm thickness and fits the gradation design for horizontal blankets set out by the United States Bureau of Reclamation (1987).

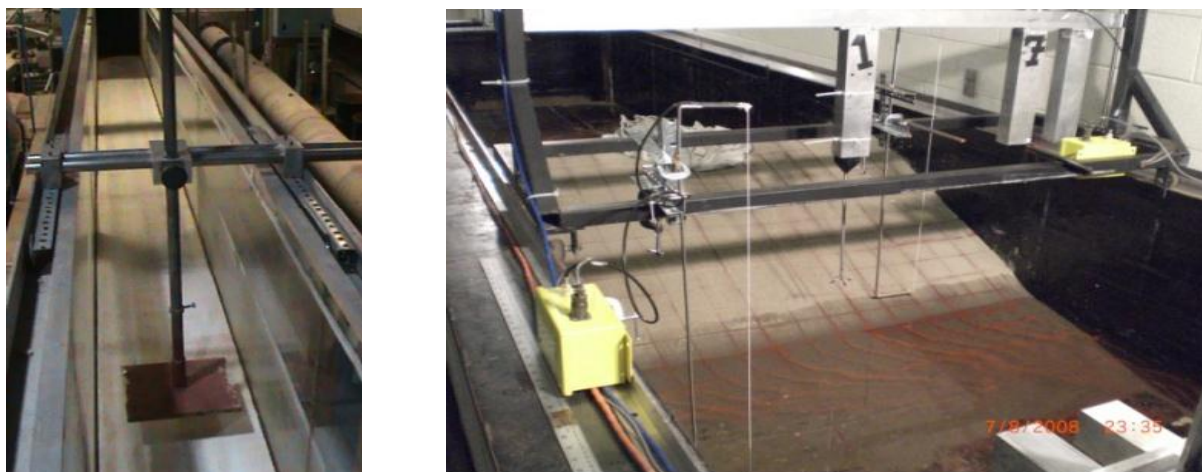


Figure 5.1: Tilting flume and large flume in the hydraulics lab at the University of Ottawa (from Al-Riffai and Nistor, 2010)

Table 5.1: Grain size distributions and dry density for embankment materials for Al-Riffai, M., 2009

Material	D ₁₀ (μm)	D ₃₀ (μm)	D ₅₀ (μm)	D ₆₀ (μm)	D ₉₀ (μm)	#Blows [Dry density (kg/m ³)]
Sand	110	160	220	260	410	2 [1496], 5 [1531], 10 [1562], 20 [1598]
Filter Medium	840	1000	1200	1300	1500	5 & 20 [1299]
Drain Medium	3000	3000	4400	4800	6300	5 & 20 [1299]

The reservoir was filled at a rate of approximately 3-4mm/min and stopped when the water surface elevation reached a point 2cm below the embankment crest. The reservoir volumes were measured at 1.4m³ for test A, 1.6m³ for test B, and 11.5m³ for test C. These volumes were calculated using the surface elevations measured by four WG-50 wave gauges. Test B also measured the piezometric pressure during filling by using existing tapping points located under the flume. The breaches were also recorded by 3 different HD video cameras. They recorded the breach evolution of the upstream, downstream, and top view for each breach. A V-notch (2cm high with a top width of 4cm) was cut into the centre of the dam crest to initiate breaching. The reservoir was then filled at a rate of 0.485L/s.

Outflow hydrographs were recorded for each test using the described wave gauges. Froude scaling was then used to scale up to the model to prototype values. However, in tests A and B the breach channel reached the side walls of the flume too quickly. This made much of the outflow hydrograph unusable since it would not give an accurate description of what would happen in the field. Therefore, these tests were not used in this thesis.

The second series of tests performed at the University of Ottawa were much more valuable to this project. The series was performed in the large flume with a 1.5m width and the dam was constructed near the flume outlet to minimize the effects the tailwater would have on breach characteristics. Please refer to Figure 5.2 to see the experimental setup. For this series,

the authors used a different construction method since they believed a vibratory load would better compact the non-cohesive material.

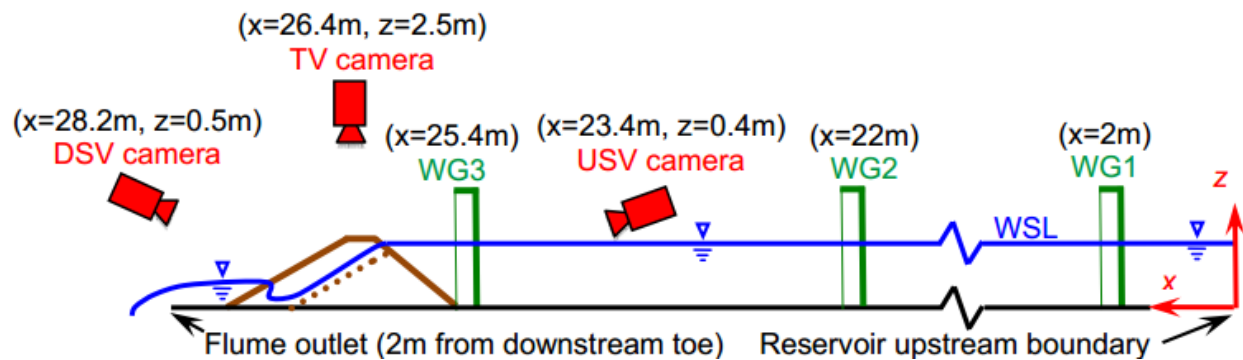


Figure 5.2: Experimental setup for series 2 (from Al-Riffai, M., and Nistor, I., 2011)

All models in series 2 were constructed to be 30cm in height. These models were made out of 6 layers, each having a height of 5cm. The authors stated that the lift height before compaction was determined based on trial and error to make sure the layer height was always consistent. The compactor that was used was also designed and constructed by Al-Riffai and Nistor (2011). It consisted of a rotary motor that was mounted on a metallic frame that was then welded on a steel plate (0.7m in length x 1.5m wide). Three different compaction efforts were used to create a high compaction, low compaction and very low compaction test. The high compaction effort was achieved by using the vibratory plate and applying a static load using a hydraulic jack until no more compaction was occurring. This was verified by a pressure gauge located on the hydraulic jack. The low compaction test was achieved by using the vibratory plate without applying a static load. Finally, the very low compaction test was achieved by using loose dry sand. Please refer to Table 5.2 and Table 5.3 for the soil characteristics.

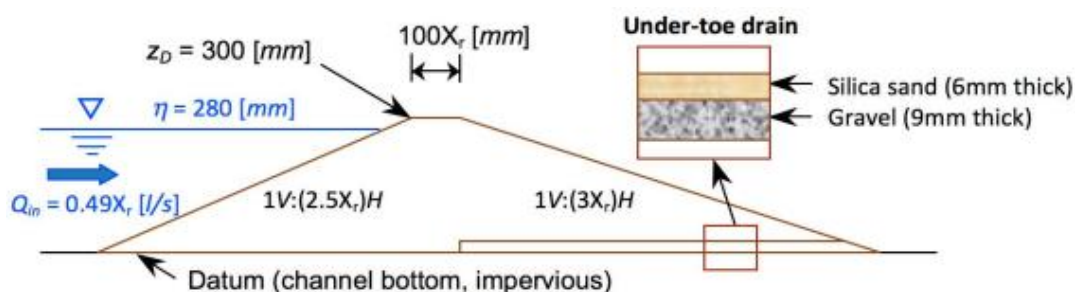


Figure 5.3: Geometry of embankment (from Al-Riffai, M., and Nistor, I., 2011)

Table 5.2: Void ratio for different soil compactions

Soil Compaction	Dry Unit Weight (kN/m ³)	Void Ratio, e
High Compaction	17.8	0.52
Low Compaction	16.8	0.61
Very Low Compaction	15.4	0.75

Table 5.3: Soil characteristics summary

Characteristic	Value
Coefficient of Curvature, C_c	1.14
Coefficient of Uniformity, C_u	2.88
D_{10} (μm)	95
D_{30} (μm)	171
D_{50} (μm)	232
D_{60} (μm)	273
D_{90} (μm)	556
% fines	3
Specific Gravity, SG	2.75
Angle of Internal Friction, Φ'	36.2

Please refer to Figure 5.4 and Figure 5.5 to see the observed discharges and water levels in the reservoir. The water levels came from WG1 and its location can be seen on Figure 5.2. The first thing to note is that the lag time until the breaching starts was not recorded during the experiments. Therefore, the authors have started the outflow hydrograph from when the breaching starts for each experiment. This makes it difficult to show a true comparison. However, if you refer to Figure 5.4 and Figure 5.5 you can see that the lower compaction test breaches quicker and therefore has a higher peak discharge. As expected, the highest compaction test takes longer to breach, has a longer breach time, and therefore a lower peak discharge. The peak discharge of the very low compaction test was recorded as $0.076 \text{ m}^3/\text{s}$. The peak discharge then drops to $0.067 \text{ m}^3/\text{s}$ for the low compaction test and to $0.065 \text{ m}^3/\text{s}$ for the high compaction test. The very low compaction test takes the shortest time to peak taking 94s as well had the shortest breaching time taking only 654s. The high compaction test takes the longest time to breach and has the longest overall breach time, taking 125s and 1007s, respectively. As expected, the low compaction test's time to breach and total breach time were in between the high and very low compaction test's breaching characteristics, taking 108s and 801s, respectively. To see a summary of the dam breaching characteristics please refer to Table 5.4.

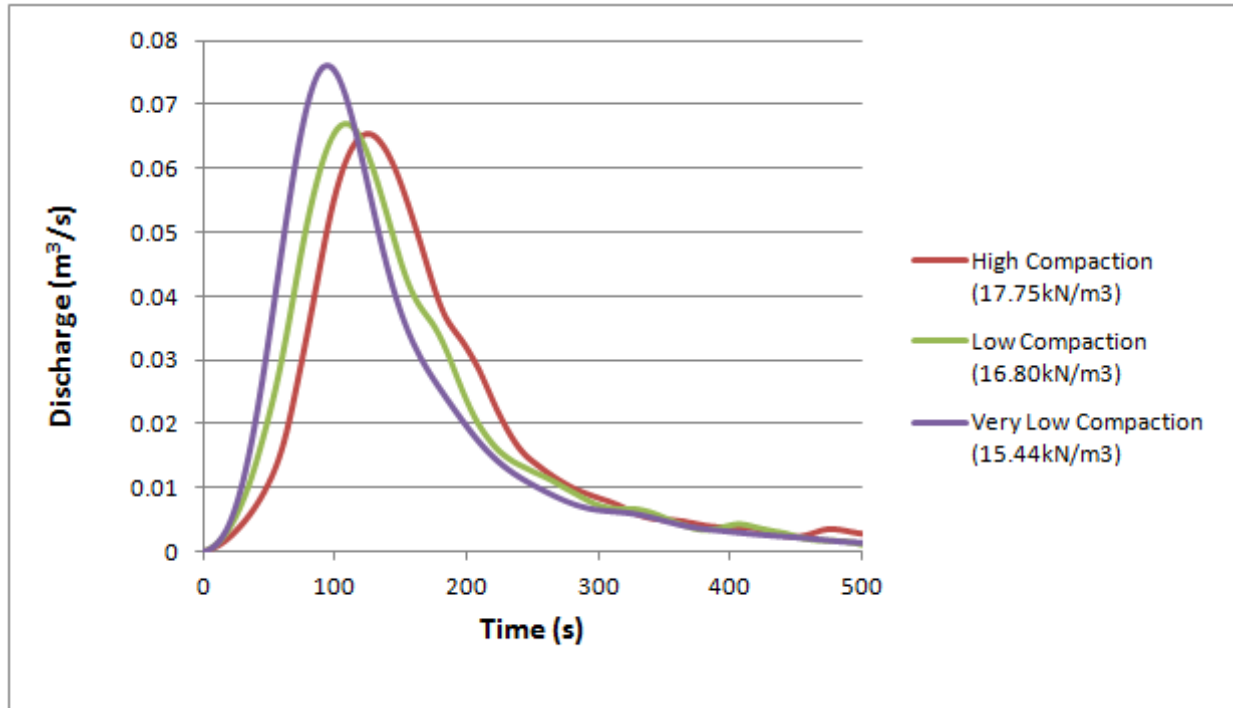


Figure 5.4: Outflow hydrograph from University of Ottawa tests

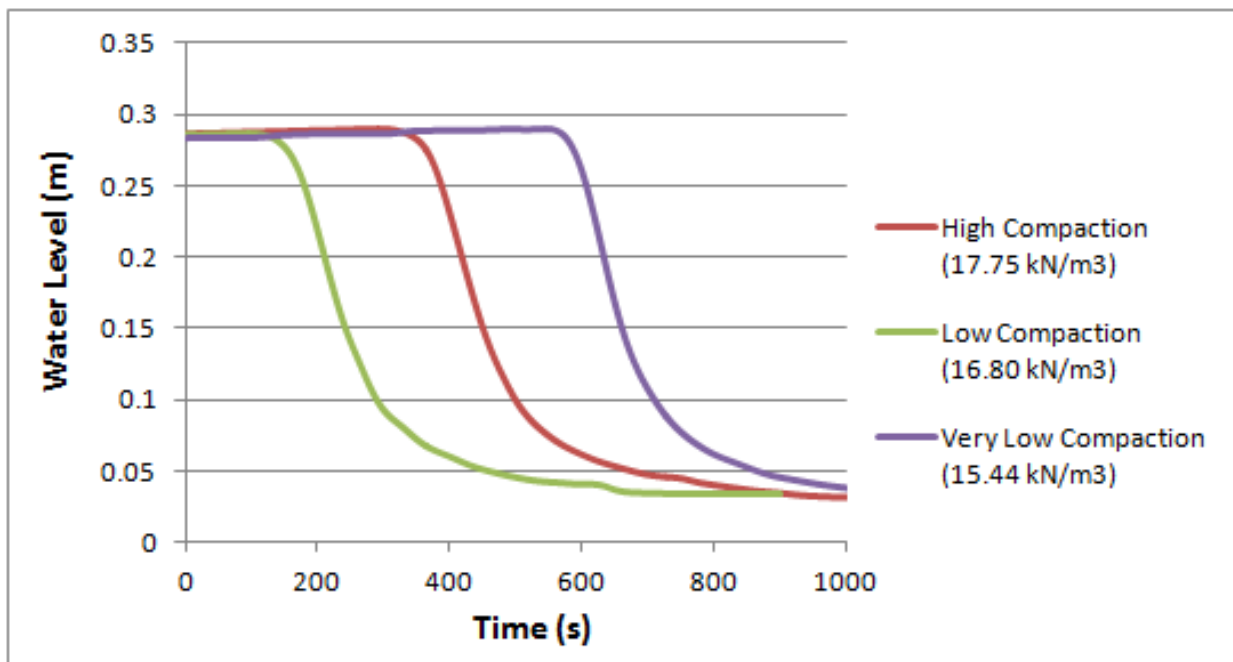


Figure 5.5: Water levels from University of Ottawa tests

Table 5.4: Breaching characteristics for University of Ottawa tests

	Peak Flow (m³/s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Surface Elevation of Water at Peak (m)	Final Surface Elevation (m)
High Compaction	0.06546	125	0	1007	28.97	0.0763
Low Compaction	0.06694	108	0	801	28.66	0.0908
Very Low Compaction	0.07622	94	0	654	28.89	0.108

5.2 Delft University of Technology Experiments

Zhu (2006) completed his doctoral thesis on breach growth in clay-dikes. In his thesis he developed a mathematical model to represent the breaching process that is present in cohesive dikes. This is a follow up to his supervisors work, Visser (1998), on breach growth in sand-dikes. The author has chosen to only look at homogeneous clay dikes at this point due to the complexity of the study. The model is based on the breaching mechanisms that have been observed in the laboratory and field. Zhu (2006) has decided not to look at the effect of protection layers or waves on the cohesive dike.

The author used the 5-stage breaching process that was described by Visser (1998). In the first stage, erosion starts along the inner slope and possibly along the dike crest if the flow velocity is high enough. This decreases the width and height of the dike in the breach. The second and third stages continue with erosion through a combination of flow shear erosion, fluidization of the surface of the slope, impinging jet scour of the dike foundation, and discrete headcut slope mass failure. At the end of stage 3 all of the dike body in the breach should have washed away. The fourth and fifth stages have the breach growing laterally due to flow shear along the side slopes and their resulting instability. How quickly the breach develops vertically depends on the erodibility of the foundation and if any toe protection is evident. For more information on the process please refer to the Dam Breach Theoretical Background section.

The author conducted laboratory experiments in the flume at the Delft University of Technology for model calibration and validation and to further understand the breaching process. These experiments will be used to help validate the numerical models used in this study. A total of 5 tests were performed. The first test was a sand-dike and the next four were constructed out of cohesive materials with different mixtures of sand, silt, and clay. Flow velocities upstream and downstream of the dam were measured and the breaching process was recorded by digital video cameras.

The author chose to breach the dikes due to overtopping. When they were overtopped, the erosion generally occurred near the toe of the dam first. This caused a steepening of the downstream slope of the dam and this eventually turned into a headcut. The headcut then played a large role in how the breach continued to erode. The author stated that the cohesiveness played a very large role in how quickly the dam breached. The sand-dike eroded much quicker and the higher clay proportions led to slower erosion rates.

The experiments were completed in two straight flumes in the fluid mechanics laboratory from June to September of 2005. Please refer to Figure 5.6 to see a photo of the two flumes at Delft University of Technology, Netherlands. The flumes were used jointly when the experiments were run. Flume 1 was the testing flume where the dikes were built and tested. Flume 2 was the “storage basin” flume and was used to help in water recirculation and as a sediment trap. Please refer to Table 5.5 for the dimensions of the two flumes and the usable water storage capacity in the second flume. Flume 1 was built with glass side walls and this allowed for visual observation of the breaching process.



Figure 5.6: Flumes used in TU Delft tests (from Zhu, 2006)

Table 5.5: TU Delft flume dimensions

	Length (m)	Width (m)	Depth (m)
Flume 1	35.5	0.8	0.85
Flume 2	27.0	2.0	0.40
Usable Water Storage Capacity	27.0	2.0	0.35

Please refer to Figure 5.7 to Figure 5.9 for the layout of the two flumes during the tests. Due to the flume only being 1m in width, the author has decided to only focus on the first 3 stages of the 5 stage breaching process. These steps include the breach until it reaches the bottom limit or foundation and does not include the horizontal widening. To decrease the width of the flume to 0.40m, a vertical wooden wall was placed in the middle of the flume. Another vertical wooden wall was then installed perpendicular to the middle wall to separate the upstream from the downstream sections of the flume.

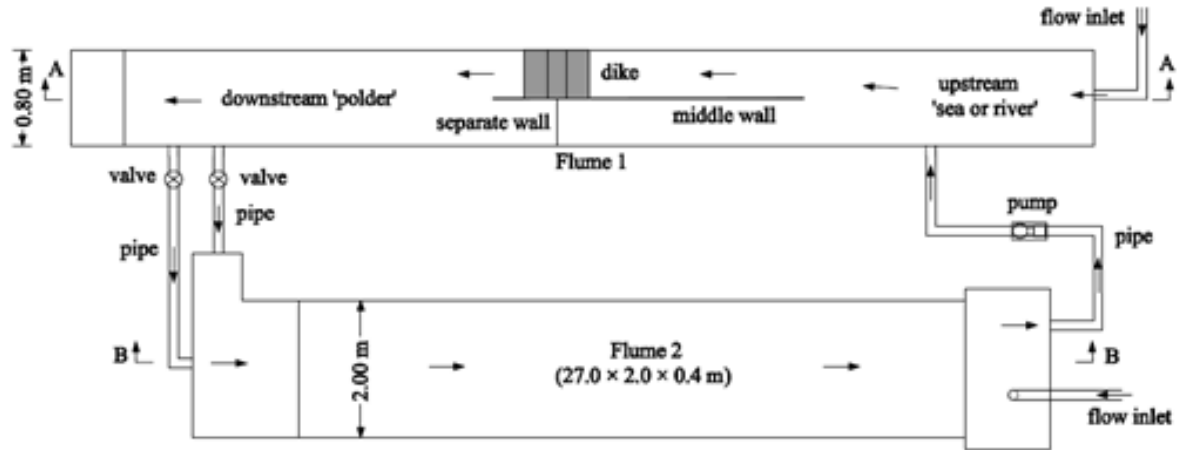


Figure 5.7: Top view of flume 1 and 2 used in TU Delft tests (from Zhu, 2006)

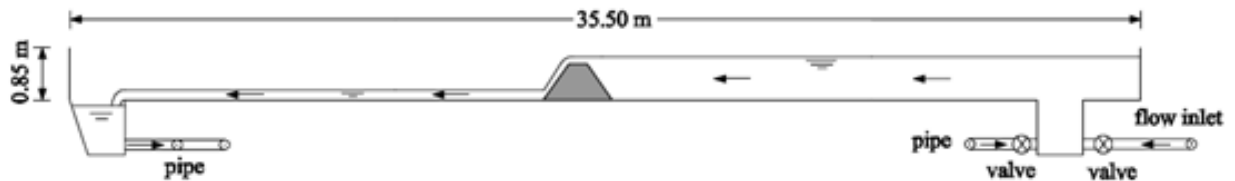


Figure 5.8: A-A longitudinal section of flume 1 used in TU Delft tests (from Zhu, 2006)

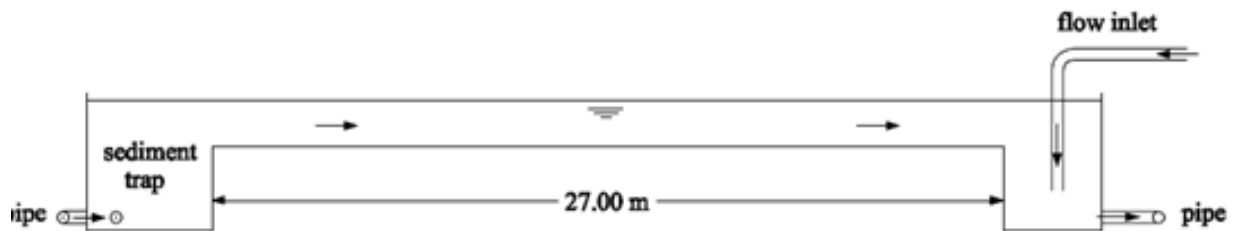


Figure 5.9: B-B longitudinal section of flume 2 used in TU Delft tests (from Zhu, 2006)

As mentioned earlier, the author constructed a total of 5 tests. The first test was constructed out of sand (T_s) and was used as a sample test and to serve as a starting point for the four clay-dike tests. The four formal tests (T_1 , T_2 , T_3 , T_4) were built out of cohesive material and the sand, silt, and clay content varied between each test. All of the tests have the same dam geometry and the dimensions can be viewed in and on Figure 5.10.

Table 5.6: TU Delft test dimensions

Parameter	Value
Dike Height	75.0 cm
Dike Crest Length	40.0 cm
Dike Crest Width	60.0 cm
Dike Inner Slope	1:2.0
Dike Outer Slope	1:2.0
Thickness of Soil Foundation	0.0 cm

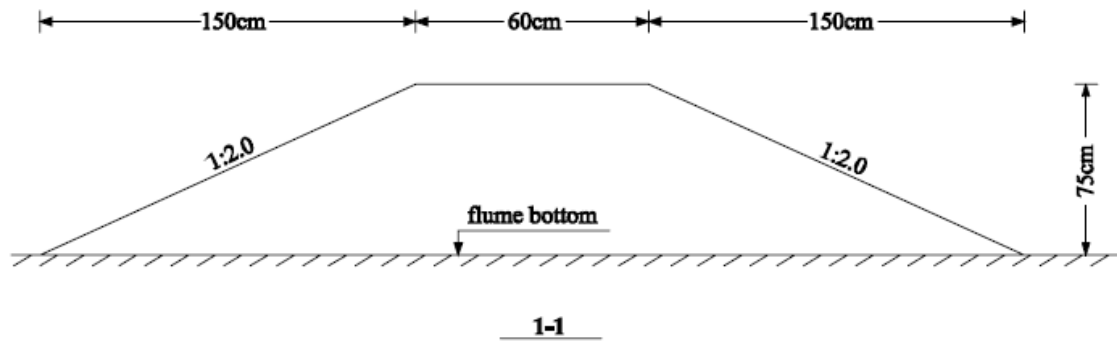


Figure 5.10: TU Delft dike dimensions (from Zhu, 2006)

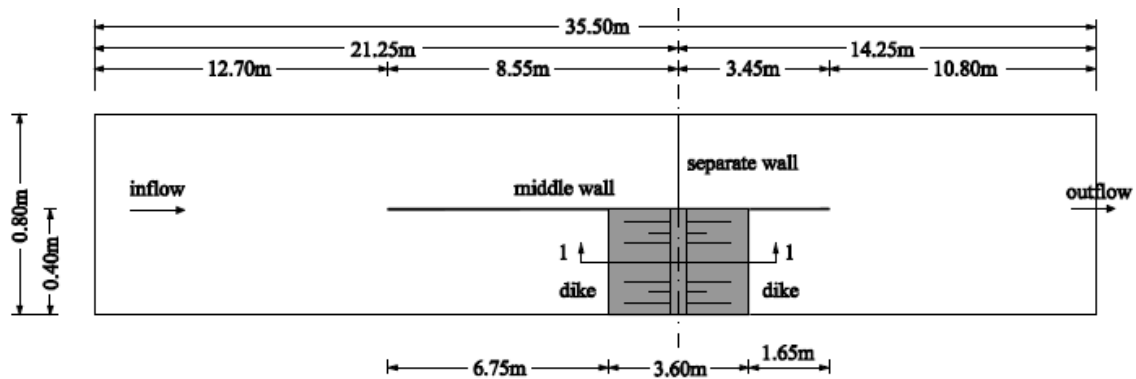


Figure 5.11: TU Delft flume set-up (from Zhu, 2006)

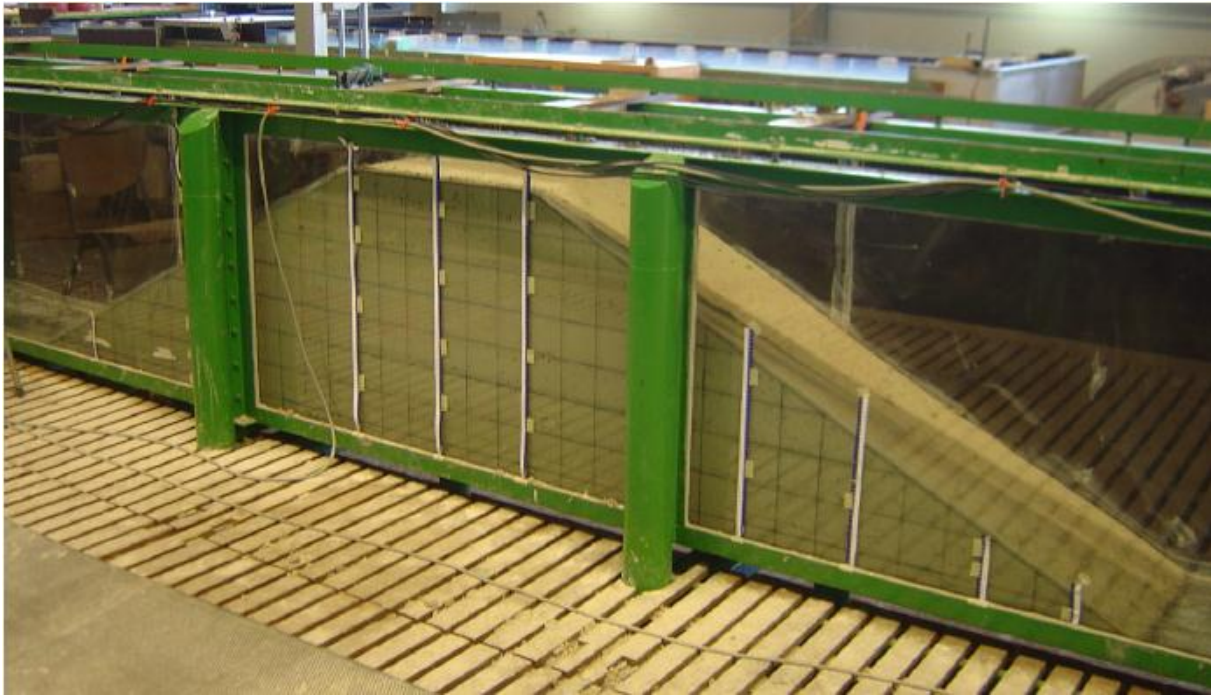


Figure 5.12: Photo of TU Delft dike (from Zhu, 2006)

A summary of the soil properties used in each of the five tests are listed in Table 5.7. A series of tests were completed to measure the soil properties of each test. The soil wet density (ρ_s) and water content (ω) were found for each layer of the dike at the time of construction. A sieve analysis was completed to determine the particle size distribution. A pycnometer was applied to find the specific gravity of the soil (G_s). The maximum dry density (ρ_{dm}) and optimum water content (ω_{opt}) were determined by Proctor compaction tests for the first formal test (T_1). Unconfined compression tests were completed by using the triaxial test apparatus and measured the undrained shear strength (c_u). Please refer to Figure 5.13 for the particle size distribution of each of the tests.

Table 5.7: Summary of TU Delft soil properties

Item		Soil Material Used for Tests				
		T_s	T_1	T_2	T_3	T_4
G_s		2.66	2.67	2.67	2.67	2.67
ρ (kg/m ³)		1650	1938	1936	1951	1948
ω (%)		20.1	17.6	16.7	16.7	15.4
C_u (kPa)			22.10	24.63	26.42	26.75
ω_{opt} (%)			11.5			
ρ_{dm} (kg/m ³)			1940			
Void Ratio		.094	0.62	0.61	0.60	0.58
Degree of Saturation (%)		57.1	75.8	73.2	74.7	70.7
Composition 1 (%)	Trip Popken Sand	100	45	50	50	50
	Millisil M10	0	35	35	35	35
	Polwhite E Kaolin	0	15	10	10	10
	Illite	0	5	5	5	5
Composition 2 (%)	Sand (>50 μ m)	99.0	50.5	54.6	54.6	54.6
	Silt (>2 μ m)	1.0	37.5	35.1	35.1	35.1
	Clay (<50 μ m)	0.0	12.0	10.3	10.3	10.3
D_{50} (μ m)		91	53	63	63	63

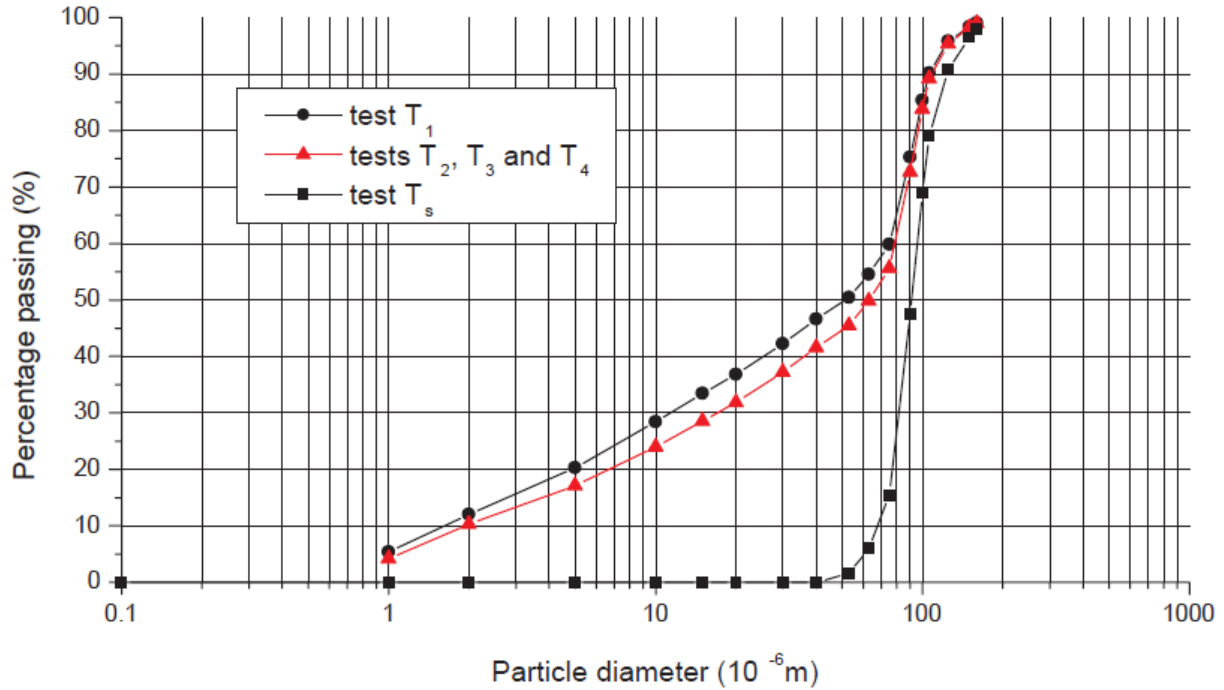


Figure 5.13: TU Delft particle size distribution (from Zhu, 2006)

The formal dikes were constructed in horizontal layers that were 0.10m in thickness and the test dike was constructed in layers of 0.20m thickness. The dikes were compacted using a hand-operated compaction roller that was 22.0cm wide and had a mass of 32kg. Before the dike construction, tests were done to see how many passes of the roller was needed to achieve the desired density. It was determined that 2 passes were needed to achieve this density. At least one sample from each layer was taken to determine the density. Another sample was taken to determine the water content.

Water levels upstream and downstream of the dike were recorded using four wave gages (G14, G15, G16, and G17). Flow velocities were measured using three electromagnetic velocity sensors (E7, E10, and E11). Video and photos of the dam breach were taken with two digital video cameras (VC1 and VC2) and photographed using two digital cameras (DC1 and DC2). The locations of these wave gages, velocity sensors, and cameras can be seen in Figure 5.14 and Figure 5.15. VC1, VC2, and DC1 were all installed on one side of the flume. Horizontal and vertical lines were then drawn on the flume's glass side wall so the breaching process could be recorded.

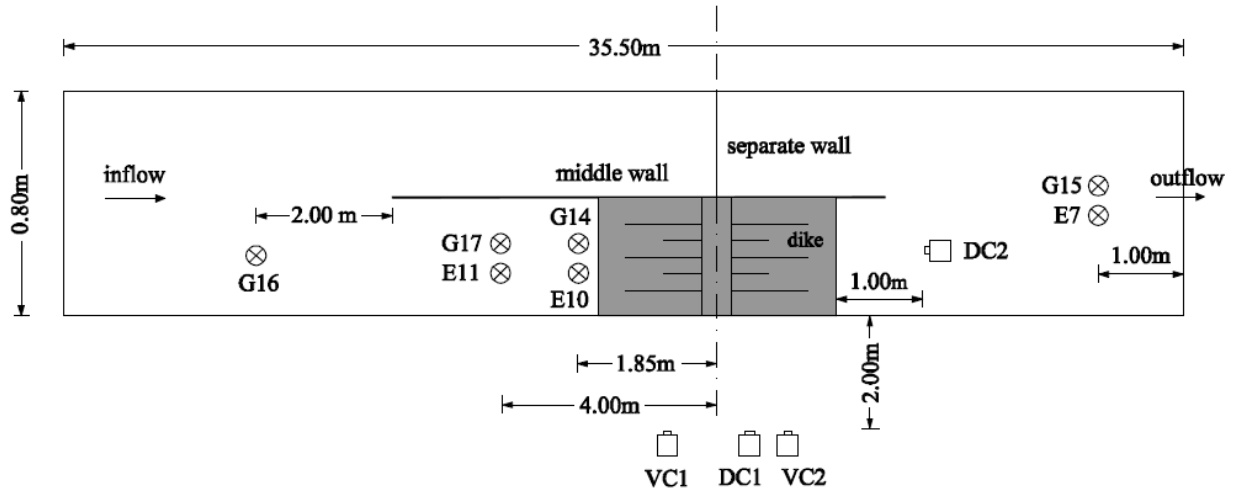


Figure 5.14: Plan view of TU Delft wave gage set up (from Zhu, 2006)

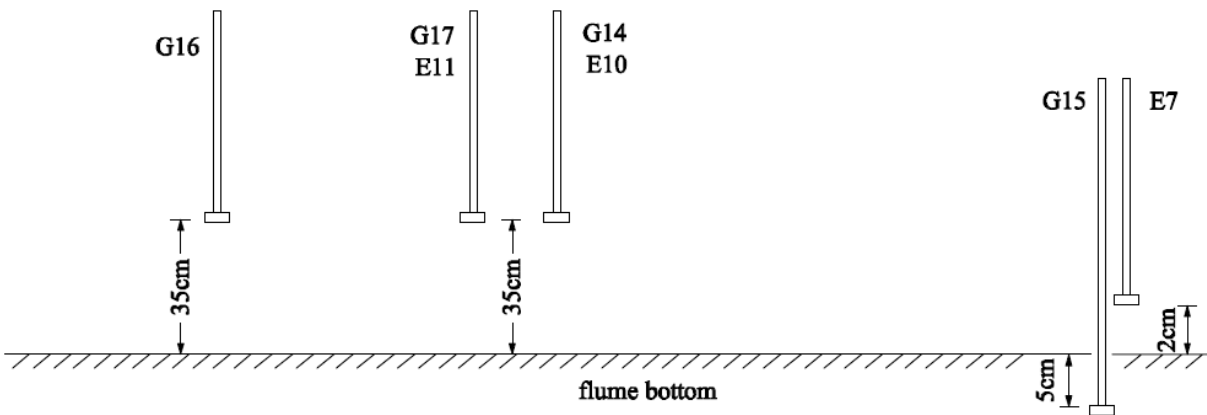


Figure 5.15: Profile view of TU Delft wave gage set up (from Zhu, 2006)

The first step to these tests was to fill the storage basin in flume 2 and the upstream section in flume 1. A wood board and sandbag were placed on the dike crest so that the water would not overtop the dike when the upstream water level was filled to 5cm above the dike crest. The wood board and sandbag were then removed and the breaching process started. Water was then re-circulated by the pipes and pump. To see the discharge, water level, and dike height for each of the first three formal tests please refer to Figure 5.16 to Figure 5.24.

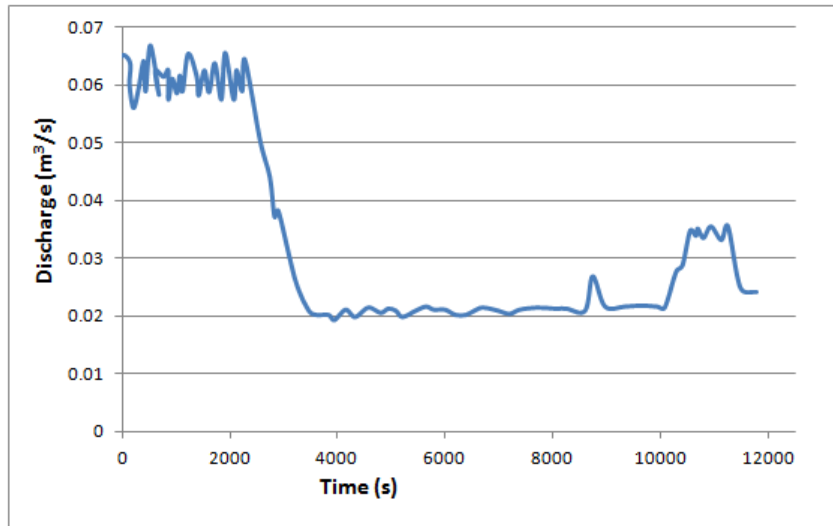


Figure 5.16: TU Delft formal test 1- discharge

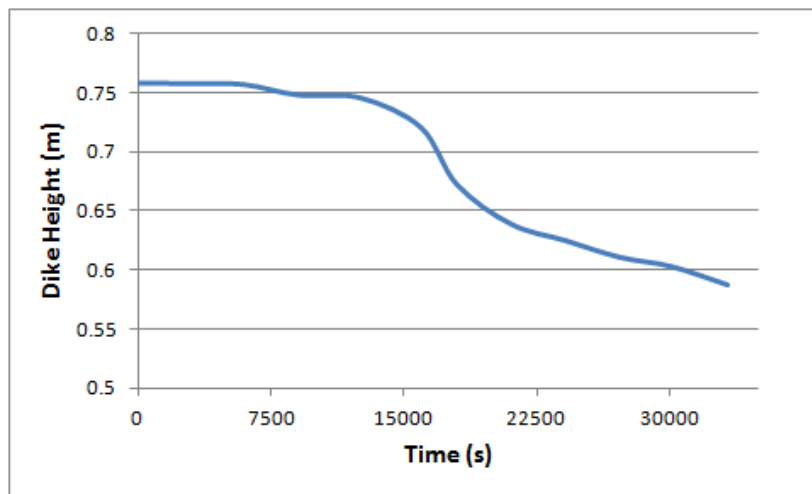


Figure 5.17: TU Delft formal test 1- dike Height

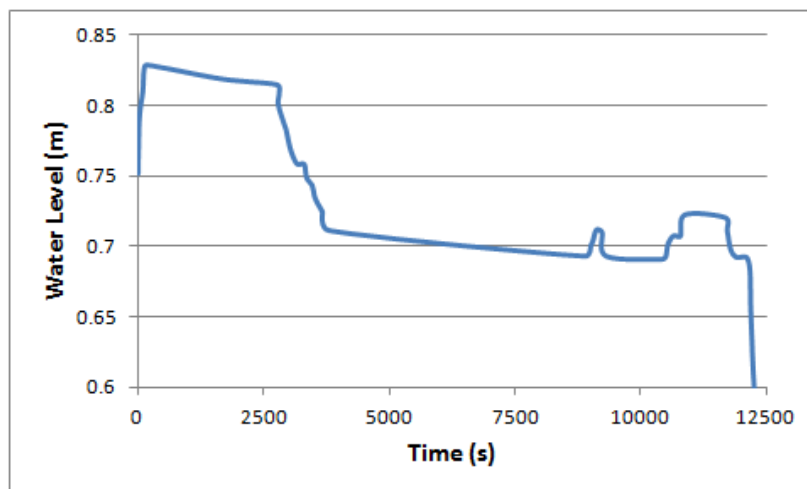


Figure 5.18: TU Delft formal test 1- water level

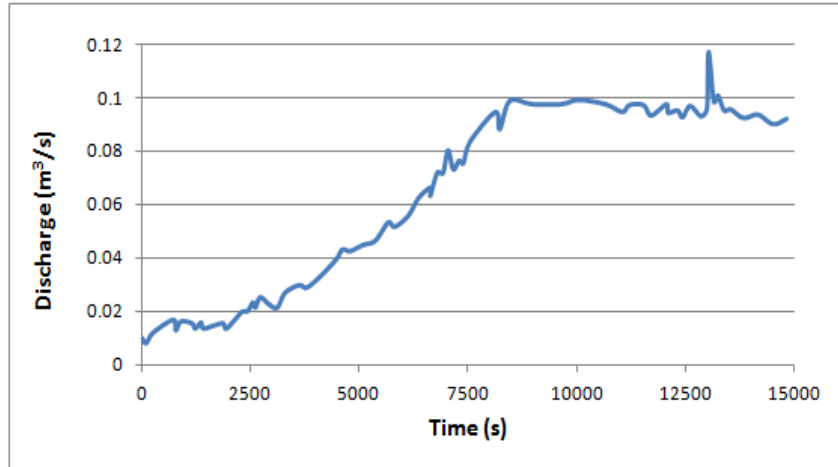


Figure 5.19: TU Delft formal test 2 – discharge

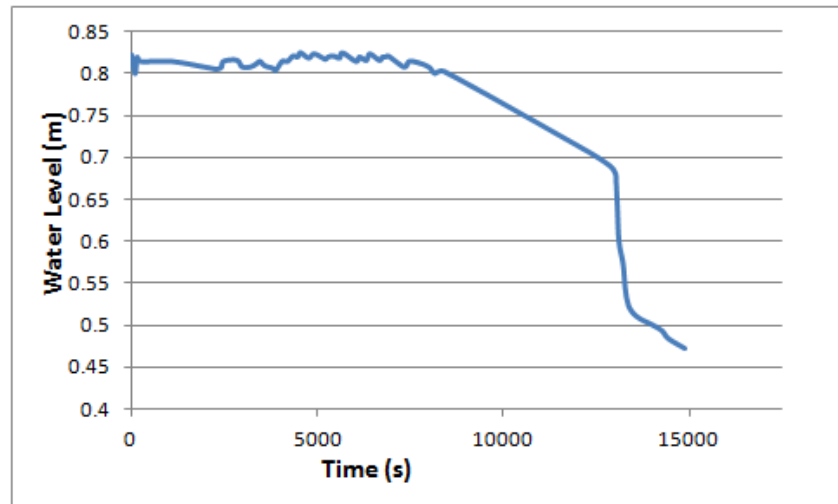


Figure 5.20: TU Delft formal test 2 - water level

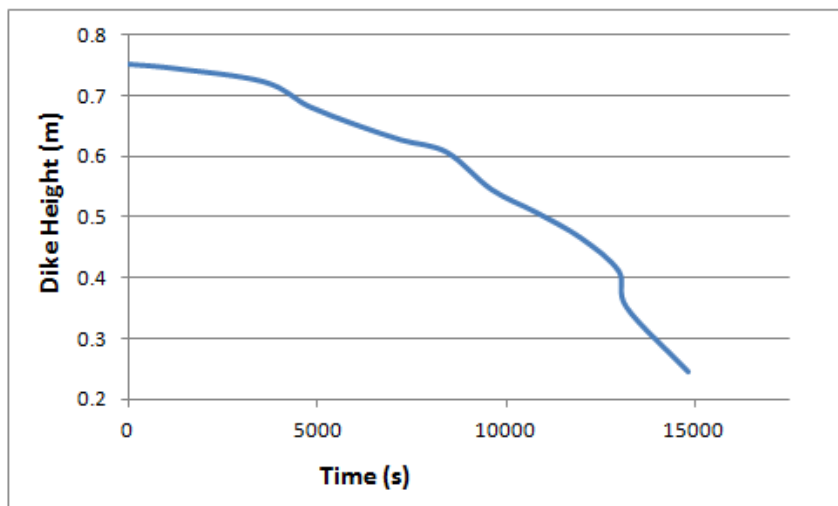


Figure 5.21: TU Delft formal test 2 - dike height

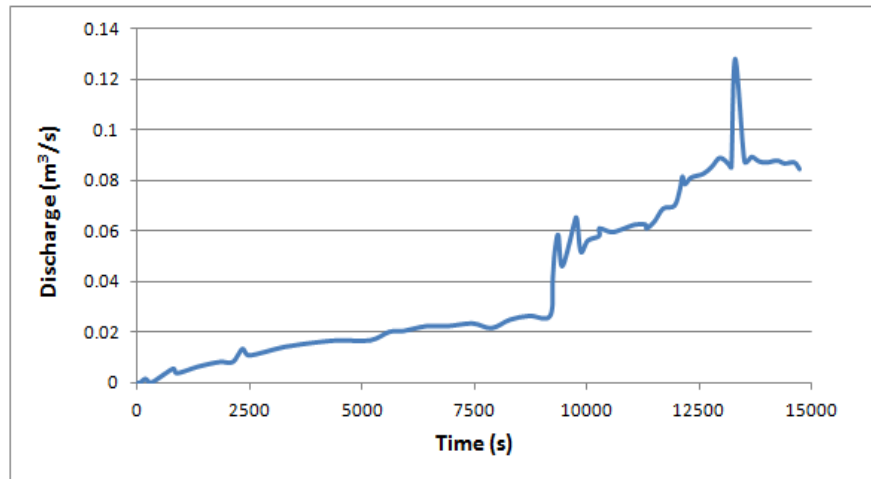


Figure 5.22: TU Delft formal test 3 – discharge

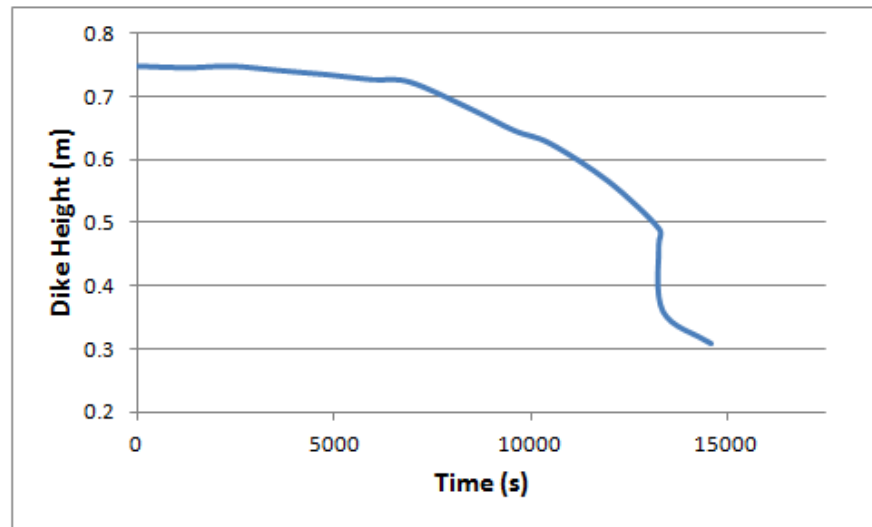


Figure 5.23: TU Delft formal test 3 - dike height

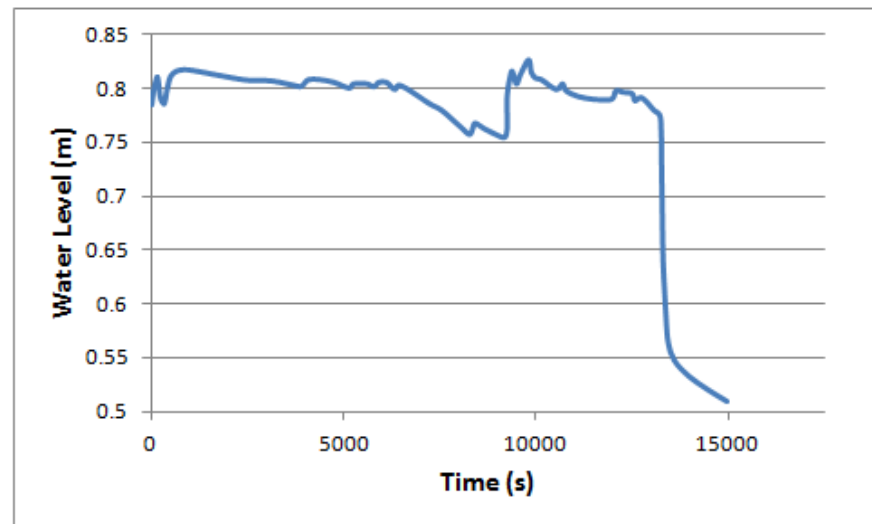


Figure 5.24: TU Delft formal test 3 - water level

5.3 IMPACT-Project Experiments

The IMPACT-Project was a study completed in Europe to assess and reduce the risk associated with extreme flooding. This extreme flooding could be caused by natural events, the failure of dams or of flood defence structures. The researchers stated that the value of dam and flood defence structures in Europe amount to many billions of Euro. They believed that it was therefore necessary to identify and manage the hazards and to have good knowledge of the structures behaviour in case of emergency situations.

The IMPACT-project was separated into five main areas of study. Four areas studied specific flood processes, while the fifth looked at the uncertainty of the processes. The four processes were breach formation, flood propagation, sediment movement, and geophysics and field data. During their breach formation study, the researchers looked at four main areas. They were:

- Field modelling of embankment failure (completed in Norway)
- Laboratory modelling of embankment failure (completed in the United Kingdom)
- Numerical modelling of embankment failure
- Breach location

For this thesis, both the results of the field modelling and laboratory modelling were compared to MIKE11 and BREACH.

5.3.1 Field Modelling of Embankment Failure

Each of the five field tests were completed in Nordland Country, Norway near the town of Mo i Rana. To see the location of the field tests please refer to Figure 5.25. The site is located about 600m downstream of the Rossvassdammen dam. The Rossvatn Reservoir and Rossaga River flow north into Sorfjorden. The spillways of the Rossvassdammen dam can be open or shut which makes it possible to control the inflow to the reservoir behind the test site.

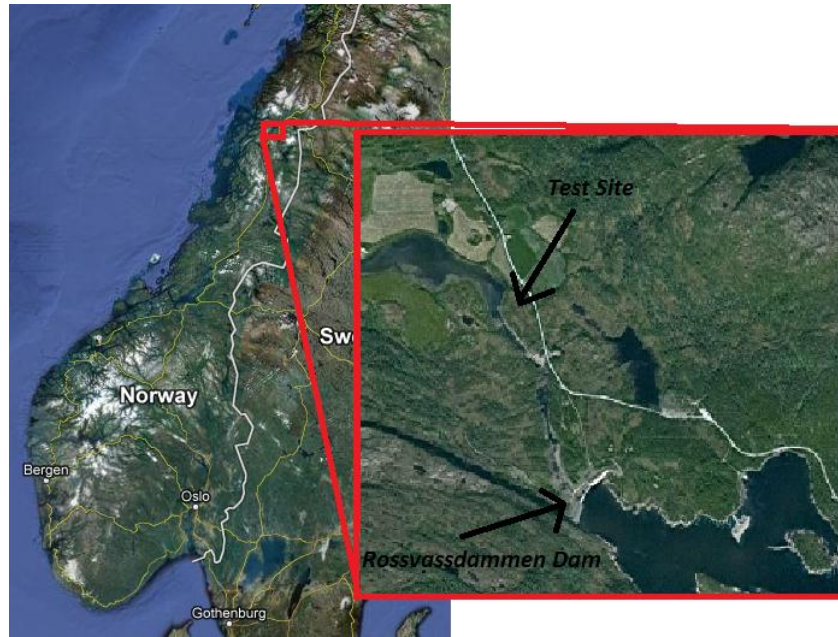


Figure 5.25: Location of IMPACT field tests

Just upstream of the test dam there is a natural weir at a level of 368 meters above sea level (m.a.s.l.) which makes the capacity of the reservoir very limited up to this elevation. Please refer to Figure 5.26 to see the capacity curve of the reservoir created for the test dam. Above 368 m.a.s.l. the capacity of the reservoir gets much larger.

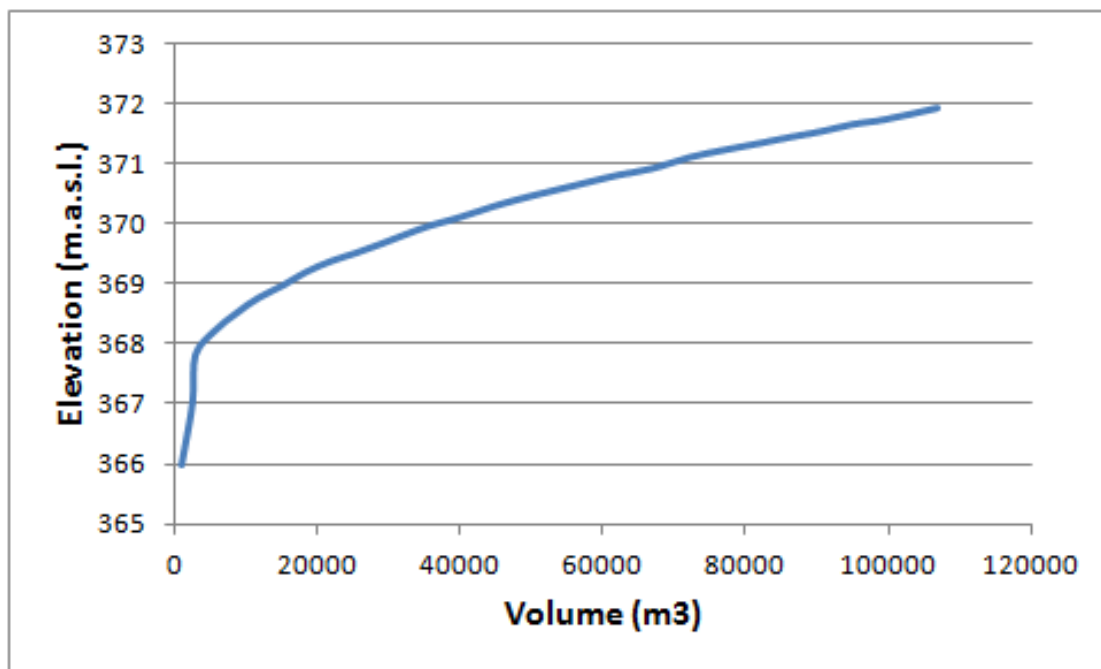


Figure 5.26: Capacity curve for reservoir used in IMPACT field tests

The following data was collected during each field test:

- Water levels (upstream and downstream)
- The amount of flow released from the reservoir to test reservoir
- The embankment pore water pressures
- Breach development
- Videos of the breach (upstream and downstream)

There were a total of five field tests completed for the IMPACT-project. The field tests varied soil properties, whether a moraine core was present, the type of breaching (overtopping or piping) and dam dimensions.

Field Test #1

The first field test was labelled as the “maximum cohesive” test because the embankment was built mainly from clay and silt. It had a D50 of approximately 0.01mm and was composed of less than 15% sand and 25% clay. This is characterised as lean clay. The main purpose of this test was to help the authors better understand the breaching characteristics of a homogeneous cohesive dam that has failed due to overtopping. To see the geotechnical characteristics of the dam please refer to Table 5.8 and for the grading curve please refer to Figure 5.27. The researchers at IMPACT provided most of the geotechnical properties however the critical shear stress needed to be calculated. The non-dimensional critical shield stress was found using the Shields Curve and the method presented by Prasuhn (1992) was used. Please refer to the section: “Methodology” for more details on the method presented by Prasuhn.

Table 5.8: Soil properties of Field Test #1

Soil Property	Value
Moisture Content	30
D50 (mm)	0.009
Porosity	0.47
Angle of friction	22.9
Cohesion (kN/m ²)	4.9
Dry Density (kN/m ³)	14.7
Specific Gravity	2.95
Critical Shear Stress	0.032

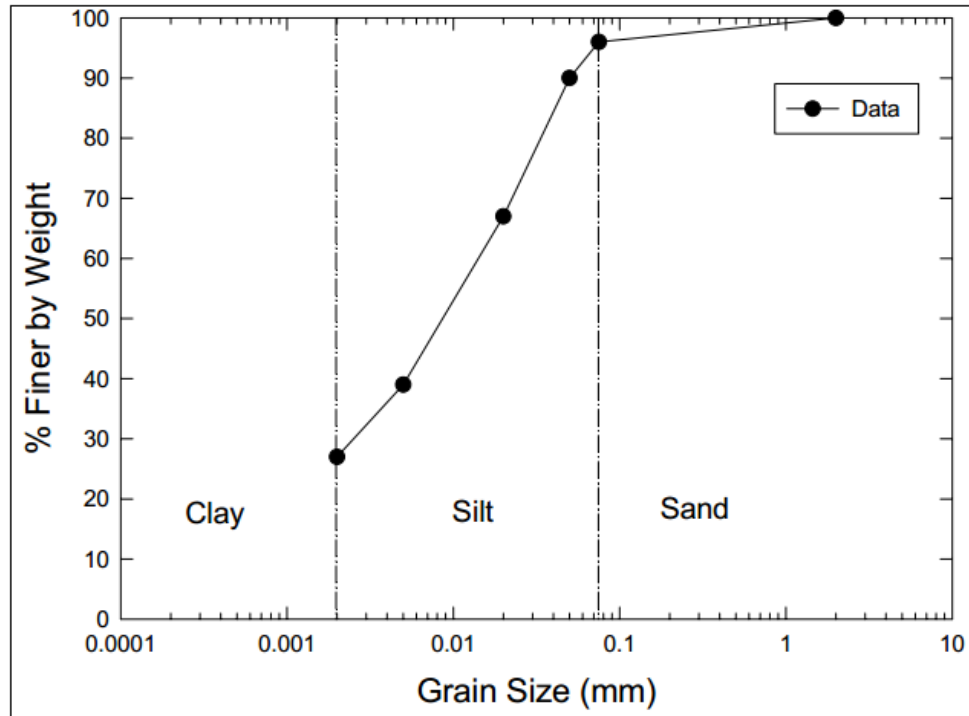


Figure 5.27: Grading curve for Field Test #1(from Morris, 2005)

The embankment had a crest level of 370.81m, crest length of 36m and crest width of 2m. Please refer to Figure 5.28 for a representation of the dam geometry. The dam had an upstream and downstream side slope of 2:1 (horizontal to vertical). The breach notch that was dug into the embankment crest to initiate a breach was 0.5m deep and had a width of 5.4m at the bottom and 8.0m at the top. Please refer to Figure 5.29 for a representation of the initial breach notch. The recorded inflow and outflow of the dam during the test can be found on Figure 5.30 and the water level and breach width during the test can be found on Figure 5.31.

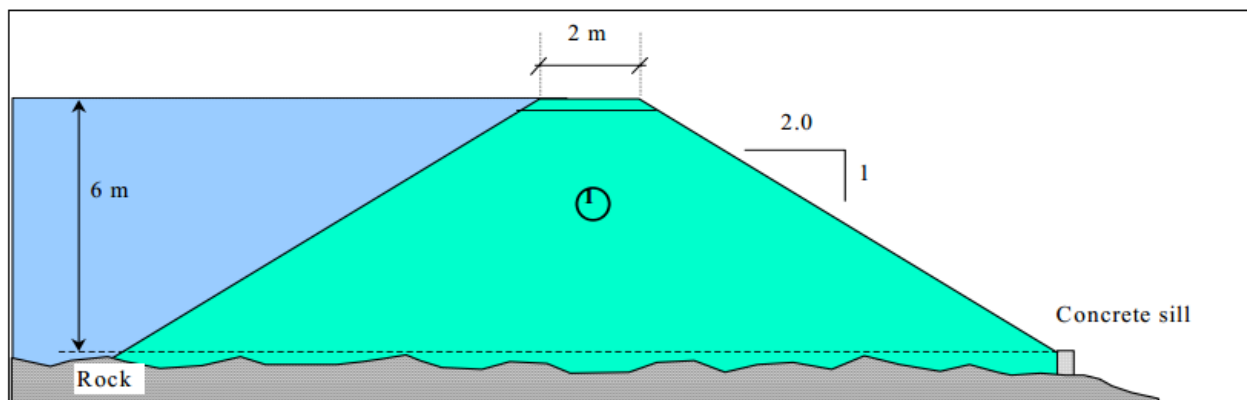


Figure 5.28: Embankment geometry of Field Test #1(from Morris, 2005)

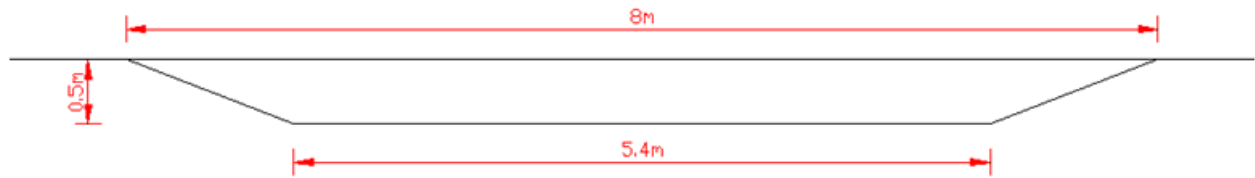


Figure 5.29: Initial breach notch for Field Test #1

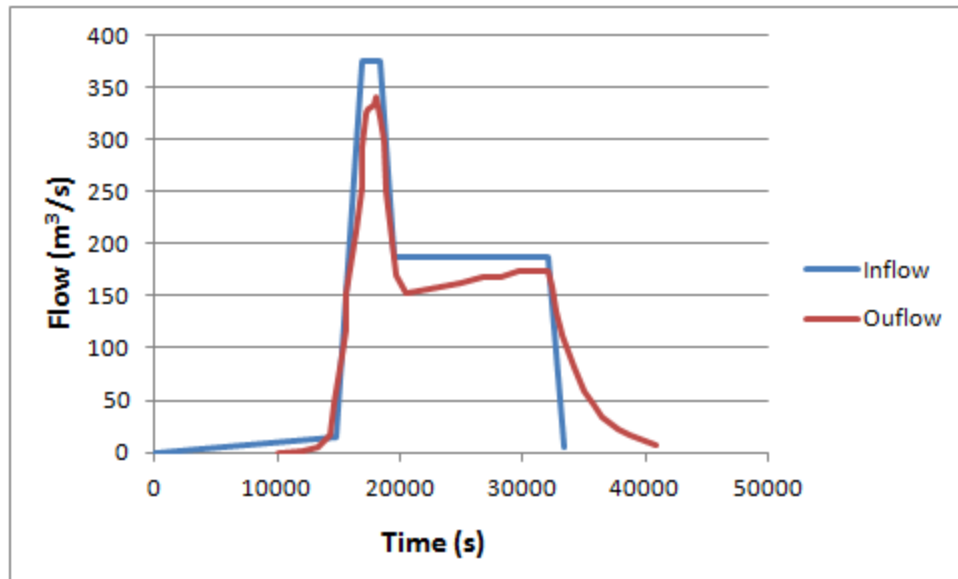


Figure 5.30: Inflow and outflow for Field Test #1

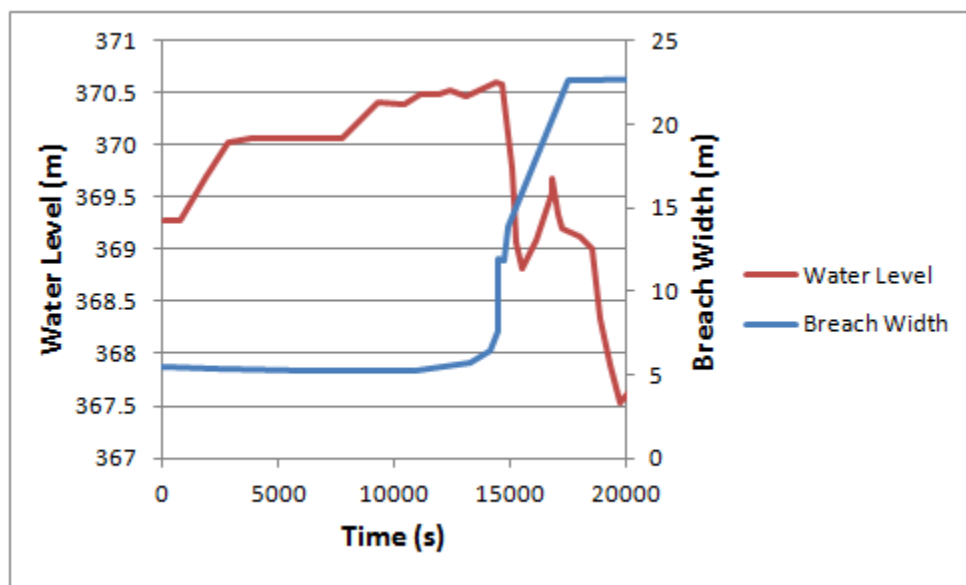


Figure 5.31: Water level and breach width for Field Test #1

Field Test #2

The second field test was known as the “minimum cohesive” test and was constructed of soil with a D50 of approximately 5mm and contained less than 5% fines. This test was used to better understand the breaching process of a homogenous, non-cohesive dam that has failed due to overtopping. To see the geotechnical characteristics of the dam please refer to Table 5.9 and for the grading curve please refer to Figure 5.32. Similar to the first field test, the researchers did not provide the non-dimensional critical shear stress for this material. The shear stress was once again calculated using the method presented by Prasuhn.

Table 5.9: Soil properties of Field Test #2

Soil Property	Value
Moisture Content	7
D50 (mm)	4.65
Porosity	0.22
Angle of friction	42
Cohesion (kN/m^2)	0.9
Dry Density (kN/m^3)	21.15
Specific Gravity	2.65
Critical Shear Stress	0.04

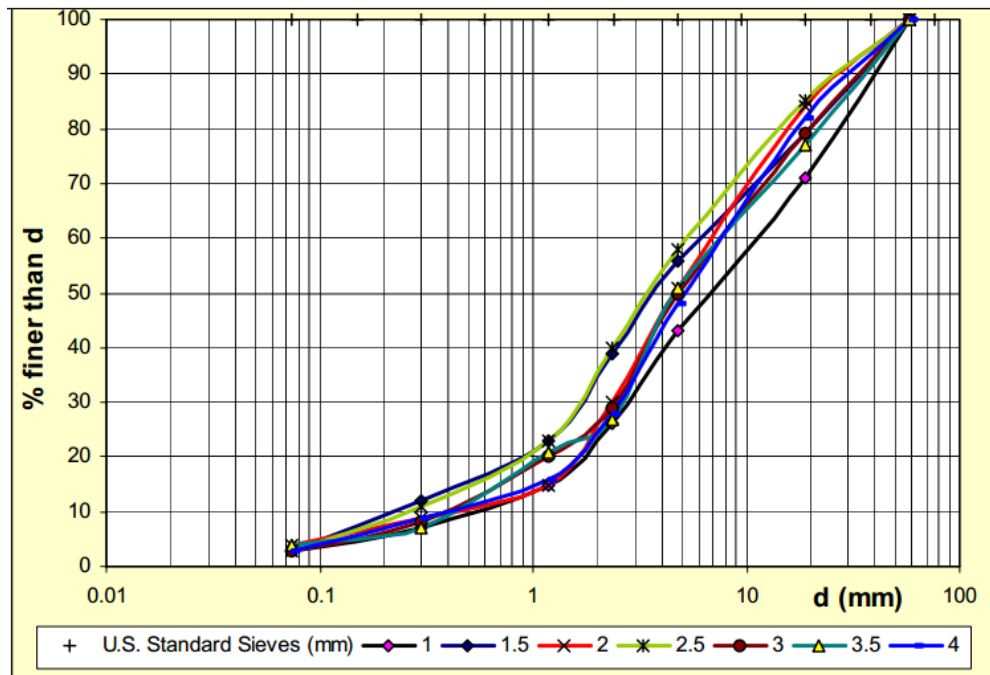


Figure 5.32: Grading curve for Field Test #2 (from Morris, 2005)

The embankment had a crest level of 369.81m, crest length of 36m and crest width of 2m. Please refer to Figure 5.33 for a representation of the dam geometry. The dam had an upstream and downstream side slope of 1.7:1 (horizontal to vertical). The breach notch that was dug into

the embankment crest to initiate a breach was 0.1m deep and had a width of 2.0m with almost vertical side slopes. The recorded inflow, outflow and water level during the test can be found on Figure 5.34.

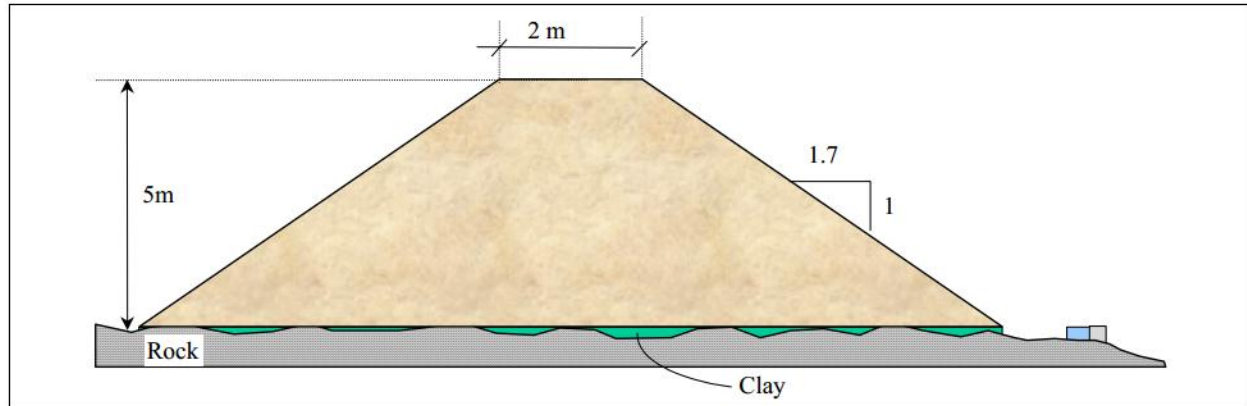


Figure 5.33: Embankment geometry of Field Test #2 (from Morris, 2005)

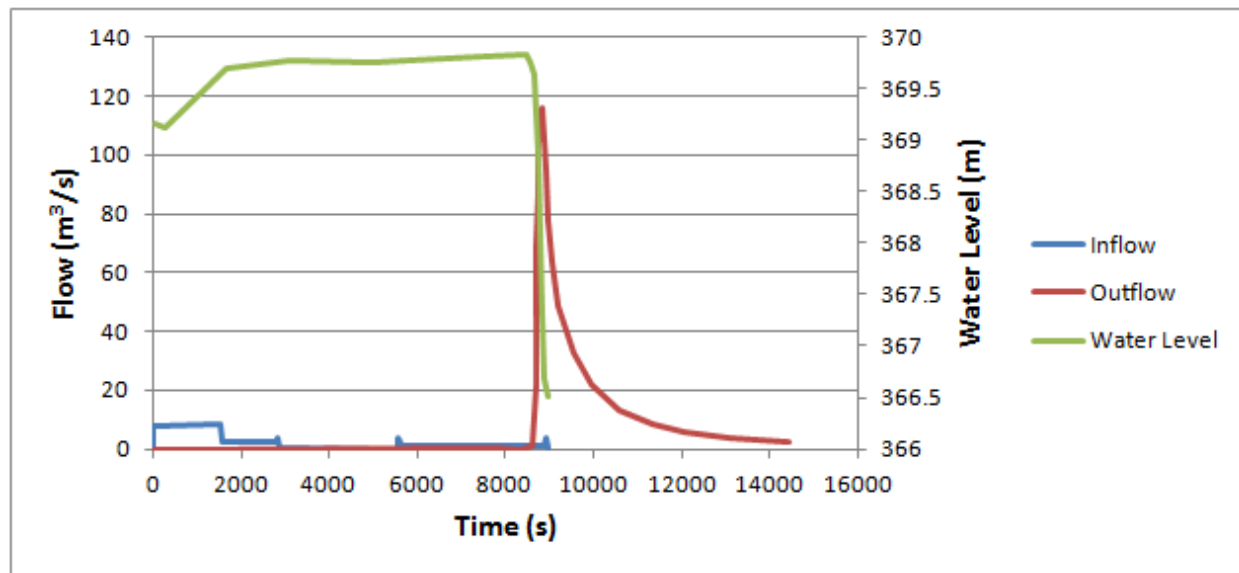


Figure 5.34: Inflow, outflow, and water level of Field Test #2

Field Test #3

The third test was constructed out of a central moraine core with rock-fill shoulders. This test was used to better understand the breach process of a composite embankment that has failed due to overtopping. To see the geotechnical characteristics of both the rock-fill and moraine used to construct this dam please refer to Table 5.10 and for the grading curves please refer to Figure 5.35. Similar to the first and second field tests, the researchers did not provide the non-dimensional critical shear stress for this material. The shear stress was once again calculated using the method presented by Prasuhn.

Table 5.10: Soil properties of Field Test #3

Soil Property	Rock-Fill Value	Moraine - Value
Moisture Content	2.6	6.0
D50 (mm)	85	7
Porosity	0.16	0.21
Angle of friction	42	42
Cohesion (kN/m^2)	0	20
Dry Density (kN/m^3)	20.8	20.5
Specific Gravity	2.70	2.65
Critical Shear Stress	0.06	0.04

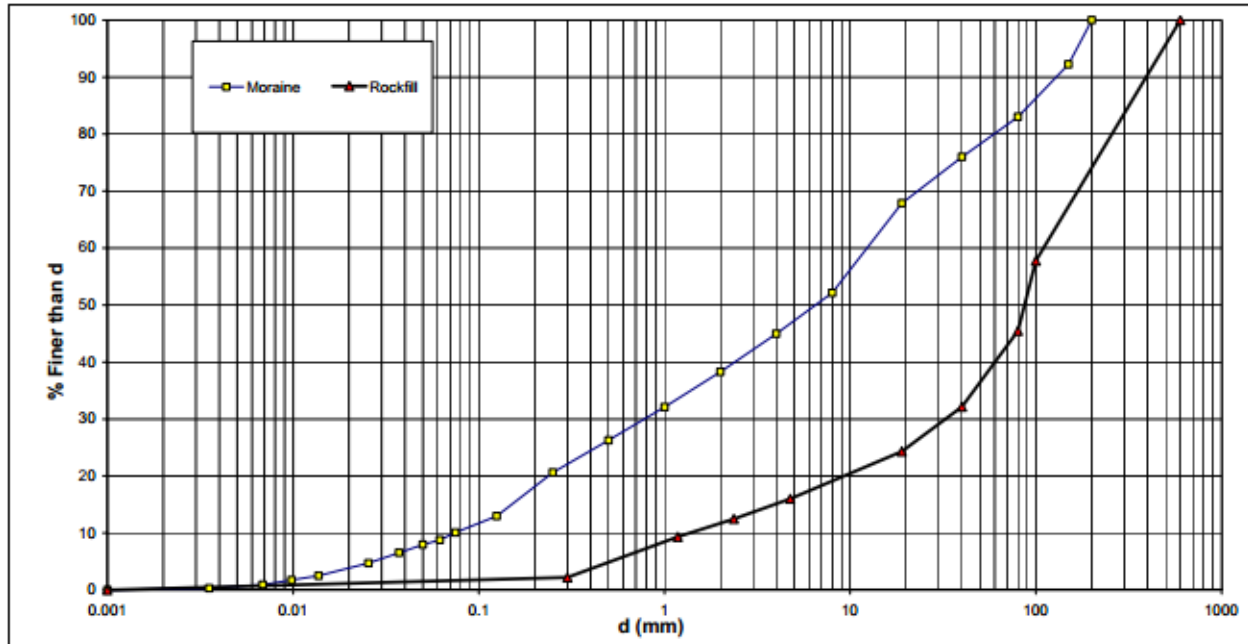


Figure 5.35: Grading curve for Field Test #3 (from Morris, 2005)

The embankment had a crest level of 370.81m, crest length of 36m and crest width of 3.0m. Please refer to Figure 5.36 for a representation of the dam geometry. It can be seen that the moraine core has a width of 1.5m and is constructed up to 0.65m below the crest width. The dam had upstream and downstream side slopes of 1.5:1 (horizontal to vertical). The breach notch that was dug into the embankment crest to initiate a breach was 0.1m deep and had a width of 2.0m with almost vertical side slopes. The recorded inflow, outflow and water level during the test can be found on Figure 5.37.

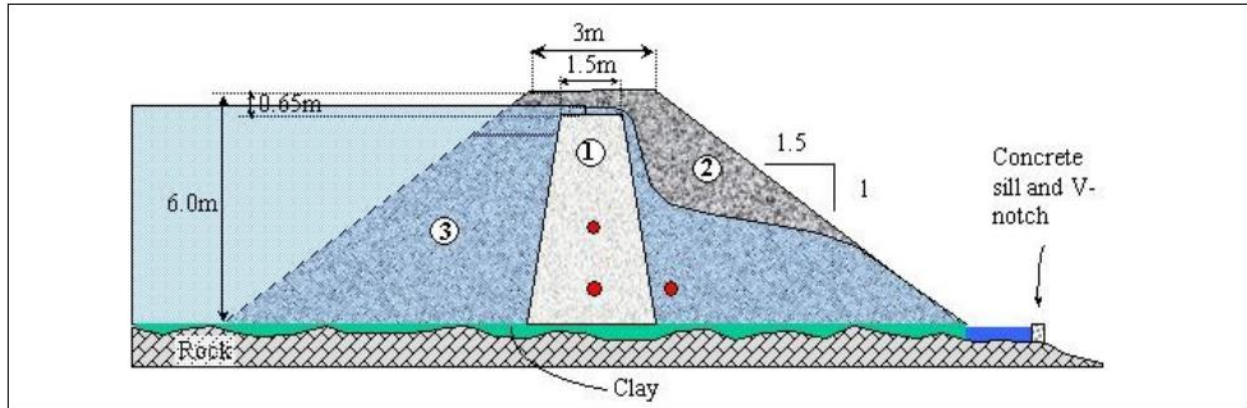


Figure 5.36: Embankment geometry of Field Test #3 (from Morris, 2005)

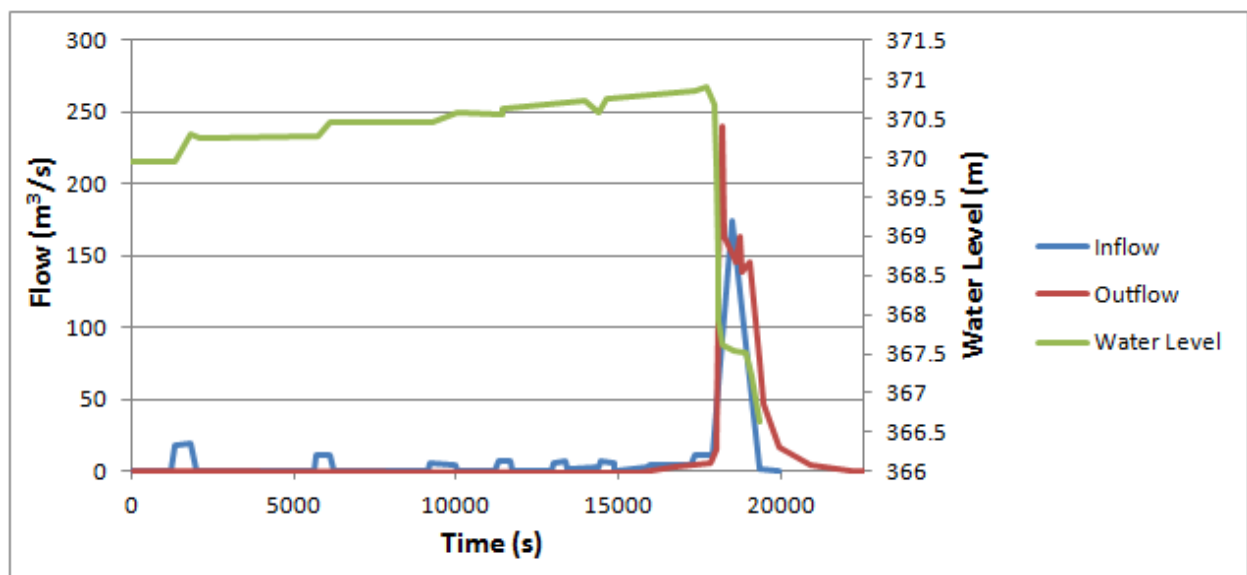


Figure 5.37: Inflow, outflow, and water level of Field Test #3

Field Test #4

The fourth field test is very similar to the third field test; however the failure is induced by piping instead of overtopping. The dam was again constructed out of a central moraine core with rock-fill shoulders. The test's purpose is to better understand the breaching process of a composite embankment that has failed due to piping. The geotechnical characteristics of the two materials can be seen in Table 5.11 and the grading curves can be seen in Figure 5.38. The non-dimensional critical shear stress was again calculated using the method outlined in Prasuhn (1992).

Table 5.11: Soil properties of Field Test #4

Soil Property	Rock-Fill Value	Moraine - Value
Moisture Content	2.6	6.0
D50 (mm)	85	7
Porosity	0.16	0.21
Angle of friction	42	42
Cohesion (kN/m^2)	0	20
Dry Density (kN/m^3)	20.8	20.5
Specific Gravity	2.70	2.65
Critical Shear Stress	0.06	0.04

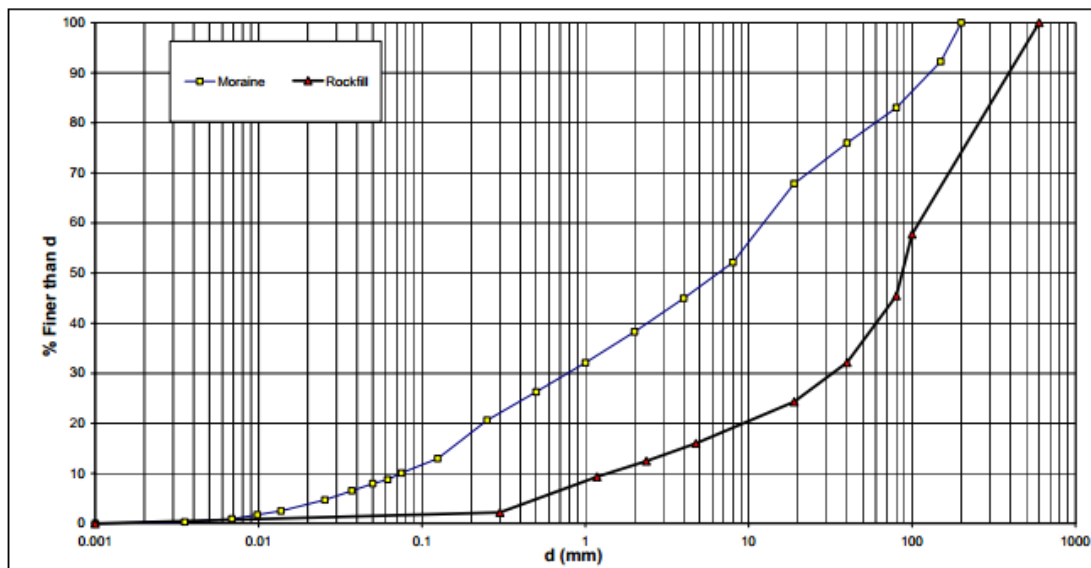


Figure 5.38: Grading curve for Field Test #4 (from Morris, 2005)

The embankment geometry was very similar to field test 3 but there were some important differences. The crest level again was set at 370.81m, with a crest length of 36m and crest width of 3.0m. Please refer to Figure 5.39 for a representation of the dam geometry. However, in this test the moraine core was constructed up to the crest. The dam had upstream and downstream side slopes of 1.5:1 (horizontal to vertical). Two different failure mechanisms were used in the test. The first trigger mechanism consisted of a pipe with a perforated top and solid ends. The bottom half of the pipe was removed and the pipe was then filled and surrounded by sand. This didn't seem to trigger a dam failure so the second trigger mechanism was used. The second mechanism was similar to the first but the sand fill surrounding the pipe extended to the crest of the dam. The initial pipe diameter was 0.215m and was placed at a level of 365.100m. Please refer to Figure 5.39 for a visual representation of the different trigger mechanisms. The recorded inflow, outflow and water level during the test can be found on Figure 5.40.

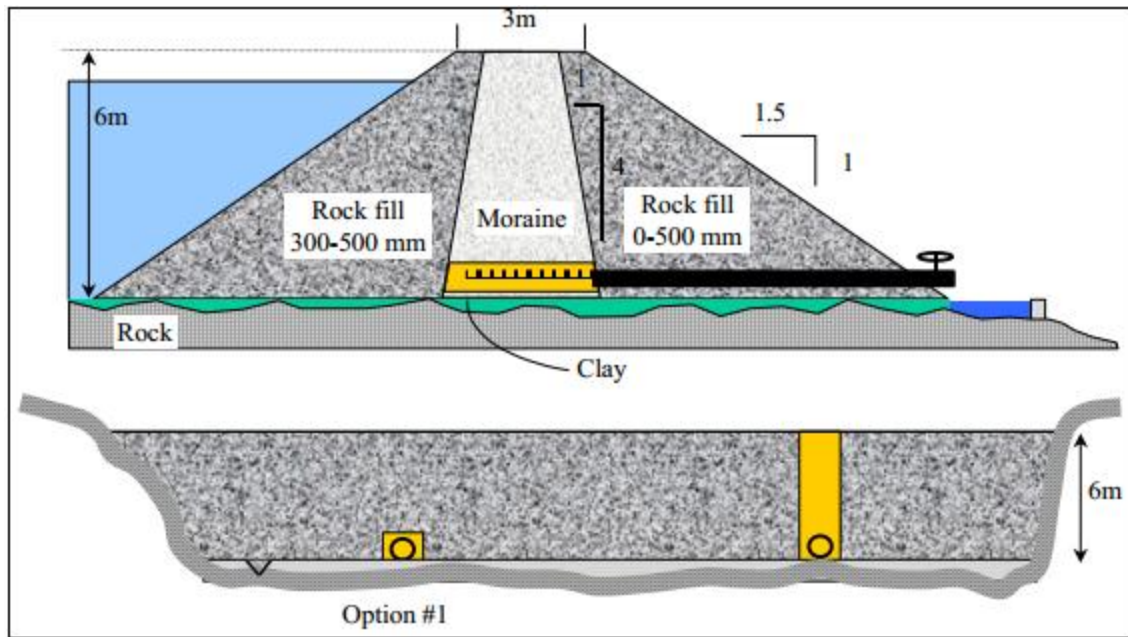


Figure 5.39: Embankment geometry of Field Test #4 (from Morris, 2005)

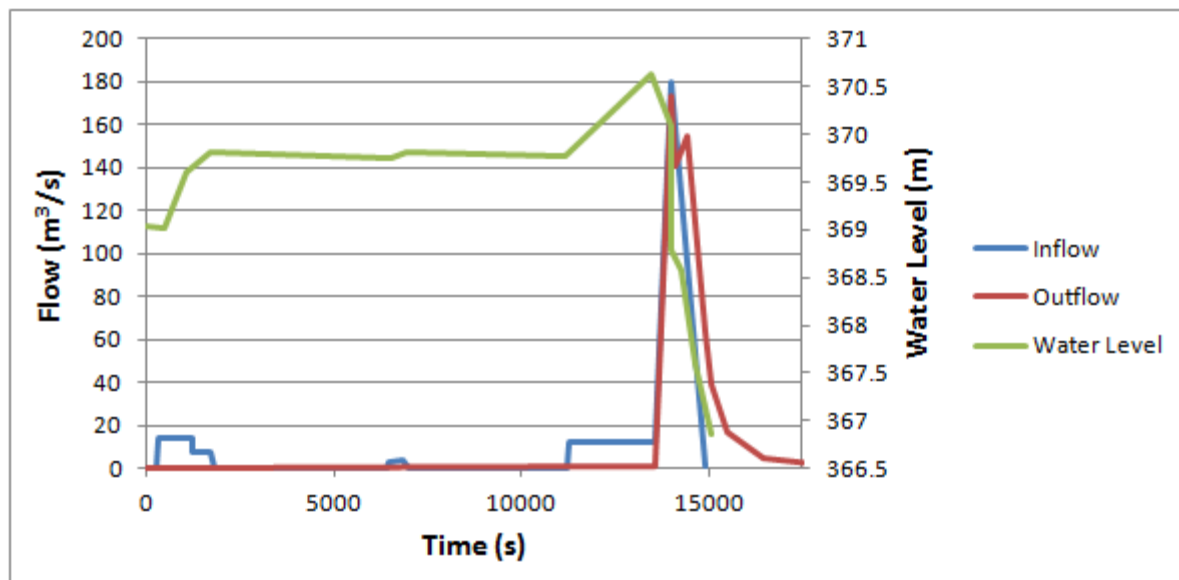


Figure 5.40: Inflow, outflow, and water level of Field Test #4

Field Test #5

The fifth and final test was built entirely out of the moraine material used in field test 3 and 4. The purpose of this test was to understand the breaching process of a homogenous embankment that has failed due to piping. The geotechnical characteristics of the moraine can be seen in Table 5.12 and the grading curve can be seen in Figure 5.41. The non-dimensional critical shear stress was again calculated using the method outlined in Prasuhn (1992).

Table 5.12: Soil properties of Field Test #5

Soil Property	Value
Moisture Content	6.0
D50 (mm)	7
Porosity	0.21
Angle of friction	42
Cohesion (kN/m^2)	20
Dry Density (kN/m^3)	20.5
Specific Gravity	2.65
Critical Shear Stress	0.04

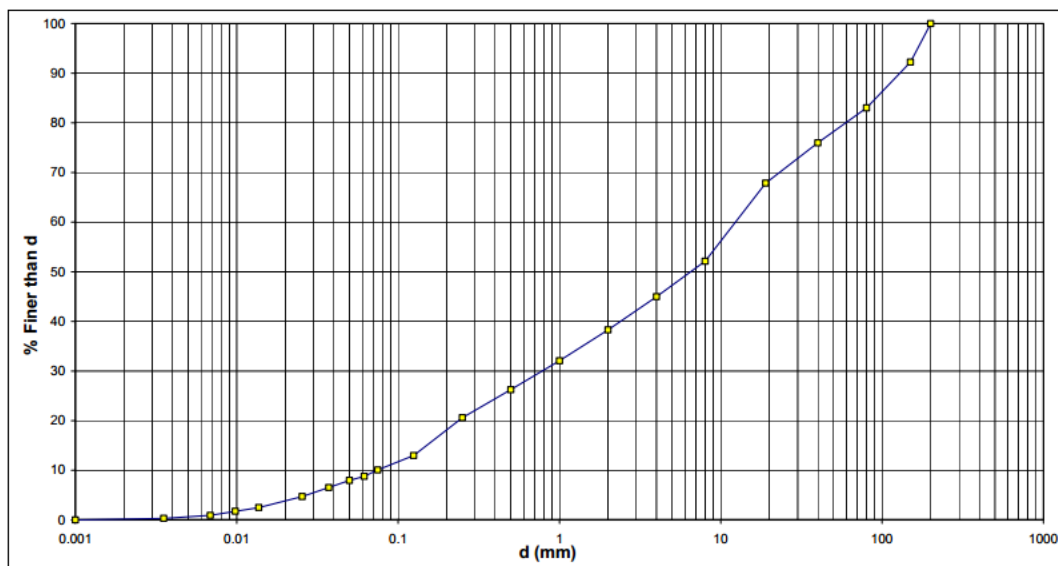


Figure 5.41: Grading curve for Field Test #5 (from Morris, 2005)

The crest level was set at 369.31m, with a crest length of 36m and crest width of 3.0m. Please refer to Figure 5.42 for a representation of the dam geometry. The dam had an upstream and downstream side slope of 1.3:1 (horizontal to vertical). However, in this test the moraine core was constructed up to the crest. The dam had an upstream and downstream side slope of 1.5:1 (horizontal to vertical). The trigger mechanism that was used to induce breaching was similar to the first mechanism discussed in field test #4. The initial pipe diameter was 0.2m and was placed at a level of 364.8m. The recorded inflow, outflow and water level during the test can be found on Figure 5.43.

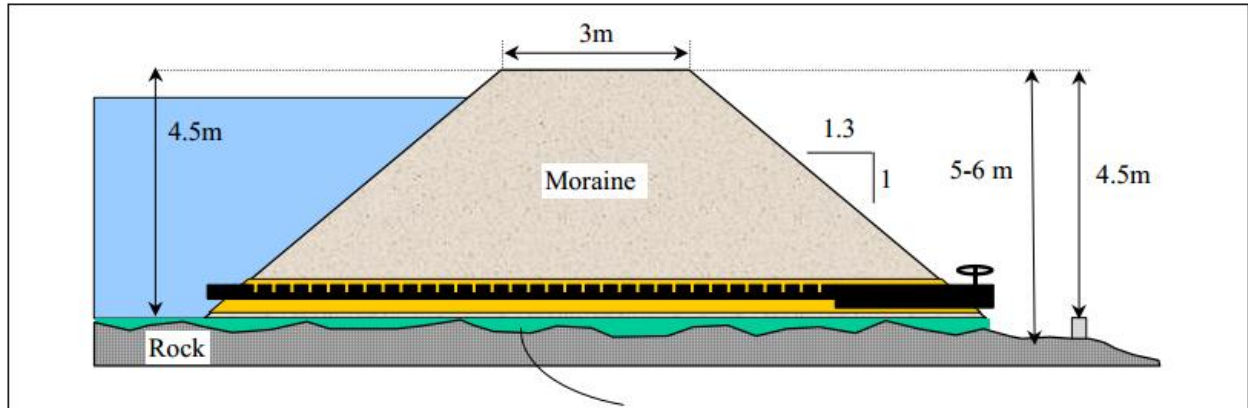


Figure 5.42: Embankment geometry of Field Test #5 (from Morris, 2005)

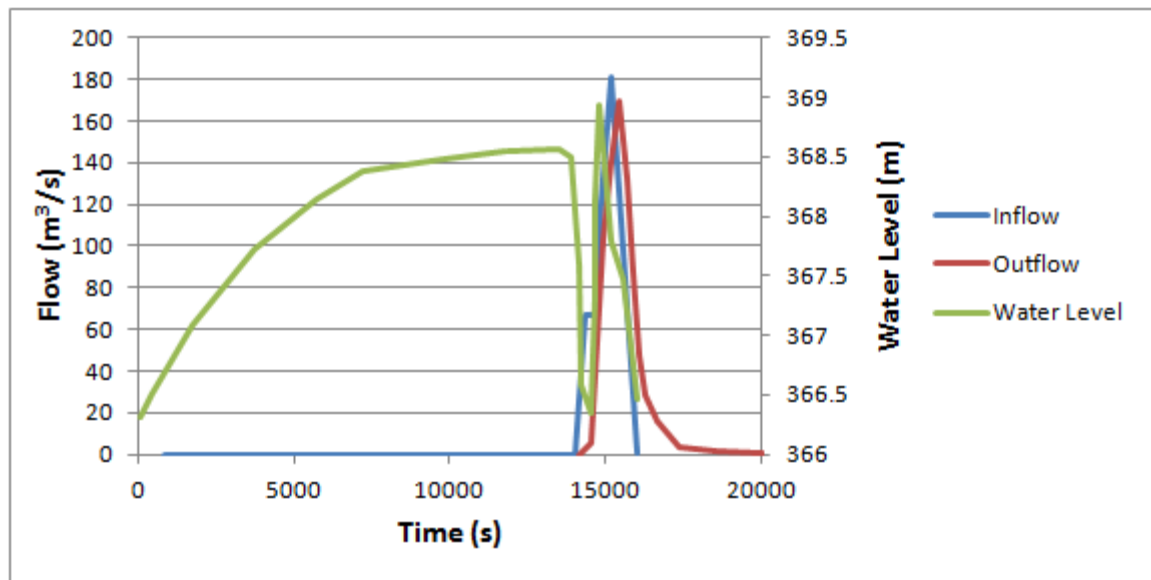


Figure 5.43: Inflow, outflow, and water level of Field Test #5

5.3.2 Laboratory Modelling of Embankment Failure

The laboratory tests were completed in a large flume at HR Wallingford in the United Kingdom. The flume was modified to fit the requirements of the embankment tests. The flume is approximately 50m long and 10m wide. There are a number of pumps equipped in the laboratory to allow over $1\text{m}^3/\text{s}$ of flow throughout the flume. There is a very large storage reservoir upstream of the flume to ensure that continuous flow during the laboratory tests is possible. There is also a large pit downstream of where the embankment is constructed to act as a sediment trap before the water is re-circulated in the sump system. The described flume was used for all of the overtopping tests (series 1 and 2). Two separate flumes were used for the piping tests (series 3) due to the nature of the tests. Series 3 tests were not used in this study so more information on these tests will not be provided.

During Series 1 and 2 tests, the following data was collected:

- Inflow into the flume
- Water levels (upstream and downstream of embankment)
- Velocity approaching embankment
- Pore water pressure in the embankment
- Photos and videos of the breach development

As mentioned earlier, there was a total of 3 different series of tests in the IMPACT laboratory study. The first series contained non-cohesive embankments that have failed due to overtopping. The second series contained cohesive embankments that have failed due to overtopping. The third and final series of tests contained embankments that have failed due to piping.

Series 1: Non-Cohesive Embankments (overtopping failure)

A total of 9 tests were completed in the first series. These tests were based around the second field test at a scale of 1:10. Each test was constructed in the flume using a non-cohesive material. The authors decided to use more than one grading of sediment to see how the results varied. The authors also varied embankment geometry, breach location and amount of seepage throughout the tests to see how the parameters varied the breaching process.

A total of 3 different sediment gradings were used in this series. The first had a uniform grading with a D_{50} of 0.70-0.90mm. The second had a uniform grading with a D_{50} of 0.25mm. The final grading had a wide grading with a D_{50} of 0.25m. The uniform grading tests used a grading curve that was as steep as possible (dependent on sediment suppliers). The wide grading tests used a combination of 4 different types of sand to match the grading distribution that was present in the field. To see the grading curves of the three different sediments please refer to Figure 5.44. For more details on the laboratory tests in series 1 please refer to Table 5.13.

Table 5.13: Impact lab test series 1 summary

Lab Test Number	Test Description	Test Objective
Lab Test #1	Trial/Test Experiment	Facility set up and trial
Lab Test #2	Second Uniform Grading	The effect of sediment uniformity
Lab Test #3	Same as Lab Test #2	To assess the repeatability of the test
Lab Test #4	Same as Lab Test #2 but with initial breach notch against abutment	To assess the effect of the breach location
Lab Test #5	Same as Field Test #2 (scale of 1:10)	A direct replication of the field event
Lab Test #6	Same material as lab test #5 but with a face slope of 1:2 instead of 1:1.7	To assess the effect of the face slope
Lab Test #7	Same material as lab test #5 but with a crest width of 0.30m instead of 0.20m	To assess the effect of the crest width
Lab Test #8	Same geometry as Lab Test #2 but using the first uniform grading	To assess the effect of the sediment size
Lab Test #9	Same geometry as lab test #2 but allowing seepage before failure	To assess the effect of seepage

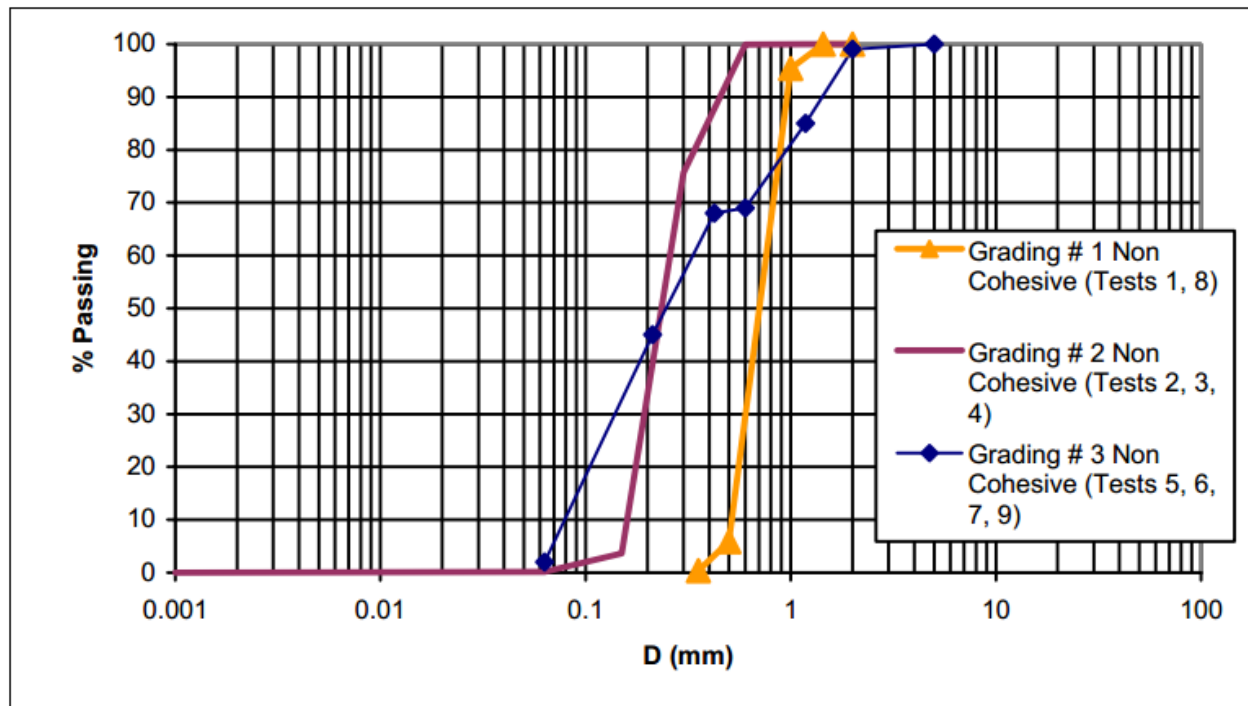


Figure 5.44: Grading curves for different sediments used in IMPACT lab tests - series 1(from Morris, 2005)

Series 2: Cohesive Embankments (overtopping failure)

Another 8 tests were completed at HR Wallingford and they were based around Field Test #1 at a scale of 1:10. Tests #10-16 were built out of clay and test #17 was built from a moraine material. To see the grading curves for clay and moraine please refer to Figure 5.45 and for more details on the series 2 laboratory tests please refer to Table 5.14 .

Table 5.14: Impact lab test series 2 summary

Lab Test Number	Test Description	Test Objective
Lab Test #10	Same as Field Test #1 (scale of 1:10)	Replicating Field Test #1
Lab Test #11	Same as Lab Test #10	To assess the repeatability of the test
Lab Test #12	Same as Lab Test #10 but compacted with half the compaction effort	To assess the effect of compaction
Lab Test #13	Same as Lab Test #10 but with optimum moisture content (only partially failed)	To assess the effect of moisture content
Lab Test #14	Remainder of Lab Test #13 (left overnight)	To assess the effect of seepage
Lab Test #15	Same as Lab Test #10 but a downstream face slope of 1:1 instead of 1:2	To assess the effect of the downstream face slope
Lab Test #16	Same as Lab Test #10 but a downstream face slope of 1:3 instead of 1:2	To assess the effect of the downstream face slope
Lab Test #17	Same as Lab Test #10 but built from moraine instead of clay	To assess the effect of construction material

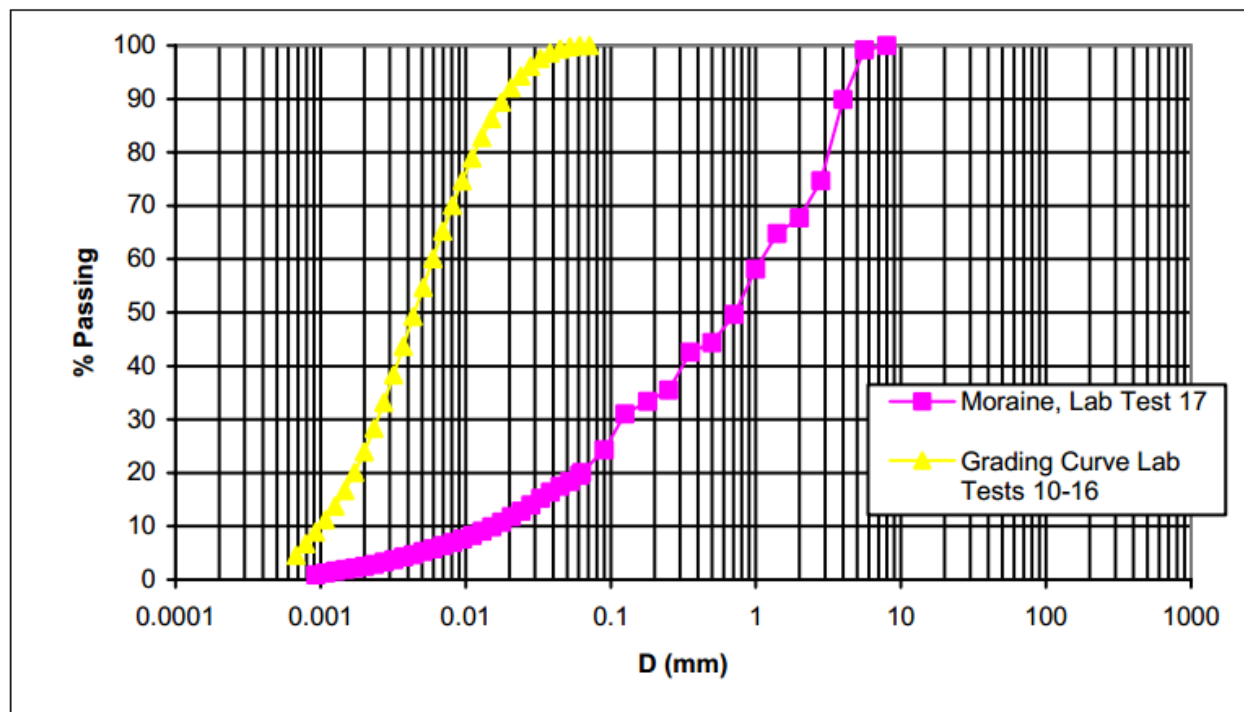


Figure 5.45: Grading curves for different sediments used in IMPACT lab tests - series 2(from Morris, 2005)

Series 3: Piping Failure

The last series of tests consisted of 5 tests. The first two tests of the series were used to support the construction of the triggering mechanism used in Field Test #5 and to give more understanding of piping initiation and formation. The last three tests were used to better understand the breaching process of piping failures and they were constructed out of material found in a local river embankment. To see the grading curves for the sediment and moraine please refer to Figure 5.46 and for more details on the series 3 laboratory tests please refer to Table 5.15.

Table 5.15: Impact lab test series 3 summary

Lab Test Number	Test Description	Test Objective
Lab Test #18	Same as Field Test #5 (scale of 1:10)	Provide information on triggering mechanisms to help in the field
Lab Test #19	Same as Lab Test #18	To assess the repeatability of the test
Lab Test #20	Material from UK river embankment	Monitor the breaching process due to failure
Lab Test #21	Material from UK river embankment	
Lab Test #22	Material from UK river embankment	

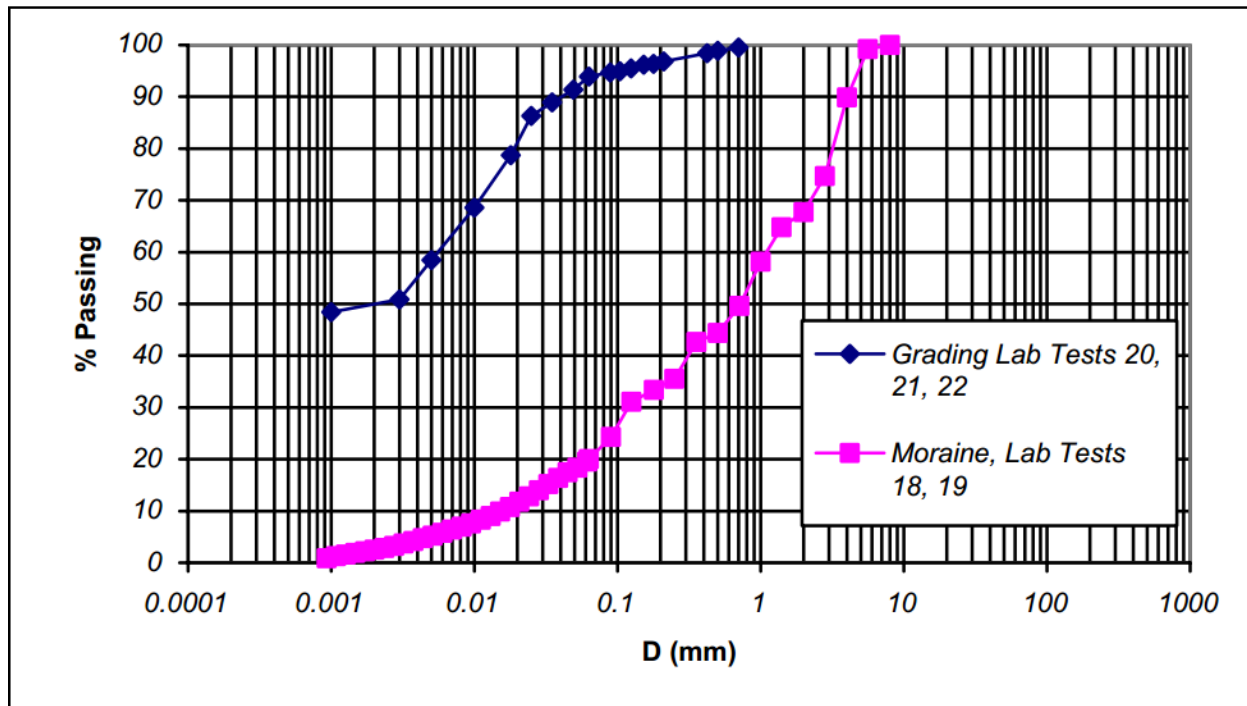


Figure 5.46: Grading curves for different sediments used in IMPACT lab tests - series 3(from Morris, 2005)

Please refer to Figure 5.47 and Figure 5.57 for a summary of the inflow, outflow, water level, and breach width for each of the laboratory tests completed in the IMPACT-Project. A comparison to the numerical model results are shown in the results section and appendix.

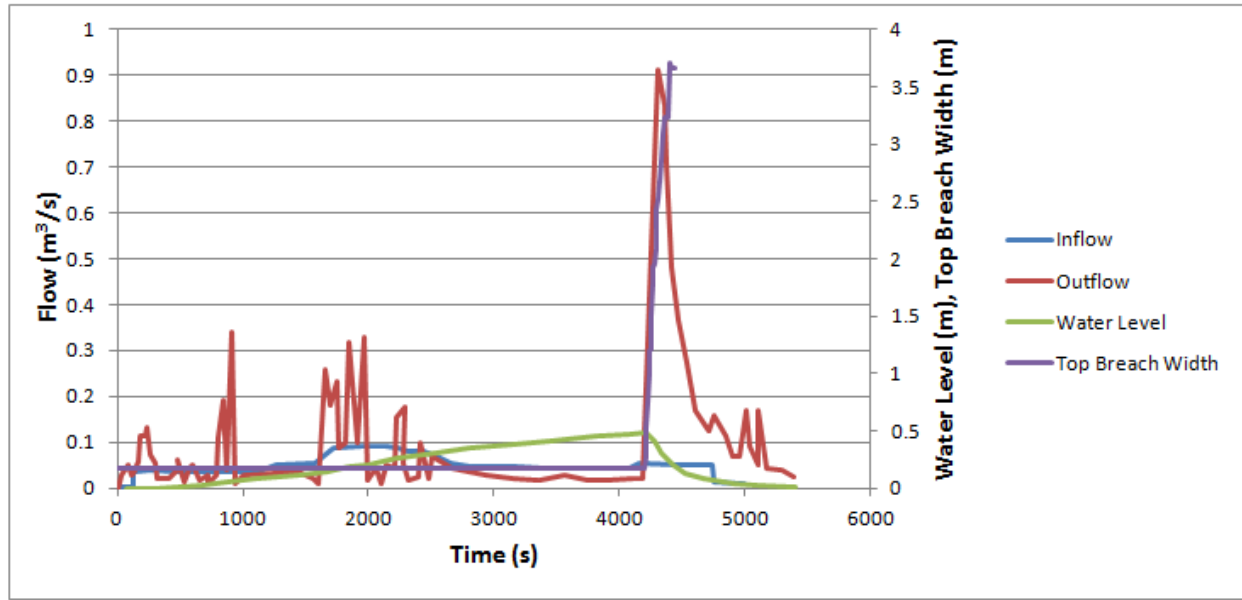


Figure 5.47: Inflow, outflow, water level and top breach width for Lab Test #2

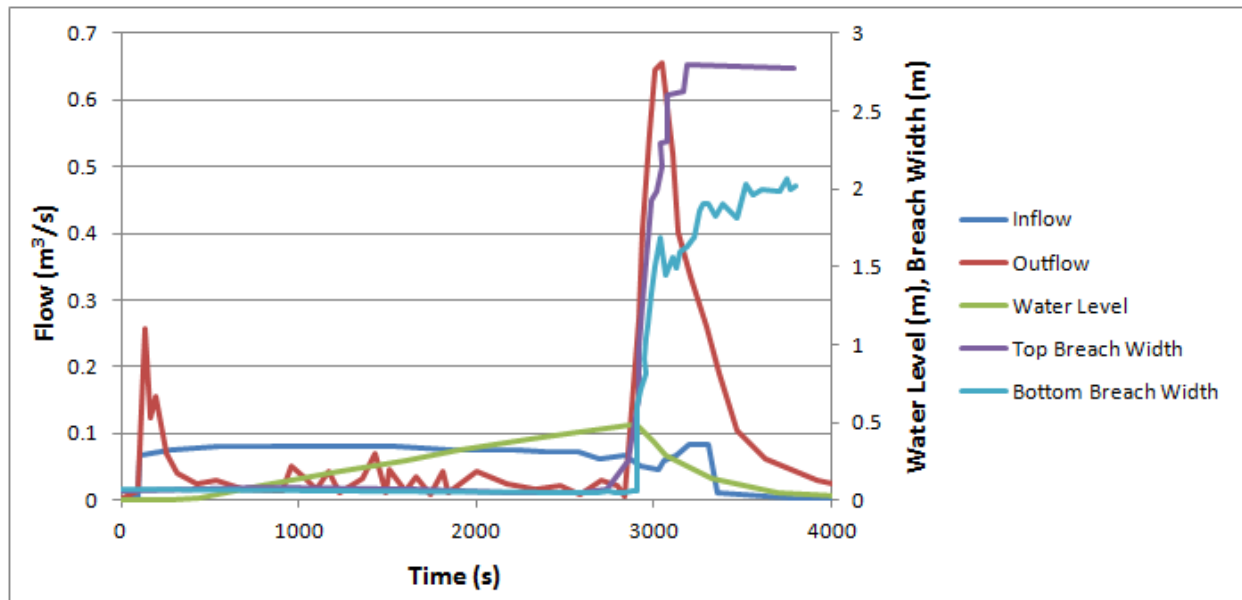


Figure 5.48: Inflow, outflow, water level and top breach width for Lab Test #4

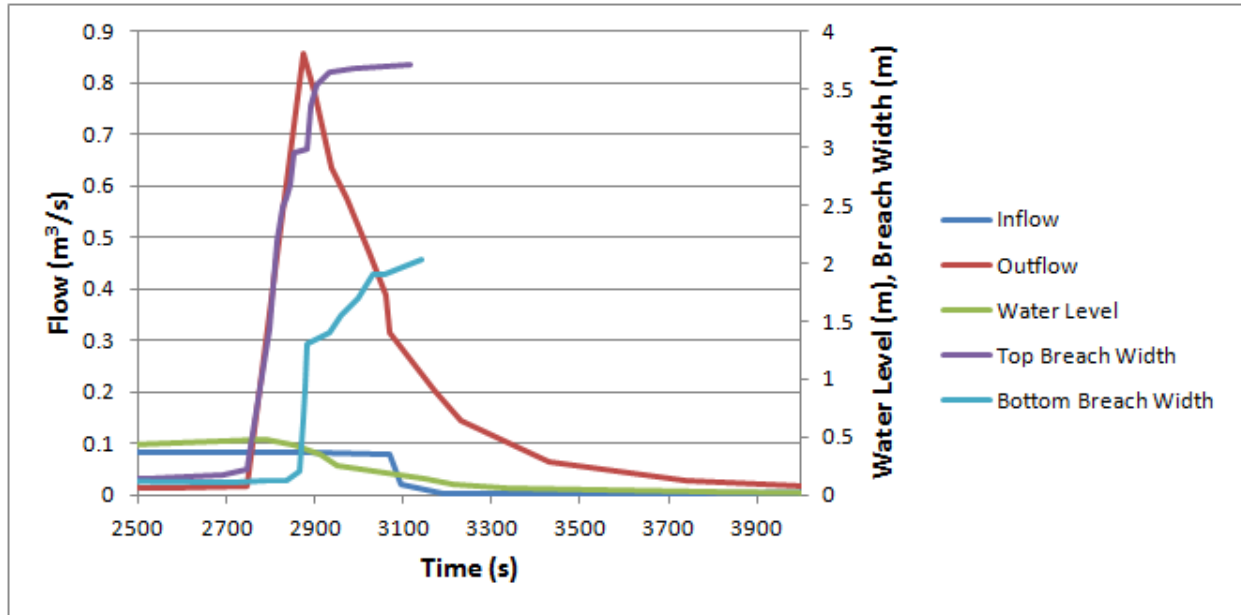


Figure 5.49: Inflow, outflow, water level and top breach width for Lab Test #5

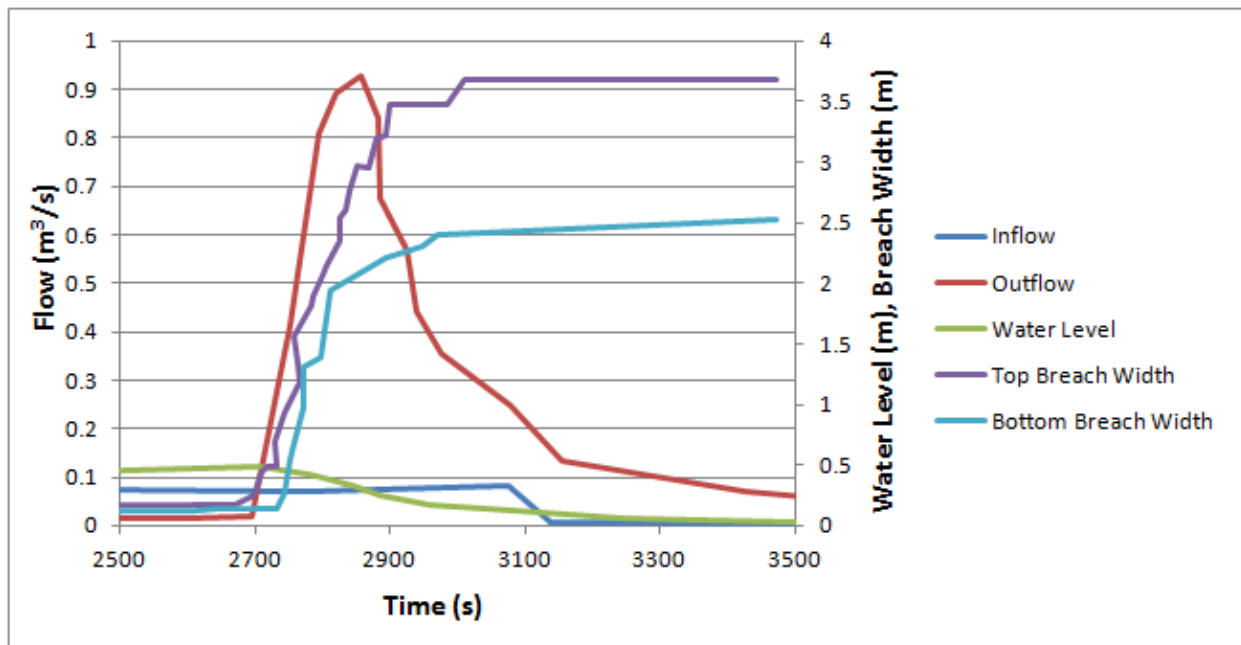


Figure 5.50: Inflow, outflow, water level and top breach width for Lab Test #6

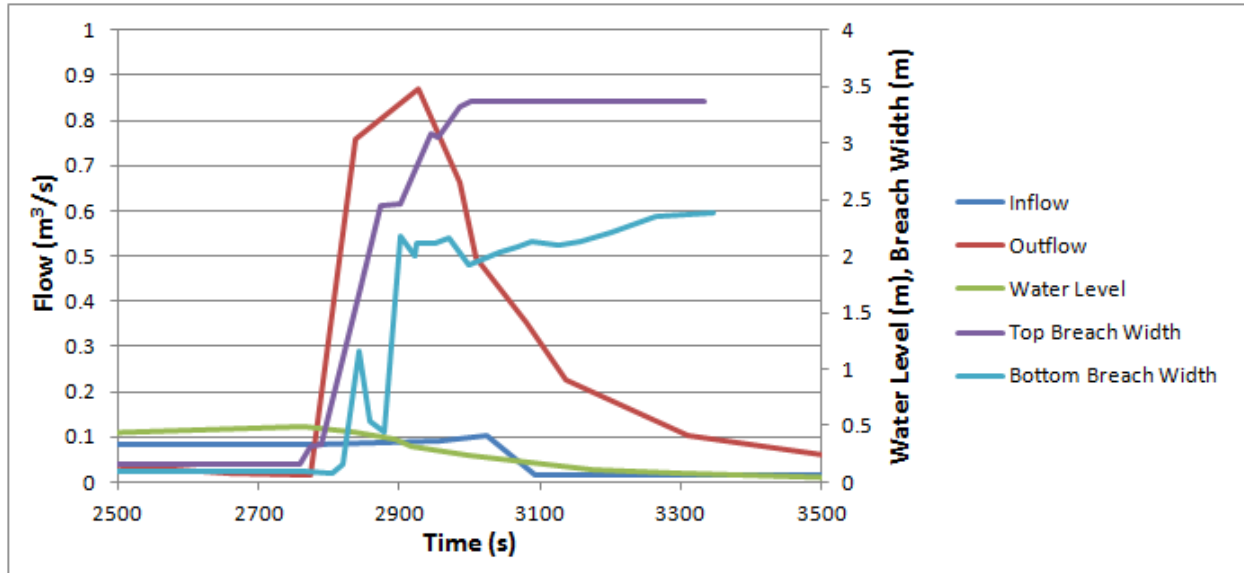


Figure 5.51: Inflow, outflow, water level and top breach width for Lab Test #7

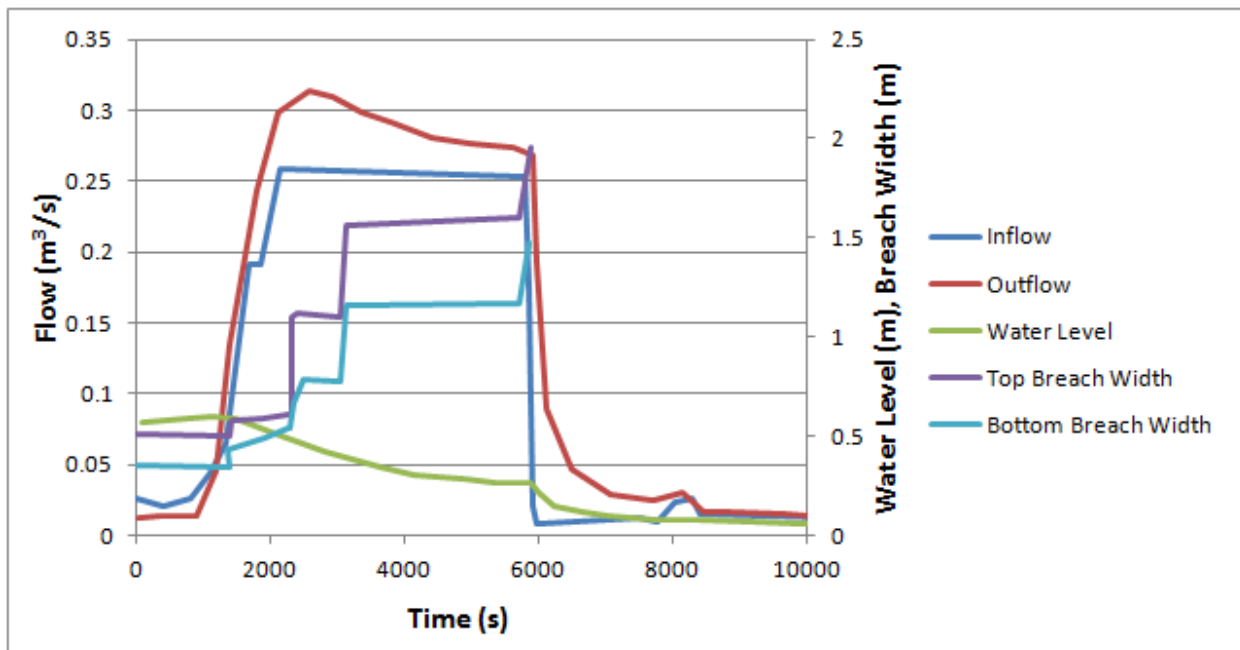


Figure 5.52: Inflow, outflow, water level and top breach width for Lab Test #10

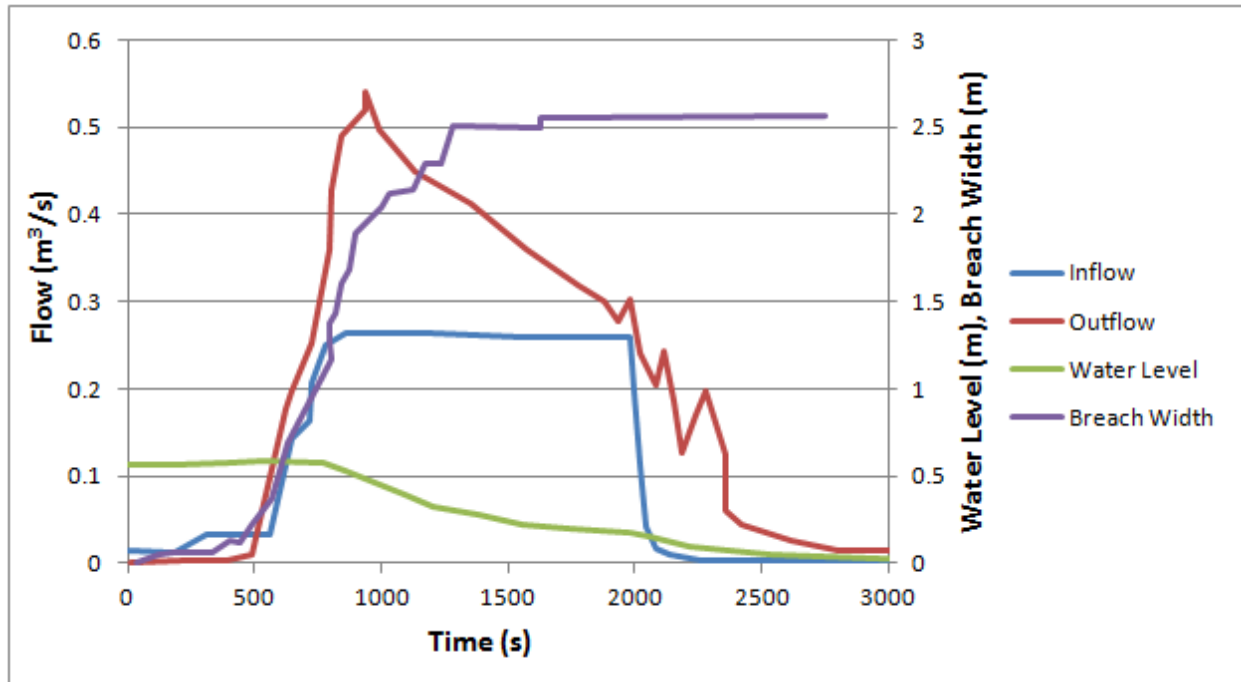


Figure 5.53: Inflow, outflow, water level and top breach width for Lab Test #12

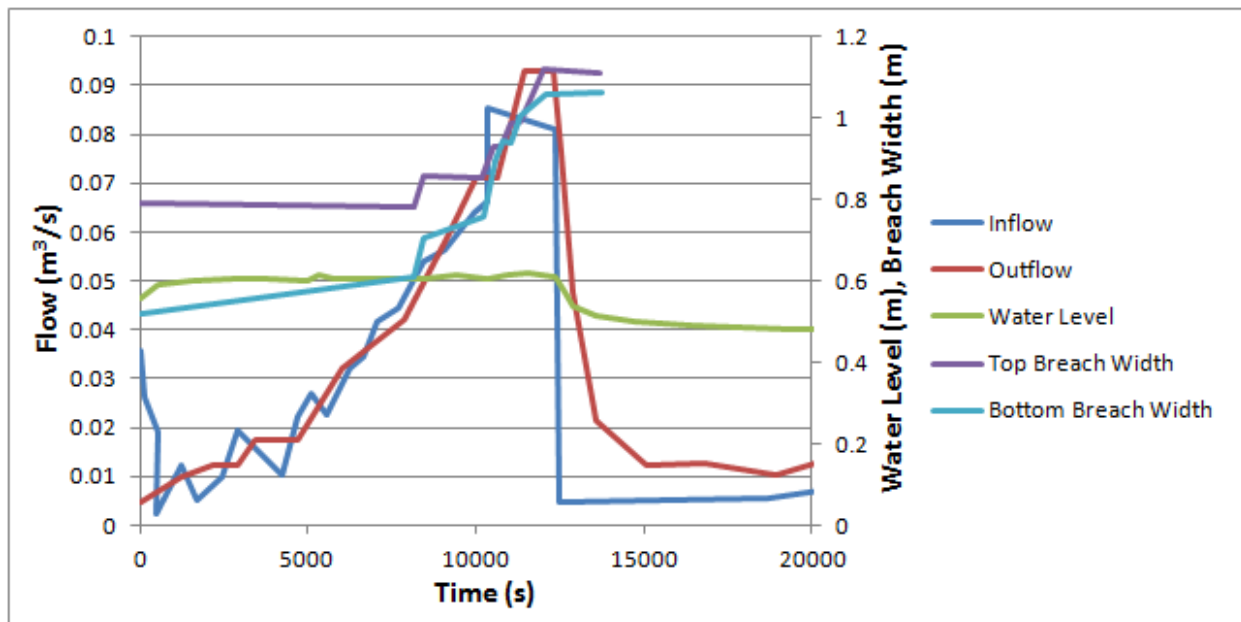


Figure 5.54: Inflow, outflow, water level and top breach width for Lab Test #13

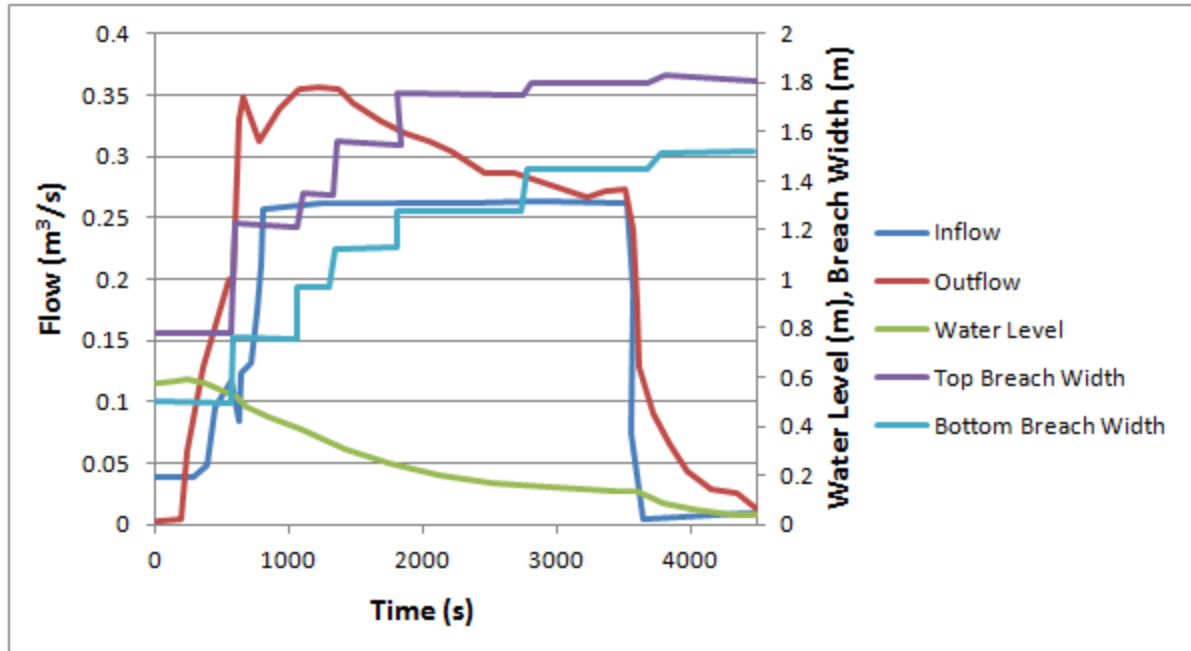


Figure 5.55: Inflow, outflow, water level and top breach width for Lab Test #15

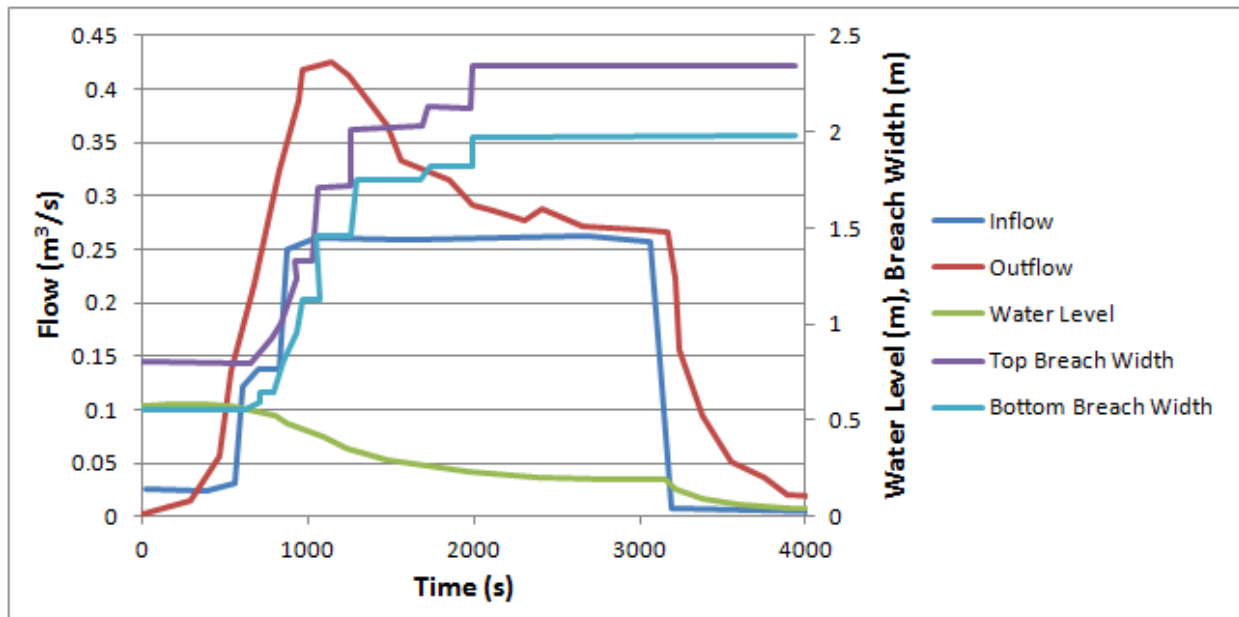


Figure 5.56: Inflow, outflow, water level and top breach width for Lab Test #16

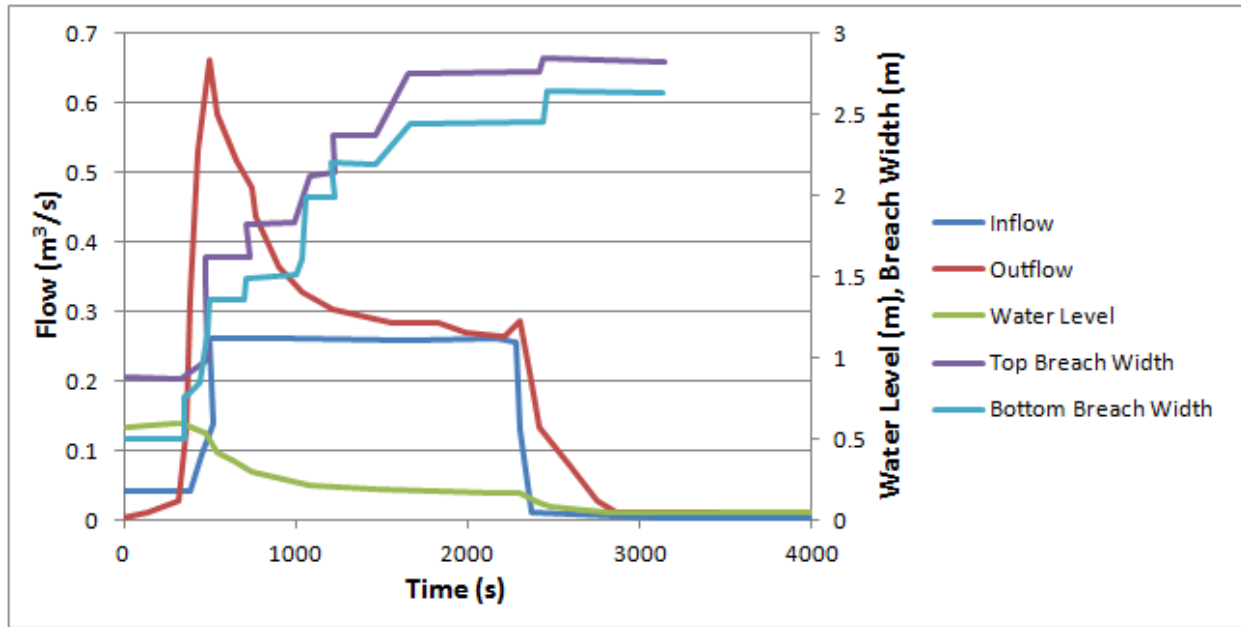


Figure 5.57: Inflow, outflow, water level and top breach width for Lab Test #17

6.0 Methodology

This section outlines the methodology used during this study. One of the main issues with this type of study was that some of the information needed to run the simulations was not available and approximations had to be made. How these approximations were made is outlined below. The method used to compare the two models and complete a sensitivity analysis is also outlined below.

6.1 Estimating Non-Dimensional Critical Shear Stress

The Non-Dimensional Critical Shear Stress, τ_c^* , can be estimated using Shields curve that was reproduced by Prasuhn (1992). To use the Shields curve shown in Figure 6.1, both the dimensionless stress, τ^* , and the boundary Reynolds number, R^* , are needed. The Shields curve is mostly made up of uniform sediments such as quartz. This makes its applicability for finer sediments debatable (Orendorff, 2010). The Boundary Reynolds number can be found using:

$$R^* = \frac{u^* D}{\nu} \quad (6.1)$$

where D is the diameter in millimeters, ν is the kinematic viscosity, and u^* is the shear velocity which can be computed using the relation below.

$$u^* = \sqrt{\tau / \rho_w} \quad (6.2)$$

where τ is the bed shear stress and ρ_w is the density of water. The best stress can be calculated using:

$$\tau = \rho g h S \quad (6.3)$$

where ρ is the density of water, g is the gravitational acceleration, h is the depth of flow, and S is the slope of the reach in question. After the Reynolds number is calculated the curve can then be followed and the non-dimensional critical stress can be estimated.

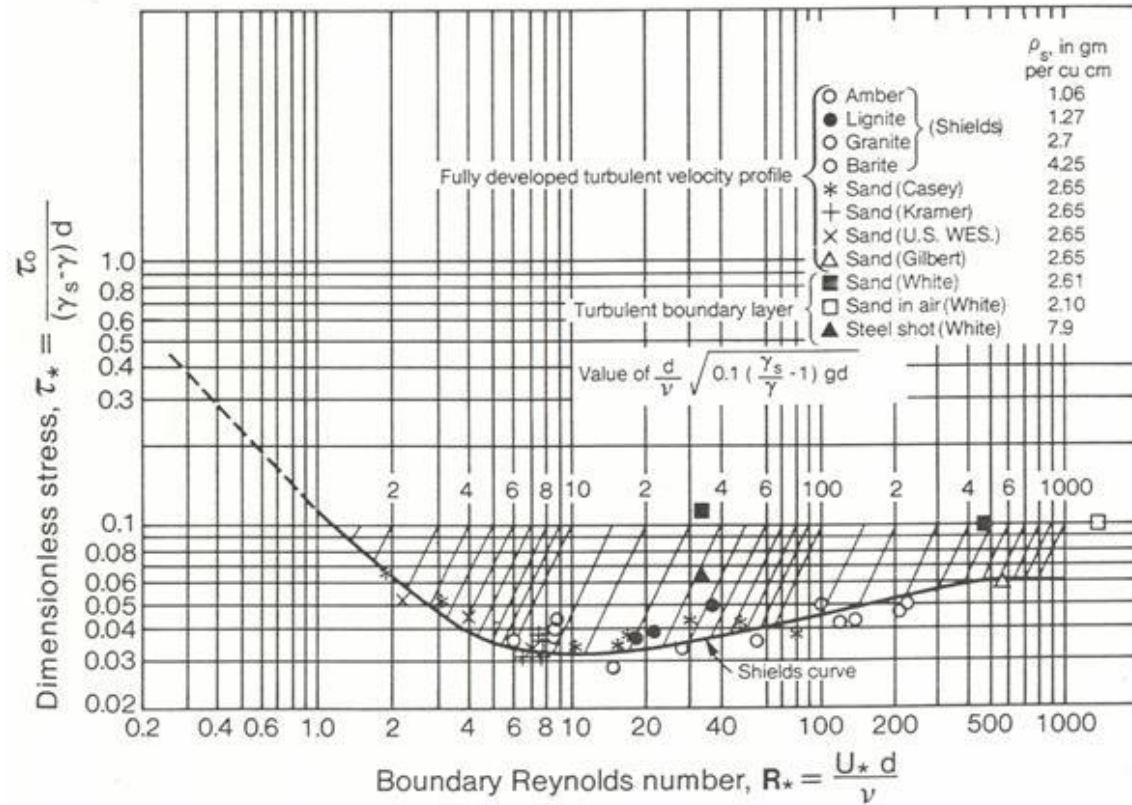


Figure 6.1: Shields curve from Prasuhn (1992)

To ensure that the approximations made using Shields curve are accurate, typical values for the critical shear stress were taken from Rennie (2011). For typical critical shear stress values please refer to Table 6.1.

Table 6.1: Typical critical shear stress values from Rennie (2011)

Soil Type	τ_c^*
Uni-granular Bed	0.06
Mixtures	0.45 (based on D_{50})
Looser Bed	0.03
Isolated Granular	0.01
Structured Bed	0.1
Gravel (7~8mm)	0.06
Sand (0.2~0.7mm)	0.03
Silt and Clay	Cohesion is important

6.2 Typical Values for Density and Specific Weight

Zhu (2010) provided many other typical values for geotechnical characteristics of different soils. An estimate for density and specific weight for sand, gravel, silt, and clay can be seen in Table 6.2.

Table 6.2: Typical density and specific weight values from Zhu (2010)

Soil Type	ρ (kg/m ³)	γ (kN/m ³)
Sand	1800	17.62
Gravel	2000	19.62
Silt	2100	20.6
Clay	1900	18.63

6.3 Typical Values for Porosity

Another geotechnical parameter approximation that Zhu (2010) provided was for the porosity of different soils. Not only did the author provide estimations for gravel, sand, silt, and clay, the author also provided porosity estimations for a sand and gravel mix. To see the approximate porosities for different soil types please refer to Table 6.3.

Table 6.3: Typical porosity values from Zhu (2010)

Soil Type	Porosity (n)
Gravel	30 - 40
Sand	35 - 50
Silt	35 - 50
Clay	33 - 60
Sand and Gravel Mix	20 - 35

6.4 Typical Values for D_{50}

Table 6.4 provided typical values for the D_{50} of a number of different soil types. When approximations were made in this study, the D_{50} was usually provided and the soil type was determined from it. Zhu (2010) has also characterised silt, sand, and gravel into sub categories to classify whether the soil is fine, medium, or coarse.

Table 6.4: Typical D50 values from Zhu (2010)

Soil Type		D ₅₀ (mm)
Clay		< 0.002
Silt	Fine	0.002 - 0.006
	Medium	0.006 - 0.02
	Coarse	0.02 - 0.06
Sand	Fine	0.06 - 0.2
	Medium	0.2 - 0.6
	Coarse	0.6 - 2.0
Gravel	Fine	2.0 - 6.0
	Medium	6.0 - 20.0
	Coarse	20.0 - 60.0
Cobbles		> 60.0

6.5 Typical Values for Cohesive Strength

Zhu (2010) has also provided typical values for the cohesive strength of different types of cohesive sediments. He has put clay into different classifications depending on the hardness and they can be seen in Table 6.5. Sand and gravel have no cohesive properties and they were therefore not included.

Table 6.5: Typical cohesive strength values from Zhu (2010)

Soil Type	Cohesive Strength (kPa)
Rock	10,000 kPa
Silt	75 kPa
Clay	10 – 20 kPa
Very Soft Clay	0 – 48 kPa
Soft Clay	48 – 96 kPa
Medium Clay	96 – 192 kPa
Stiff Clay	192 – 384 kPa
Very Stiff Clay	384 – 766 kPa
Hard Clay	> 766 kPa

6.6 Typical Values for Internal Angle of Friction

Multiple authors have provided typical values for the internal angle of frictions of different sediment types. Peck et al (1974) and Meyerhof (1956) have provided typical values based on the density of sand. Both studies used a qualitative approach instead of a quantitative approach. It is therefore somewhat difficult to differentiate between loose or very loose sand. Peck *et al.*'s values for the angle of internal friction were typically a bit smaller than Meyerhof's

values. Zhu provided the internal angle of friction based on soil type as well as soil density. His values were generally easier to use because he defined the density of each classification earlier. To see a comparison of the values from the different authors please refer to Table 6.6 and Table 6.7.

Table 6.6: Typical internal angle of friction values from Peck et al. (1974) and Meyerhof (1956)

Density of Sand	Φ (degrees) from Peck et al. (1974)	Φ (degrees) from Meyerhof (1956)
Very Loose	< 29	< 30
Loose	29 – 30	30 – 35
Medium	30 – 36	35 – 40
Dense	36 – 41	40 – 45
Very Dense	> 41	> 45

Table 6.7: Typical internal angle of friction values from Zhu (2010)

Soil Type	Φ (degrees)
Rock	30
Sand	30 – 40
Gravel	35
Sandy-Silt	34
Clay	20
Loose Sand	30 – 35
Medium Sand	40
Dense Sand	35 – 45
Gravel with Some Sand	34 – 48
Silt	26 – 35

6.7 Calculating Void Ratio/Porosity

When using BREACH or MIKE11, it was often necessary to convert the porosity to the void ratio or vice versa. The following equations allow for such a conversion:

$$n = \frac{e}{1+e} \quad (6.4)$$

$$e = \frac{n}{1-n} \quad (6.5)$$

where e is the void ratio and n is the porosity.

6.8 Comparing Results

To compare the results from the two different models a method very similar to the method shown in Morris (2000) was used. Multiple dam breach characteristics were chosen to be compared and these were outlined earlier. Some of these characteristics include the peak flow, time to peak flow, breach time, lag time, final breach width, breach width at peak, and water level. For more information on what these characteristics represent please refer to section “2.3 Typical Dam Breach Hydrograph and Characteristics”. For each test, the model results were recorded. A non-dimensional percent error was then calculated for each dam breach characteristic. The tests and models were compared to each other and it was determined whether one model worked more effectively in certain situations. To calculate the non-dimensional percent error the following equation was used:

$$\% \text{ Error} = \frac{\text{Numerical Model Result} - \text{Field Result}}{\text{Field Result}} \times 100\% \quad (6.6)$$

A sensitivity analysis was also completed to determine which parameters within the models are most sensitive. This sensitivity analysis was very similar to the one presented in Al-Riffai *et al.* (2007) and shows how changing a specified parameter will, in turn, change the peak outflow and other breaching characteristics. Due to the fact that many parameters are dependent on other parameters, multiple parameters were changed when a sensitivity analysis was completed on a single geotechnical parameter. The geotechnical parameters were estimated for sand, silt, clay, and gravel and these parameters were used to complete the analysis. The parameters that caused the largest change in the dam breach characteristics could then be classified as the most sensitive parameters. The total variation a parameter caused in a main dam breaching characteristic was recorded and the parameters were ranked from most to least sensitive. Typical parameter values were the same as what was used by Riffai *et al.* (2007) during their sensitivity analysis.

7.0 Results

Below are typical results from each series of tests. Each test shows the discharge hydrograph comparing the simulations by MIKE11 and BREACH to the results recorded in the lab or field. When available, a comparison of the water level in the reservoir, breach depth, and breach width is provided. At the end of the section, a summary of the percent errors for each test will be presented. For all figures showing the outflow hydrographs for different tests as well as tables comparing their characteristics please refer to the Appendix section located at the end of this thesis.

7.1 University of Ottawa Test Comparison

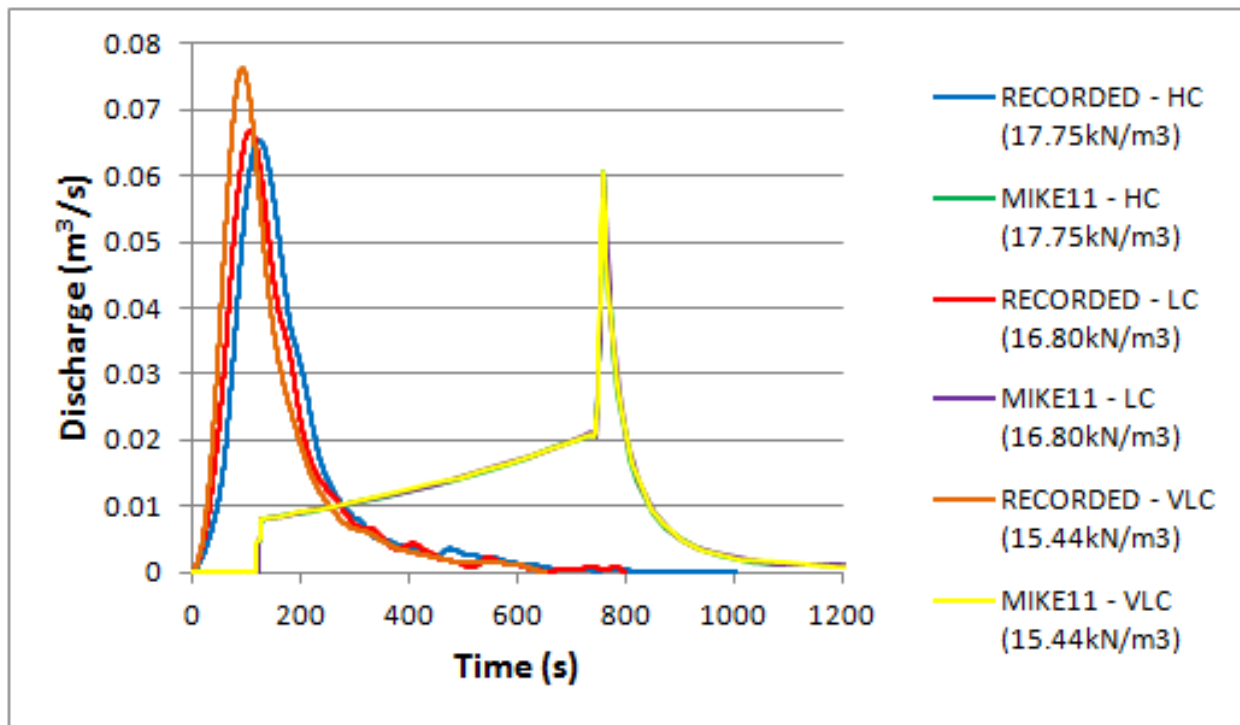


Figure 7.1: University of Ottawa test – discharge – recorded vs. MIKE11 with varying compaction before adjustment

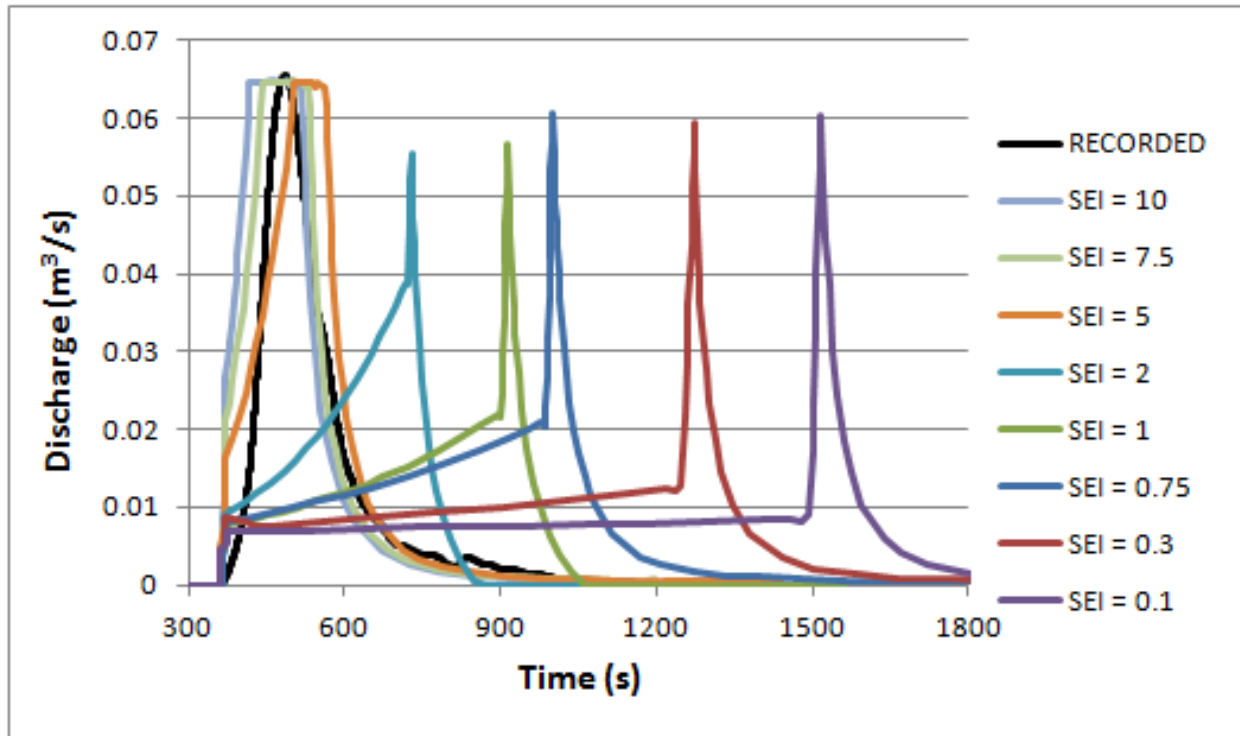


Figure 7.2: University of Ottawa test – discharge – recorded vs. MIKE11 with varying Side Erosion Index

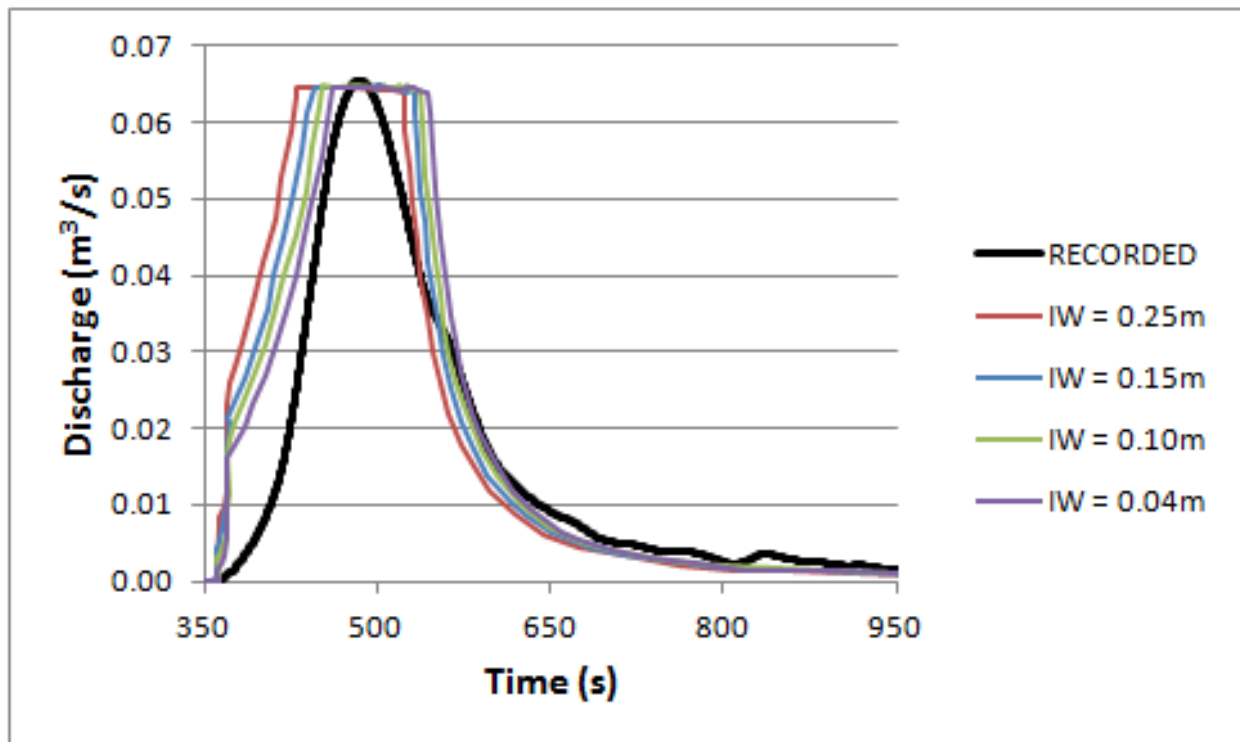


Figure 7.3: University of Ottawa test – discharge – recorded vs. MIKE11 with varying initial breach notch width

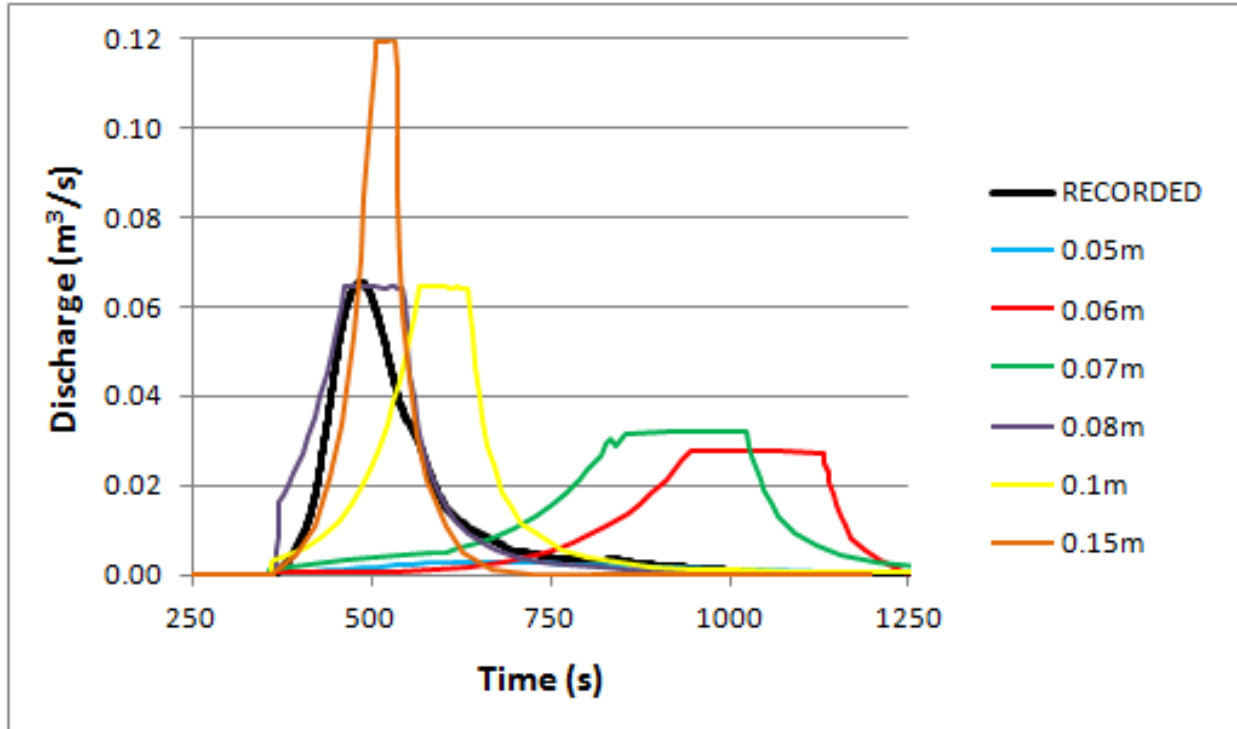


Figure 7.4: University of Ottawa test – discharge – Recorded vs. MIKE11 with varying initial breach notch depth

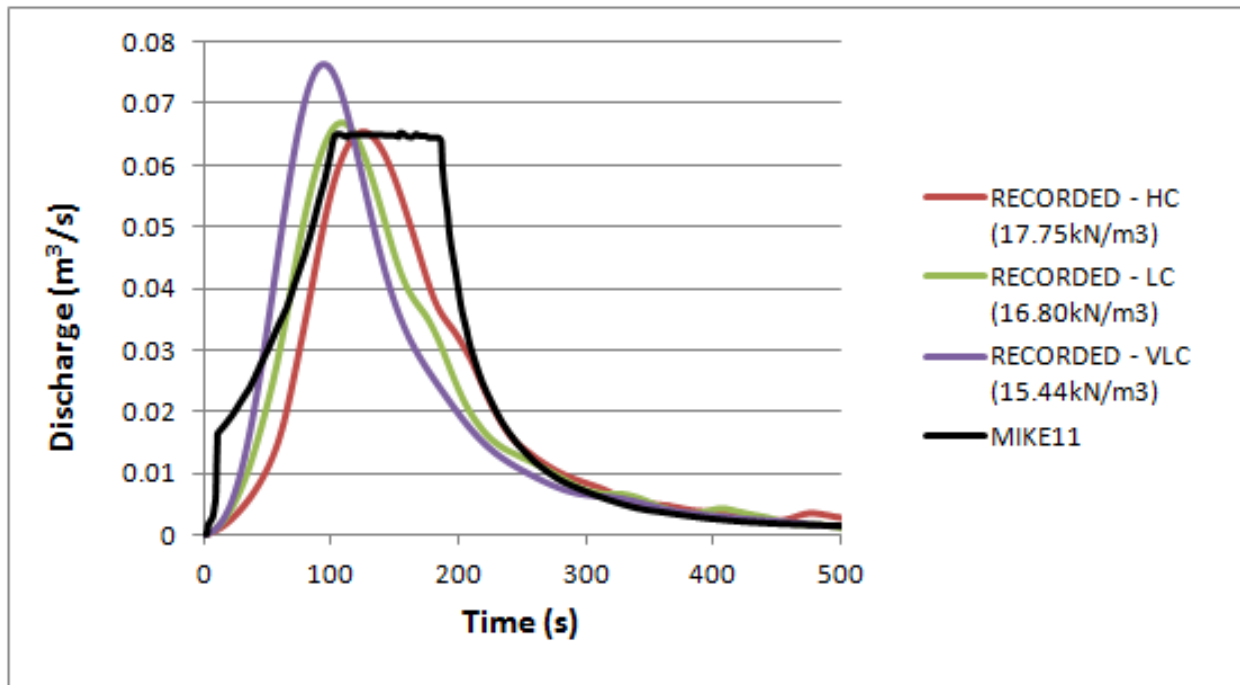


Figure 7.5: University of Ottawa test – discharge – Recorded vs. MIKE11 with varying compaction after adjustment

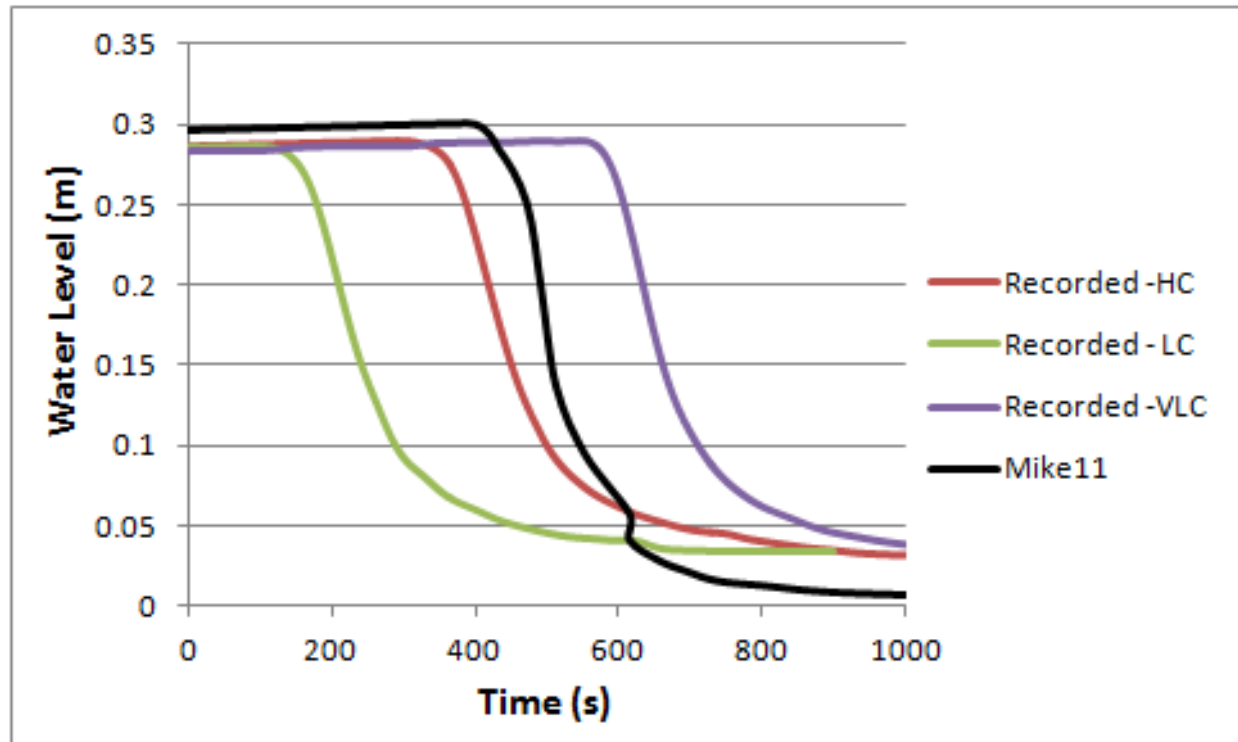


Figure 7.6: University of Ottawa Test – water level – recorded vs. MIKE11 with varying compaction after adjustment

Table 7.1: University of Ottawa comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Surface Elevation of Water at Peak (m)	Final Surface Elevation (m)
High Compaction	0.06546	125	443	1007	28.97	0.0763
Low Compaction	0.06694	108	542	801	28.66	0.0908
Very Low Compaction	0.07622	94	495	654	28.89	0.108
MIKE11	0.0648	194	23760	720	30	0.05
Error - HC	1.01%	55.20%	5263.43%	28.50%	3.56%	34.47%
Error - LC	3.20%	79.63%	4283.76%	10.11%	4.68%	44.93%
Error - VLC	14.98%	106.38%	4700.00%	10.09%	3.84%	53.70%

7.2 IMPACT Field Test Comparison

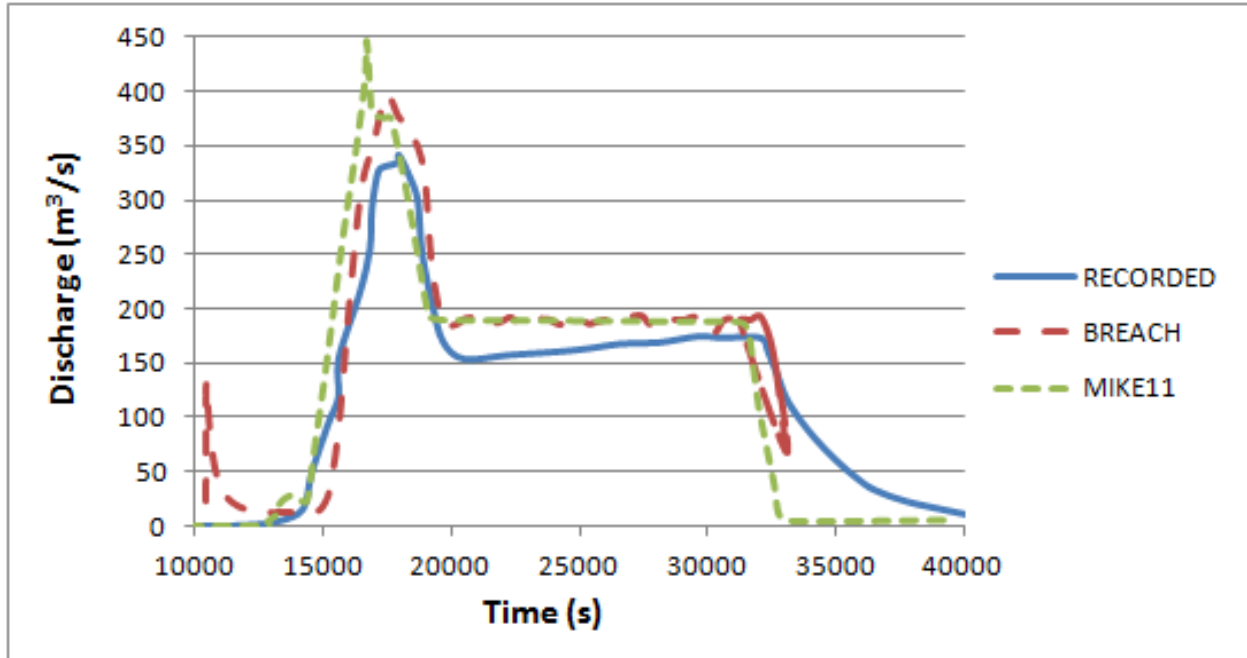


Figure 7.7: IMPACT Field Test #1- discharge - recorded vs. BREACH vs. MIKE11

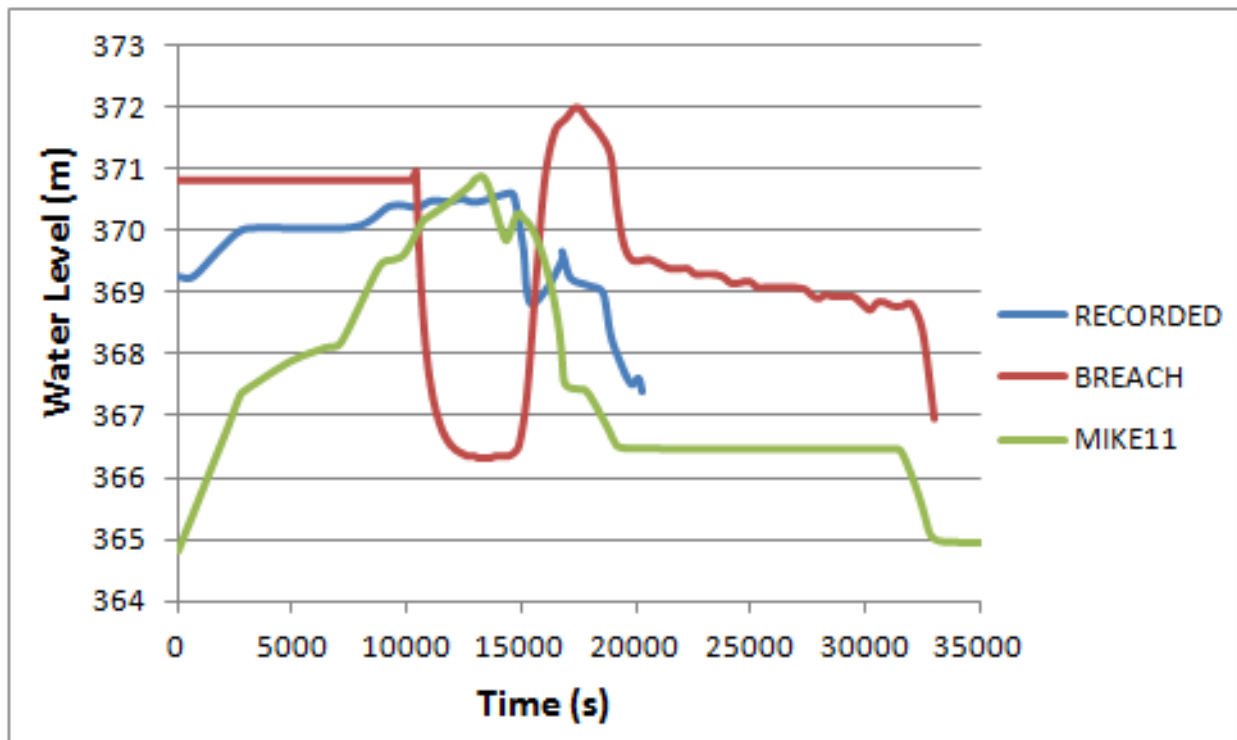


Figure 7.8: IMPACT Field Test #1 - water level - recorded vs. BREACH vs. MIKE11

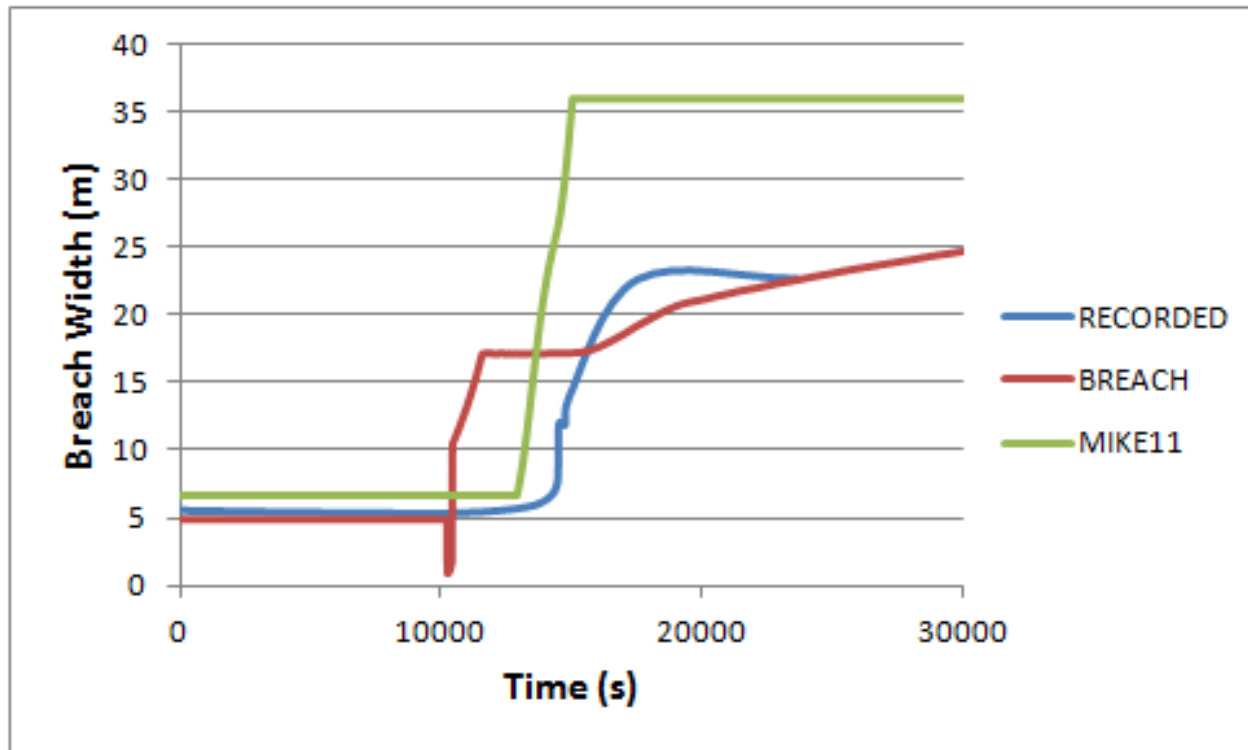


Figure 7.9: IMPACT Field Test #1 - breach width - recorded vs. BREACH vs. MIKE11

Table 7.2: IMPACT Field Test #1 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)		Breach Time (s)		Breach depth at Peak (m)	Final Water depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	340.40	17979.66	12040.67		28881.252		NA	NA	22.7	22.7
BREACH	396.29	17427.60	10206.00	14893.20	22831.2	18144	6	6	19.23	25.45
Error	16.42%	3.07%	15.24%	23.69%	20.95%	37.18%	NA	NA	15.29%	12.11%
MIKE11	445.06	16698.78	12881.92		20396.412		3.11	6	36	36
Error	30.74%	7.12%	6.99%		29.38%		NA	NA	58.59%	58.59%

7.3 IMPACT Laboratory Tests Comparison

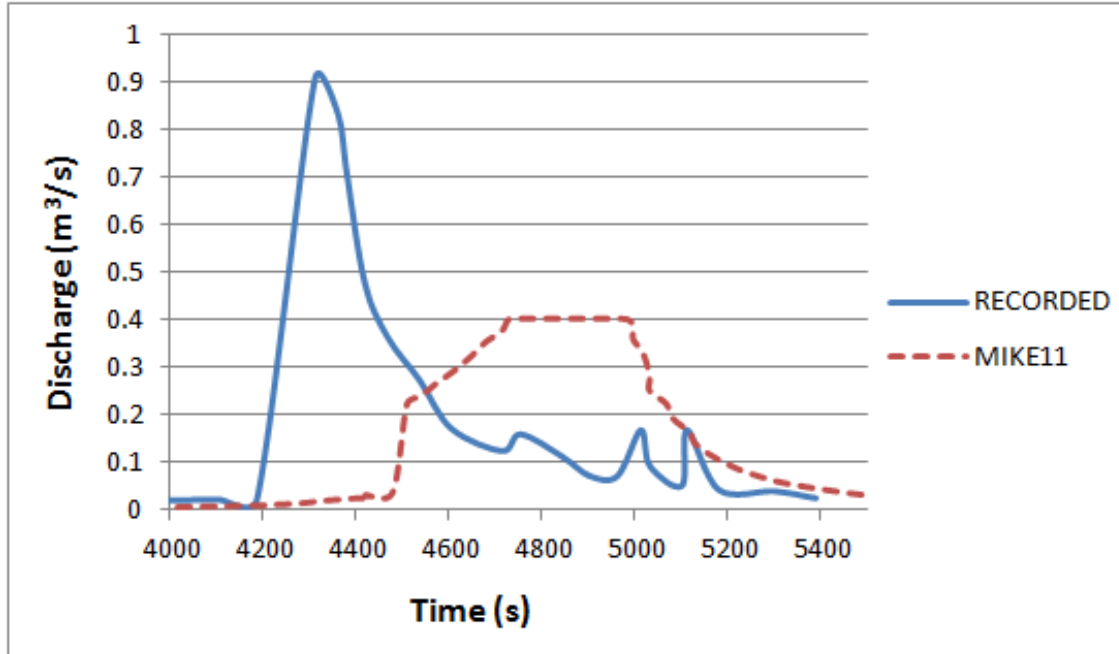


Figure 7.10: IMPACT Lab Test #2- discharge - recorded vs. MIKE11

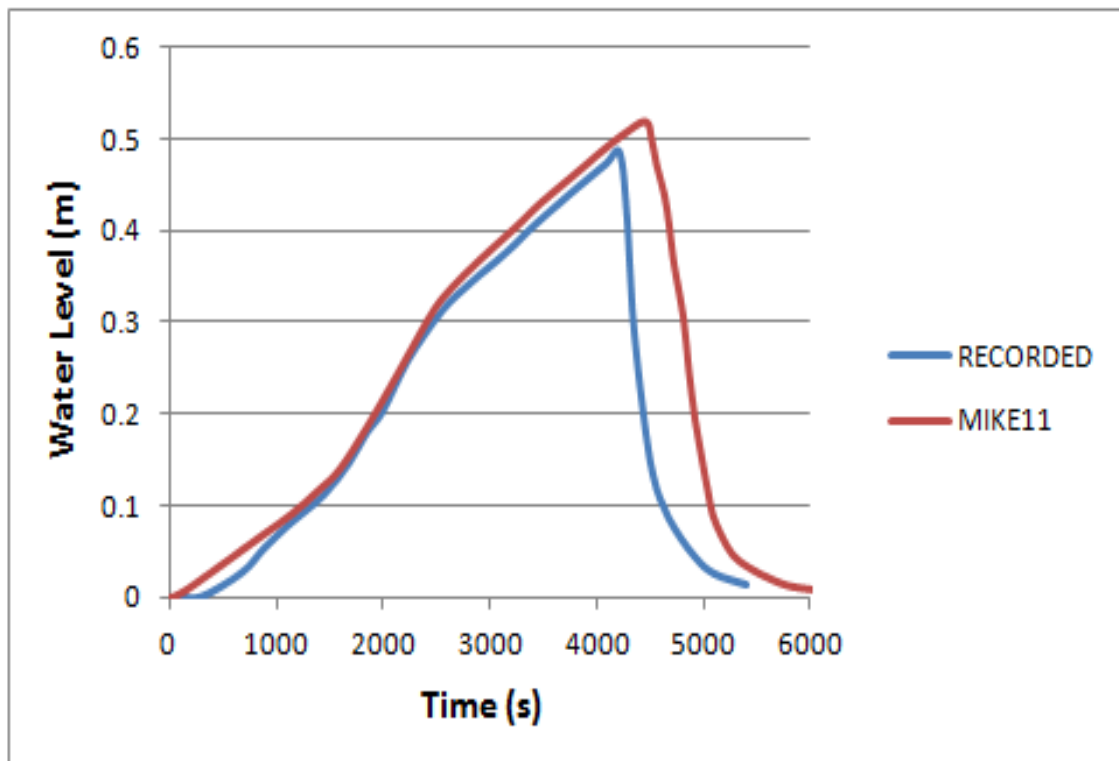


Figure 7.11: IMPACT Lab Test #2- water level - recorded vs. MIKE11

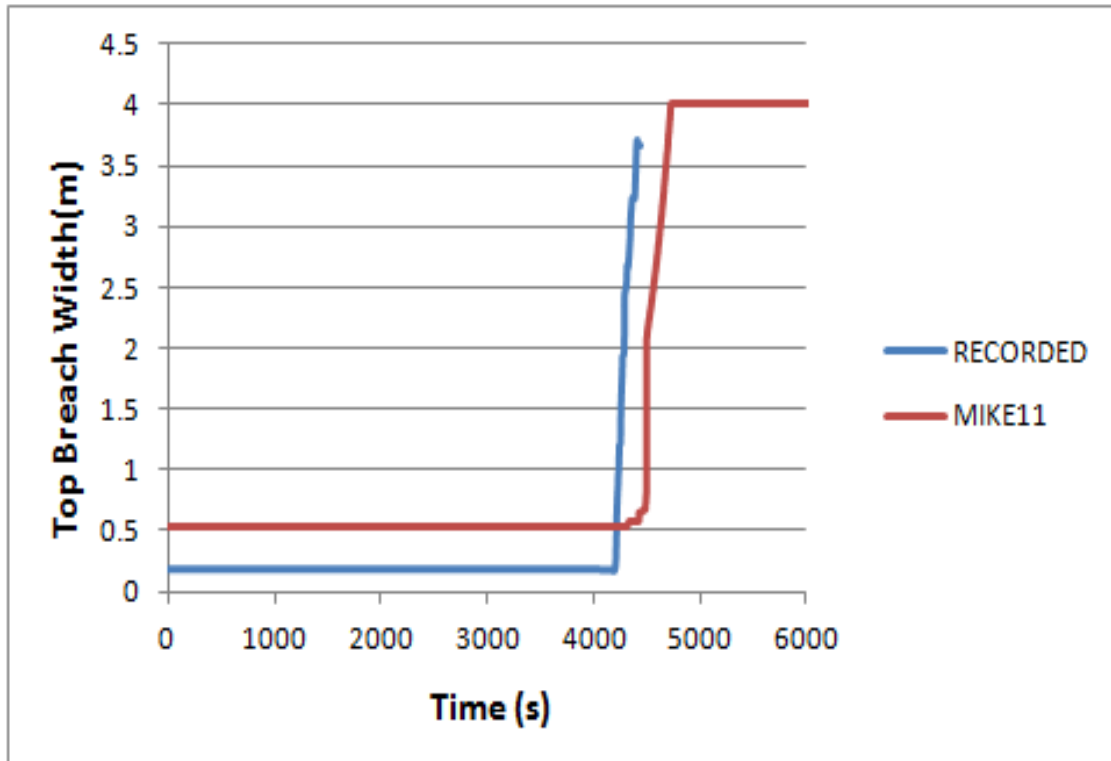


Figure 7.12: IMPACT Lab Test #2- top breach width - recorded vs. MIKE11

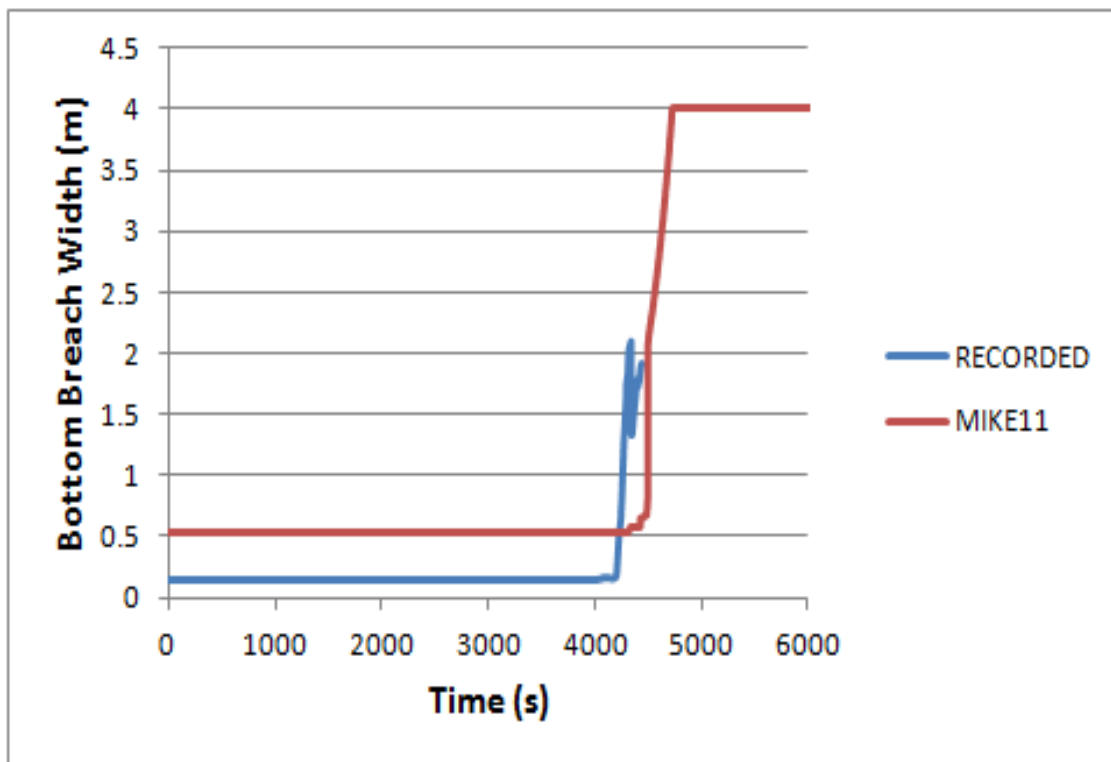


Figure 7.13: IMPACT Lab Test #2- Bottom breach width - recorded vs. MIKE11

Table 7.3: IMPACT Lab Test #2 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.91	4320.00	4185.01	1202.80	0.35	0.49	NA	0.50	2.75	2.75
MIKE11	0.40	4756.82	4267.40	1536.48	0.37	0.52	0.30	0.50	4.00	4.00
Error	56.04%	10.11%	1.97%	27.74%	5.71%	6.12%	NA	0.00%	45.45%	45.45%

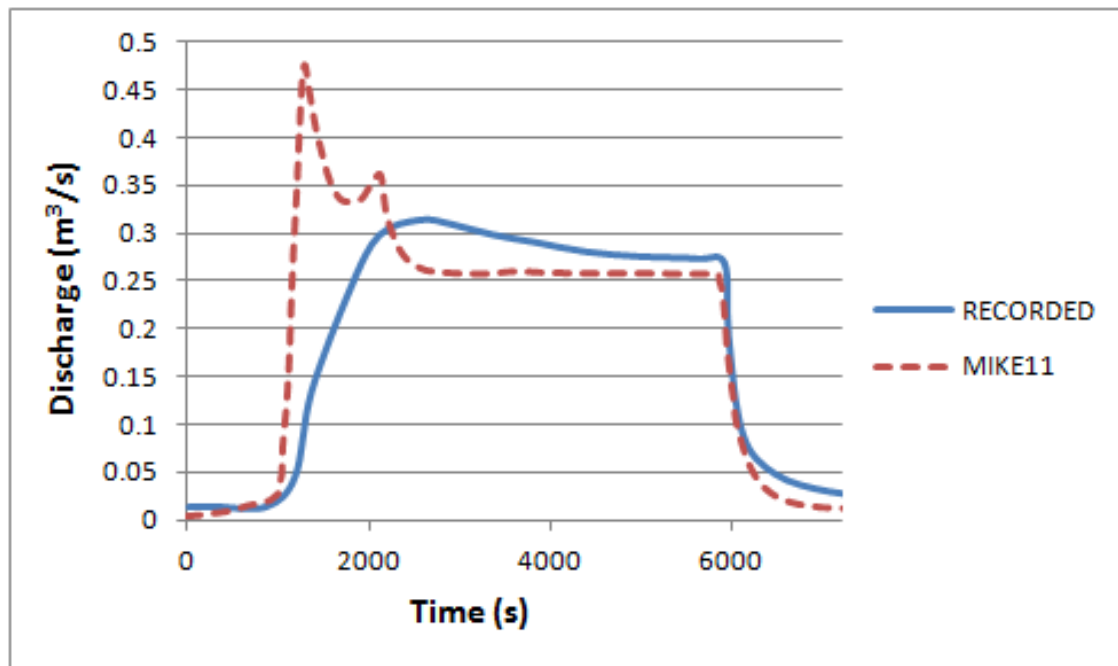


Figure 7.14: IMPACT Lab Test #10- discharge - recorded vs. MIKE11

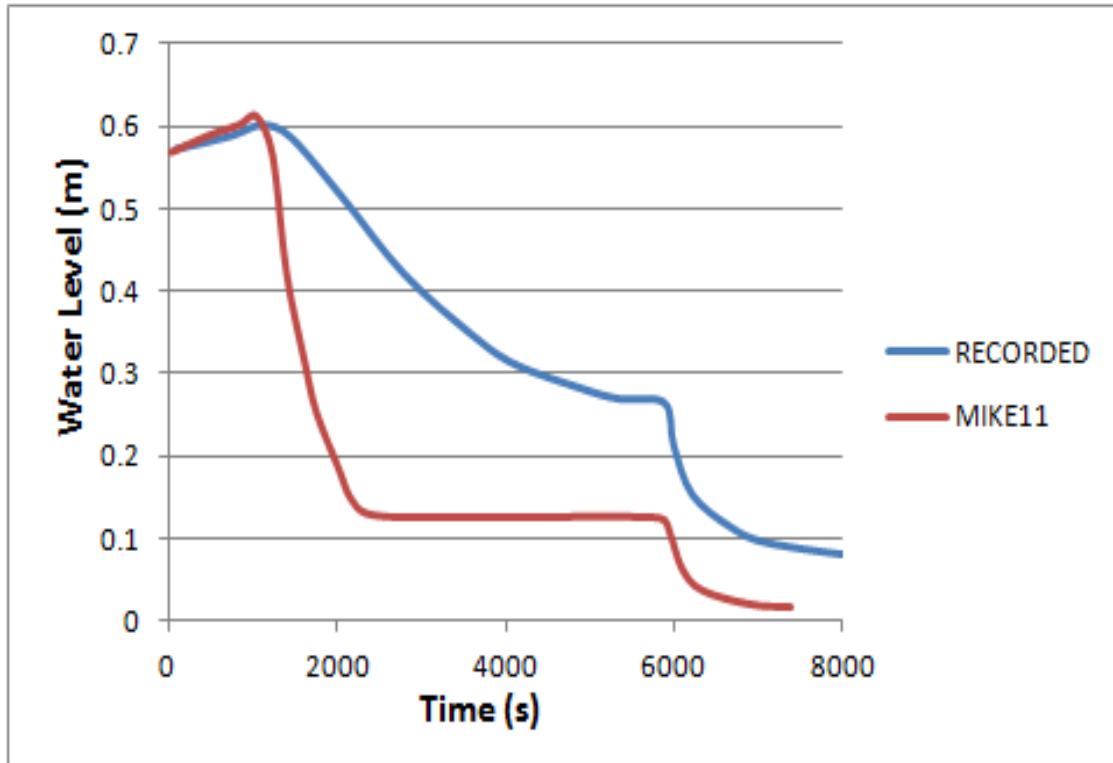


Figure 7.15: IMPACT Lab Test #10- Water level - recorded vs. MIKE11

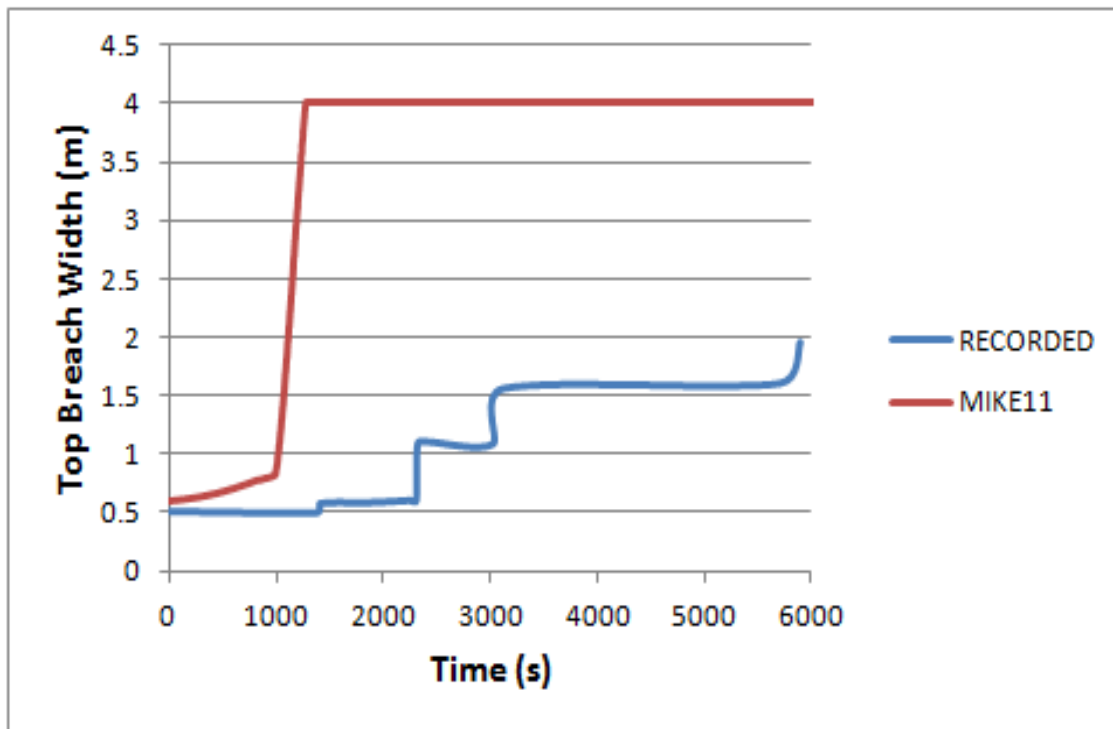


Figure 7.16: IMPACT Lab Test #10- top breach width - recorded vs. MIKE11

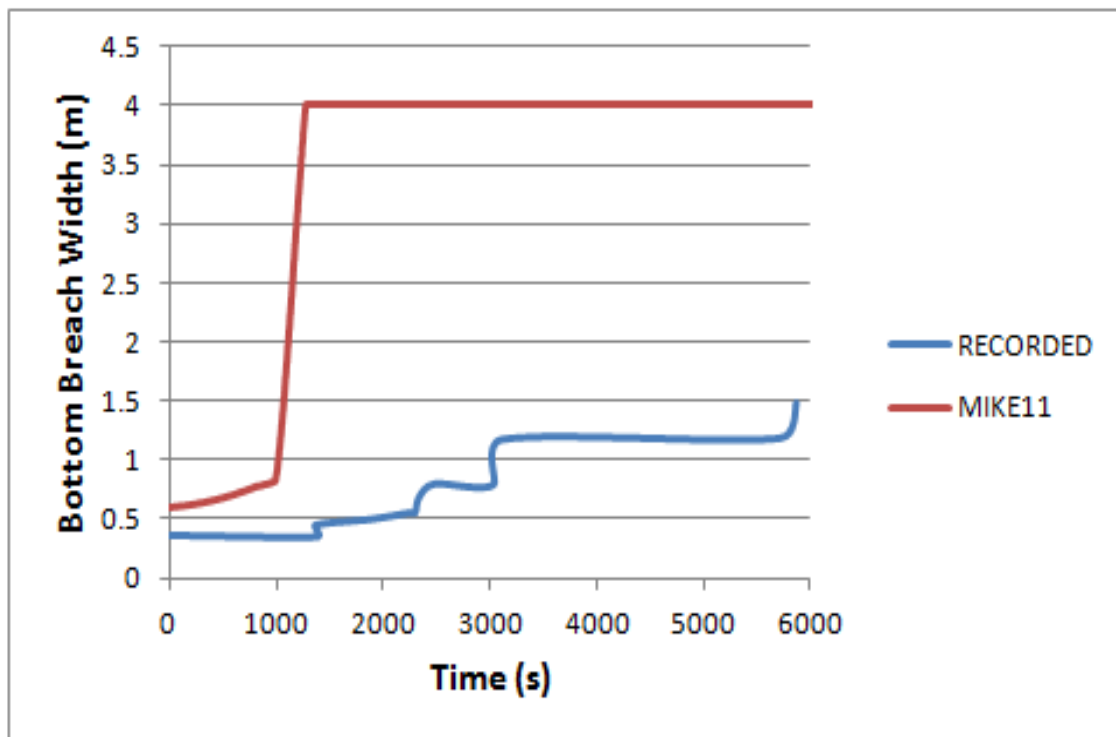


Figure 7.17: IMPACT Lab Test #10- Bottom breach width - recorded vs. MIKE11

Table 7.4: IMPACT Lab Test #10 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak	Peak Water Level	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.31	2412.00	1191.72	4922.01	0.46	NA	NA	0.55	1.07	1.85
MIKE11	0.48	1299.61	1057.01	5033.034	0.5	0.61	0.293	0.6	4	4
Error	53.48%	46.12%	11.30%	2.26%	8.70%	NA	NA	9.09%	273.83%	116.22%

7.4 Delft University of Technology Test Comparison

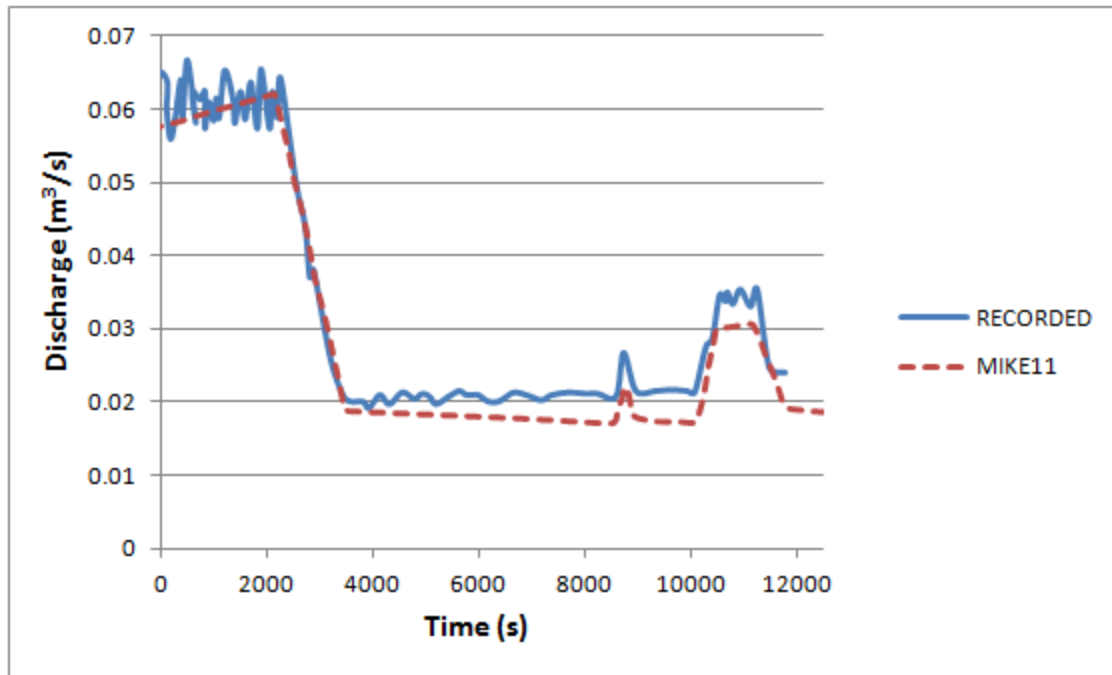


Figure 7.18: Delft University of Technology Lab Test #1 discharge results

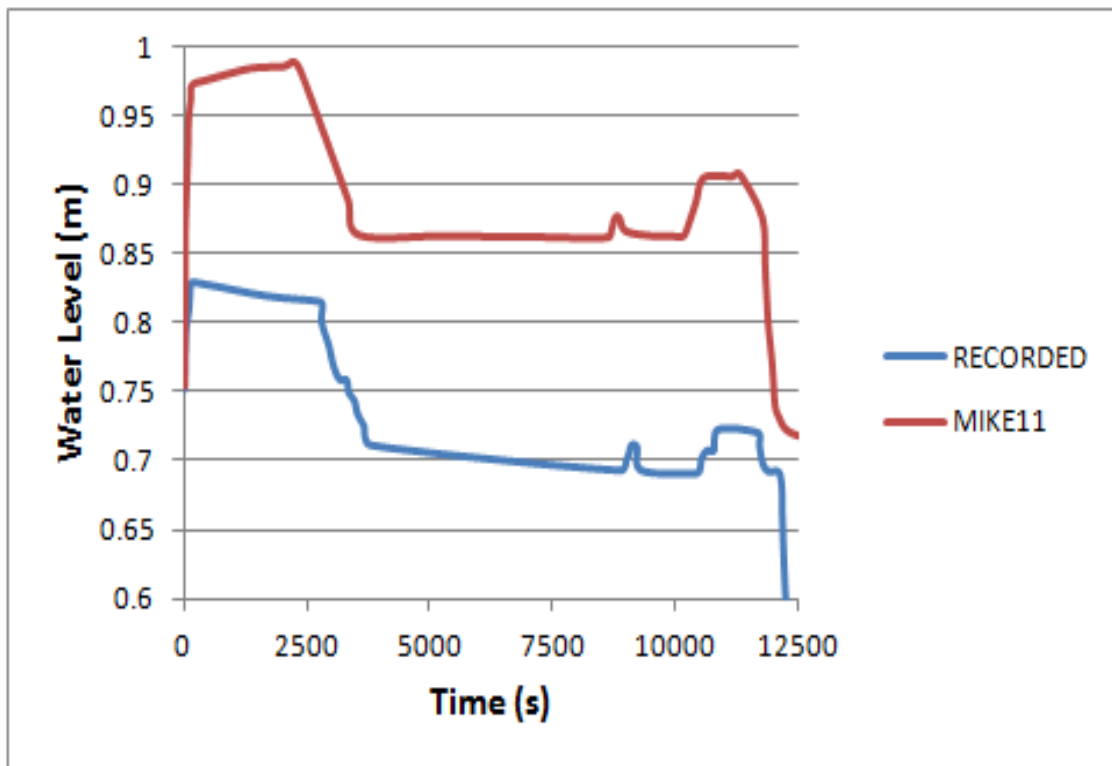


Figure 7.19: Delft University of Technology Lab Test #1 water level results

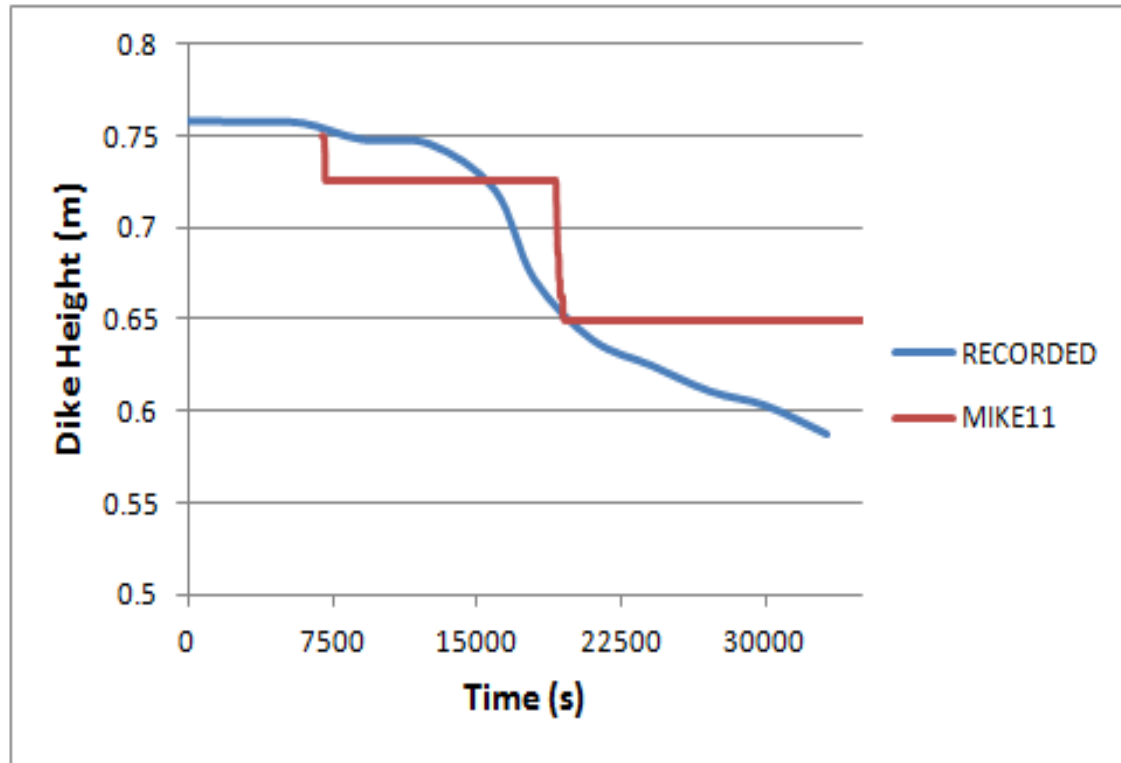


Figure 7.20: Delft University of Technology Lab Test #1 dike height results

Table 7.5: Delft University of Technology Lab Test #1 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)
RECORDED	0.067	519.9	0.840	0.845	0.100	0.275
MIKE11	0.062	2095.2	0.907	0.910	0.124	0.200
Error	7.20%	303.00%	7.98%	7.69%	24.00%	27.27%

Table 7.6: Comparison of IMPACT field test's percent errors

Tests	Peak Outflow Percent Error		Time to Peak Percent Error		Lag Time Percent Error		Breach Time Percent Error		Breach Depth at Peak Percent Error		Final Water depth Percent Error		Breach Width at Peak Percent Error		Final Breach Width Percent Error	
	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH	MIKE11	BREACH
Impact Field Test 1	30.74%	16.42%	7.12%	3.07%	6.99%	23.69%	29.38%	37.18%	NA	NA	NA	NA	58.59%	15.29%	58.59%	12.11%
Impact Field Test 2	25.04%	18.68%	71.43%	41.63%	72.66%	40.66%	71.77%	78.91%	0.44%	98.80%	51.06%	51.06%	NA	NA	140.00%	11.00%
Impact Field Test 3	12.44%	37.63%	1.76%	6.08%	0.11%	8.00%	34.23%	74.70%	37.41%	178.11%	3.27%	3.27%	NA	NA	NA	NA
Impact Field Test 4	5.49%	26.75%	8.59%	97.95%	9.58%	99.55%	47.16%	74.87%	52.81%	5.98%	39.90%	3.63%	NA	NA	NA	NA
Impact Field Test 5	3.68%	35.74%	1.40%	98.60%	2.07%	99.93%	22.62%	61.48%	141.61%	0.00%	62.91%	4.76%	NA	NA	NA	NA

Table 7.7: Comparison of IMPACT laboratory test's percent errors

Tests	Peak Outflow Percent Error	Time to Peak Percent Error	Lag Time Percent Error	Breach Time Percent Error	Water Level at Peak Percent Error	Peak Water Level Percent Error	Breach Depth at Peak Percent Error	Final Breach Depth Percent Error	Breach Width at Peak Percent Error	Final Breach Width Percent Error
Non-Cohesive Tests										
IMPACT Lab Test 2	56.04%	10.11%	1.97%	27.74%	5.71%	6.12%	NA	0.00%	45.45%	45.45%
IMPACT Lab Test 4	69.51%	45.62%	0.44%	33.85%	2.94%	8.33%	NA	0.00%	104.16%	66.67%
IMPACT Lab Test 5	76.52%	35.05%	1.09%	71.14%	11.43%	10.42%	NA	0.00%	56.25%	36.52%
IMPACT Lab Test 6	77.80%	36.02%	6.19%	69.97%	6.67%	7.14%	NA	0.00%	35.59%	29.03%
IMPACT Lab Test 7	76.72%	36.70%	1.69%	261.86%	0.00%	8.16%	NA	0.00%	56.86%	37.93%
Cohesive Tests										
IMPACT Lab Test 10	53.48%	46.12%	11.30%	2.26%	8.70%	NA	NA	9.09%	273.83%	116.22%
IMPACT Lab Test 12	30.24%	0.53%	49.45%	26.50%	10.64%	6.90%	NA	N/A	101.01%	56.86%
IMPACT Lab Test 13	278.97%	68.71%	8.64%	14.49%	1.67%	24.59%	NA	N/A	85.35%	273.83%
IMPACT Lab Test 15	122.91%	40.62%	16.89%	12.24%	0.00%	0.00%	NA	N/A	172.11%	131.21%
IMPACT Lab Test 16	37.80%	7.32%	138.16%	3.87%	37.50%	7.46%	NA	0.00%	135.44%	73.16%
IMPACT Lab Test 17	57.51%	127.61%	42.87%	63.88%	55.87%	21.53%	NA	N/A	47.96%	22.32%

Table 7.8: Comparison of TU Delft laboratory test's percent errors

Tests	Peak Flow Percent Error	Time to Peak Percent Error	Water Level at Peak Percent Error	Peak Water Level Percent Error	Breach depth at Peak Percent Error	Final Breach Depth Percent Error
TU Delft Test 1	7.20%	303.00%	7.98%	7.69%	24.00%	27.27%
TU Delft Test 2	6.08%	23.88%	22.34%	14.60%	0.97%	48.83%
TU Delft Test 3	6.98%	0.32%	9.84%	1.80%	28.06%	46.59%

7.5 Sensitivity Analysis

A main objective of this section was to run a sensitivity analysis on different parameters used in the two models to see which influenced the discharge hydrograph and breaching characteristics the most. Please refer to the “Methodology” section for information on how a parameter’s sensitivity rank is determined. An example of the sensitivity analysis completed on the non-dimensional critical shear stress for MIKE11 can be seen in Figure 7.21 to Figure 7.23. All other results for the sensitivity analysis can be found in the appendix on Figure 12.52 to Figure 12.92. The parameters were ranked, as outlined in the methodology section, for both MIKE11 and BREACH and summarized in Table 7.9 and Table 7.10.

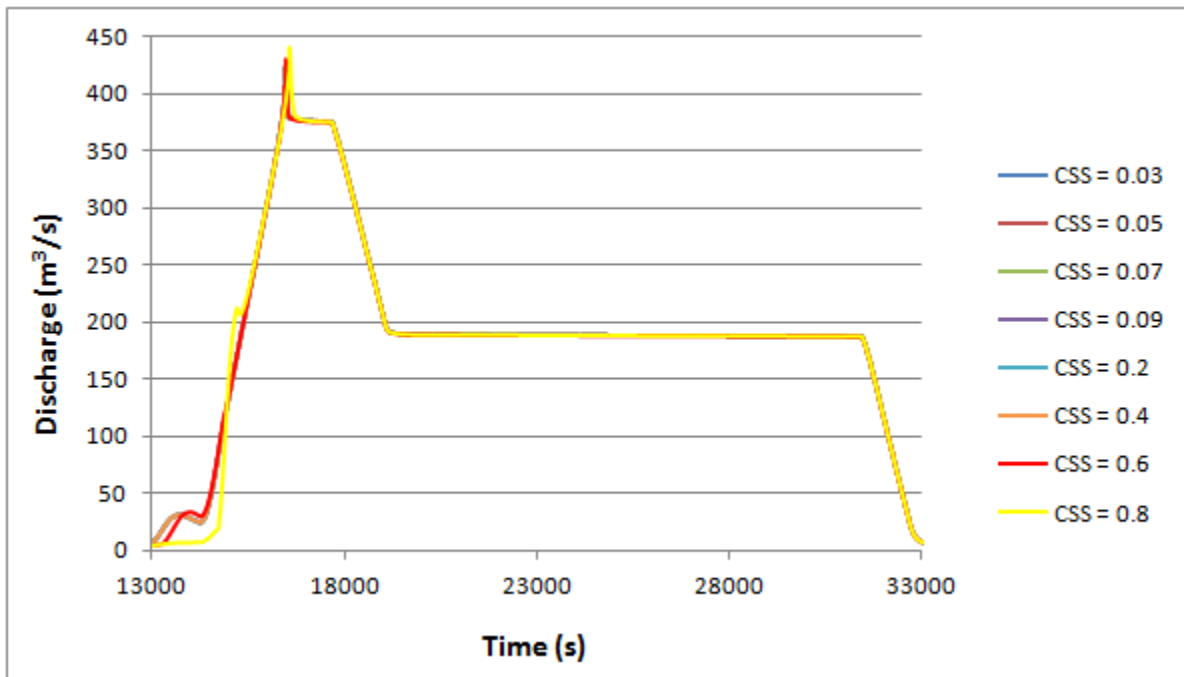


Figure 7.21: MIKE11 sensitivity analysis – non-dimensional critical shear stress

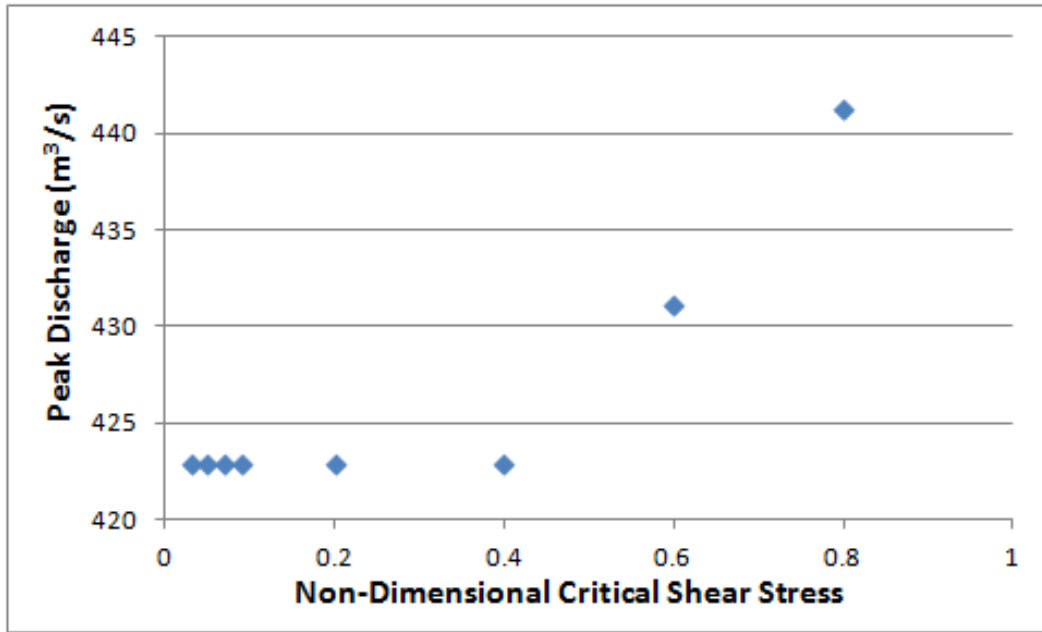


Figure 7.22: MIKE11 sensitivity analysis - non-dimensional shear stress - peak discharge

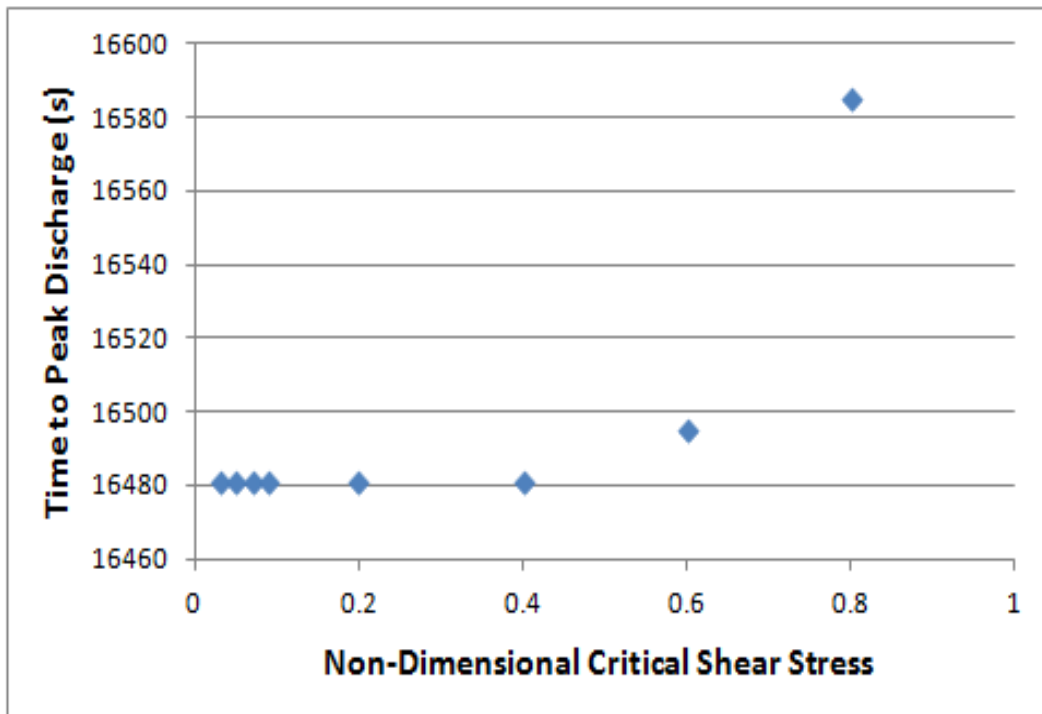


Figure 7.23: MIKE11 sensitivity analysis - non-dimensional shear stress - time to peak discharge

Table 7.9: Parameter sensitivity by rank - MIKE11

Parameter	Peak Discharge Variation (m ³ /s)	Time to Peak Discharge Variation (s)	Rank Average
Non-Dimensional Critical Shear Stress	422.9 - 441.2 = 18.3	16585.2 - 16480.8 = 104.4	7
Sensitivity Rank	7	7	
Crest Width	449.2 - 423.6 = 25.6	17067.6 - 16567.2 = 500.4	6
Sensitivity Rank	6	6	
D50	494.3 - 81.3 = 413.0	18795.6 - 16426.8 = 2368.8	1
Sensitivity Rank	1	1	
Porosity	448.2 - 419.1 = 29.1	17366.4 - 16480.8 = 885.6	5
Sensitivity Rank	5	5	
Side Slopes	447.7 - 325.9 = 121.8	17719.2 - 16430.4 = 1288.8	3
Sensitivity Rank	2	4	
Upstream Side Slopes	414.5 - 381.5 = 33.0	17798.4 - 16430.4 = 1368.0	2.5
Sensitivity Rank	3	2	
Downstream Side Slopes	414.5 - 381.5 = 33.0	17798.4 - 16430.4 = 1368.0	2.5
Sensitivity Rank	3	2	

Table 7.10: Parameter sensitivity by rank - BREACH

Parameter	Peak Discharge Variation (m ³ /s)	Time to Peak Discharge Variation (s)	Average Rank
Internal Angle of Friction	216.65 - 137.00 = 79.66	10425.6 - 10414.8 = 10.8	4.5
Sensitivity Rank	3	6	
Unit Weight	194.34 - 150.84 = 43.49	10522.8 - 10360.8 = 162.0	5
Sensitivity Rank	5	5	
Gradation	390.63 - 133.88 = 254.94	17337.6 - 10414.8 = 6922.8	2
Sensitivity Rank	2	2	
Upstream Side Slopes	464.06 - 391.57 = 72.49	18154.8 - 17427.6 = 727.2	4
Sensitivity Rank	4	4	
Downstream Side Slopes	478.36 - 131.98 = 346.37	17571.6 - 10414.8 = 7156.8	1
Sensitivity Rank	1	1	
Sediment Type	179.10 - 164.66 = 14.44	11379.6 - 10418.4 = 961.2	4.5
Sensitivity Rank	6	3	

8.0 Discussion

As mentioned in the Methodology section, the models were compared using a non-dimensional percent error and the comparison was similar to the same procedure used by Morris (2005) who evaluated different dam breach modelling software. Please refer to Table 7.1 to Table 7.10 and Table 12.1 to Table 12.15 for a comparative summary of each test. The comparative summary includes many dam breach characteristics, such as, peak flow, time to peak flow, lag time, breach time, water level at peak, final water level and the associated percent error. Each experiment will be discussed in further detail throughout this section.

8.1 University of Ottawa Laboratory Tests

The first series of experiments that will be discussed was completed in the Hydraulics Laboratory at the University of Ottawa. This series included three different compaction efforts: high compaction, low compaction, and very low compaction. The high compaction test had a specific weight of 17.75kN/m^3 , the low compaction test had a specific weight of 16.80kN/m^3 , and the very low compaction test had a specific weight of 15.44kN/m^3 . When looking at the results of the University of Ottawa experiments it can be seen that as the compaction increased by 8.8% and 15%, the peak discharge decreased by 12.2% and 14.1%, respectively. At the same time, the time to peak discharge increased by 15.2% and 33.1%, respectively. This is generally what is expected.

While reproducing the University of Ottawa physical experiments with the two numerical models, some difficulties were encountered. Firstly, due to the small size of the laboratory experiments, BREACH would not reproduce the dam breach. This has been observed and documented by researchers in the field of dam breaching as being a problem often encountered when using BREACH. Therefore, the laboratory experiments could not be used for the validation of BREACH. When MIKE11 was used to breach the high compaction dam studied at the University of Ottawa there was a noticeable lag in breach time. This can be observed in Figure 7.1. When the breach initiated, at roughly 150 seconds into the experiment, the discharge increased almost instantly to $0.009\text{ m}^3/\text{s}$. It then took over 600 seconds to reach a discharge of $0.02\text{ m}^3/\text{s}$. The simulation then took another 40-50 seconds to reach a peak discharge of $0.06\text{ m}^3/\text{s}$. The simulation by MIKE11 was similar to the experimental results, except for the 600 second delay it took MIKE11 to reach a discharge of $0.02\text{ m}^3/\text{s}$. This was likely due to the Side Erosion Index used in the simulation. The Side Erosion Index is very important when calculating the breach width and the amount of erosion during a simulation. It is a multiplication factor that is used to calculate the breach width erosion rates from breach depth predictions. In physical experiments which are completed in the field or laboratory, the researcher has no means to record or estimate this parameter. It is therefore left up to the modeller to estimate a meaningful, physically-based value for Side Erosion Index. Literature on the topic suggests that a value between 0.5 and 1.0 should be used. When higher values are selected for the Side Erosion Index, the breach width erosion rates are greater and the peak discharge generally arrives

in a shorter amount of time when compared to lower Side Erosion Index values. In Figure 7.1, based on values found in literature, the Side Erosion Index was initially approximated to be 0.75. However, after completing a sensitivity analysis on the Side Erosion Index, it was observed that a value closer to 7.5 generates a much more accurate result. To see a graphical representation of the sensitivity analysis completed on the Side Erosion Index the reader is directed to Figure 7.2. It is quite evident from this figure that a Side Erosion Index between 0.5 and 1.0 will not breach the dam quickly enough. However, when the Side Erosion Index is as large as 10.0, the dam breach occurs too quickly. Therefore, a Side Erosion Index of 7.5 was used for the remainder of the MIKE11 simulations and calibration was conducted periodically to ensure an accurate estimate is still being used. Another main issue when using MIKE11 is that the initial breach notch used in the recorded laboratory experiments will not initiate a breach when using the MIKE11 model. During the high compaction laboratory experiment, completed at the University of Ottawa, a V-Notch of 4cm wide and 2cm deep was used. When different breach notch widths were used in the simulation not much of a change occurred in terms of the dam breach characteristics. However, when the initial breach depth was varied, there was a large change in the outflow hydrograph produced by MIKE11. It should also be noted that MIKE11 only allows for a rectangular breach notch to initiate breaching rather than the triangular one used in the experiment. The first estimate used for the breach notch depth was 1cm, which was an average depth of the recorded breach notch. After referring to Figure 7.4, it can be observed that a breach was not initiated until the breach depth was at least 6.0cm deep, 5.0cm deeper than what was recorded in the laboratory experiment. It was not until the initial breach notch depth was increased to 8cm that the outflow hydrograph was similar to the one produced in the laboratory.

Another important point to note is that the outflow hydrograph produced by MIKE11 did not change when there was a change in porosity and compaction. It can be observed from Figure 7.5 that the hydrograph produced by MIKE11 does not change when different compaction levels were used and it is similar to the high compaction test completed in the laboratory. Figure 7.6 shows a comparison of the water levels between the MIKE11 simulation and the laboratory results. The results from MIKE11 are once again closest to the high compaction test results. This is expected, however, it must be noted that the water level and outflow hydrograph are somewhat related: the water level decreasing at a faster rate is a result of the water leaving the reservoir at a faster rate and therefore the discharge through the dam breach will be greater.

The results of the University of Ottawa tests are summarized in Table 7.1. As mentioned earlier, the lag times for this test were not accurate and, therefore, should not be used to measure the accuracy of a model. The peak flow estimates were fairly accurate for all of the compaction types but it was especially accurate when compared to the high compaction laboratory test. When comparing the peak flow results of the high compaction laboratory test to the peak flow generated by MIKE11, there was an error of approximately 1.0%. This error was increased to approximately 15% for the very low compaction test. For the amount of time it took until the peak flow was reached, the high compaction laboratory test once again had the best results when compared to the MIKE11 simulation, with a relative error of about 55%. The error increased to

106% for the very low compaction laboratory test. This error should be much lower since a value slightly below the peak flow, occurred almost one minute before the recorded results. After referring to Figure 7.5, it can be observed that the peak flow produced by MIKE11 occurs for over 100 seconds. If the time to peak was approximated using the graph, there would be much less error since a value near the 110 second mark would be used. The total breach time estimate from MIKE11 also had a lower than average error when compared to other laboratory tests. There was approximately a 10% error for both, the low compaction and very low compaction laboratory tests when compared to the MIKE11 simulation. The high compaction laboratory test reproduced by MIKE11 had an error of 28.5% for the breach time when compared to the recorded laboratory results. When the breach time ends, is often dependent on the researcher. This is because some researchers will end the breach time when the inflow into the reservoir equals the discharge through the breach, while others will observe the outflow hydrograph and make approximations from it. Therefore, the amount of error produced by MIKE11 will vary. MIKE11's estimation for the water level in the reservoir at the time of peak discharge was also very good. There was an error between 3.5% and 4.7% for each of the high compaction, low compaction, and very low compaction tests. The estimated final water level in the reservoir had an about average accuracy when compared to other laboratory simulations. There was an average relative error of 43% but the difference in water level was only a couple centimeters.

8.2 IMPACT-Project Field Tests

The next series of tests that were studied, were the Field Tests completed as part of the IMPACT-Project. Since the field tests were larger in size, both BREACH and MIKE11 were able to simulate the outflow hydrograph and breaching characteristics. Field test #1 was labelled, the “maximum cohesive” test and was built out of lean clay. After referring to Figure 7.7 the reader can observe that both BREACH and MIKE11 performed accurately. Both models produced somewhat accurate lag time and time to breach estimations when compared to the recorded field results. BREACH, however, shows a discharge peak just after the 10,000 second mark that was not recorded during the experiment or shown in the MIKE11 simulation. BREACH and MIKE11 were both conservative when it came to estimating the peak flow. BREACH produced a peak flow of $396.29 \text{ m}^3/\text{s}$ and MIKE11 produced a peak flow of $445.06 \text{ m}^3/\text{s}$ while the recorded flow during the experiment peaked at $340.40 \text{ m}^3/\text{s}$. This resulted in an error of 16.4% and 30.74% for BREACH and MIKE11, respectively. As mentioned earlier, it can be noted that both models produce a somewhat accurate time to peak discharge: BREACH with only 3.07% error and MIKE11 with 7.12%. The MIKE11 simulation for field test #1 has an error of only 7% when referring to lag time compared to the BREACH simulation which is over double the error, at 15.24% and 23.69% for the first and second peak discharges, respectively. When the total breach time is evaluated, there is a bit more error present when comparing the model results to the recorded results. This is due to both models decreasing the

discharge at a faster rate than what was recorded in the field. Referring to Figure 7.7, it can be observed that after the inflow stops, around the 34000 second mark, discharge estimations from both models decrease rapidly as the discharge from the field test decreases at a much more gradual rate. This results in a 37% error in breach time for BREACH and a 29% error in breach time for MIKE11 when compared to the recorded field results. The water levels in the reservoir and breach width during the experiment can be observed in Figure 7.8 and Figure 7.9, respectively. When referring to the water level, it can be observed that MIKE11 is more accurate than BREACH when comparing them to the recorded results. The first small peak that is shown in the discharge hydrograph for BREACH is also very apparent when looking at the water level. It is impossible to make a full comparison though since the recorded water level and breach width are only available for the first 20,000 seconds of the test. The breach width simulated by BREACH, however, looks more accurate than the breach width produced over time by MIKE11. The breach width in MIKE11 increases at a very rapid rate until it reaches the maximum breach width of 36m. This is likely due to the increased Side Erosion Index that is being used. If a lower side erosion index is used, the amount of error in the breach width simulated by MIKE11 would likely decrease.

The next test in the IMPACT-Project is Field Test #2. This was known as the “minimum cohesive” test and was constructed out of soil with a D50 of approximately 5mm. The outflow hydrograph comparison between BREACH, MIKE11, and the recorded field results can be observed in Figure 12.1. As would be expected, the recorded total breaching time is much less due to the non-cohesive properties of the soil. Both models were somewhat accurate when producing the general shape of the discharge hydrograph. However, the lag times produced by BREACH and MIKE11 were much shorter than what was recorded in the field. It is important to note that once again, MIKE11 would not breach field test #2 using the breach notch dimensions that were recorded in the field. A depth of 0.1m was used in the field but a depth between 1.45m and 1.50m was needed in MIKE11 for a somewhat accurate breach to occur. A breach depth of 1.50m was used since it produced the most similar shape to what was recorded in the field. Please refer to Figure 12.2 to see how the discharge hydrograph simulated by MIKE11 changes with a varying initial breach notch depth. Figure 12.3 shows the recorded water level in the reservoir compared to the water level results produced by MIKE11 and BREACH. The first 2000 seconds shown on the graph for MIKE11 is the reservoir filling before the breaching occurs and should be disregarded. How quickly the water level decreases, is comparable for what was recorded in the field and what was produced by the two models. When referring to Figure 12.1, BREACH produces less error than MIKE11 when compared to recorded results except when looking at the breach depth at peak discharge. MIKE 11 produces almost an exact breach depth at peak discharge estimation when compared to the recorded field results and has an error of 0.44%.

The third field test from the IMPACT-Project was a composite structure using a central moraine core with rock-fill shoulders. BREACH has the option to enter both an outer and inner core so entering the information into the model was not problematic. MIKE11, however, only

gives you the option to enter one material. It was assumed that the central moraine core properties would be more influential in the dam breaching process so its characteristics were used. Another option would have been to take an average of the moraine core and rock-filled shoulder properties and use them for the single material. Figure 12.4 shows the outflow hydrograph produced by the two models compared to what was recorded in the field and the comparison summary can be observed in Table 12.2. It is noted when referring to Figure 12.4, that the general shape of the outflow hydrograph was not modelled very accurately by either numerical model. The breach simulated by BREACH occurred almost 1500 seconds after the breach occurred in the field. MIKE11 breached approximately 500 seconds after the recorded field test breached. The peak discharge produced by MIKE11 was much more accurate than the peak discharge modelled by BREACH when compared to the field results. The peak discharge in the field was $242.00 \text{ m}^3/\text{s}$ and the peak discharge produced by MIKE11 and BREACH was $272.10 \text{ m}^3/\text{s}$ and $150.93 \text{ m}^3/\text{s}$, respectively. This equates to an error of 12.44% and 37.63% for MIKE11 and BREACH, respectively. It should be mentioned that MIKE11 gave a conservative estimate while the estimate by BREACH was much less than what was expected. MIKE11 also had a more accurate time to peak estimate and only produced 1.76% error compared to the 6% error produced by BREACH. The lag time estimate by MIKE11 was also very accurate and only differed from the recorded amount by less than 20 seconds. When referring to Figure 12.4, it can be noted that MIKE11 and BREACH take much less time to fully breach the dam when compared to the recorded field results. The field test took approximately 2900 seconds from the start of the breach to the end of the breach. MIKE11 took only 1900 seconds for the total breach time and BREACH took even less time, taking only 730 seconds for the total breach time. Figure 12.6 shows the water levels recorded in the field for field test #3 compared to the results from the two models. It can be observed from this figure that the models are fairly accurate when modelling the water level. Once again, MIKE11 shows the water level rising until it reaches the dam crest height. This is the reservoir filling before the dam breach begins and is inputted by the user. If preferred, the reservoir could also be filled earlier in the test.

To ensure that a correct Side Erosion Index is still being used in MIKE11, another calibration of the Side Erosion Index parameter is completed and the results are shown in Figure 12.5. Once again, the best results come from a Side Erosion Index of 7.5. When the Side Erosion Index is above or below 7.5, the time to peak discharge is increased and a lag can visibly be observed in the figure. Therefore, a Side Erosion Index of 7.5 will continue to be used.

The fourth field test in the IMPACT-Project is very similar to the third test except the failure was induced by piping instead of overtopping. The outflow hydrograph for this test can be seen in Figure 12.7. Once again, MIKE11 produced a much better general shape of the hydrograph when compared to the field results. It is also very important to note that the breach induced by BREACH occurred too quickly and the hydrograph had to be lagged to show the graphs in the same figure. The BREACH hydrograph was lagged about 14000 seconds on the figure and actually breaches only 288 seconds after test start, compared to 14040 seconds recorded in the field and 15246 seconds simulated by MIKE11. MIKE11 also has a much more

accurate peak flow with only a 6% error compared to a 27% error for BREACH when compared to the field results. The water level at peak discharge and the peak water level comparisons are shown on Figure 12.8. It can be observed that the peak water level simulated by MIKE11 was more than 2m higher than the dam crest. This was a main problem that was encountered when breaching a dam due to piping. Another parameter that was needed to calibrate MIKE11 was the collapse ratio. When the ratio between the diameter of the pipe and the distance from the top of the dam to the top of the pipe is larger than the collapse ratio, the pipe collapses. A calibration was completed to ensure that the dam did not collapse right away but eventually did. During the calibration, the collapse ratio was varied from 0.1 to 500 but the outflow hydrograph did not change. After referring to Figure 12.8 it can be observed that the embankment in MIKE11 never collapsed since the water level stays at the height of the dam crest after the breaching process has occurred. This requirement makes it very difficult to model a piping failure because this ratio should change for the geometry of the dam and the types of soil being used. Therefore, a collapse ratio is very difficult to estimate for new dam breaches. MIKE11 also requires a calibration coefficient. This is a multiplication factor used to adjust the calculated change in pipe radius. A very large calibration coefficient (~500) was used during these simulations since the pipe radius was not increasing quickly enough with a lower factor. DHI Water & Environment (2009) has indicated that there is no easy way of determining this calibration coefficient and a calibration should be completed for an accurate estimation. Therefore, the coefficients used are very much dependent on the user and can very much change the outcome of the hydrograph.

The fifth and final field test in the IMPACT-Project was very similar to the fourth field test but the embankment was constructed entirely out of the moraine material. This test also induced a piping failure. Once again, the failure induced by BREACH occurred almost instantly and therefore had to be lagged 15000 seconds on the discharge hydrograph, as observed on Figure 12.9. This seems to be a common occurrence when using BREACH to simulate a piping failure. The initial pipe diameter recorded to induce breaching in the field was 0.2m. However, the size of initial pipe diameter would not cause the dam to fail when using MIKE11. When the initial pipe diameter was increased to 1.75m the dam would eventually collapse. The general shape of the breach outflow hydrograph is again more accurately predicted by MIKE11 than by BREACH when compared to the recorded field results. After referring to Table 12.4, it can be observed that MIKE11 produces less error than BREACH for peak flow, time to peak, lag time, and breach time when compared to the recorded field results. However, the water levels in the reservoir during the simulation are predicted more accurately by BREACH than MIKE11. The same issues occurred with the water levels in MIKE11 that occurred during the fourth field test. Once again, the water levels in MIKE11 rose much higher than the dam crest level before the dam breached.

Table 7.6 shows a comparison of all the percent errors for each IMPACT field test. When looking at the peak discharge characteristic, BREACH was more accurate than MIKE11 for the homogenous overtopping tests (field tests 1 and 2). However, MIKE11 performed more accurately than BREACH when the moraine material was used and when a piping failure

occurred. MIKE11 was also much more effective than BREACH when calculating the time characteristics of the simulations. MIKE11 had much less error than BREACH when calculating time to peak, lag time, and total breach time. When calculating the breach depth, MIKE11 was more accurate than BREACH when an overtopping failure occurred but was less accurate when piping failure occurred, as mentioned earlier. The final water depth was simulated equally by the two models when an overtopping failure occurred but BREACH was slightly superior to MIKE11 when simulating the final water depth during a piping failure.

8.3 IMPACT-Project Laboratory Tests

The IMPACT-Project laboratory tests were the next series of tests that were simulated. Much like the University of Ottawa laboratory tests, BREACH was not able to simulate an embankment failure because of the small size. Therefore, only a validation of MIKE11 was possible. Two different series of tests within the IMPACT laboratory test project were simulated. The first series contained tests 1-9 and a non-cohesive material was used. The second series contained tests 10-17 and a cohesive material was used. The tests varied geotechnical parameters and geometry as explained in the “Physical Model Experiments” section.

Laboratory Test #1 was a trial experiment and was used to set-up the facility and to ensure everything was running correctly. Therefore, all the data that was needed to make an accurate comparison was not available. No comparisons were made for the first laboratory test.

Laboratory Test #2 is the first test with available data that will be used for validation of MIKE11. As mentioned earlier, this is a non-cohesive test and it uses sediment with a uniform grading. Figure 7.10 shows the discharge hydrograph comparison for the recorded laboratory test and the simulation by MIKE11. It can be observed that MIKE11’s discharge almost “tops out” and does not reach the peak that it does in the lab. Consequently, an error of approximately 56% occurs. The time to peak and lag time simulated by MIKE11 is much more accurate and has an error of only 10.11% and 1.97%, respectively when compared to the recorded laboratory test. The reservoir drainage time estimated by MIKE11 increased due to having a lower peak discharge, which resulted in a longer total breach time. This is very evident on Figure 7.10. Figure 7.11 shows a comparison for water levels in the reservoir between what was recorded in the lab and what was simulated in MIKE11. The MIKE11 simulation very accurately describes the water level during the test and results in an error of approximately 5-6% over the duration. The percent error can be viewed in Table 7.3. When looking at the breach widths in Figure 7.12 and Figure 7.13 it can be observed that MIKE11 does a much better job simulating the top breach width than the bottom breach width. The simulation by MIKE11 increases the width of the bottom breach too much and almost doubles the actual width that was recorded in the lab.

Laboratory Test #3 was used to assess the repeatability of the test and therefore the same parameters were used. It would be redundant to validate MIKE11 using this test as the results

were the same as in Laboratory Test #2. Therefore, Laboratory Test #3 was excluded from this thesis.

The fourth laboratory test in the IMPACT-Project was the same as Laboratory Test #2 but the initial breach notch was against an abutment. This was to assess the effect of the breach location during a dam failure. Within MIKE11, the model assumes that the breach notch is always in the centre of the embankment crest and does not give an option to change its location. Therefore, this test is used to see how accurately the model can represent a failure even if the breach notch location is not in the centre of the dam. Once again, the discharge hydrograph produced by MIKE11 seems to “top out” and does not reach the peak discharge that was recorded in the lab. The amount of peak flow error in this test is higher than what was seen in Laboratory Test #2. The increase in error, however, is understandable since MIKE11 could not accurately model where on the dam crest the breach was initiating. The amount of lag time until the breach initiated was well represented by MIKE11 producing only a 0.44% error when compared to the recorded results. There was an increased amount of error in the time to peak discharge and this is due to MIKE11 not reaching the peak discharge recorded in the laboratory. As mentioned earlier, the simulation tends to reach a smaller maximum limit and it takes more time to reach that limit. The results from the laboratory tend to show that the embankments fail at a much more rapid rate even though they are not using cohesive sediments. It was noted during the field tests that the cohesive dams tend to fail at a more rapid rate compared to the non-cohesive dams that fail at a more gradual rate. MIKE11 once again accurately estimated the water level during the simulation and only resulted in an error between 3% and 8% when compared to the recorded results. The breach width during the simulation, for the most part, was greater than what was recorded in the lab. The simulated top breach width reached the maximum width possible in the flume but in reality the top breach only reached approximately 2.75m. As mentioned in earlier tests, this increase in breach width is likely due to the Side Erosion Index that was chosen.

Laboratory Test #5 was a direct replication of the second IMPACT-Project field test at a scale of 1:10. Once again, the peak discharge simulated by MIKE11 did not reach $0.2\text{m}^3/\text{s}$ even though in reality, the discharge reached a peak greater than $0.8\text{m}^3/\text{s}$. To achieve this peak discharge, the simulation would have to breach the dam at a much quicker rate. Therefore, Figure 12.15 shows the outflow hydrograph with a side erosion index of both 7.5 and 75. As mentioned earlier, a Side Erosion Index of 7.5 was used for the simulations as determined by calibration. However, a higher Side Erosion Index would cause a more rapid dam breach and therefore the Side Erosion Index was re-calibrated to ensure an accurate value was used. It should be noted from Figure 12.15 that increasing the Side Erosion Index will not increase the peak discharge but it will decrease the time to peak. Therefore, a Side Erosion Index of 7.5 will continue to be used for this study. Please refer to Figure 12.17 and Figure 12.18 for the comparison of the simulated and recorded top breach widths and bottom breach widths, respectively. It can be observed that MIKE11 does a better job at simulating these breach widths when compared to past IMPACT-Project laboratory test simulations. This is mostly due to the

wider breach widths that were recorded in the laboratory. During this experiment, the breach widths reached almost 3.75m compared to the 4.0m simulated by the model.

The sixth IMPACT-Project laboratory test used the same materials that were used in Laboratory Test #5 but the geometry was changed slightly. A face slope of 1:2 was used instead of using a face slope of 1:1.7. The same problem that has occurred in the other tests of this series has occurred again – the simulated peak discharge by MIKE11 is too low. This leads to a peak discharge error of almost 78%. The estimated lag time from MIKE11 was fairly accurate with an error of approximately 6% when compared to the recorded results. The estimated water levels by MIKE11 were also very close to what was recorded, having an error of only 6% to 7%. The breach width results are shown in Figure 12.21 and Figure 12.22. The breach width simulations by MIKE11 were more accurate than some of the earlier laboratory test simulations and had approximately the same accuracy as laboratory test #5. The percent error for all the dam breach characteristics of the sixth laboratory test can be observed in Table 12.7.

Much like the sixth laboratory test, the seventh laboratory test used the same material that was used in Laboratory Test #5 but part of the geometry was also changed. For this test, the crest width was changed from 0.20m to 0.30m. The same problem observed in the first IMPACT-Project laboratory test occurred again here. The simulated peak discharge by MIKE11 was much lower than what was recorded in the laboratory and the general shape of the discharge hydrograph, observed in Figure 12.23, was much more gradual than the recorded hydrograph. The estimated lag time for this test had an error of only 1.7% when compared to recorded results. The error associated with the breach time was much greater than other time characteristics due to the more gradual discharge hydrograph. The estimated water levels from MIKE11 were very accurate in this test averaging between 0% and 8% error when compared to recorded water levels.

Laboratory Test #10 was the first test of the second series of laboratory tests completed in the IMPACT-Project. The second series of tests was known as the cohesive series because the embankments were constructed out of either clay or moraine. Laboratory Test #10 was a replication of Field Test #1 at a scale of 1:10. A comparison of the discharge hydrographs produced by MIKE11 and recorded in the laboratory can be observed in Figure 7.14. The general shape of the hydrograph in this test was simulated much more accurately by MIKE11 compared to the hydrographs simulated for the first series of the IMPACT-Project laboratory tests. MIKE11 was conservative when estimating the peak discharge and produced a higher peak discharge than what was recorded in the laboratory. From 2500s to 7000s the general shape of the recorded hydrograph was very well represented by MIKE11. The water level simulated by MIKE11 also lowers before it did during the recorded laboratory test. This, however, could be explained due to the larger peak discharge that occurred in the simulation. Even though the general shape of the hydrograph seemed to be more accurate for this test, the calculated error is much larger. To view the comparison and error present in the main dam breach characteristics for this test, please refer to Table 7.4. There was a large amount of error when comparing the simulated and recorded breach widths and this can be observed in Figure 7.16 and Figure 7.17.

As mentioned earlier, the large amount of error present in the simulated breach widths by MIKE11 is likely due to the Side Erosion Index that was chosen. If a smaller Side Erosion Index was used, the breach widths simulated by MIKE11 would not be as large which would therefore result in a smaller error.

Laboratory Test #12 was very similar to Laboratory Test #10 except half of the compaction effort was used. This caused the embankment to fail at a quicker rate with much less lag time and this can be viewed in Figure 12.27. As observed in Table 12.9, the amount of error in this test was much less than what has been seen in other IMPACT-Project laboratory tests. The time to peak discharge was almost identical and produced an error of 0.53%. MIKE11 once again produced a conservative estimate for the peak discharge and resulted in an error of approximately 30% when compared to the recorded laboratory results. The water levels simulated by MIKE11 were also very close to what was recorded in the laboratory and can be observed on Figure 12.28. Again, the breach widths simulated by MIKE11 were much larger than what was recorded in the laboratory and this is likely due to the large Side Erosion Index that was used for the simulations.

Some of the largest error within this series resulted from Laboratory Test #13. This test mimicked Laboratory Test #10 except optimum moisture content was used for this test. During this laboratory test, the researchers were not able to complete the test due to time constraints and the test had to be paused over night. This resulted in the experiment being shut down earlier than expected. The experiment was continued the next day but was labelled Laboratory Test #14. Laboratory Test #14 was not used in this study due to the fact that the results from the beginning of the test were not present. After referring to Figure 12.30, it was determined that the test did not fully breach in the laboratory but it did during the MIKE11 simulation. This resulted in a large deviation for the dam breaching characteristics between the two. The peak discharge simulated by MIKE11 was $0.35\text{m}^3/\text{s}$ whereas the peak discharge that occurred in the laboratory was only $0.09\text{m}^3/\text{s}$ and they occurred 3,863s and 12,348s after the start of the test, respectively. It is possible that if the laboratory test was run for longer it may have fully breached the embankment; however, this was not likely the case due to the length of time it had already ran. A large deviation in peak discharge results in a large deviation in water level as well. It can also be observed in Figure 12.31 that the dam never fully breached because the recorded water level never falls below the 0.5m mark while the water level lowers to just above 0.1m in the MIKE11 simulation. The recorded breach widths in Figure 12.32 and Figure 12.33 also only increase by a small amount over the duration of the test. To view a full comparison of the results and associated errors please refer to Table 12.10.

The general shape of the discharge hydrograph estimated by MIKE11 for Laboratory Test #15 was also fairly accurate when compared to recorded results. However, there was a spike near the beginning of the breach on the simulated discharge hydrograph that was not recorded in the laboratory test. Laboratory Test #15 was similar to Laboratory Test #10 but had a downstream face slope of 1:1 instead of 1:2. When referring to Figure 12.35 it can be noted that the spike in the discharge hydrograph, simulated by MIKE11, also lowers the water level at a

quicker rate when compared to the recorded results. This spike was likely due to the increased breach width that can be viewed on Figure 12.35 and Figure 12.36. These figures show that MIKE11 increased the breach width at a very rapid pace until the maximum breach width of 4m was reached. The increase in breach width that occurred during the recorded laboratory experiment was much more gradual and only reached a total width of 1.73m. The time related characteristics for Laboratory Test #15 were fairly accurate when compared to other IMPACT laboratory tests and the associated error can be observed in Table 12.11.

Laboratory Test #16 was one of the more accurate tests in this series. This test was very similar to Laboratory Tests #10 and #15 but had a downstream face slope of 1:3 instead of 1:2 and 1:1, respectively. The peak discharge estimated by MIKE11 was once again greater than what was recorded in the laboratory but the general shape of the hydrograph was very well represented when compared to previous laboratory tests. The time to peak discharge was very similar between the MIKE11 simulation and the recorded laboratory test which resulted in only a 7.3% error. The total breach time was also very similar between the MIKE11 simulation and recorded laboratory experiment and resulted in an error of only 3.87%. When referring to the discharge hydrograph on Figure 12.38 it can be observed that the laboratory test started to breach earlier than what was observed during MIKE11's simulation. This resulted in a fairly large amount of error in lag time when compared to other tests in this series. When the breaching officially begins is often determined by the researcher and therefore this error may not be as large as shown. Once again, the breach widths predicted by MIKE11 are larger than what was recorded in the lab and the amount of associated error for the breach widths and other characteristics can be observed in Table 12.12.

The final laboratory experiment used from the IMPACT-Project was Laboratory Test #17. This test mimics Laboratory Test #10 but moraine was used to construct the embankment instead of clay. Please refer to Figure 12.42 to see the discharge hydrograph comparison between the MIKE11 simulation and recorded laboratory results. It can be noted from this figure that the results of this test are opposite from what has occurred in other IMPACT-Project laboratory tests. In this test, the recorded dam breach has a spike on the discharge hydrograph much like what MIKE11 was simulating in Laboratory Test #15. However, in this case MIKE11 does not produce a spike on the discharge hydrograph. Other than the spiked area, MIKE11 does represent the outflow hydrograph shape well when compared to other laboratory tests. Figure 12.43 shows the water level simulated by MIKE11 compared to the recorded water level in the laboratory. It reveals that the embankment in MIKE11 never fully breached when constructed out of moraine but the embankment in the laboratory did. This would explain the absence of the spiked peak discharge in the MIKE11 simulation that was recorded in the lab. The breach widths shown in Figure 12.44 and Figure 12.45 are also much narrower in the MIKE11 simulation since they never fully breached. Since MIKE11 did not breach, most of the breaching characteristics had a large amount of error and this can be observed in Table 12.13. The lowest amount of error for a breaching characteristic was for the peak water level and there was still an error of 21.53%. The error then rose as high as 127% for the time to peak characteristic.

Table 7.7 shows a comparison of the IMPACT laboratory test's percent errors. The table is split into two sections: Non-Cohesive Tests and Cohesive Tests. When the figures containing the outflow hydrographs for all of the tests are reviewed, MIKE11 tends to simulate the general shape of the hydrograph more accurately for the cohesive tests. The non-cohesive test simulations tend to be more gradual with a lower peak than what was recorded in the laboratory. Some of the percent errors in Table 7.7 may be misleading. As mentioned earlier, there was a spike in the discharge hydrograph for a few of the cohesive laboratory simulations. This resulted in a large amount of error, especially in Laboratory Tests #13 and #15. If it was not for these discharge spikes, the percent error would be much less. The cohesive series simulations by MIKE11 were also fairly conservative. With the exception of Laboratory Test #17, all of the discharge estimations were slightly higher than what was recorded in the Laboratory. This was not the case during the non-cohesive tests as most of the simulations predicted the discharge to be less than what was recorded. It can be noted that the lag time was simulated much better for non-cohesive tests than cohesive tests and every non-cohesive test was under 6.2% error. The lowest amount of error observed in the cohesive tests for lag time was under 9% and rises as high as almost 140% for Laboratory Test #16. The increase in error results from MIKE11 not starting to breach the embankment quick enough. The embankments in the laboratory usually breached much sooner than what were simulated by MIKE11 for the cohesive tests. Since a high Side Erosion Index was used, it makes sense that MIKE11 also predicted more accurate breach widths for the non-cohesive tests. The high Side Erosion Index that was used causes the breach to erode away faster in a horizontal direction. Since non-cohesive sediments erode away quicker than cohesive sediments, the breach widths in the lab would be wider for non-cohesive tests. For the most part, MIKE11 would simulate the breach width to be as wide as possible and therefore the non-cohesive tests simulated by MIKE11 would be more similar.

8.3 Delft University of Technology Laboratory Tests

The next series of tests that were analysed were performed on dikes by a team from Delft University of Technology, Netherlands (TU Delft). There were a total of three tests that were completed in the University's Hydraulics Laboratory. The tests varied the soil composition, water content, and void ratio as outline in Table 5.7.

The first test completed at Delft University of Technology had the largest amount of voids, used optimum water content and contained more clay and silt than tests two and three. As observed in Figure 7.18, MIKE11 simulated the embankment breach accurately when compared to recorded results. The peak flow differed by only 7% ($0.005\text{m}^3/\text{s}$) and this could very well be due to the interference that was received while recording the laboratory test. For a comparison showing the errors of each dam breach characteristic please refer to Table 7.5. The general shape of the recorded discharge hydrograph was also accurately simulated by MIKE11 when compared to other tests reviewed in this thesis. The time required to reach the peak discharge produced an error of over 300% but this is very misleading. If the reader is referred to the

discharge hydrograph shown in Figure 7.18, a small spike can be observed at the very beginning of the recorded laboratory test. If this spike is treated as an anomaly and not considered, the amount of error would be significantly less. The water level during the recorded experiment was also simulated fairly well by MIKE11 and only contained an error of 7% to 8%. Referring to the water levels throughout the duration of the test, on Figure 7.19, it can be observed that the recorded water levels are approximately 5-10cm lower than what was recorded in the laboratory. The difference in water level is likely because MIKE11 allows the water level to rise higher than the crest of the dam. This problem has already been analysed and discussed earlier in this thesis since it was encountered in the IMPACT-Project Laboratory tests as well. The remaining dike height, which can be viewed in Figure 7.20, was also accurately represented by MIKE11 when compared to the recorded results. The figure shows that MIKE11 simulated the height of the embankment to erode or collapse in stages. These stages occur very quickly and stay around the average value of remaining dike height that was recorded in the laboratory. Overall, most of the characteristics for this test were simulated adequately when compared to other tests.

The second test completed at Delft University of Technology was very similar to the first however the void ratio was lowered slightly and there was more sand in the soil composition. The discharge hydrograph comparison between the MIKE11 simulation and recorded laboratory test can be observed in Figure 12.46. This simulation was not as accurate as what was produced in the first Delft University of Technology laboratory test. The simulated peak flow was still fairly accurate and had an error of approximately 6% when compared to recorded results. However, when looking at the discharge hydrograph it can be noted that the simulation by MIKE11 did not breach the embankment until almost 5000 seconds after when the breach occurred in the laboratory. There was also a fair amount of error (23.88%) in the time it took to reach the peak discharge. When referring to the water levels during the experiment, in Figure 12.47, it can be observed that MIKE11 simulates a higher water level near the end of the experiment when compared to the recorded laboratory results.

The most accurate simulation by MIKE11 for a Delft University of Technology test was completed for their third test. The researchers lowered the void ratio even more for this test and therefore it contained the lowest void ratio out of all the Delft University of Technology tests. After referring to the discharge hydrograph comparison in Figure 12.49, it can be noted that the general shape of the hydrograph was almost perfectly replicated by the MIKE11 model when compared to the recorded results. Even though there was still approximately a 7% error on the peak discharge estimation, the general shape of the hydrograph was really well represented. The time to peak estimation by MIKE11 was also very accurate and only had an error of 0.32% when compared to the laboratory results. After referring to Table 12.15 it can be viewed that the water level estimations by MIKE11 were also accurate with an error between 2% to 10% when compared to recorded results. The recorded water levels shortly after the experiment started, seen in Figure 12.50, were a bit higher than what was simulated for the duration of the experiment but were almost identical at the time of peak discharge. There were some issues with the breach depth once again. As mentioned in the earlier Delft University of Technology tests,

MIKE11 tends to simulate the dike height in stages. The erosion or embankment collapse within these stages occurs very quickly. The average simulated dike height was much closer to what was recorded in the laboratory, but this was not represented in the error shown on Table 12.15.

When referring to Table 7.8, a comparison of the error associated with the Delft University of Technology simulations can be reviewed. As mentioned earlier, the best simulation by MIKE11 was for the third and final test. Consistently, there was a small amount of error for all of the embankment breaching characteristics with the exception of the breach depth. The breach depth was the only breach characteristic for this series of tests not represented well by MIKE11. MIKE11 also estimated the water levels well for both the first and third tests but produced between 14% and 20% of error for the second test. The success of these tests was similar to the success observed with the cohesive tests from the IMPACT-Project.

Within all of the tests, the high amount of error associated with the breach widths could also be due to the difficulty of accurately measuring the bottom width during simulation. Researchers have encountered many problems while trying to measure the widths during simulations because the bottom widths cannot be seen visually. Many researchers, as discussed in the literature review, have tried using different methods to measure the breach widths. Some of these methods include dyeing the sand different colours and placing sensors inside the embankment during construction. Problems with these methods are still encountered and therefore error during the tests should be expected.

8.4 Sensitivity Analysis

Another large objective of this study was to complete a sensitivity analysis on different parameters used in the two models to see how they influenced the discharge hydrograph and the characteristics associated with it. Figure 7.21 to Figure 7.23 and Figure 12.52 to Figure 12.92 show the results of the sensitivity analysis for all of the parameters that were varied. If the reader refers to Table 7.9 and Table 7.10, the parameters are ranked from most sensitive to least sensitive for each model.

The sensitivity analysis was only performed on IMPACT Field Test #1. The reason why this test was selected for the sensitivity analysis is because of the success both MIKE11 and BREACH had simulating it. A total of 7 different parameters were used in the sensitivity analysis for MIKE11 and 6 different parameters were used for BREACH.

The most sensitive parameter in MIKE11 was the D50. This was anticipated since the D50 determines the cohesive properties of the soil and MIKE11 does not ask for the cohesive strength like BREACH does. The discharge hydrographs with varying D50 can be viewed in Figure 12.55. After reviewing the figure it can be seen that the D50 plays a very large role in both the shape of the hydrograph and the breaching characteristics associated with it. Figure 12.56 shows that as the D50 increases, the peak discharge does not necessary increase or decrease. The lowest peak discharges come from using sediments with a D50 around the 0.1mm to 1mm range. Figure 12.57 shows that as the sediment size is increased, the amount of time to

the peak discharge is also increased. Overall, the sediment size causes a peak discharge variation of $413.00\text{m}^3/\text{s}$ which is a much larger of variation than any other parameter has caused. The time to peak discharge variation was also much larger than any other variation shown by other parameters, increasing by almost 2400 seconds.

The second most sensitive parameter in MIKE11 was determined to be a tie between the upstream and downstream slope. The discharge hydrographs for varying upstream and downstream slopes can be observed on Figure 12.64 and Figure 12.67, respectively. As expected, when the upstream and downstream slopes are increased (horizontal to vertical) the peak discharge decreases. This decrease can be observed in Figure 12.65 and Figure 12.68 with a total variation of approximately $33.0\text{m}^3/\text{s}$. The shape of the outflow hydrograph also becomes more gradual and therefore the time to the peak discharge also decreases with an increase in slope (horizontal to vertical). The total variation in time to peak discharge caused by a change in slope can be viewed in Figure 12.66 and Figure 12.69 with a total variation of just below 1400 seconds.

When both the upstream and downstream slopes are varied at the same time, the hydrographs produced by MIKE11 reacts slightly less. The overall peak discharge varies more but the time to peak discharge varies less and therefore the total sensitivity is ranked slightly lower. Much like when increasing the upstream or downstream slope independently, increasing both of them at the same time causes the peak discharge to decrease and the shape of the hydrograph to become more gradual. This can be observed in Figure 12.61 and Figure 12.62. The total variation in peak discharge was $121.8\text{m}^3/\text{s}$ and the time to peak discharge had a variation of less than 1300 seconds.

The porosity was the fifth most sensitive parameter found in the MIKE11 model. When the porosity was increased, the peak discharge tended to increase and the time to peak discharge decreased. The variations can be seen on Figure 12.59 and Figure 12.60. There was a total peak discharge variation of $29.1\text{m}^3/\text{s}$ and a total time to peak discharge variation of 886 seconds.

The crest width and non-dimensional critical shear stress were the least sensitive parameters within the MIKE11 model. It should be noted that when the critical shear stress was varied all of the parameters that were related to it were not varied. One great example of this would be the D50. The critical shear stress is very dependent on the sediment size however this stayed constant throughout the test. Therefore, the critical shear stress' sensitivity analysis shows when only the critical shear stress is changed. The outflow hydrographs simulated by MIKE11 were surprisingly not overly sensitive to a change in crest width. Please refer to Figure 12.52 for the discharge hydrograph variations corresponding to a change in crest width. With an increase in crest width, it can be observed that the peak discharge decreases slightly and the time to peak also increases slightly. However, this variation was very small when compared to the other parameters. The change in crest width resulted in a total variation of $25.6\text{m}^3/\text{s}$ for peak discharge and $500.4\text{m}^3/\text{s}$ for time to peak discharge.

As mentioned earlier, a total of six different parameters were varied in the BREACH sensitivity analysis. This sensitivity analysis was much less successful than the one completed

for MIKE11 because there were some problems with the model. In some cases, the model would not run for certain parameter values even though they were reasonable approximations. In other cases, the peak discharge and total breaching time would be much less than what was expected. With the exception of the sensitivity analysis completed on the upstream side slope, there was at least one simulation from each sensitivity analysis that created an inadequate result. To avoid running into any errors during simulations, four different sediments were used and their properties were estimated from Zhu (2010). To see the different sediments and their corresponding parameters please refer to Table 12.16. The outflow hydrographs produced by BREACH for each of the sediments were much different than what was expected. The peak discharges were very low, with a highest peak discharge of $179.10 \text{ m}^3/\text{s}$ for clay and a lowest peak discharge of $164.44 \text{ m}^3/\text{s}$ for sand. To see the outflow hydrographs produced by BREACH for the sensitivity analysis completed on the four different sediments please refer to Figure 12.89. When reviewing the hydrographs shown in Figure 12.89, the variations in peak discharge and time to peak discharge were the least out of all the different parameters that were varied for BREACH.

The most sensitive parameter in BREACH was determined to be the downstream slope. To see the outflow hydrograph produced by BREACH with a varying downstream slope please refer to Figure 12.77. The downstream slope was the most sensitive parameter because not all of the simulations would reach the expected peak discharge. For example, with a downstream side slope (horizontal to vertical) of 2.5, the peak discharge would only reach $131 \text{ m}^3/\text{s}$. This was much different than the $478.36 \text{ m}^3/\text{s}$ that was estimated for a downstream side slope of 20.0. Therefore, there was a total variation of $346.37 \text{ m}^3/\text{s}$. Since there was such a large variation in the peak discharge there was also a very large variation in the time to peak discharge. The total time to peak discharge variation was over 7000 seconds and much higher than most other parameters.

Gradation (D90/D30) was also a parameter that was very sensitive in BREACH. However, the outflow hydrographs had the same issue that was seen when the downstream slope was varied. There were simulations with high peak discharges that reached $390.63 \text{ m}^3/\text{s}$ but there were simulations with discharges that only reached as high as $133.88 \text{ m}^3/\text{s}$. To see the sensitivity of the outflow hydrograph when the gradation was varied, please refer to Figure 12.83. It should be noted that when the D90/D30 was increased to a large number, such as 75, the model would only simulate the first section of the breach before shutting down.

The upstream side slope was the fourth most sensitive parameter in this analysis. Each simulation was accurate when compared to expected values and therefore, it resulted in a lower sensitivity rank. When referring to Figure 12.74 through Figure 12.76 it can be observed that there is no trend that occurs with a varying upstream slope. The peak discharge looked random as the upstream side slope was increased. When the upstream side slope was varied from 2.5 to 50, there was a variation of $72.49 \text{ m}^3/\text{s}$ for peak discharge and a variation of 727.2 seconds for the time to reach peak discharge.

The angle of internal friction and unit weight were also included in the sensitivity analysis. The same issue occurred with these parameters as it did when the sediment type was

varied. The parameters did not cause BREACH to fully simulate a dam failure and the model ended before it should have. Therefore, the amount of variation was similar to the amount observed when the sediment type was changed. Please refer to Figure 12.80 and Figure 12.86 to see the sensitivity of the outflow hydrograph when the internal angle of friction and unit weight were varied, respectively.

9.0 Conclusions

Overall, this study was successful in completing its initial main objectives. The main objective of this study was to validate two dam breach models that are currently used in the industry. Several newly available laboratory experiments (provided by the University of Ottawa, Canada, Delft University of Technology, Netherlands and the IMPACT-Project, United Kingdom) and a relatively new field experimental program (provided by the IMPACT-Project, Norway) provided enough additional data to assist in the validation of these numerical models. A sensitivity analysis was also completed to determine the most sensitive parameters of the two numerical models (BREACH and MIKE11).

A total of three non-cohesive dam overtopping breaching tests were conducted in the Hydraulics Laboratory of the University of Ottawa. These tests varied the compaction of the dam models: high compaction, low compaction, and very low compaction. Another three tests used in this study were provided by the Delft University of Technology, Netherlands. These tests were completed on cohesive dikes with varying geotechnical properties that also failed due to overtopping. The final set of tests that were used were taken from the IMPACT-Project. The IMPACT-Project tests consisted of five non-cohesive soil laboratory tests, six cohesive soil laboratory tests, and five field tests that varied cohesive and non-cohesive sediments in the construction of the dams. For all the laboratory tests and for three of the field tests, the dams failed due to overtopping while for the other two field tests, the dams failed due to piping. The tests varied the geotechnical and geometric properties of the dam models.

BREACH has shown that it could not accurately model the laboratory tests. Due to the small size of the dam and its impoundment reservoir, the model could not initiate the breach. Instead, the model simulated the outflow through the dam to equal the inflow into the reservoir. In most laboratory cases, the inflow would be very small and therefore the breach outflow hydrograph was not accurately modelled. Alternatively, MIKE11 modelled the laboratory tests much more accurately comparing to BREACH. The general shape of the recorded outflow hydrograph was better simulated by MIKE11 for the tests employing a dam including cohesive materials. This was because MIKE11 has a tendency to make the shape of the non-cohesive outflow hydrographs more gradual with a larger total breaching time than what was recorded in the laboratory experiments. However, MIKE11 did better approximate the breach width when modelling dams using non-cohesive materials. This is likely due to the relatively high Side Erosion Index value that was obtained through calibration. A high side erosion index value would cause the modelled breach width to develop more quickly when compared to the breach depth.

Both BREACH and MIKE11 were able to simulate dam breaching for the recorded field tests. It was found that:

1. BREACH performed slightly better than MIKE11 when simulating the peak discharge for the overtopping tests using homogenous material dams.

2. MIKE11 better approximated the peak discharge comparing to BREACH when moraine material was used for the models and when a failure occurred due to piping.
3. MIKE11 was also more accurate than BREACH when simulating other dam breaching characteristics such as the time to peak discharge, lag time, and total breach time. MIKE11 performed more accurately than BREACH when calculating the breach depth during simulation in overtopping mode failures.
4. BREACH was more accurate than MIKE11 when calculating the breach depths for a piping failure. To initiate a piping failure when using MIKE11, the initial pipe diameter had to be increased significantly while BREACH simulated the dam breach with the recorded pipe diameter.

A successful sensitivity analysis was completed for MIKE11 during this study. However, several weaknesses of BREACH were identified during the sensitivity analysis. The sensitivity analysis was completed on the first IMPACT-Project field test since both models were able to simulate it fairly well. The most sensitive parameter within MIKE11 was found to be the sediment grain size (D_{50}). The upstream and downstream side slopes were found to be equally sensitive and were the second most sensitive parameters of all the parameters tested for MIKE11. The porosity, crest width, and non-dimensional critical shear stress proved to be the least sensitive parameters within MIKE11. As mentioned earlier, there were some problems with BREACH that caused the simulation to occasionally end before an accurate breach of the embankment was completed. This caused large variations in the results and led to some of the parameters appearing like they were more sensitive than they might actually be. To try and resolve this problem, different types of sediments were chosen from Zhu (2010) and sediment parameters were varied at the same time during the analysis. The most sensitive parameter in BREACH was the downstream side slope. Both the gradation and internal angle of friction were found to be more sensitive comparing to the variation of the upstream side slope but less sensitive than the downstream side slope. The least sensitive parameter in BREACH was determined to be the unit weight.

All things considered, this study will hopefully provide additional useful information for the numerical modelling of dam breaching. The study will present future researchers in the field of dam breaching an indication of the possible models that need improvement and models that are performing adequately. This study is therefore a pertinent step for future researchers to improve numerical models and the understanding of the complex of dam breaching characteristics.

10.0 Recommendations for Future Work

This study is only a small part of a very large and ongoing study on dam breaching that is being carried out at the University of Ottawa. As mentioned earlier, detailed and well-controlled physical experiments on dam breaching have been scarce up to this point. However, a number of physical experiments have been completed recently and this significantly contributed to the development of a data base of dam breaching. With these new experiments, a number of questions and concerns are presented and therefore, many areas of dam breach modelling can hopefully be improved.

- One main conclusion drawn from this study is that there are still a number of important dam breach parameters that researchers have not yet properly investigated. Significantly more physical modelling experiments need to be completed so that numerical modellers can better understand the dam breaching phenomena.
- Present numerical models do not take into account the location, shape, or imperfection of the initial breach of real dams. For example, MIKE11 assumes the initial breach notch to be rectangular in shape and located at the midpoint of the embankment crest. However, it is observed in many of the tests and in accidental dam overtopping, that this is not the case. In laboratory tests different shapes are regularly used to initiate the breach such as, v-notch, and circular. In reality, it is also unlikely that a real dam breach initiates at the mid-point of the dam crest. It was shown in the IMPACT-Project laboratory tests that the breaching location does change the outflow hydrograph. Therefore, researchers should try to incorporate the randomness of breaching initiation in future numerical models and investigate characteristics on the initial breach in order to determine the worst case scenario of a dam breach.
- As mentioned by Orendorff (2009), more full scale tests need to be conducted to determine the effects of scaling on embankment dam testing. It is very difficult to scale the sediment sizes in laboratory models due to the cohesive properties of smaller sediments. Therefore, more full scale tests need to be completed to determine the effect scaling has on laboratory experiments.
- Multiple materials are often used to construct earthen embankment dams. However, MIKE11 only allows the user to use one type of sediment for the entire dam. This limits the model to breaching only homogeneous dams or causes the user to make possibly unrealistic assumptions. Unlike many of the laboratory and field tests completed in the past, most dams are constructed with a clay or moraine core and covered with a sand or gravel shoulder. These composite structures breach much differently than a homogenous dam.

- There have been many dam breach numerical models that have been developed by researchers in the field that are unavailable to use through free source code. The developers have not made these models available for industry use and therefore it is very difficult for other researchers to validate or evaluate the software's accuracy and performance. There may also be models that have been developed that produce much more accurate results than what is currently being used by emergency management agencies. However, since these models have not been made available for use, emergency management agencies often use less accurate models. It is therefore suggested for future researchers to try and test the validity of some of these models which were not included in this study.

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12.0 Appendix

12.1 IMPACT Field Test Comparisons

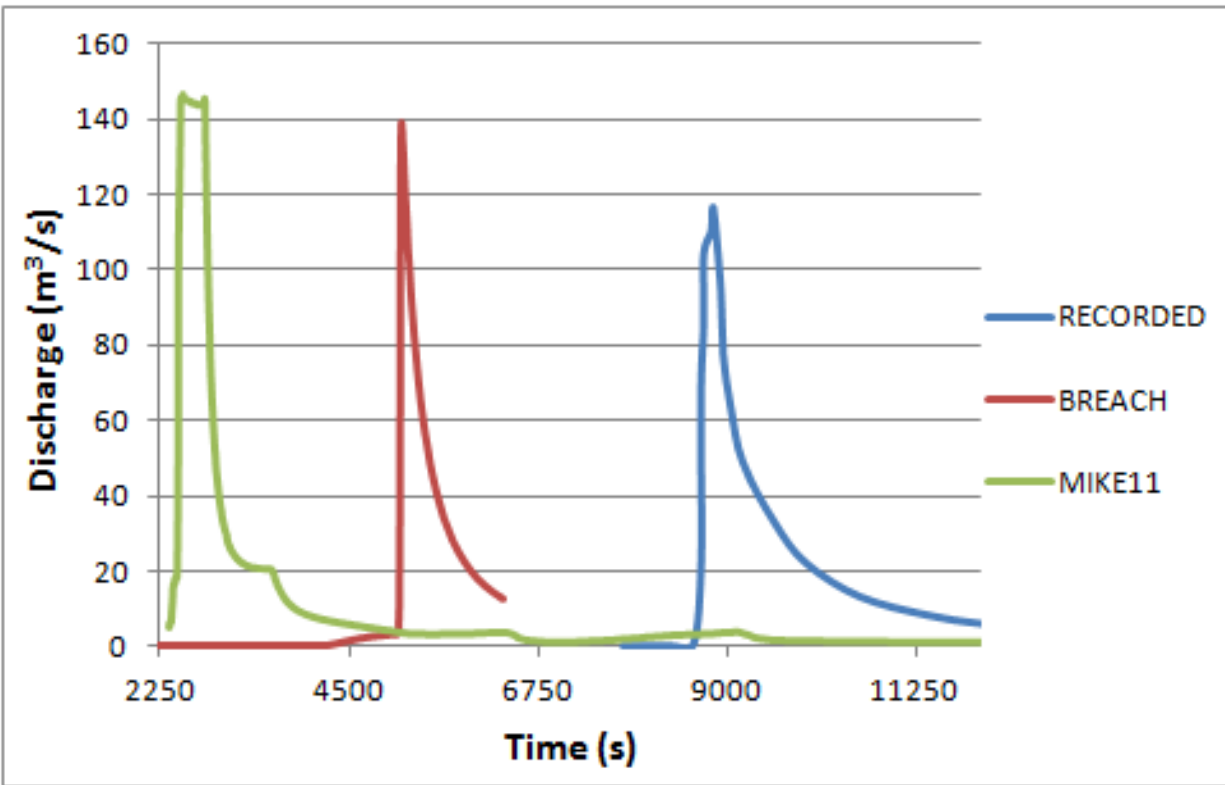


Figure 12.1: IMPACT Field Test #2 - discharge - recorded vs. BREACH vs. MIKE11

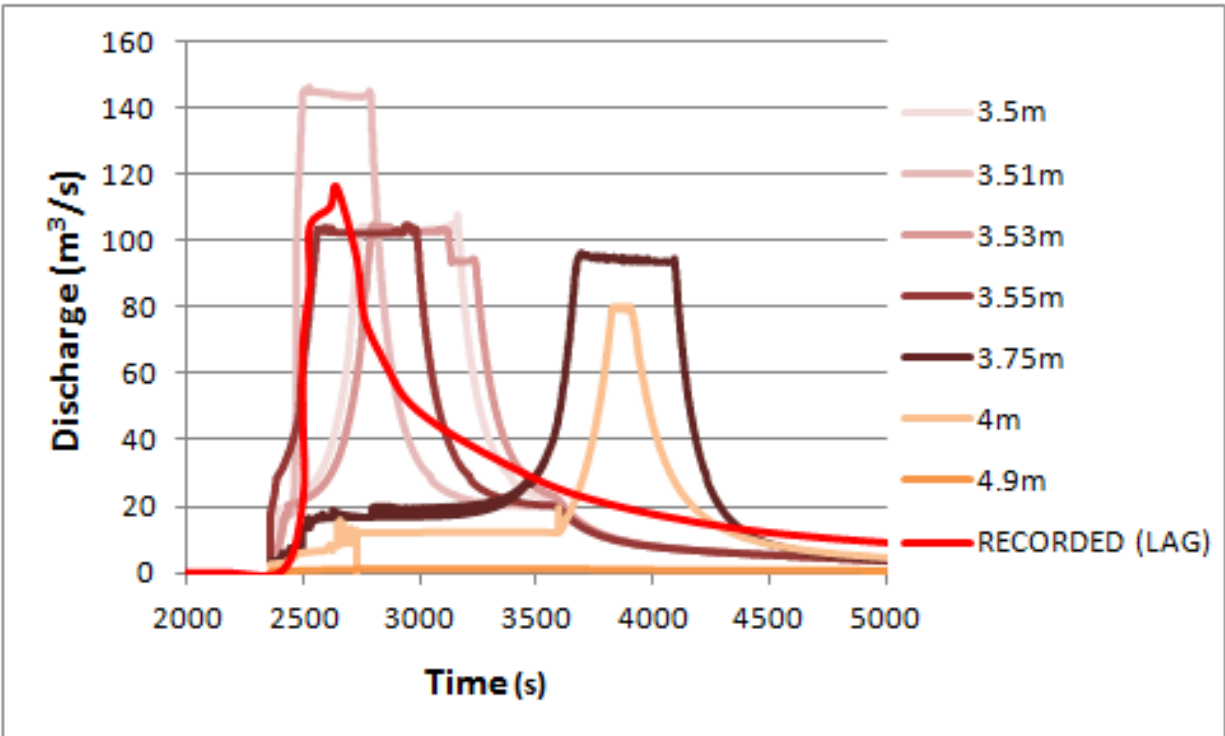


Figure 12.2: IMPACT Field Test #2 - discharge - recorded vs. MIKE11 with varying initial breach notch depth

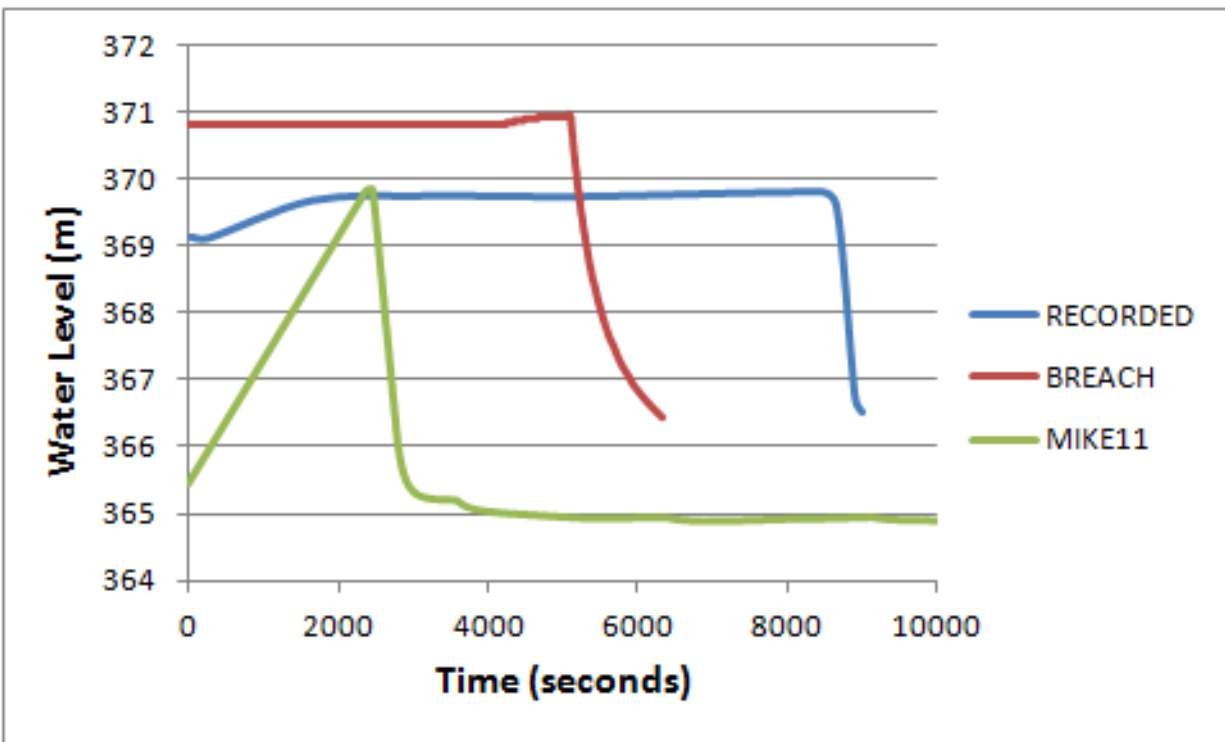


Figure 12.3: IMPACT Field Test #2- water level - recorded vs. BREACH vs. MIKE11

Table 12.1: IMPACT Field Test #2 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Breach depth at Peak (m)	Final Water depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	117.00	8820.00	8608.70	5855	2.5	3.31	NA	15
BREACH	138.86	5148.00	5108.00	1234.8	4.97	5	12.89	13.35
Error	18.68%	41.63%	40.66%	78.91%	98.80%	51.06%	NA	11.00%
MIKE11	146.30	2520.00	2354.00	1652.8	2.489	5	36	36
Error	25.04%	71.43%	72.66%	71.77%	0.44%	51.06%	NA	140.00%

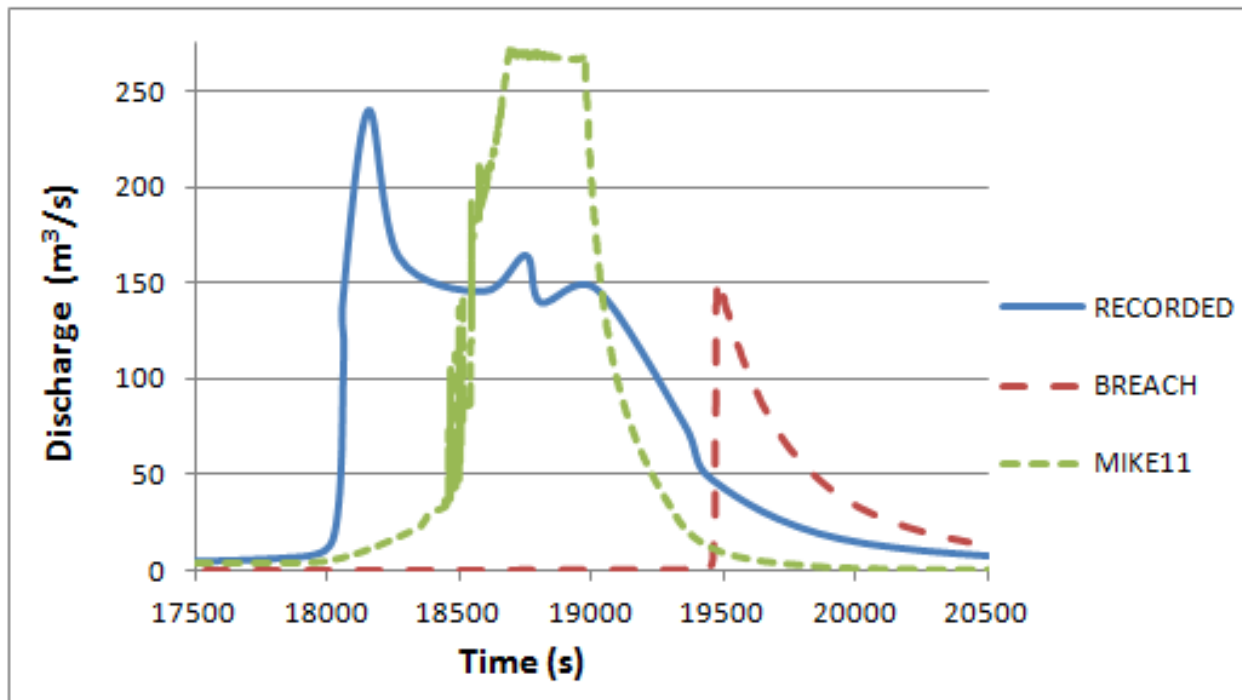


Figure 12.4: IMPACT Field Test #3- discharge - recorded vs. BREACH vs. MIKE11

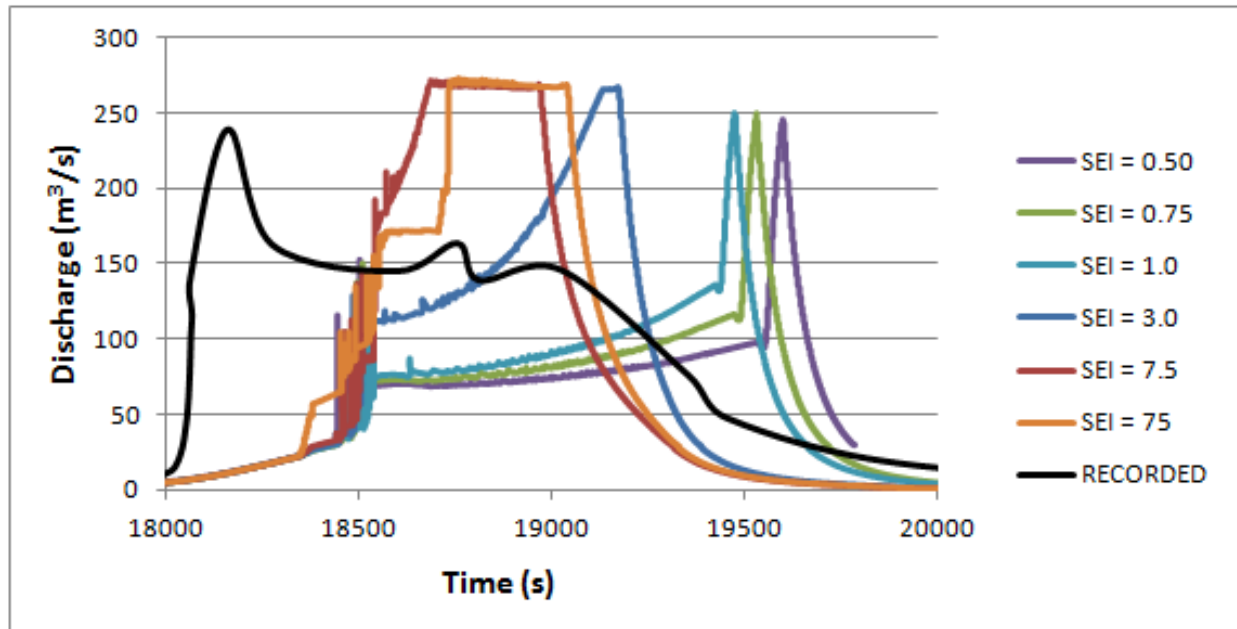


Figure 12.5: IMPACT Field Test #3 - discharge - recorded vs. MIKE11 with varying side erosion index

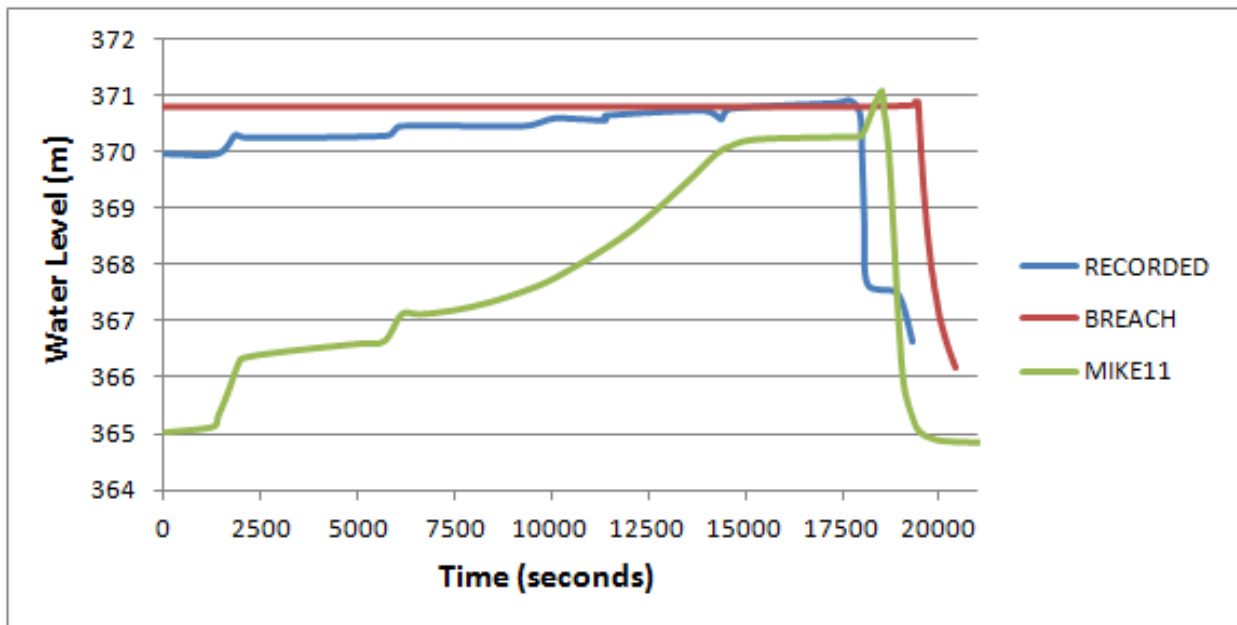


Figure 12.6: IMPACT Field Test #3- water level - recorded vs. BREACH vs. MIKE11

Table 12.2: IMPACT Field Test #3 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Breach depth at Peak (m)	Final Breach depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	242.00	18360.00	18019.60	2888.9	2.01	5.81	NA	NA
BREACH	150.93	19476.00	19461.60	730.8	5.59	6	9.72	10.24
Error	37.63%	6.08%	8.00%	74.70%	178.11%	3.27%	NA	NA
MIKE11	272.10	18684.00	18000.00	1900	2.762	6	36	36
Error	12.44%	1.76%	0.11%	34.23%	37.41%	3.27%	NA	NA

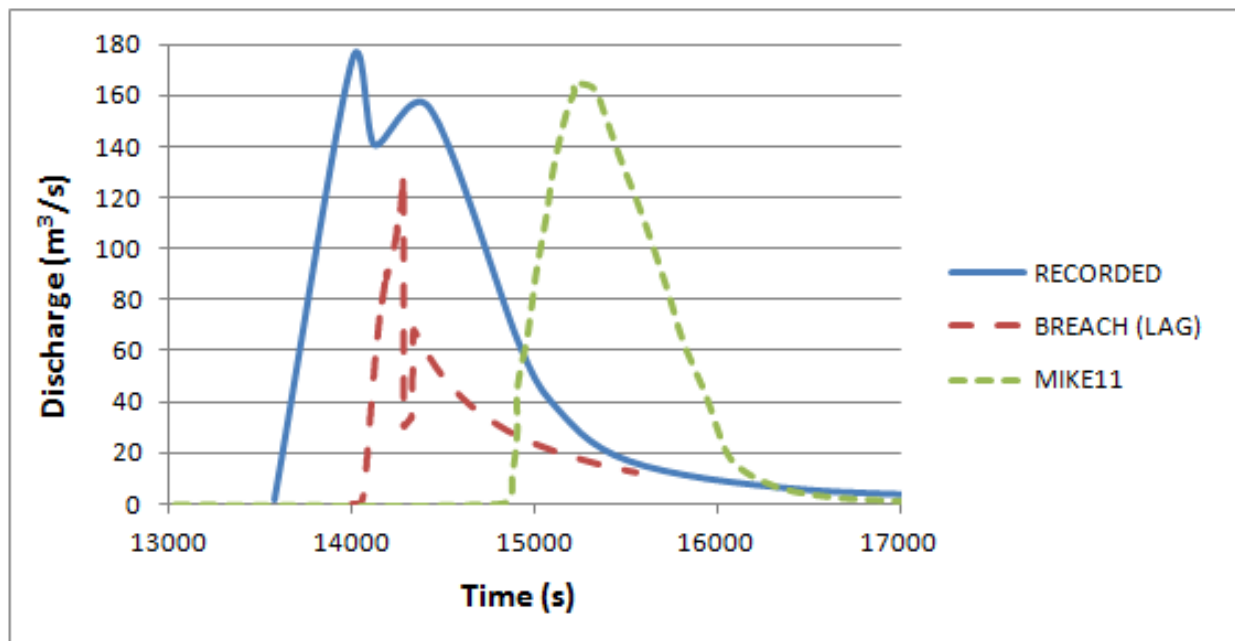


Figure 12.7: IMPACT Field Test #4- discharge - recorded vs. BREACH vs. MIKE11

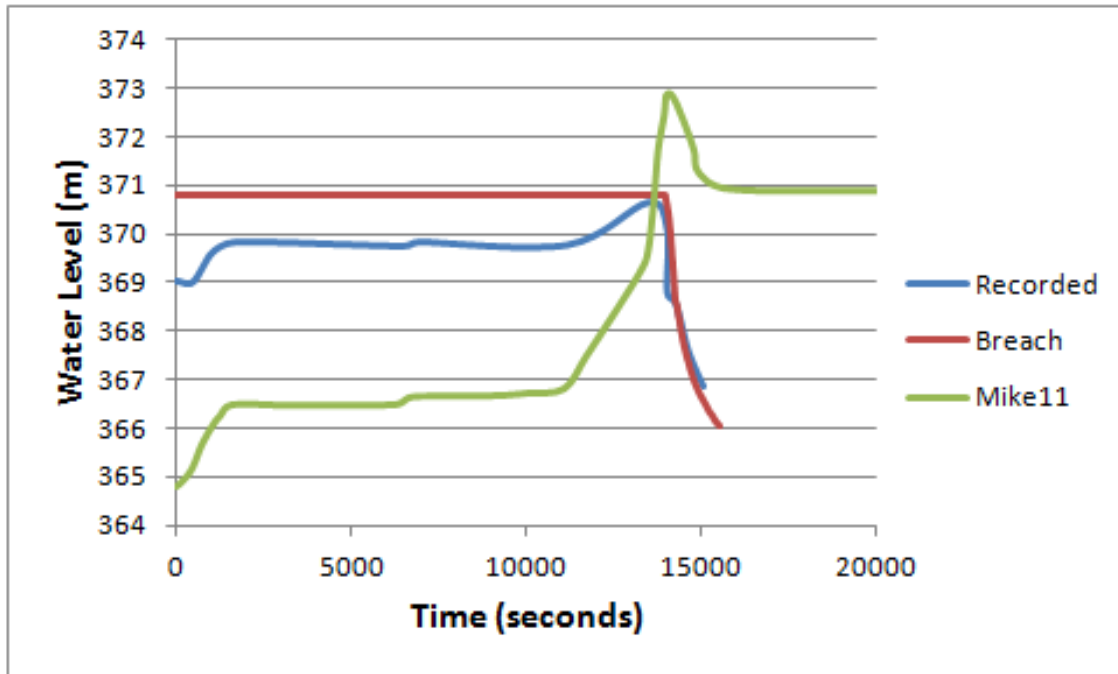


Figure 12.8: IMPACT Field Test #4- water level - recorded vs. BREACH vs. MIKE11

Table 12.3: IMPACT Field Test #4 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)
RECORDED	174.00	14040.00	13575.8	4727.2	4.09	5.79
BREACH	127.45	288.00	61.20	1188	3.8456	6
Error	26.75%	97.95%	99.55%	74.87%	5.98%	3.63%
MIKE11	164.46	15246.36	14876.28	2498.04	6.25	8.1
Error	5.49%	8.59%	9.58%	47.16%	52.81%	39.90%

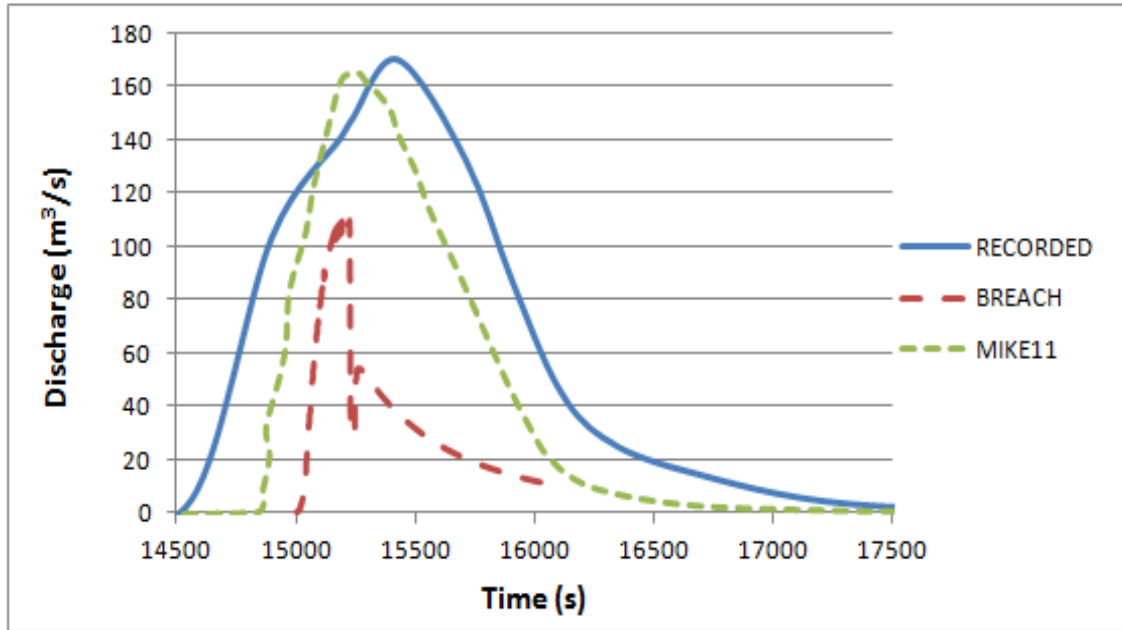


Figure 12.9: IMPACT Field Test #5- discharge - recorded vs. BREACH vs. MIKE11

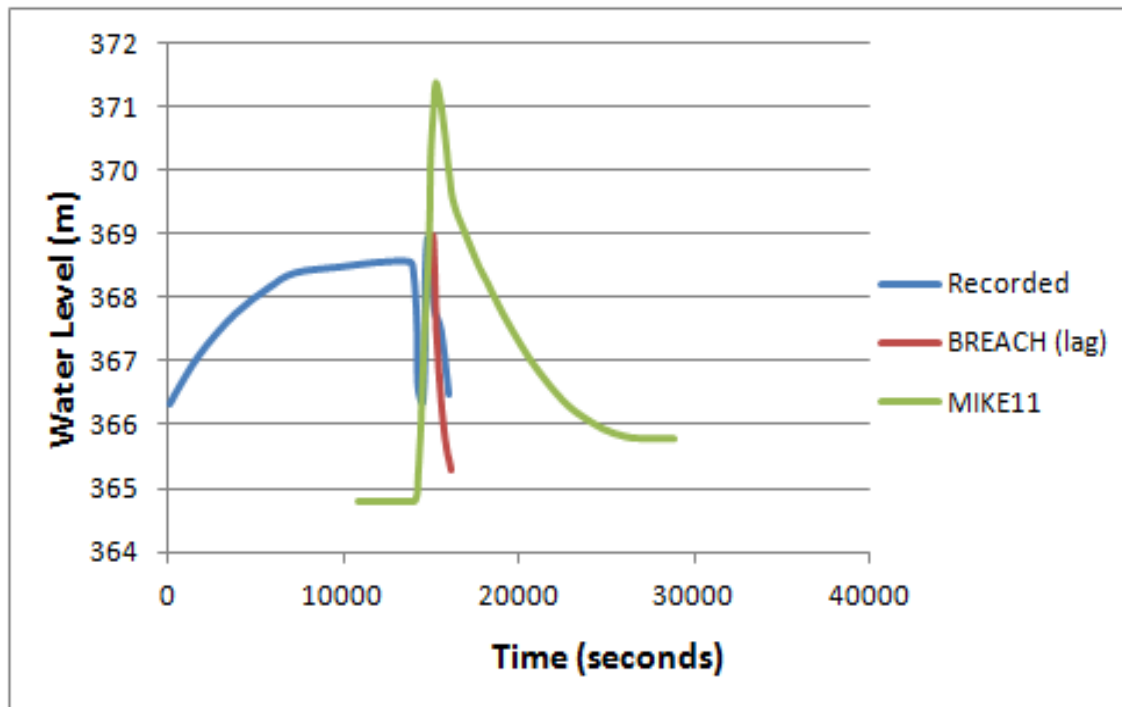


Figure 12.10: IMPACT Field Test #5- water level - recorded vs. BREACH vs. MIKE11

Table 12.4: IMPACT Field Test #5 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)
RECORDED	171.00	15480.00	14559.4	2777.8	2.69	3.99
BREACH	109.89	216.00	10.00	1070	2.69	4.18
Error	35.74%	98.60%	99.93%	61.48%	0.00%	4.76%
MIKE11	164.71	15262.56	14860.44	2149.56	6.5	6.5
Error	3.68%	1.40%	2.07%	22.62%	141.64%	62.91%

12.2 IMPACT Non-Cohesive Lab Test Comparisons

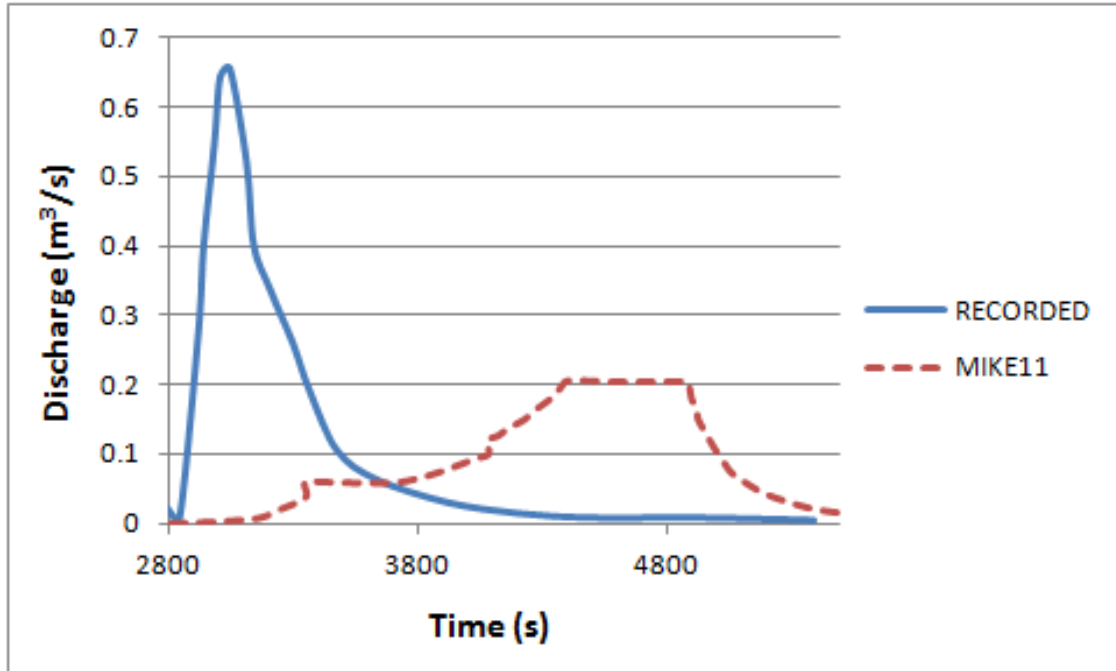


Figure 12.11: IMPACT Lab Test #4- discharge - recorded vs. MIKE11

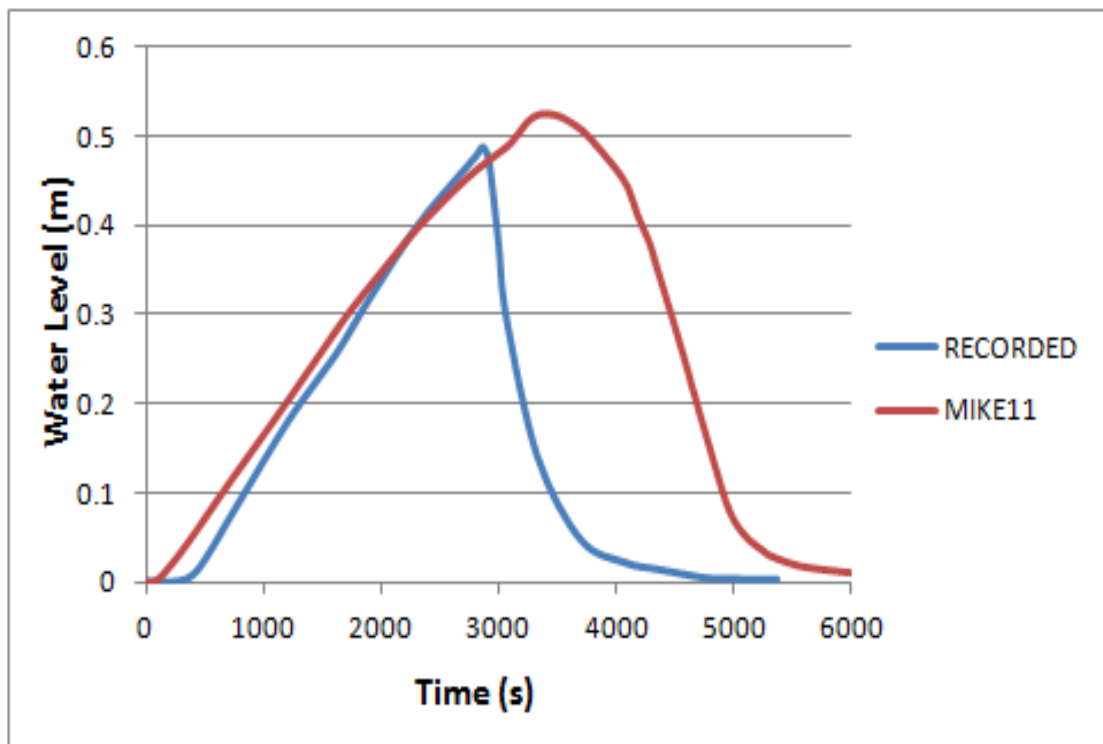


Figure 12.12: IMPACT Lab Test #4- water level - recorded vs. MIKE11

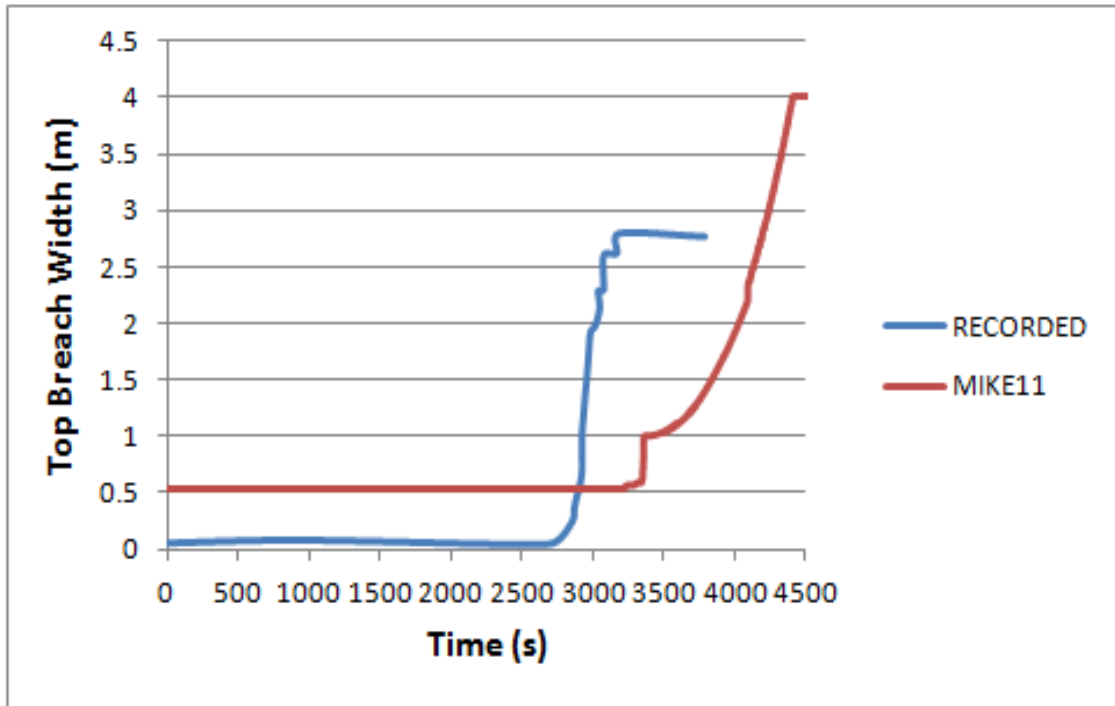


Figure 12.13: IMPACT Lab Test #4- top breach width - recorded vs. MIKE11

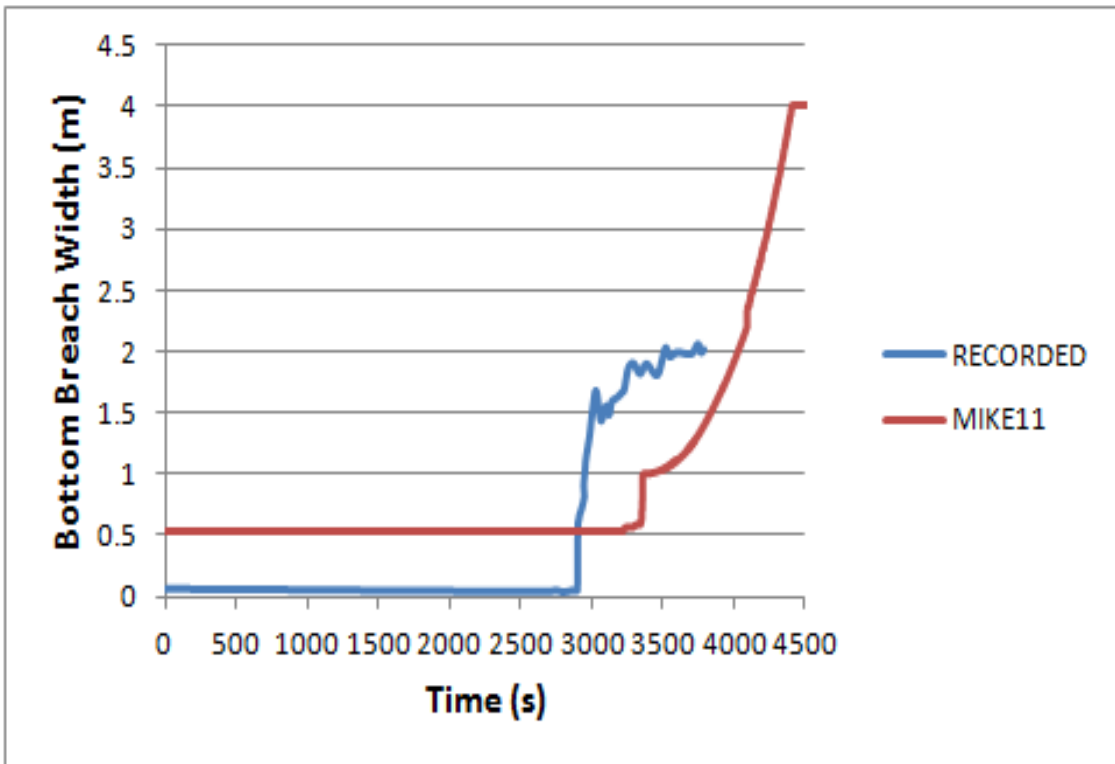


Figure 12.14: IMPACT Lab Test #4- bottom breach width - recorded vs. MIKE11

Table 12.5: IMPACT Lab Test #4 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak	Peak Water Level	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.67	3024.00	2840.00	4109.9133	0.34	0.48	NA	0.5	1.9	2.4
MIKE11	0.20	4403.48	2827.61	2718.6552	0.33	0.52	0.273	0.5	3.879	4
Error	69.51%	45.62%	0.44%	33.85%	2.94%	8.33%	NA	0.00%	104.16%	66.67%

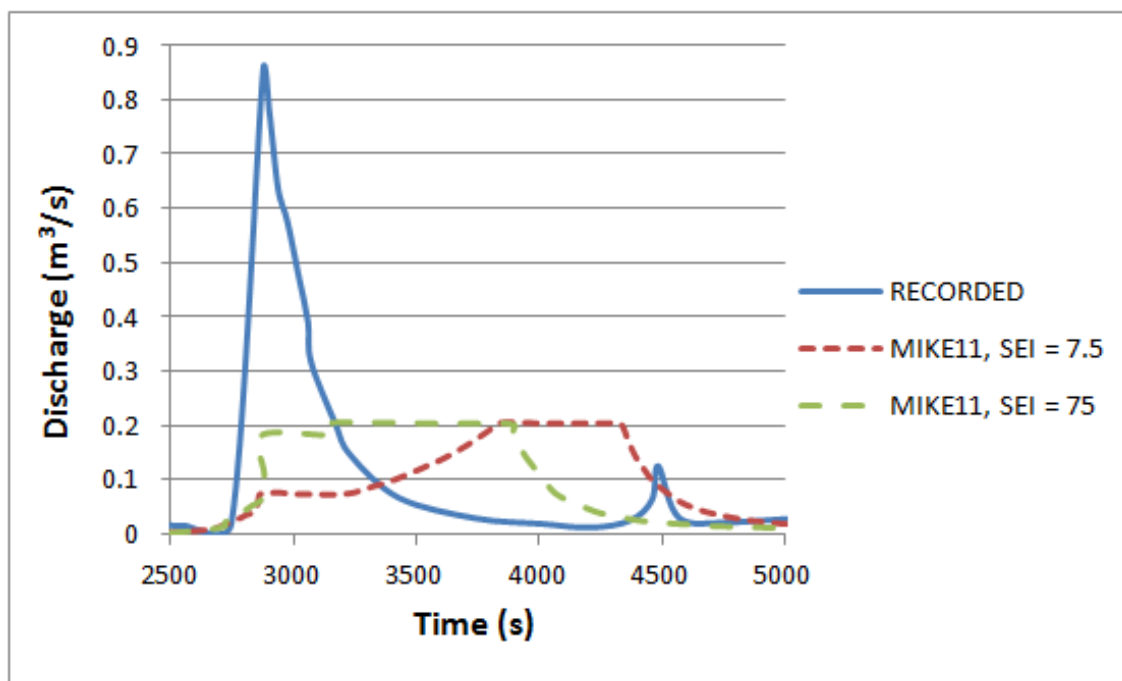


Figure 12.15: IMPACT Lab Test #5- discharge - recorded vs. MIKE11

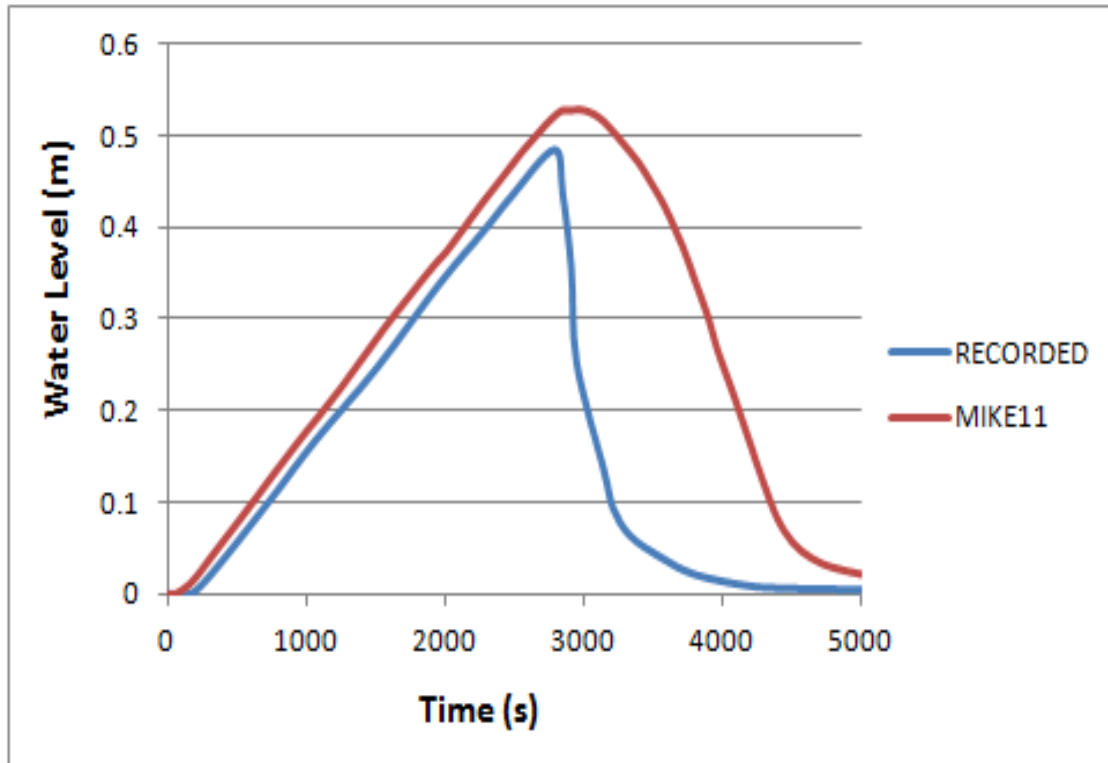


Figure 12.16: IMPACT Lab Test #5- water level - recorded vs. MIKE11

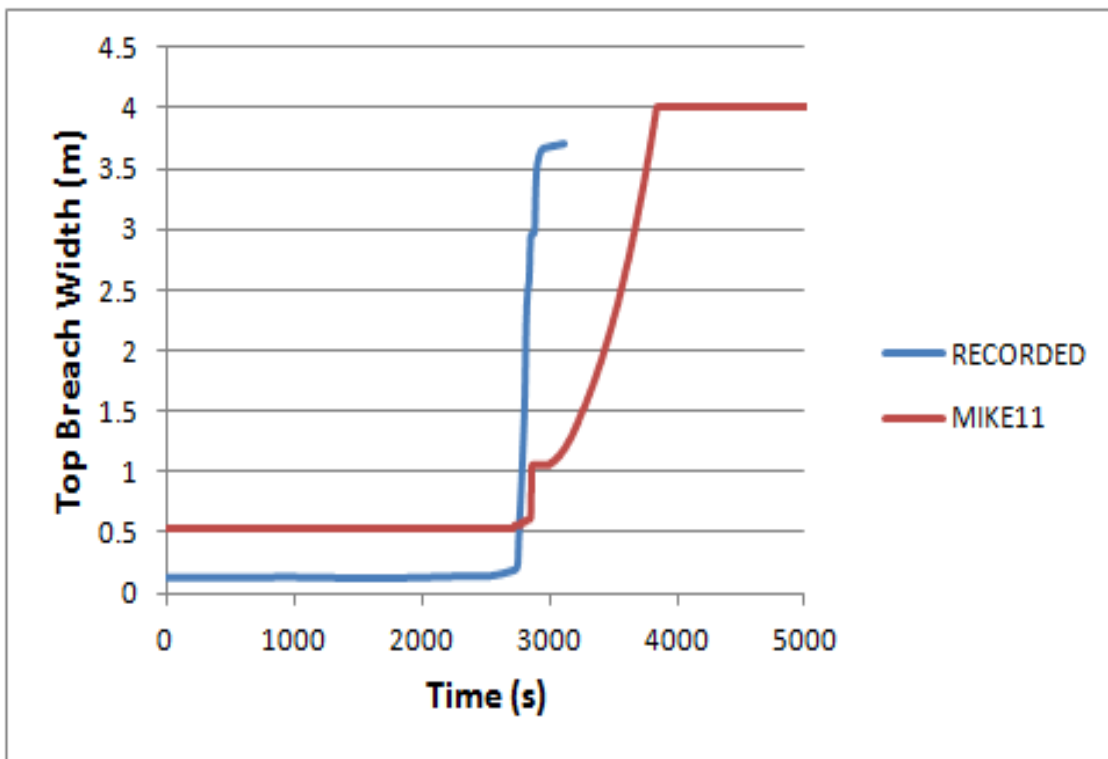


Figure 12.17: IMPACT Lab Test #5- top breach width - recorded vs. MIKE11

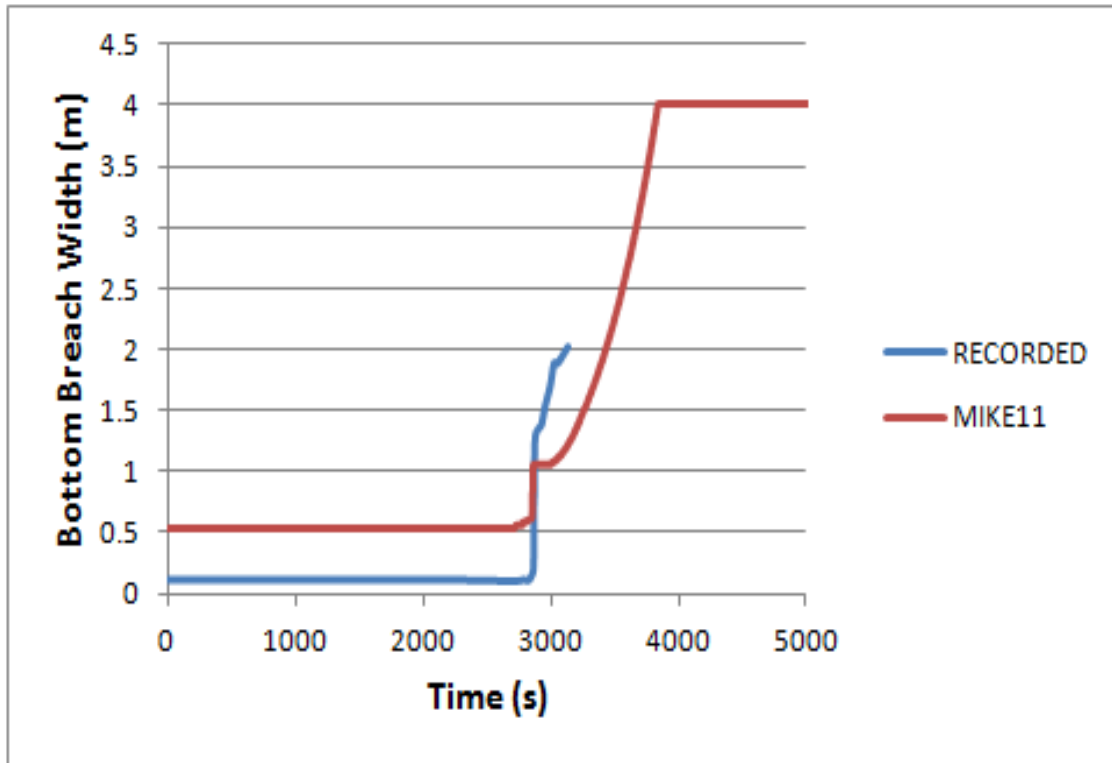


Figure 12.18: IMPACT Lab Test #5- bottom breach width - recorded vs. MIKE11

Table 12.6: IMPACT Lab Test #5 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak	Peak Water Level	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.87	2880.00	2745.46	1695.65	0.35	0.48	NA	0.5	2.56	2.93
MIKE11, SEI = 7.5	0.204295	3889.55	2715.65	2901.895	0.31	0.53	0.303	0.5	4	4
MIKE11, SEI = 75	0.205139	3184.61	2798.381	2353.831	0.43	0.525	0.182	0.5	4	4
Error	76.52%	35.05%	1.09%	71.14%	11.43%	10.42%	NA	0.00%	56.25%	36.52%

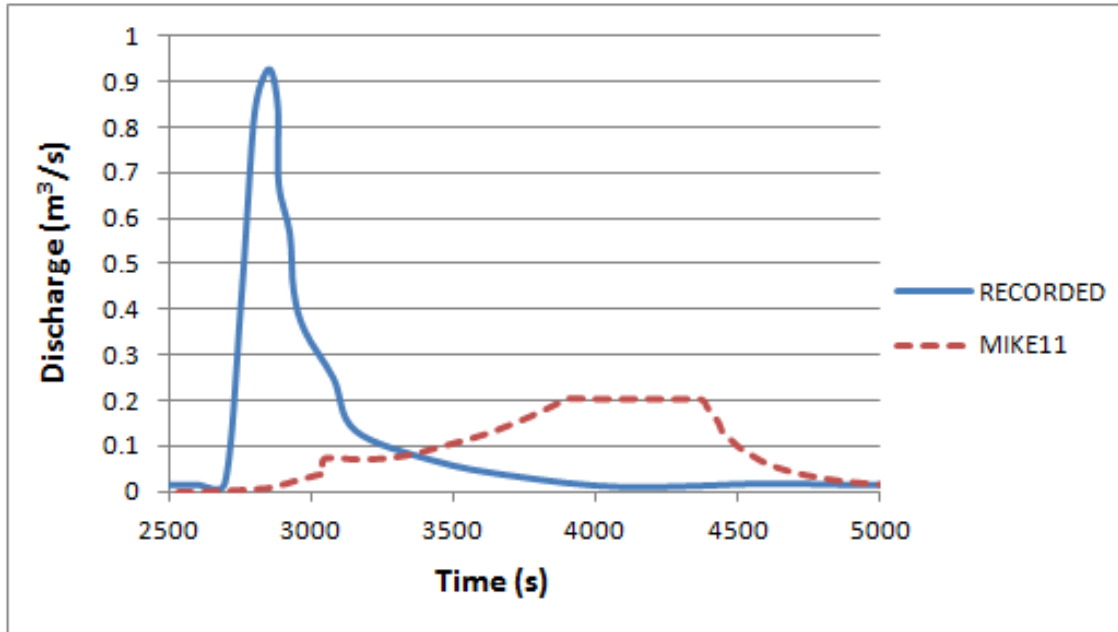


Figure 12.19: IMPACT Lab Test #6- discharge - recorded vs. MIKE11

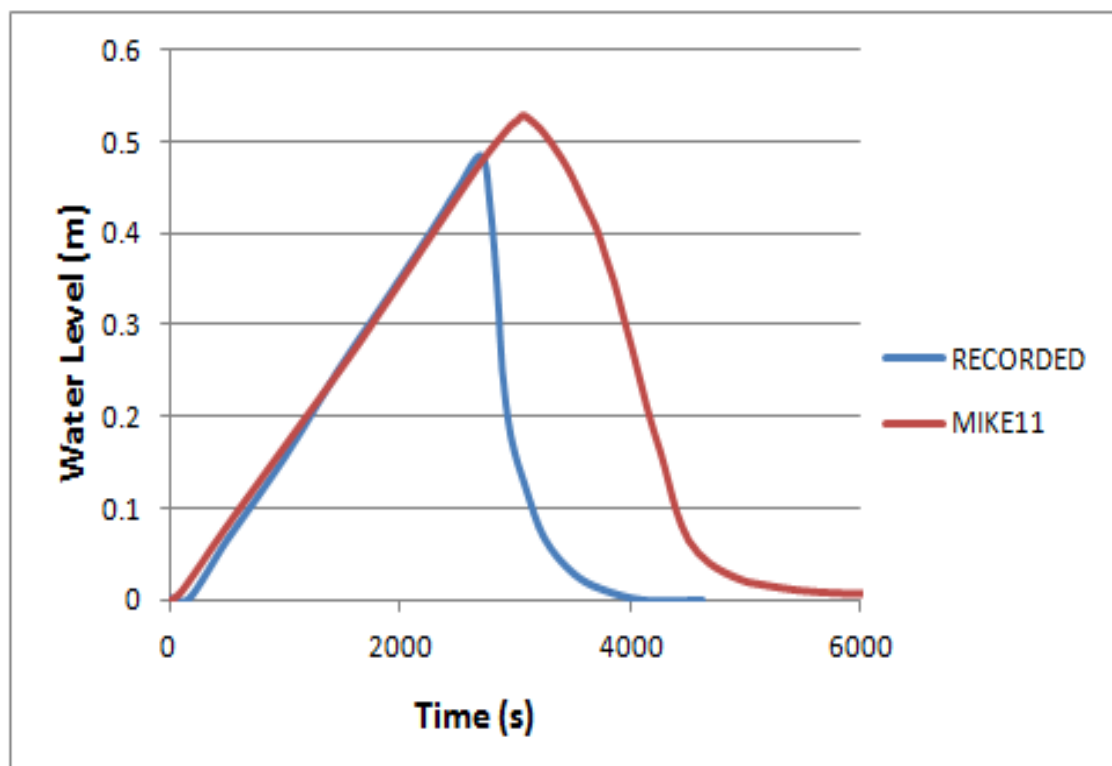


Figure 12.20: IMPACT Lab Test #6- water level - recorded vs. MIKE11

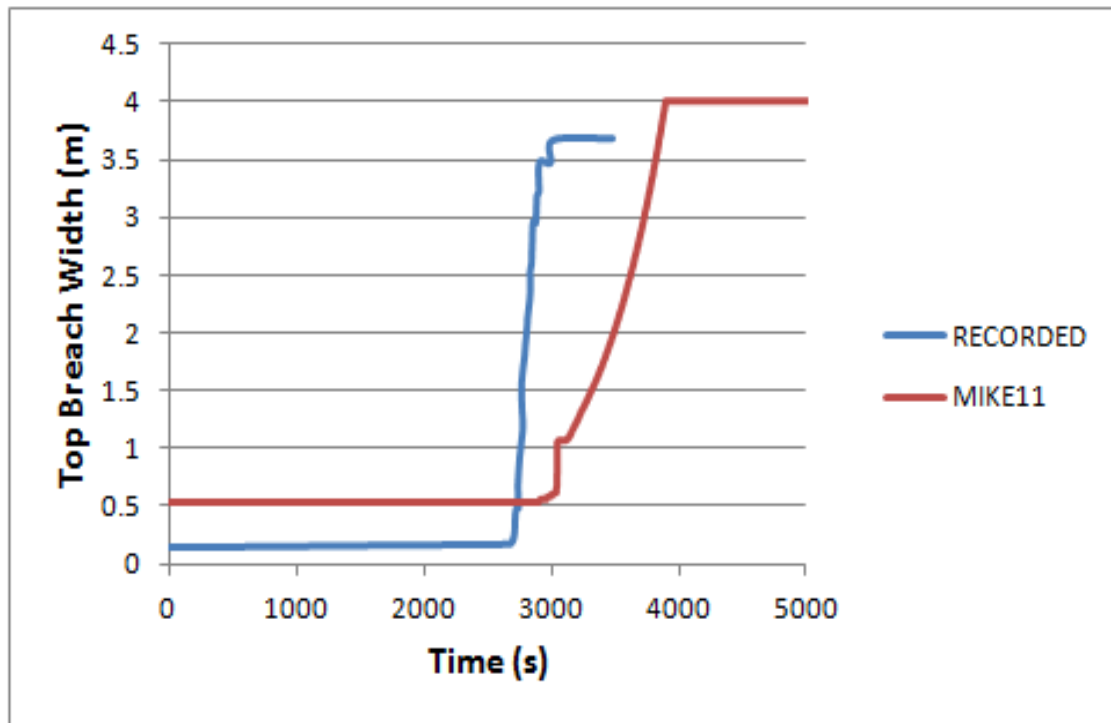


Figure 12.21: IMPACT Lab Test #6- top breach width - recorded vs. MIKE11

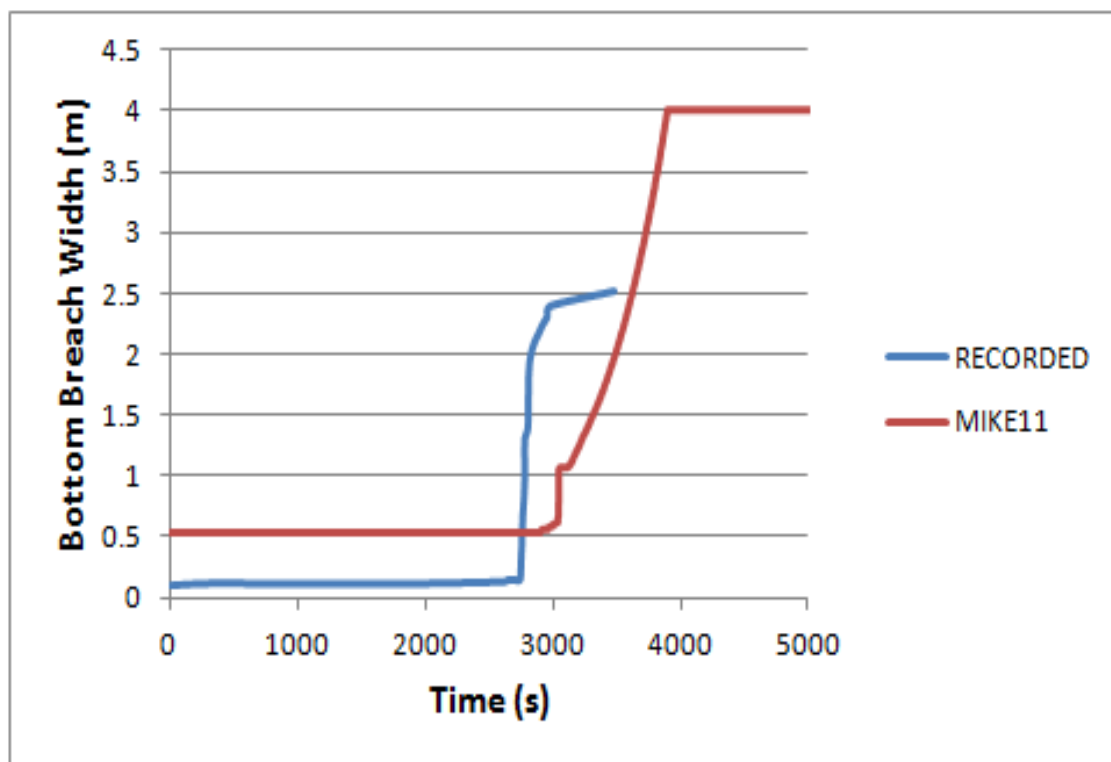


Figure 12.22: IMPACT Lab Test #6- bottom breach width - recorded vs. MIKE11

Table 12.7: IMPACT Lab Test #6 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak	Peak Water Level	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.92	2880.00	2695.35	1403.86	0.3	0.49	NA	0.5	2.9	3.1
MIKE11	0.20	3917.23	2862.07	2386.192	0.32	0.525	0.276	0.5	3.932	4
Error	77.80%	36.02%	6.19%	69.97%	6.67%	7.14%	NA	0.00%	35.59%	29.03%

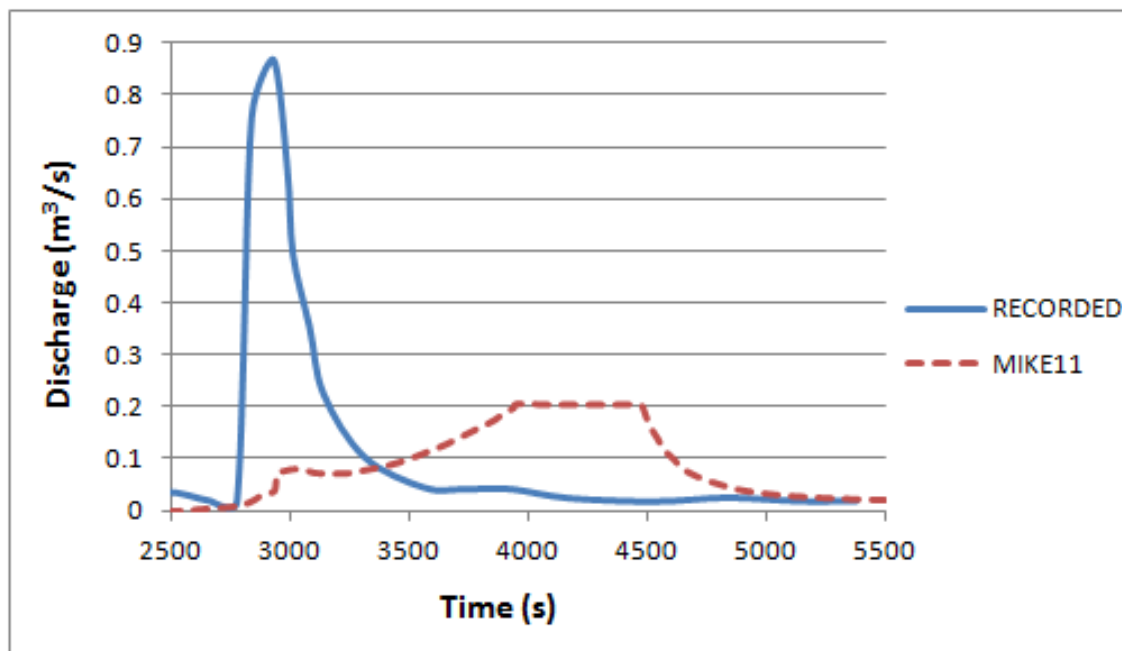


Figure 12.23: IMPACT Lab Test #7- discharge - recorded vs. MIKE11

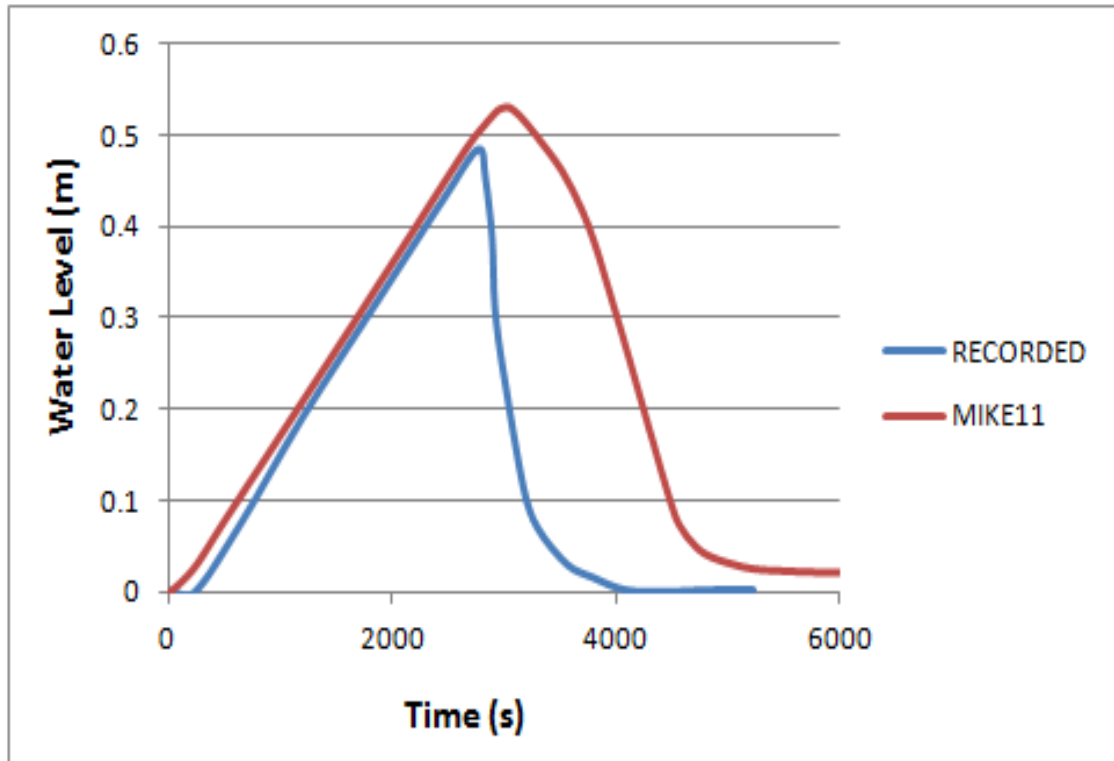


Figure 12.24: IMPACT Lab Test #7- water level - recorded vs. MIKE11

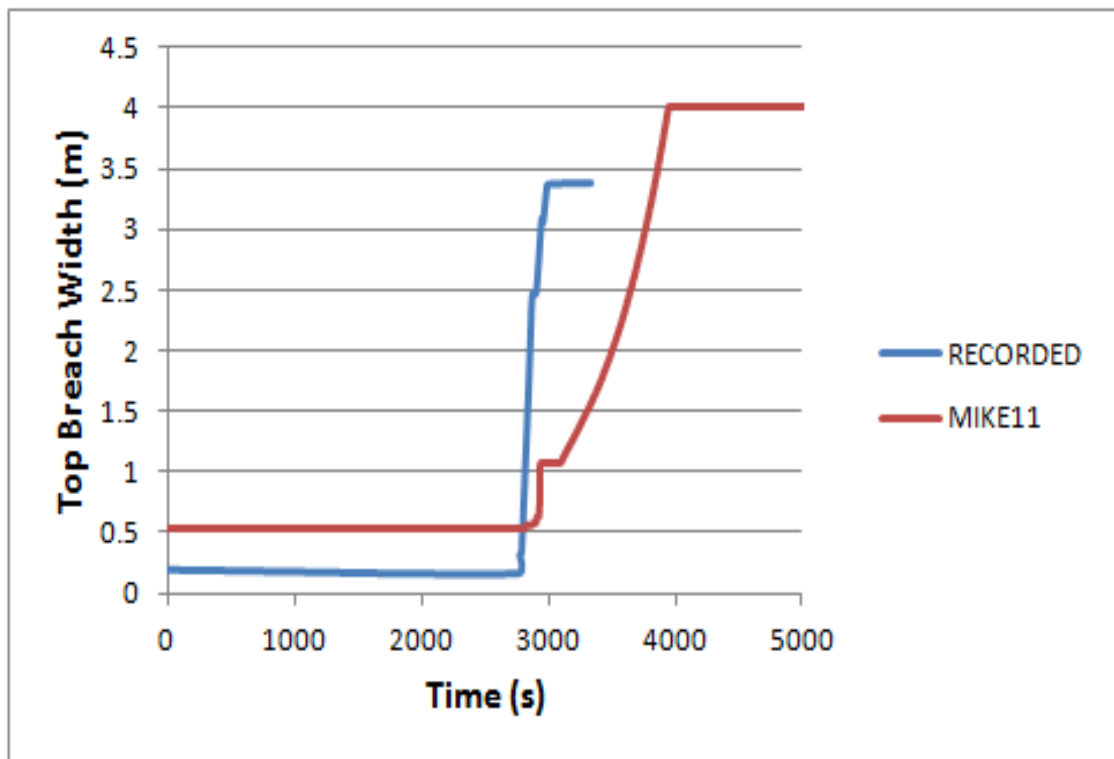


Figure 12.25: IMPACT Lab Test #7- top breach width - recorded vs. MIKE11

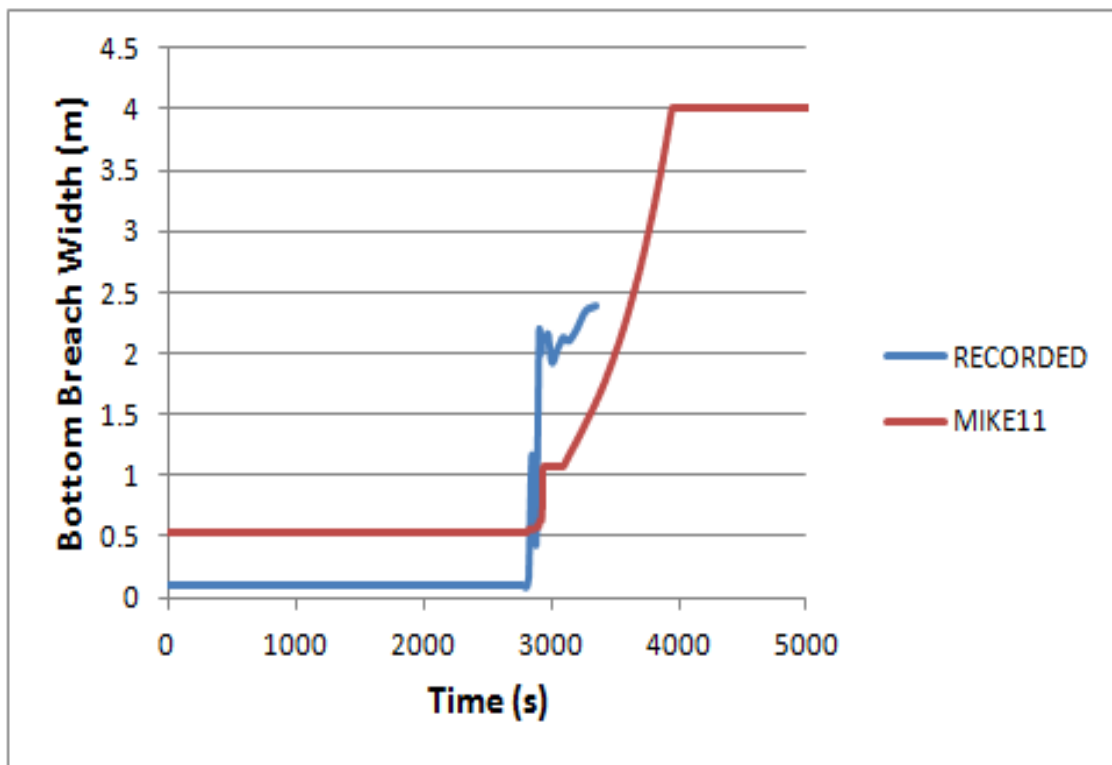


Figure 12.26: IMPACT Lab Test #7- bottom breach width - recorded vs. MIKE11

Table 12.8: IMPACT Lab Test #7 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak	Peak Water Level	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.88	2916.00	2773.85	787.46	0.31	0.49	NA	0.5	2.55	2.9
MIKE11	0.20	3986.24	2727.05	2849.53	0.31	0.53	0.297	0.5	4	4
Error	76.72%	36.70%	1.69%	261.86%	0.00%	8.16%	NA	0.00%	56.86%	37.93%

12.3 IMPACT Cohesive Lab Test Comparisons

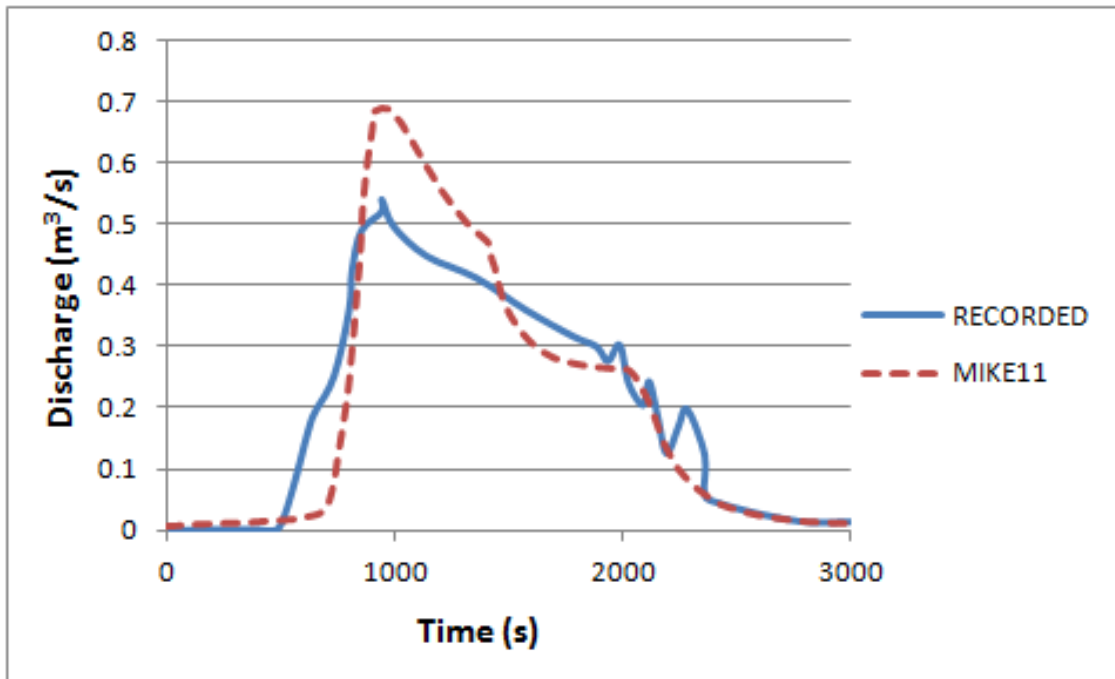


Figure 12.27: IMPACT Lab Test #12- discharge - recorded vs. MIKE11

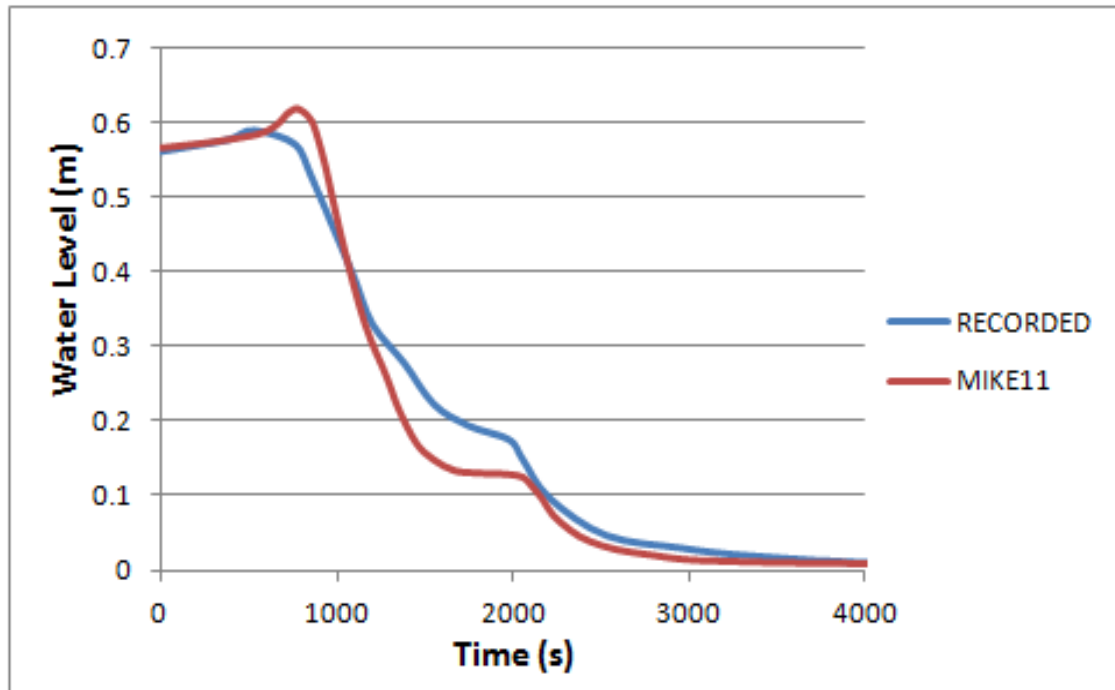


Figure 12.28: IMPACT Lab Test #12- water level - recorded vs. MIKE11

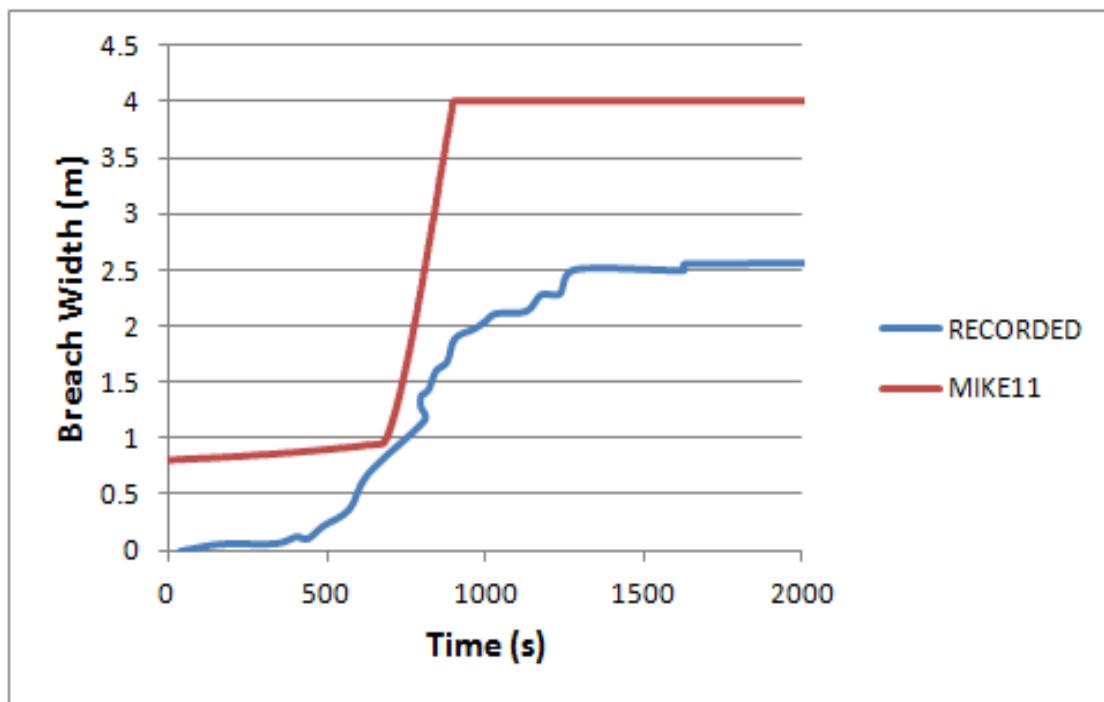


Figure 12.29: IMPACT Lab Test #12- breach width - recorded vs. MIKE11

Table 12.9: IMPACT Lab Test #12 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.53	936.00	491.78	2471.384	0.47	0.58	NA	N/A	1.99	2.55
MIKE11	0.69	940.96	734.98	1816.3872	0.52	0.62	0.322	0.5	4	4
Error	30.24%	0.53%	49.45%	26.50%	10.64%	6.90%	NA	N/A	101.01%	56.86%

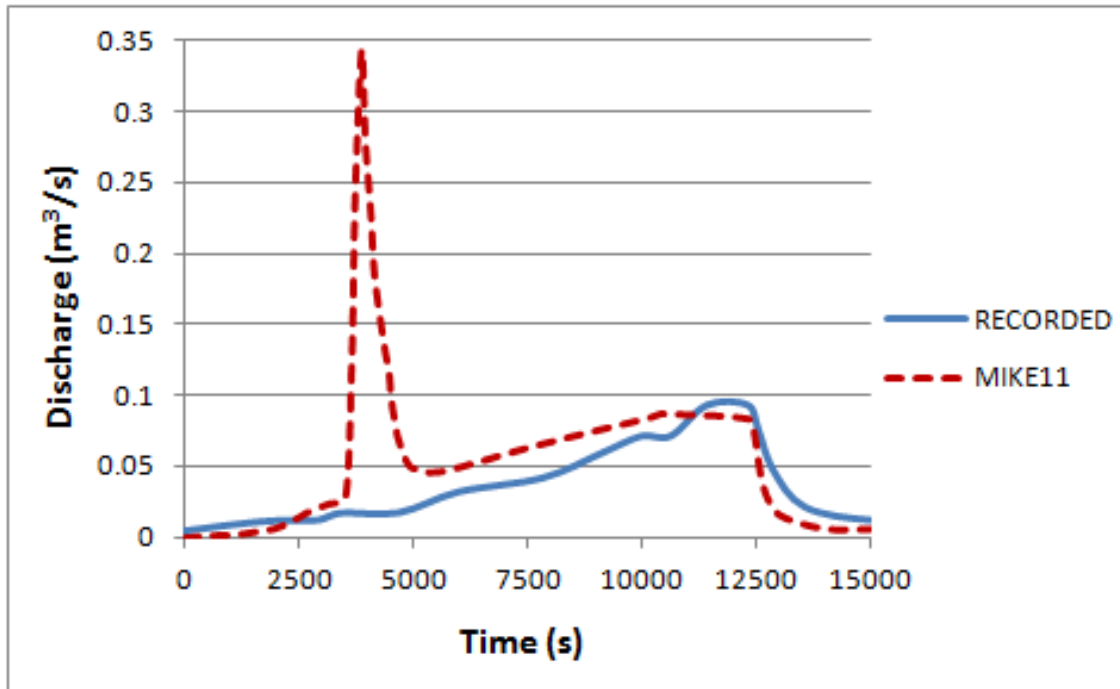


Figure 12.30: IMPACT Lab Test #13- discharge - recorded vs. MIKE11

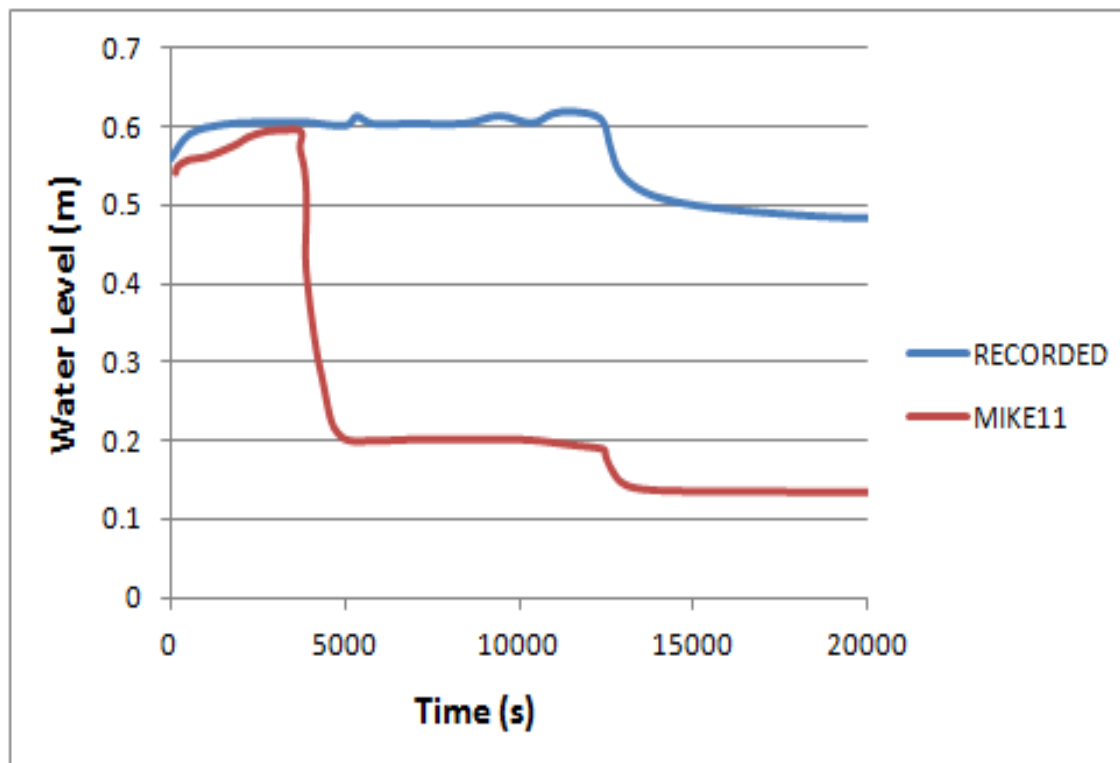


Figure 12.31: IMPACT Lab Test #13- water level - recorded vs. MIKE11

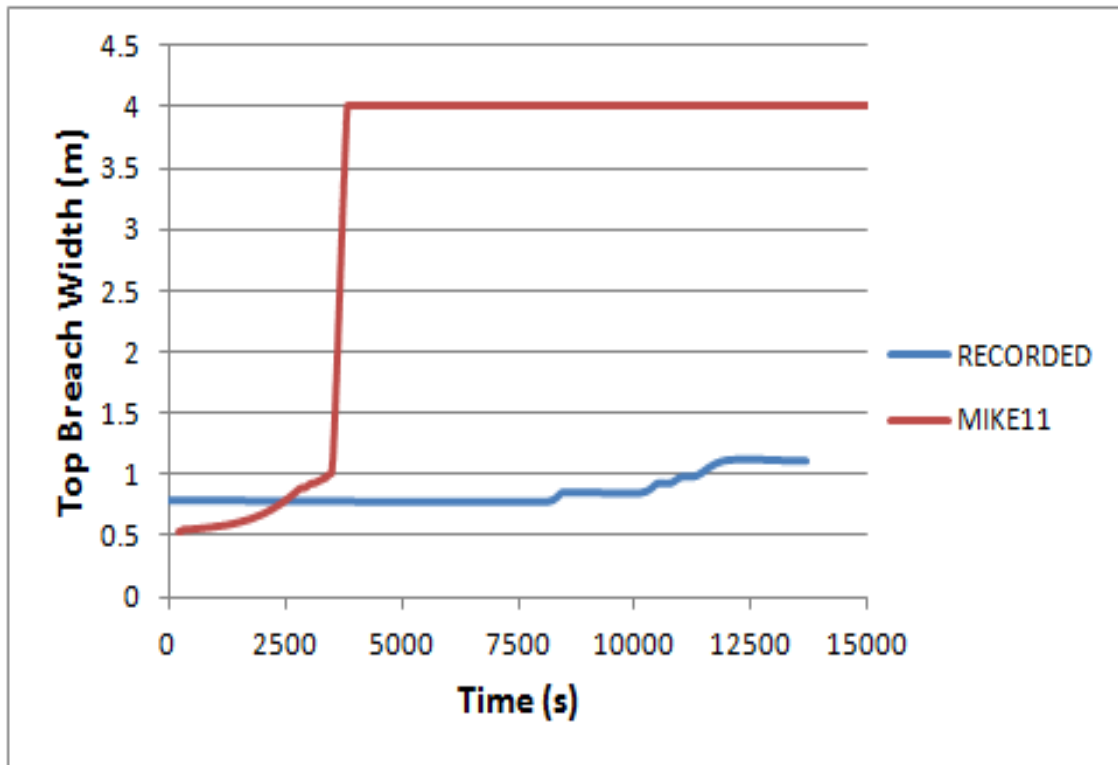


Figure 12.32: IMPACT Lab Test #13- top breach width - recorded vs. MIKE11

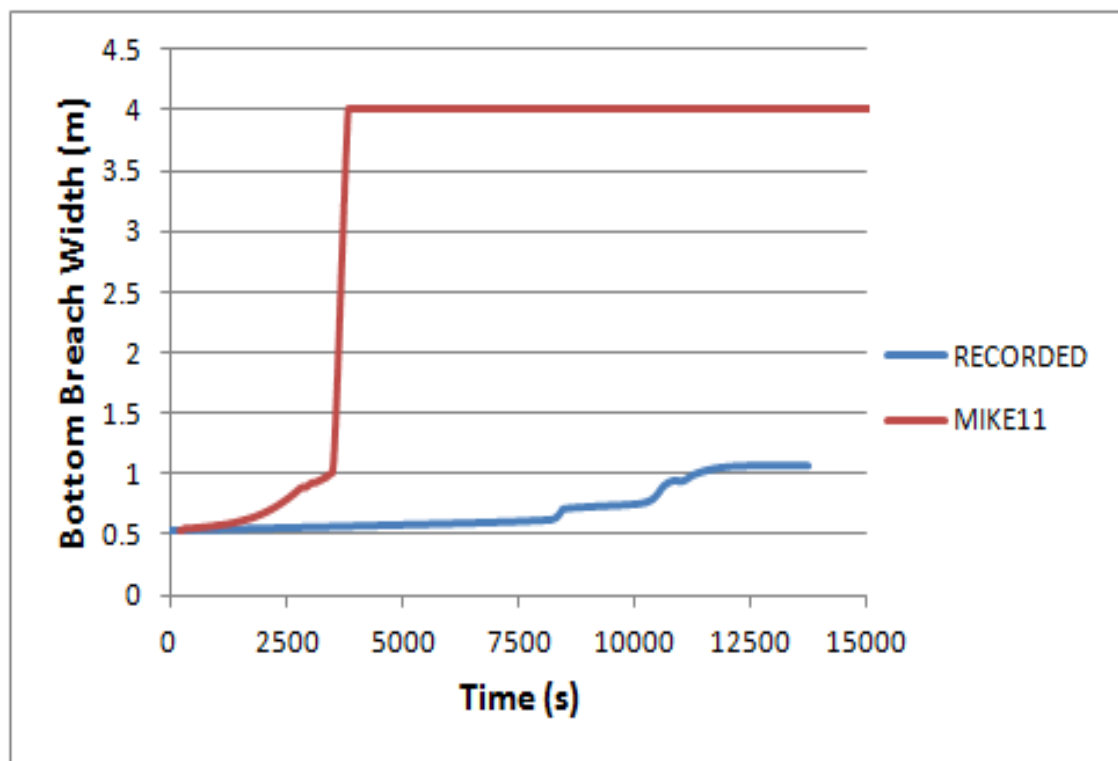


Figure 12.33: IMPACT Lab Test #13- bottom breach width - recorded vs. MIKE11

Table 12.10: IMPACT Lab Test #13 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.09	12348.00	2137.11	12918.29	0.60	0.61	NA	0.15	1.01	1.07
MIKE11	0.34	3863.12	2321.69	11046.95	0.59	0.46	0.14	0.54	1.87	4.00
Error	278.97%	68.71%	8.64%	14.49%	1.67%	24.59%	NA	N/A	85.35%	273.83%

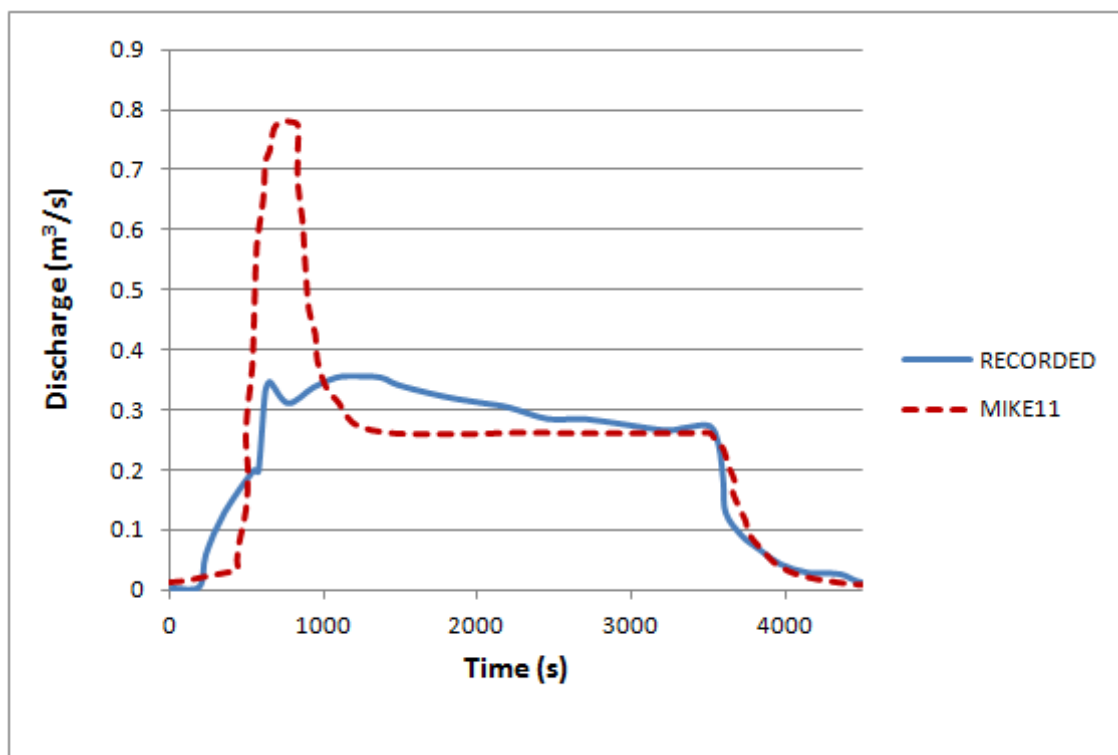


Figure 12.34: IMPACT Lab Test #15- discharge - recorded vs. MIKE11

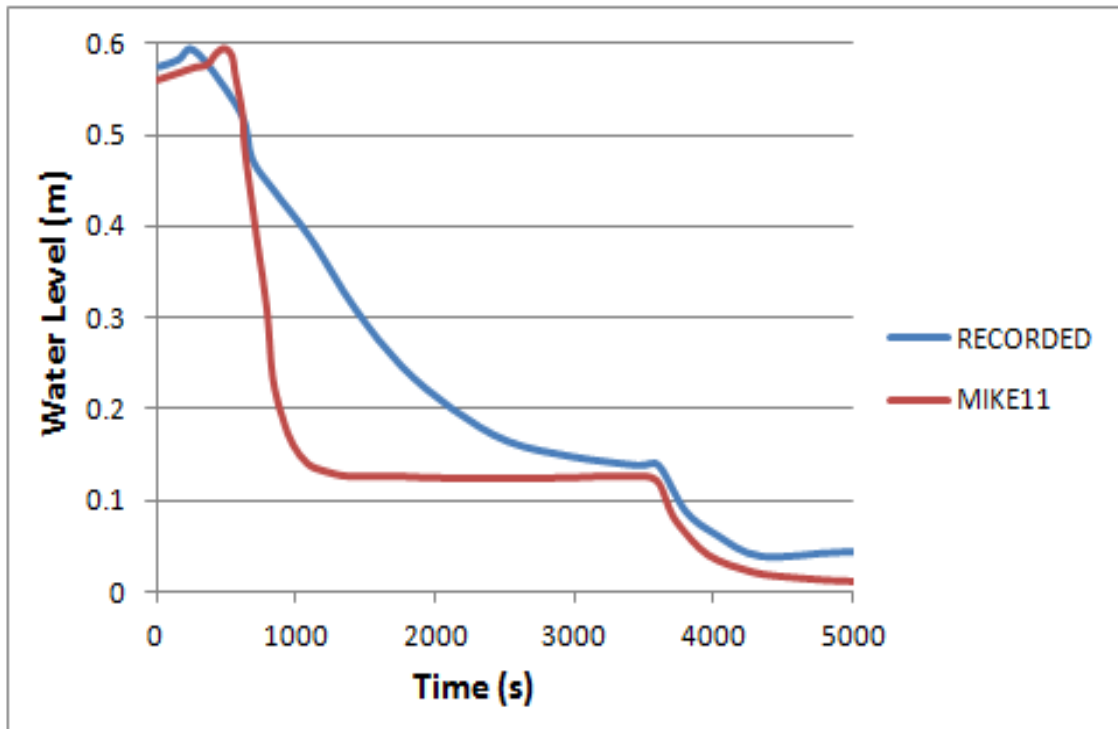


Figure 12.35: IMPACT Lab Test #15- water level - recorded vs. MIKE11

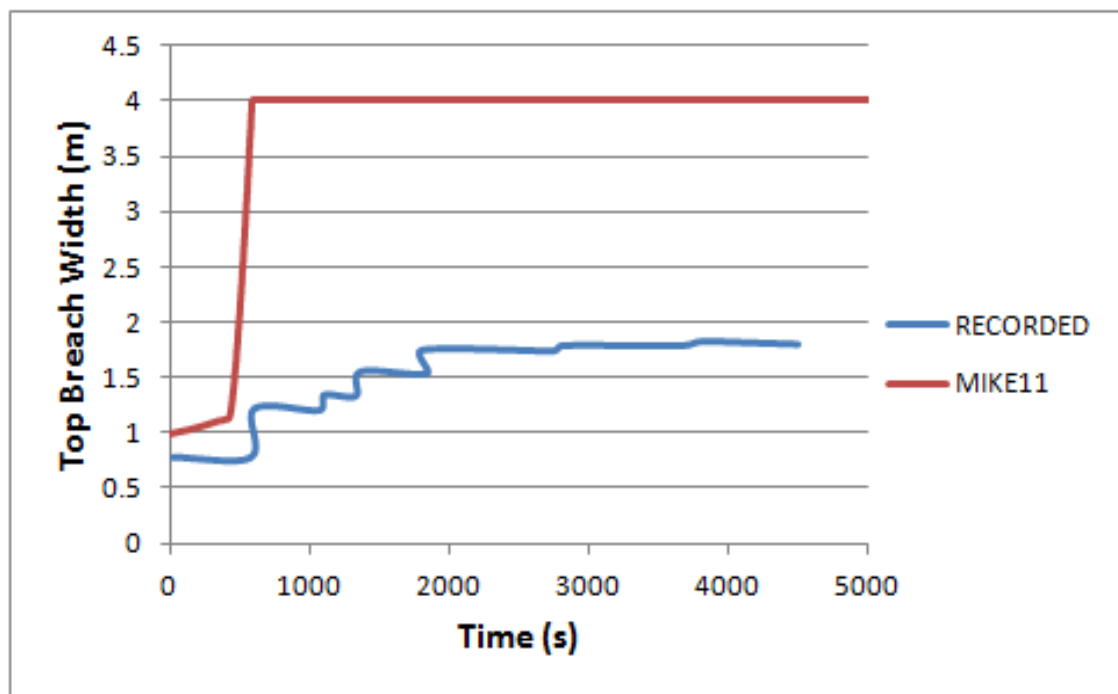


Figure 12.36: IMPACT Lab Test #15- top breach width - recorded vs. MIKE11

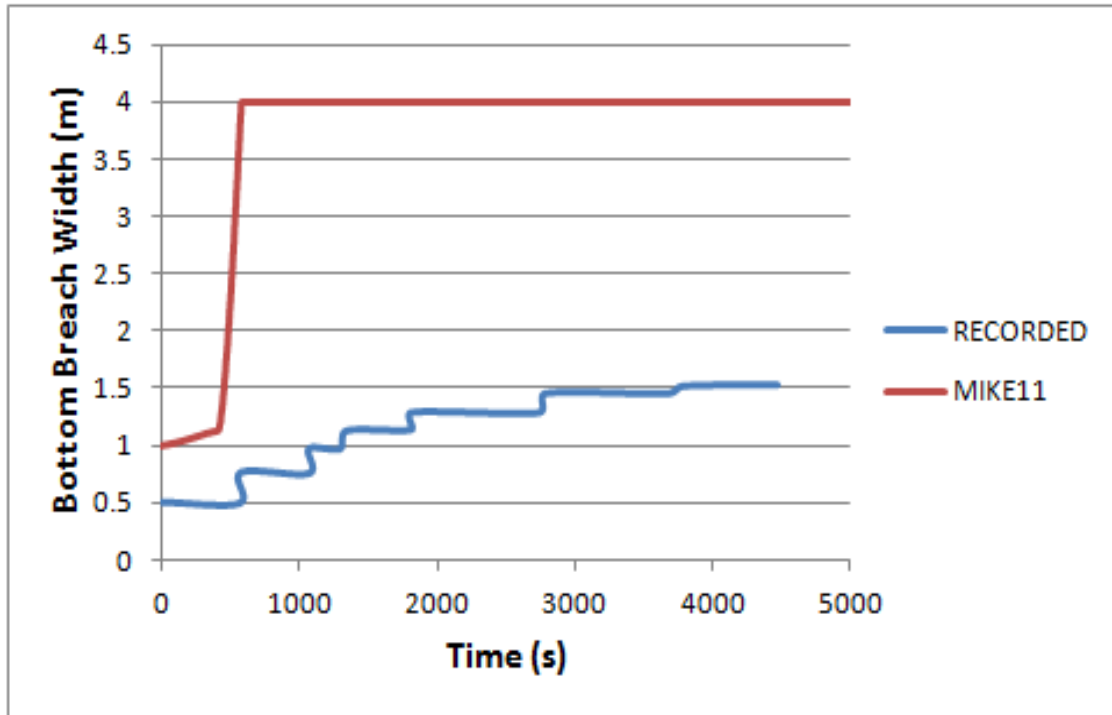


Figure 12.37: IMPACT Lab Test #15- bottom breach width - recorded vs. MIKE11

Table 12.11: IMPACT Lab Test #15 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.35	1260.00	543.60	3697.92	0.35	0.59	NA	0.6	1.47	1.73
MIKE11	0.78	748.23	451.76	4150.584	0.35	0.59	0.52	0.6	4	4
Error	122.91%	40.62%	16.89%	12.24%	0.00%	0.00%	NA	N/A	172.11%	131.21%

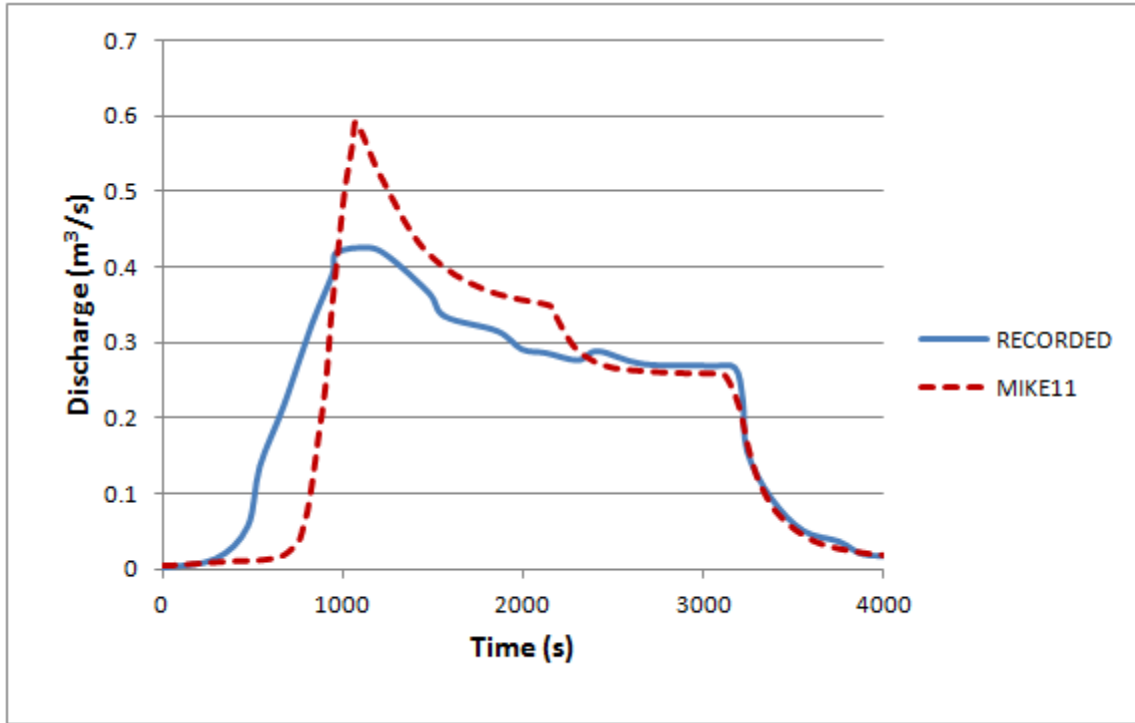


Figure 12.38: IMPACT Lab Test #16- discharge - recorded vs. MIKE11

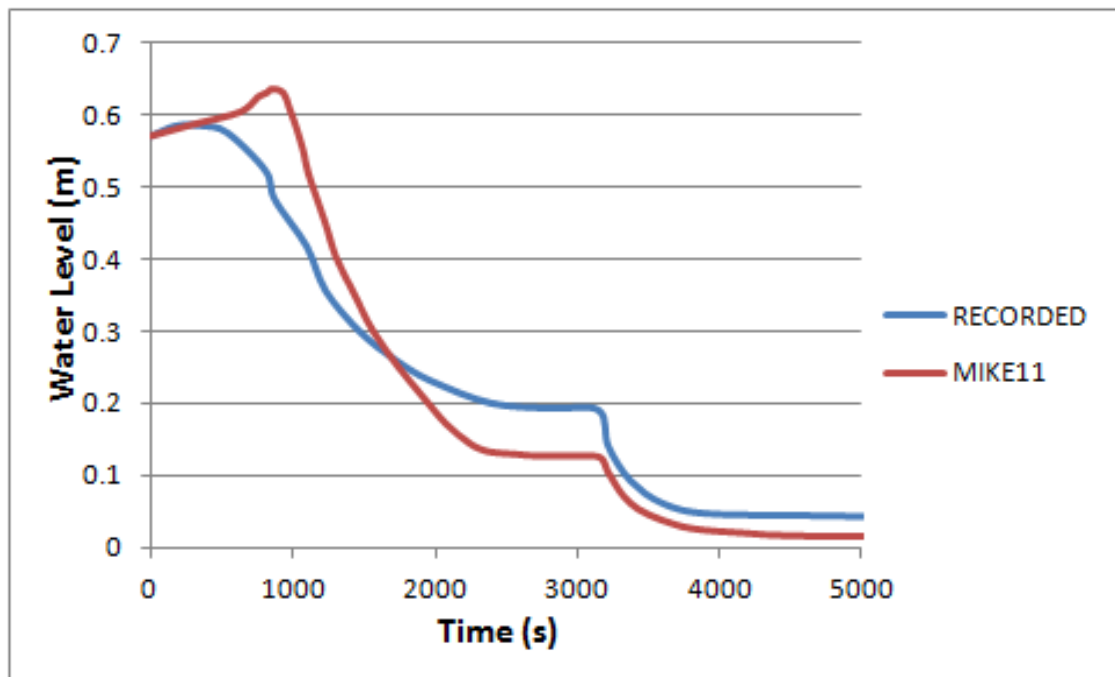


Figure 12.39: IMPACT Lab Test #16- water level - recorded vs. MIKE11

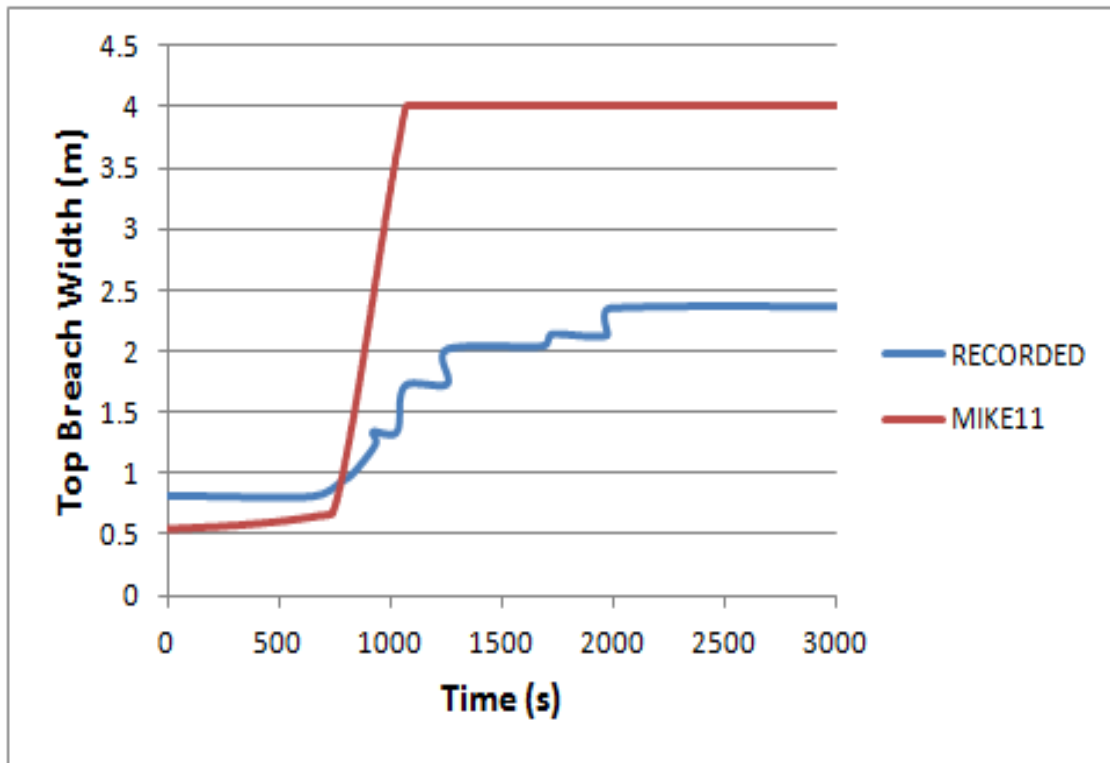


Figure 12.40: IMPACT Lab Test #16- top breach width - recorded vs. MIKE11

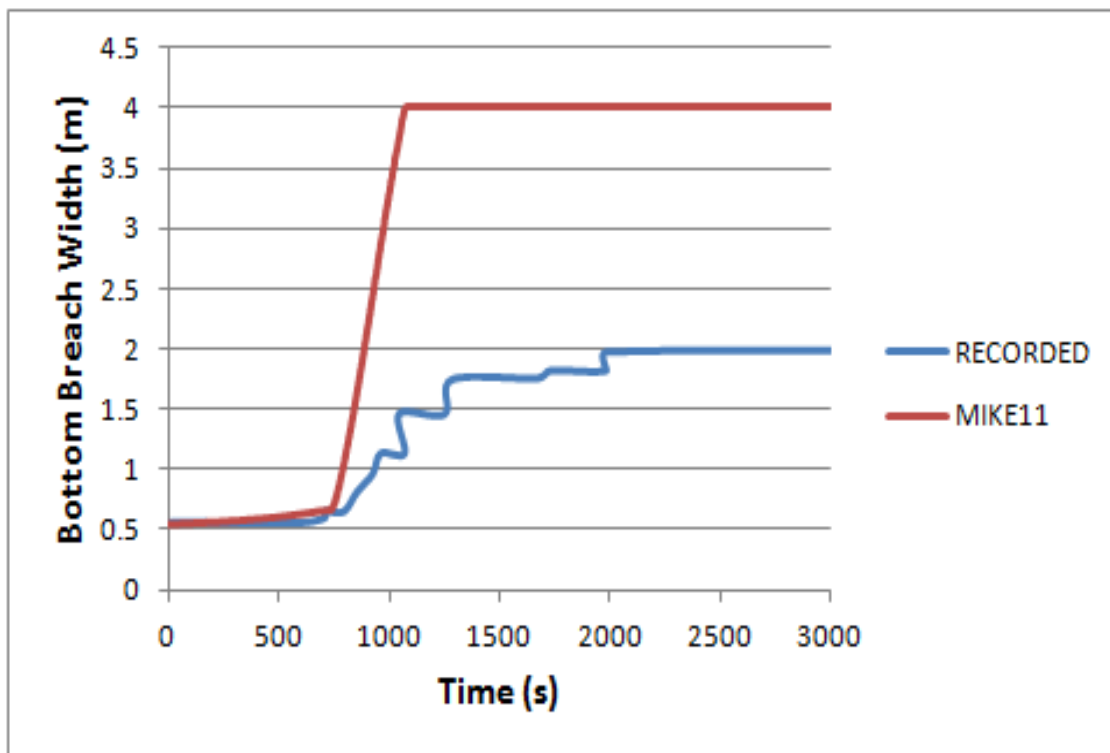


Figure 12.41: IMPACT Lab Test #16- bottom breach width - recorded vs. MIKE11

Table 12.12: IMPACT Lab Test #16 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.43	1152.00	290.50	3597.77	0.40	0.59	NA	0.60	1.69	2.31
MIKE11	0.59	1067.70	691.86	3458.51	0.55	0.63	0.28	0.60	3.98	4.00
Error	37.80%	7.32%	138.16%	3.87%	37.50%	7.46%	NA	0.00%	135.44%	73.16%

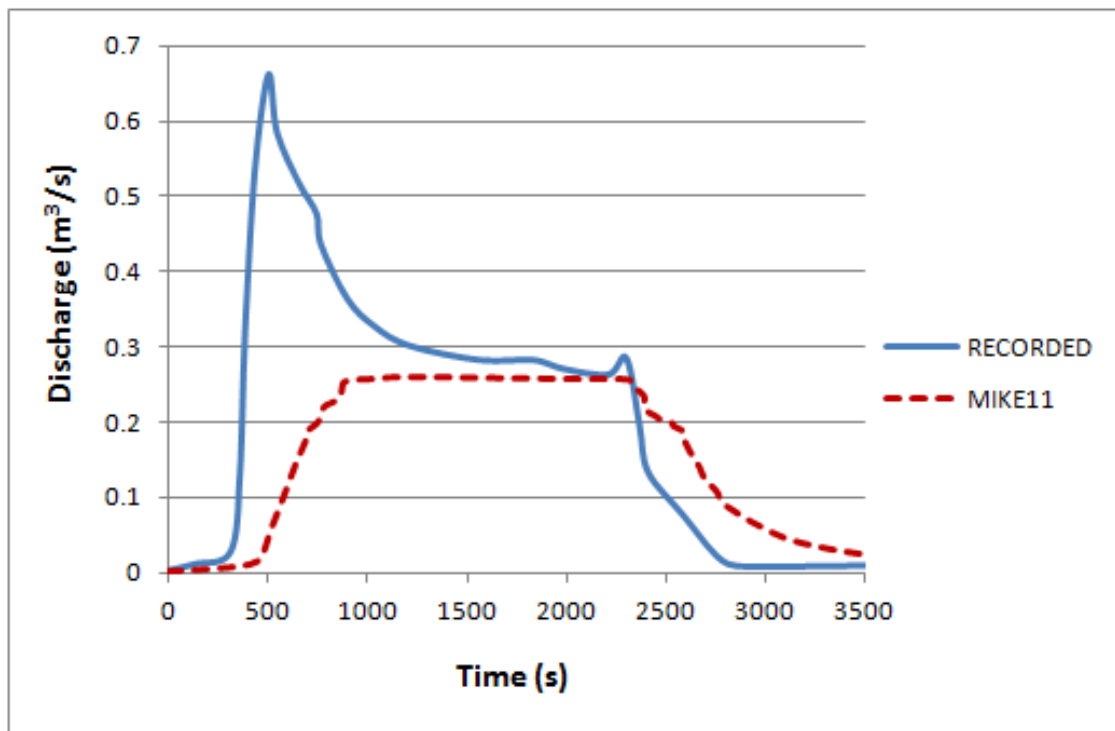


Figure 12.42: IMPACT Lab Test #17- discharge - recorded vs. MIKE11

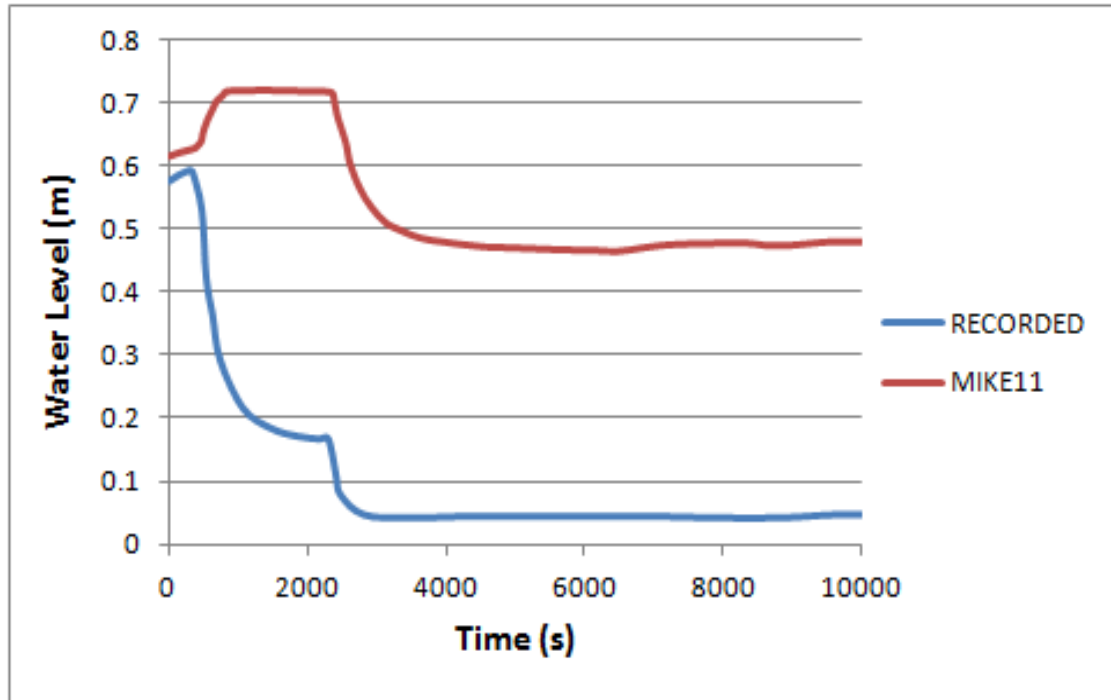


Figure 12.43: IMPACT Lab Test #17- water level - recorded vs. MIKE11

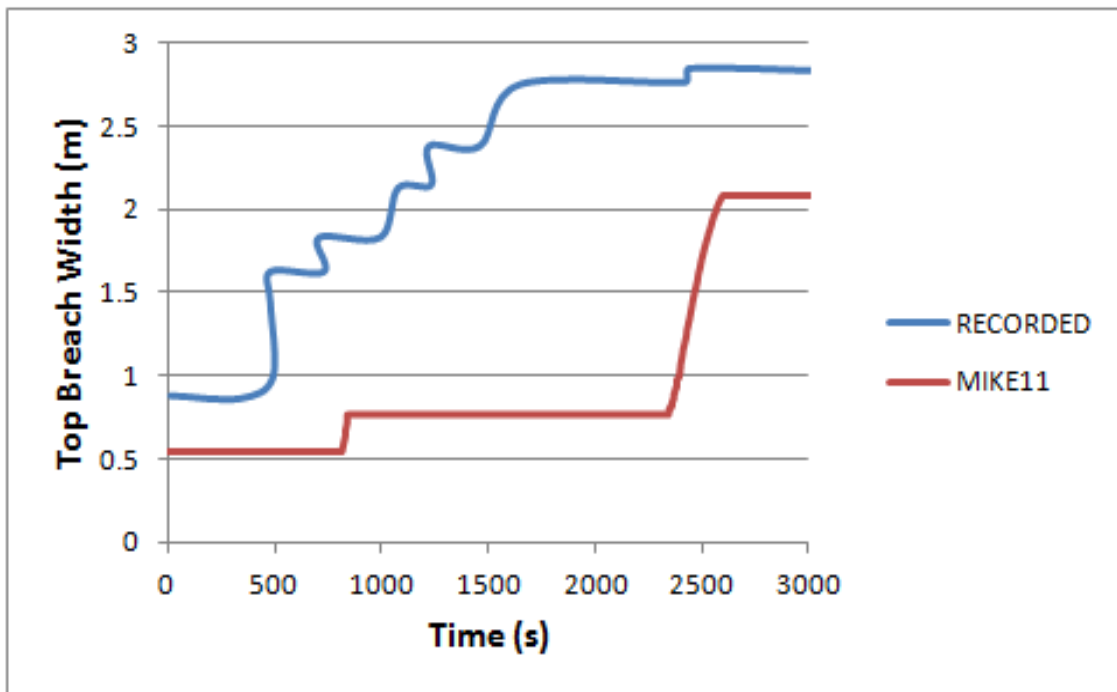


Figure 12.44: IMPACT Lab Test #17- top breach width - recorded vs. MIKE11

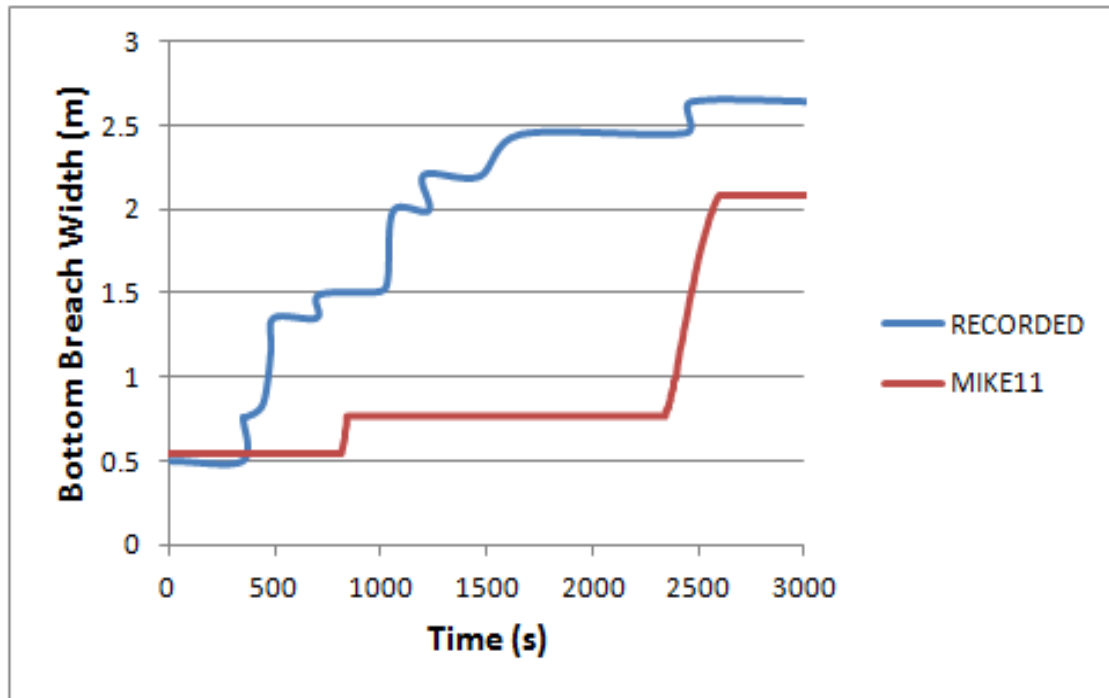


Figure 12.45: IMPACT Lab Test #17- bottom breach width - recorded vs. MIKE11

Table 12.13: IMPACT Lab Test #17 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Lag Time (s)	Breach Time (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)	Breach Width at Peak (m)	Final Breach Width (m)
RECORDED	0.61	504.00	313.35	2435.703	0.46	0.59		0.6	1.47	2.71
MIKE11	0.26	1147.15	447.67	3991.709	0.717	0.717	0.065	0.154	0.765	2.105
Error	57.51%	127.61%	42.87%	63.88%	55.87%	21.53%	NA	N/A	47.96%	22.32%

12.4 Delft University of Technology Test Comparisons

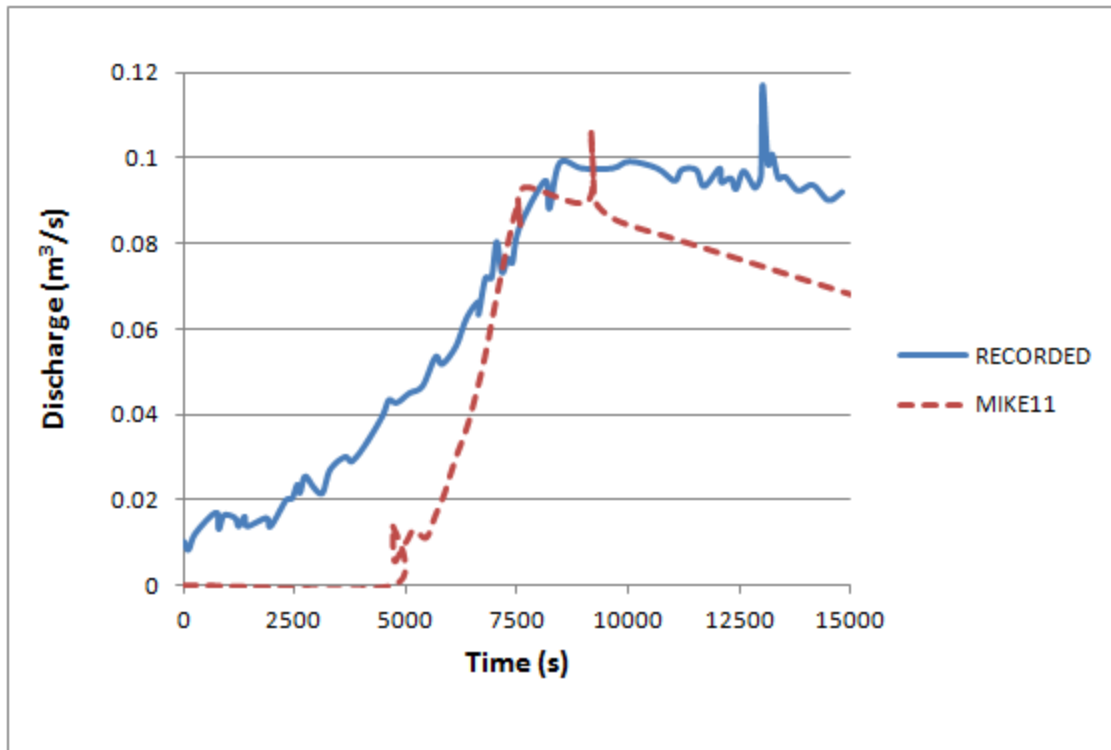


Figure 12.46: Delft University of Technology Lab Test #2 discharge results

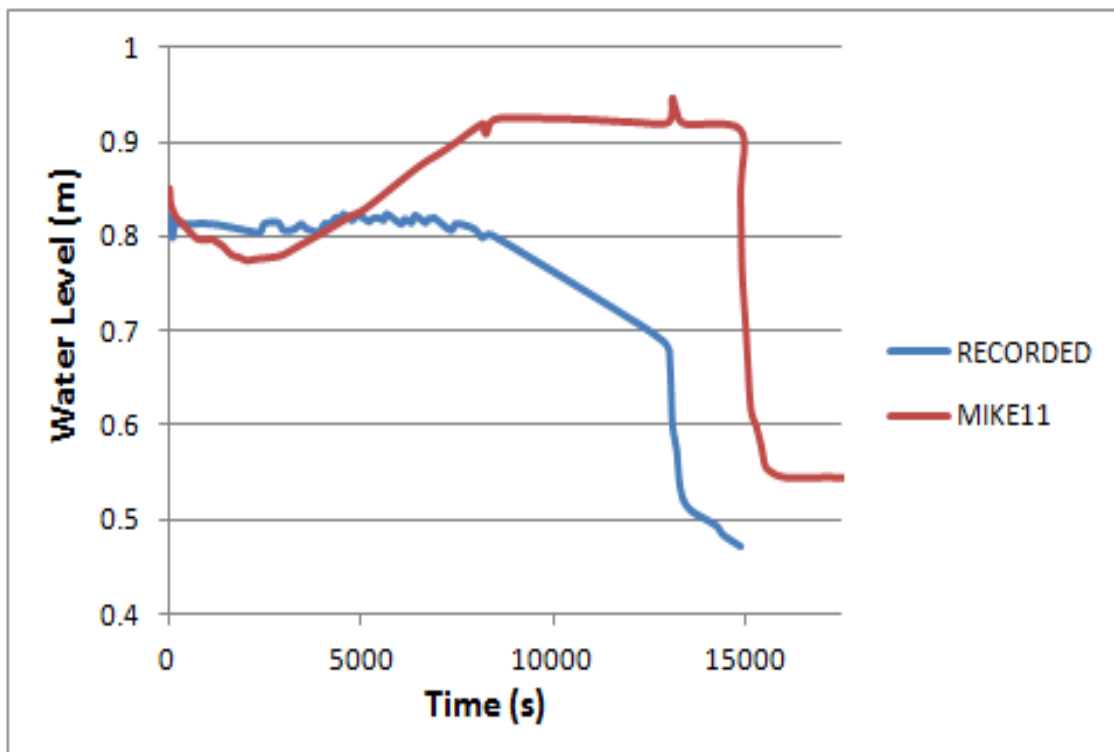


Figure 12.47: Delft University of Technology Lab Test #2 water level results

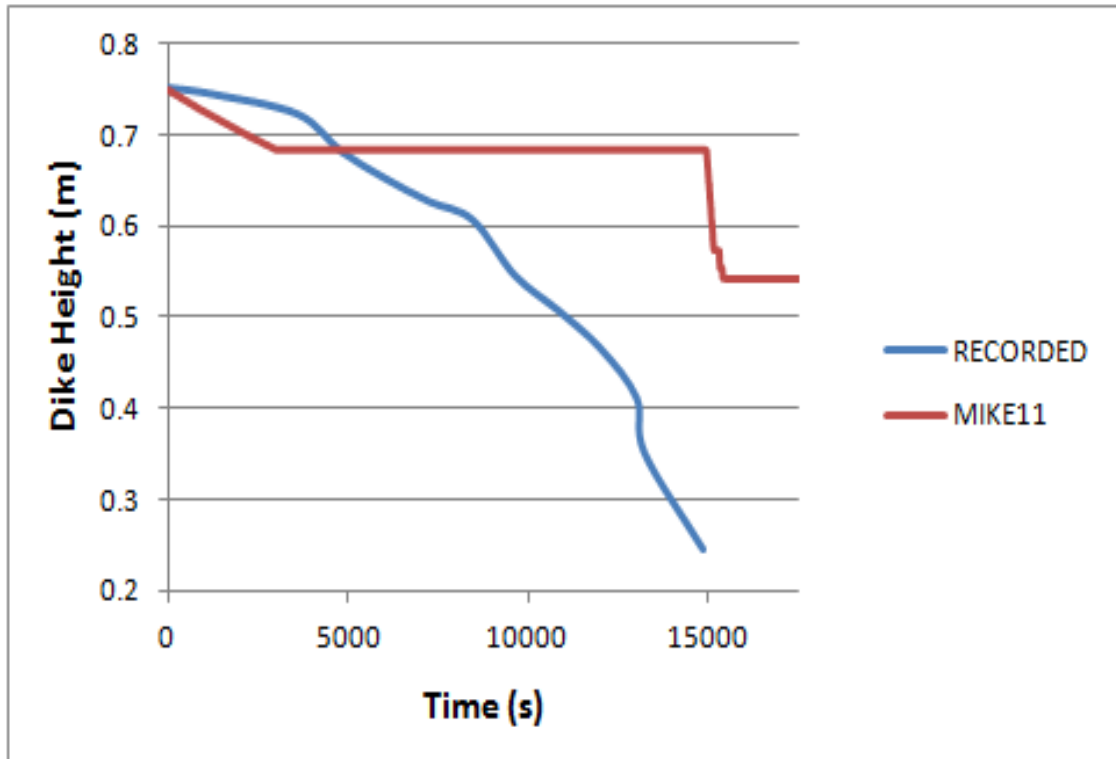


Figure 12.48: Delft University of Technology Lab Test #2 dike height results

Table 12.14: Delft University of Technology Lab Test #2 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)
RECORDED	0.099	10036.6	0.752	0.829	0.310	0.600
MIKE11	0.093	7639.6	0.920	0.950	0.307	0.307
Error	6.08%	23.88%	22.34%	14.60%	0.97%	48.83%

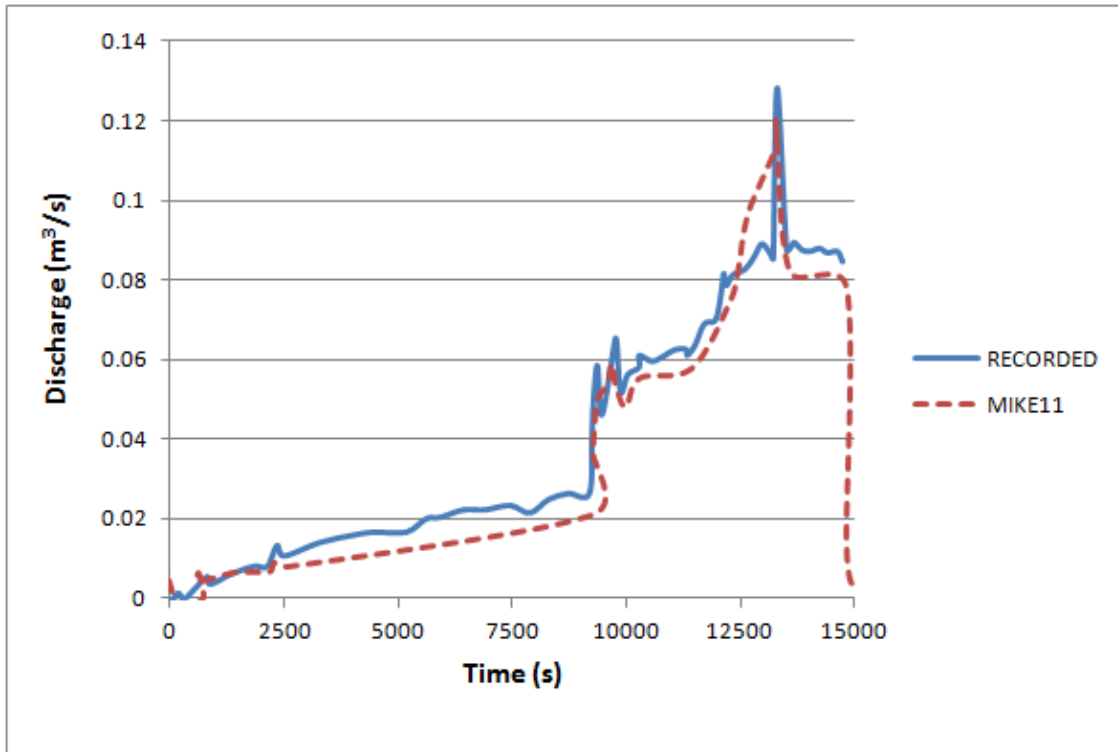


Figure 12.49: Delft University of Technology Lab Test #3 discharge results

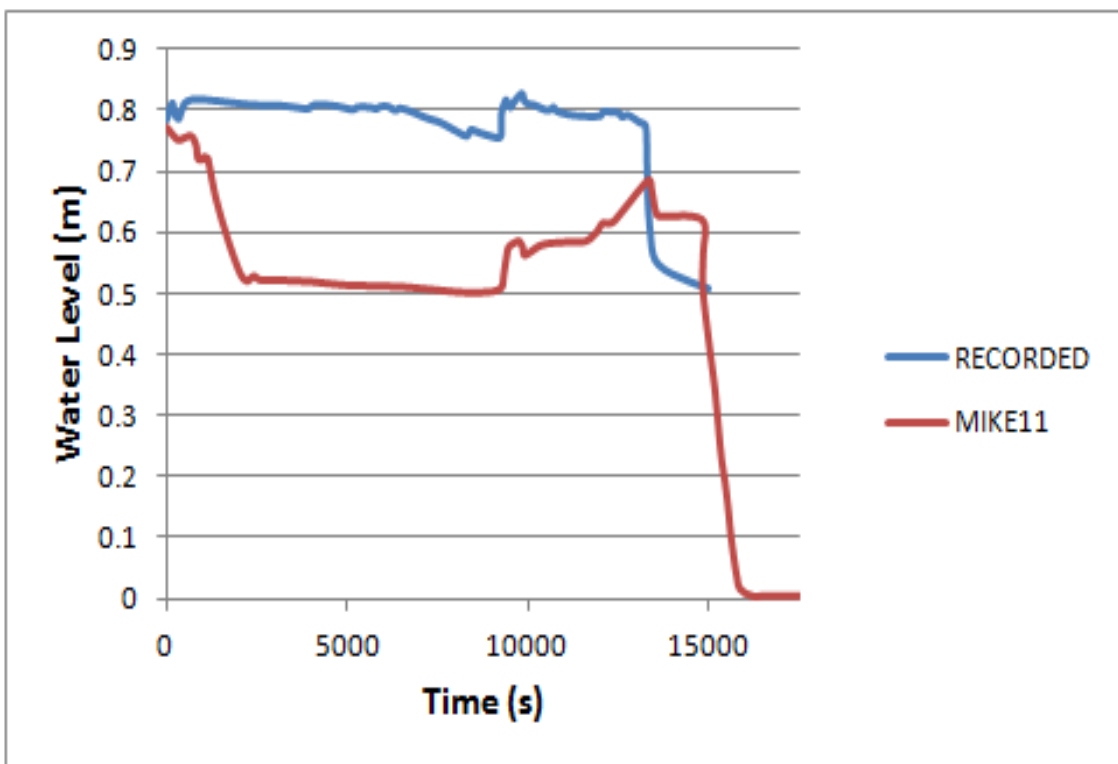


Figure 12.50: Delft University of Technology Lab Test #3 water level results

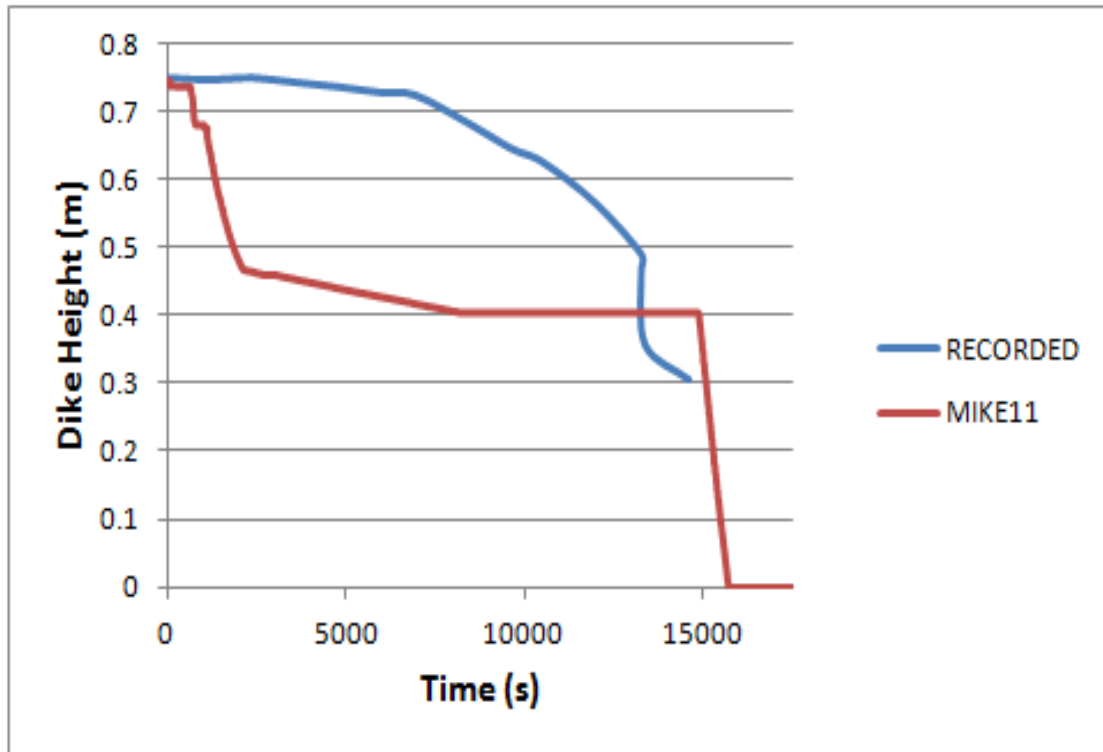


Figure 12.51: Delft University of Technology Lab Test #3 dike height results

Table 12.15: Delft University of Technology Lab Test #3 comparison

	Peak Flow (m ³ /s)	Time to Peak (s)	Water Level at Peak (m)	Peak Water Level (m)	Breach depth at Peak (m)	Final Breach Depth (m)
RECORDED	0.128298	13310.6	0.62	0.835	0.43	0.54
MIKE11	0.11934	13268.52	0.681	0.85	0.446	0.446
Error	6.98%	0.32%	9.84%	1.80%	28.06%	46.59%

12.5 Sensitivity Analysis

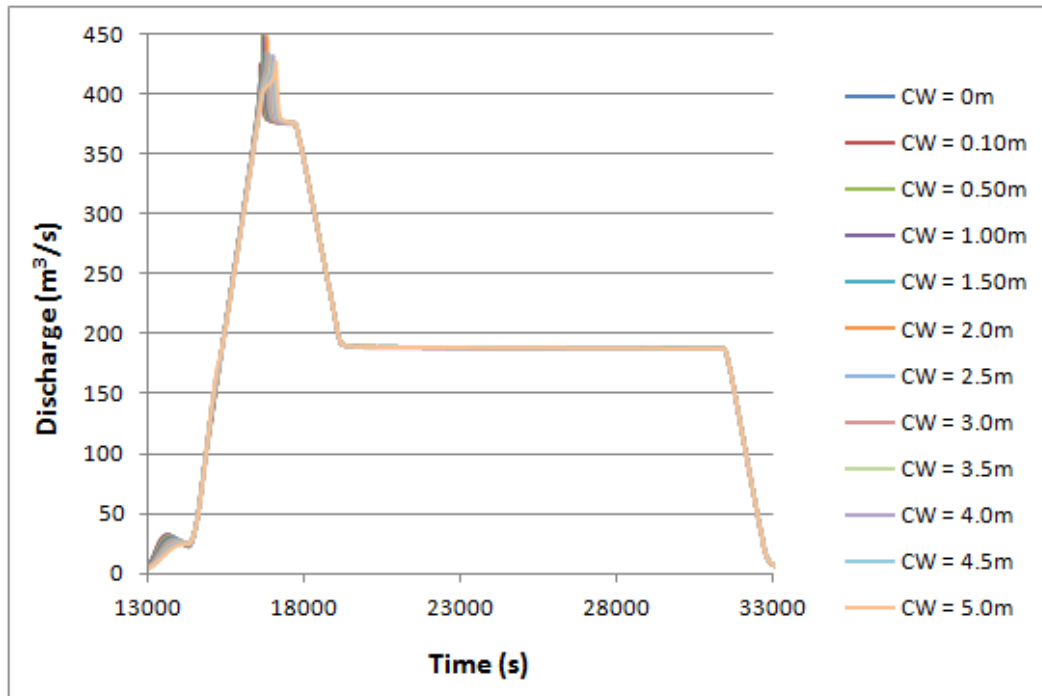


Figure 12.52: MIKE11 sensitivity analysis – crest width

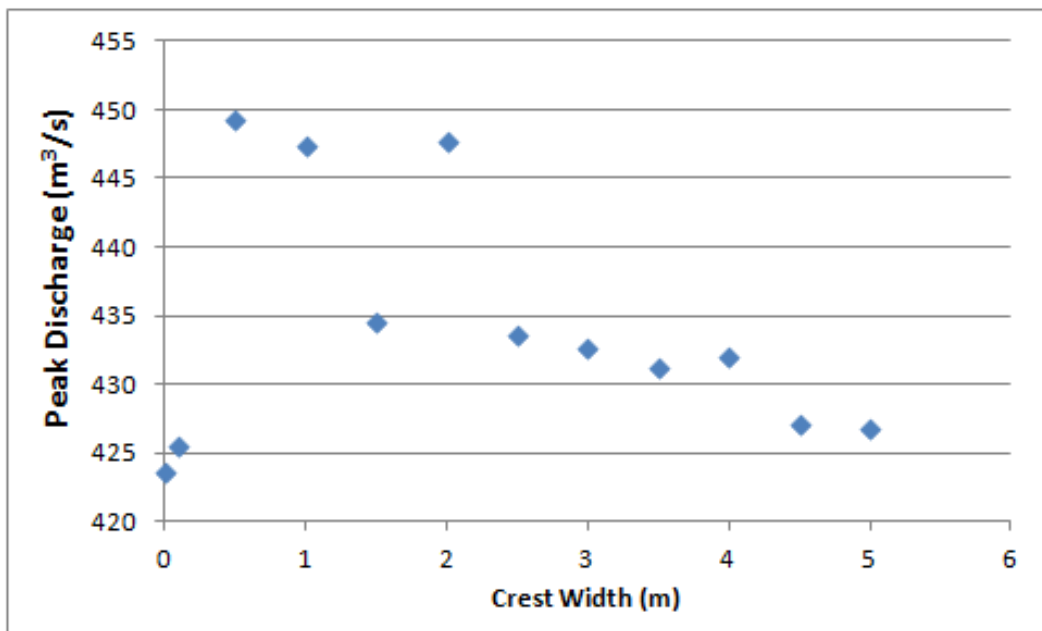


Figure 12.53: MIKE11 sensitivity analysis – crest width - peak discharge

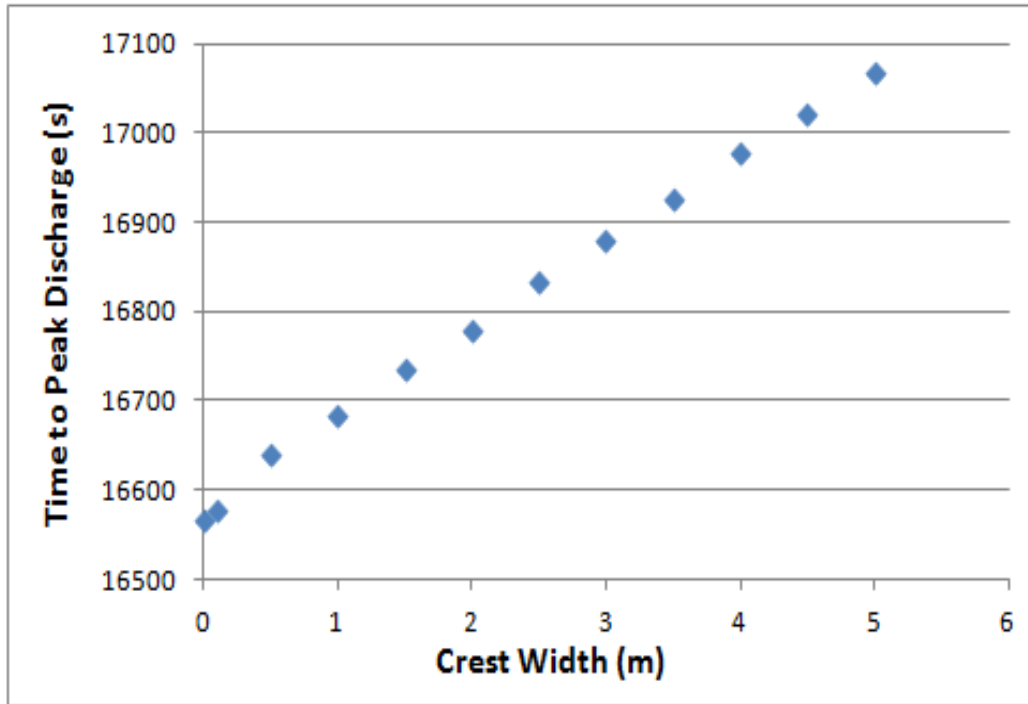


Figure 12.54: MIKE11 sensitivity analysis – crest width - time to peak discharge

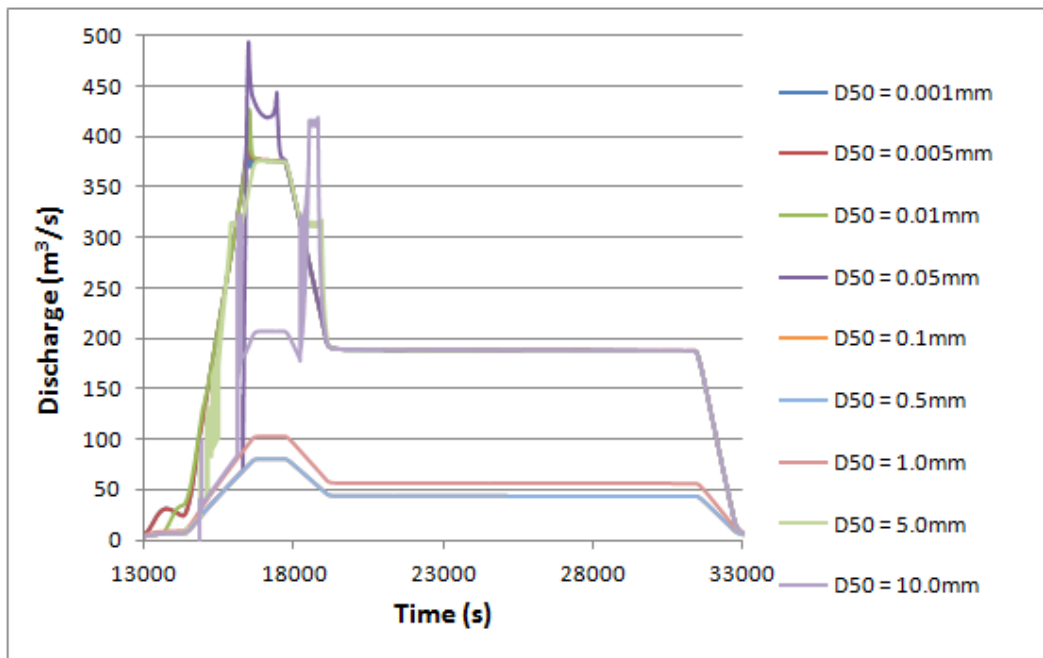


Figure 12.55: MIKE11 sensitivity analysis – D50 (median grain size)

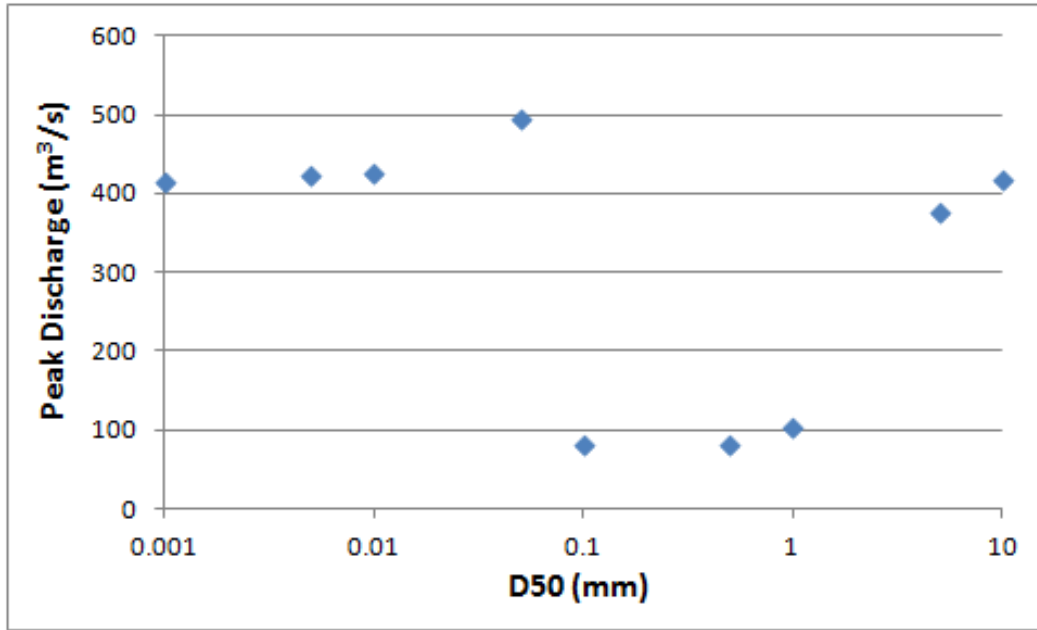


Figure 12.56: MIKE11 sensitivity analysis – D50 (median grain size) - peak discharge

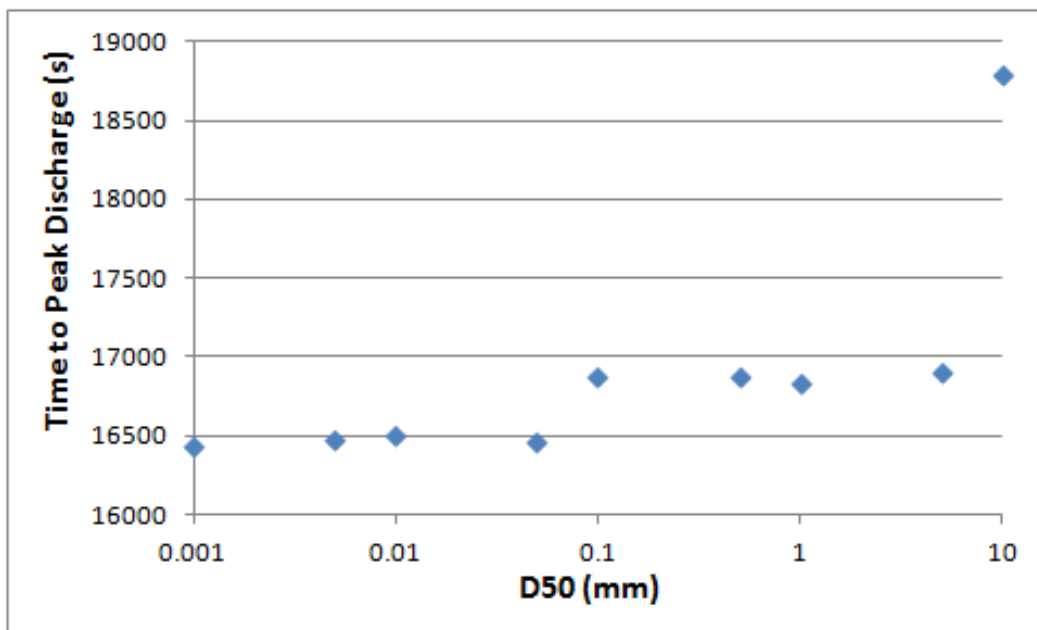


Figure 12.57: MIKE11 sensitivity analysis – D50 (median grain size) - time to peak discharge

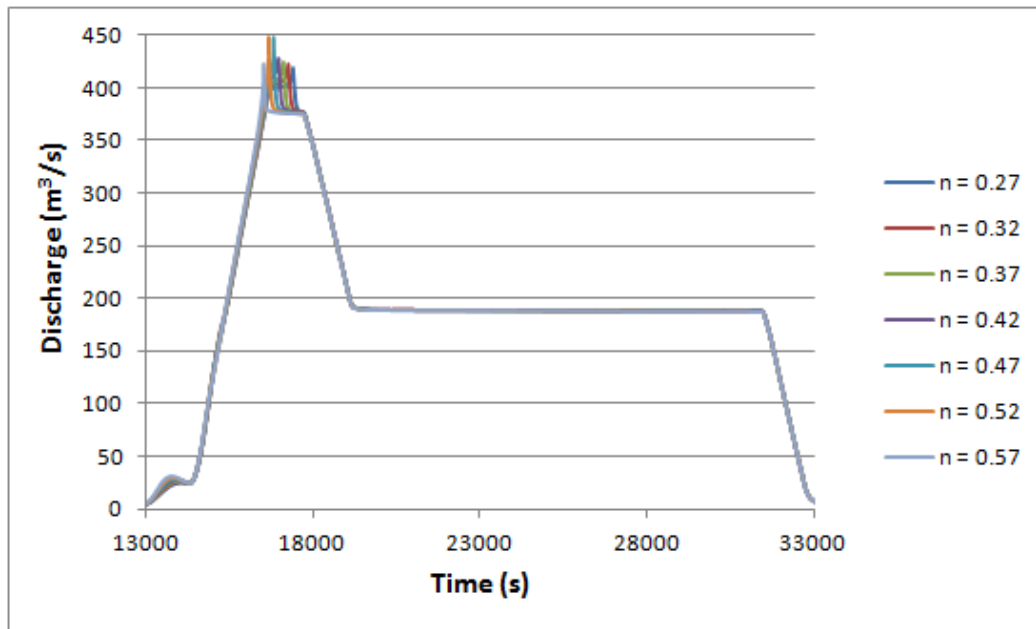


Figure 12.58: MIKE11 sensitivity analysis - porosity

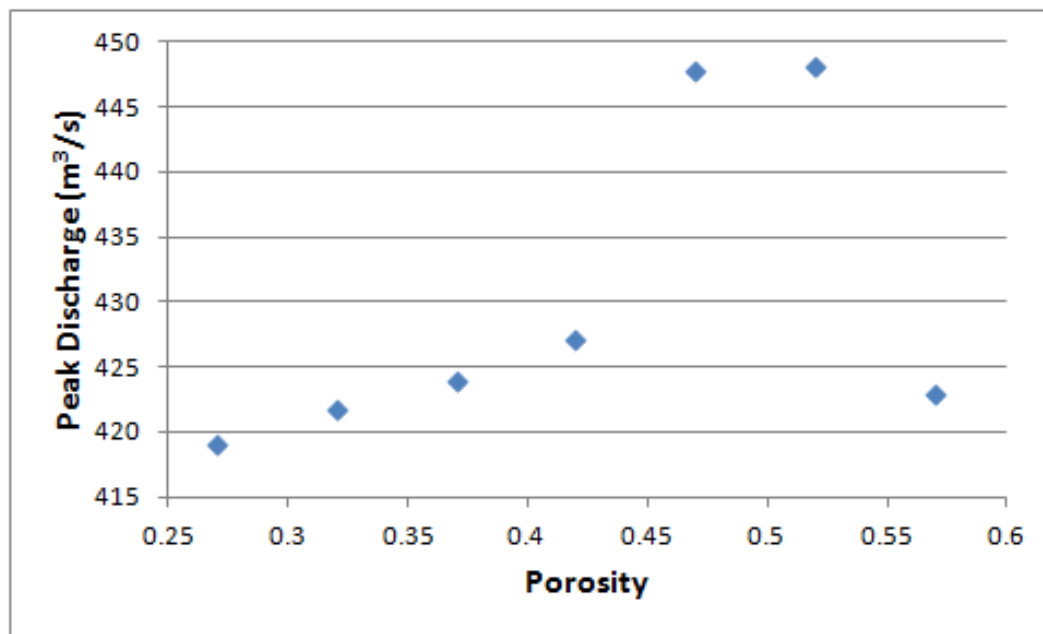


Figure 12.59: MIKE11 sensitivity analysis - porosity - peak discharge

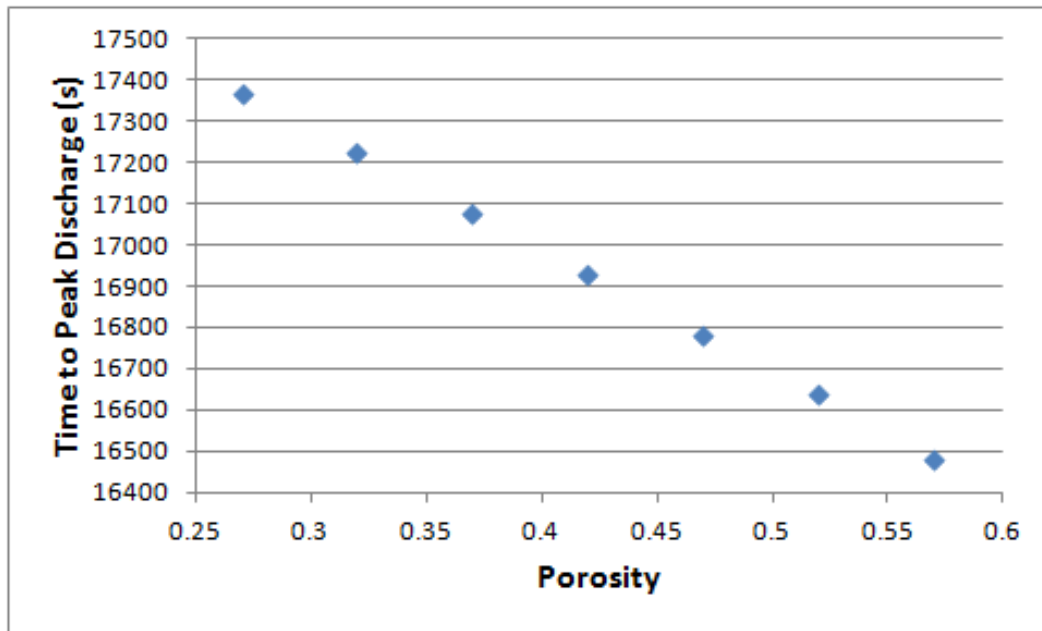


Figure 12.60: MIKE11 sensitivity analysis - porosity - time to peak discharge

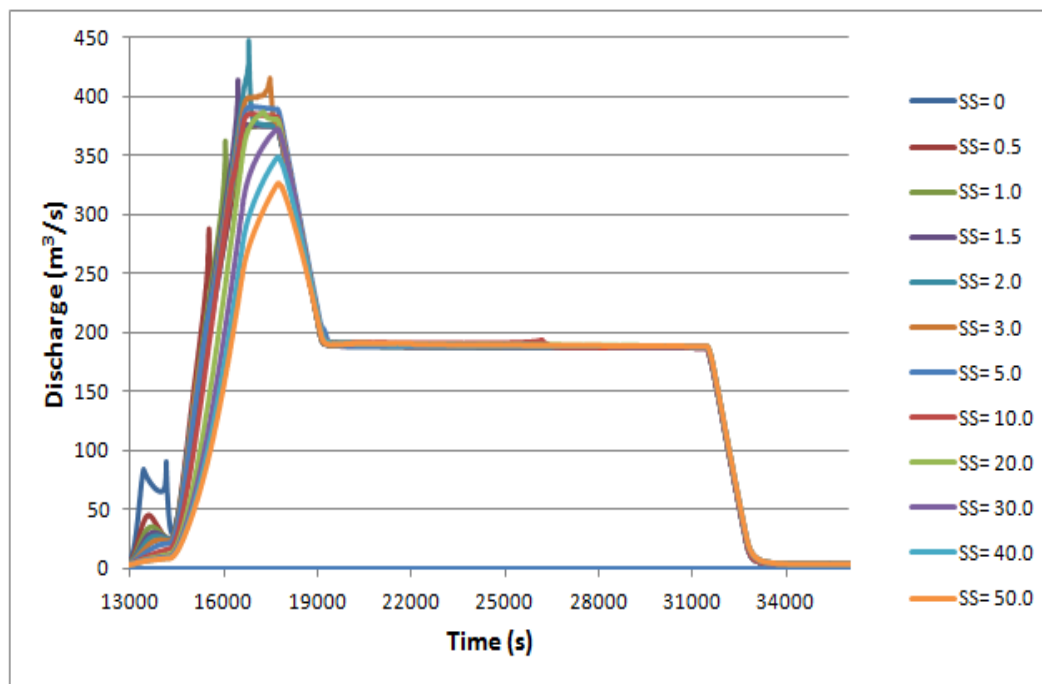


Figure 12.61: MIKE11 sensitivity analysis - side slopes (ss=h:v)

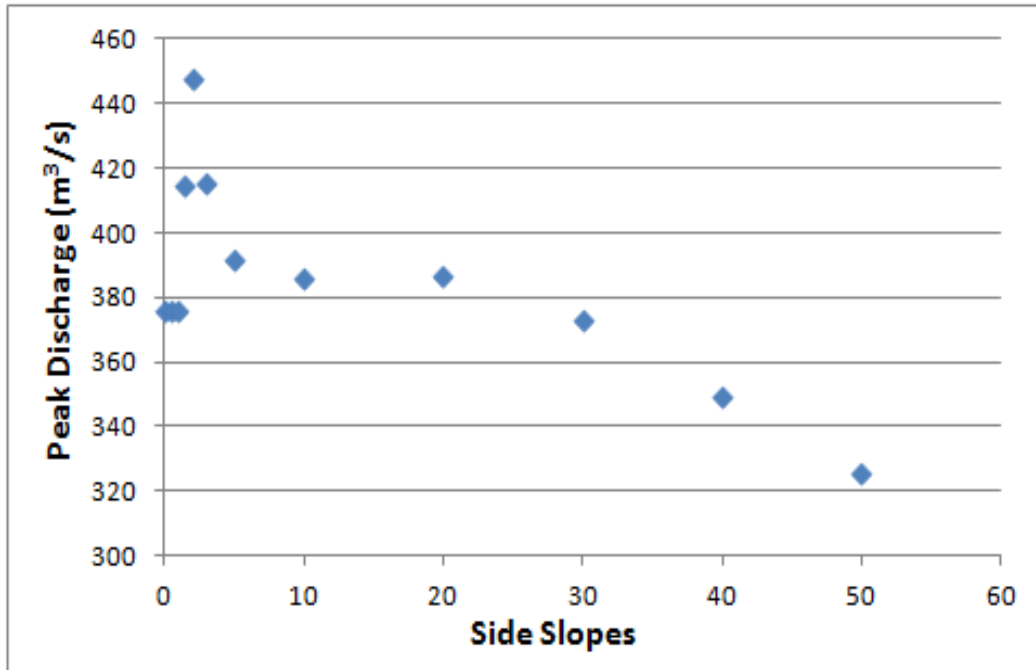


Figure 12.62: MIKE11 sensitivity analysis - side slopes (ss=h:v) - peak discharge

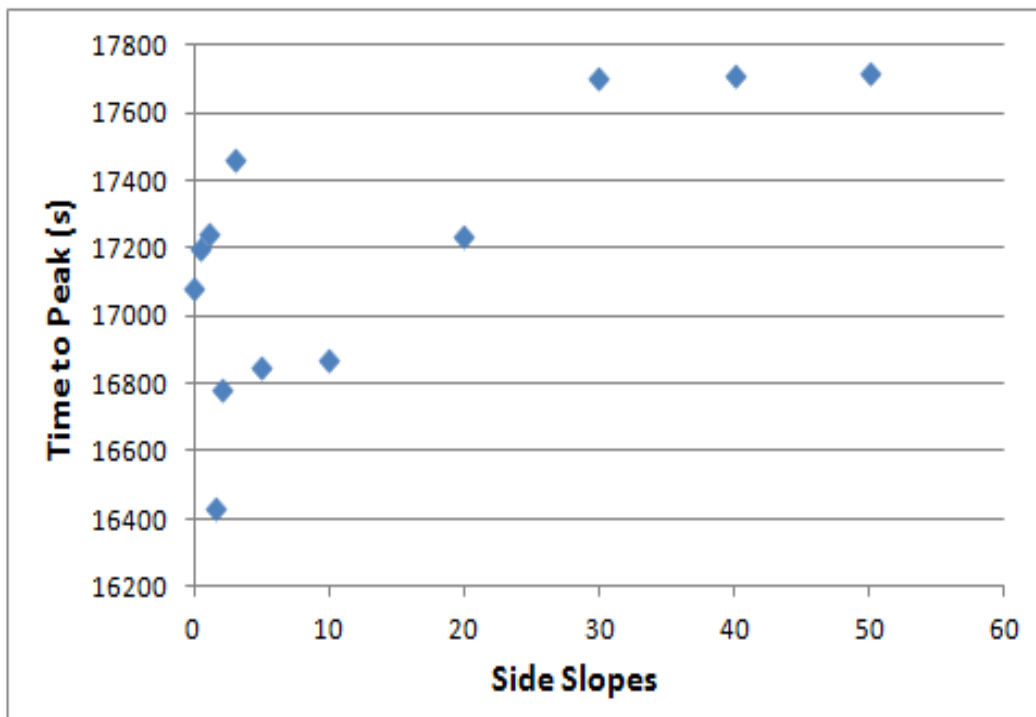


Figure 12.63: MIKE11 sensitivity analysis - side slopes (ss=h:v) - time to peak discharge

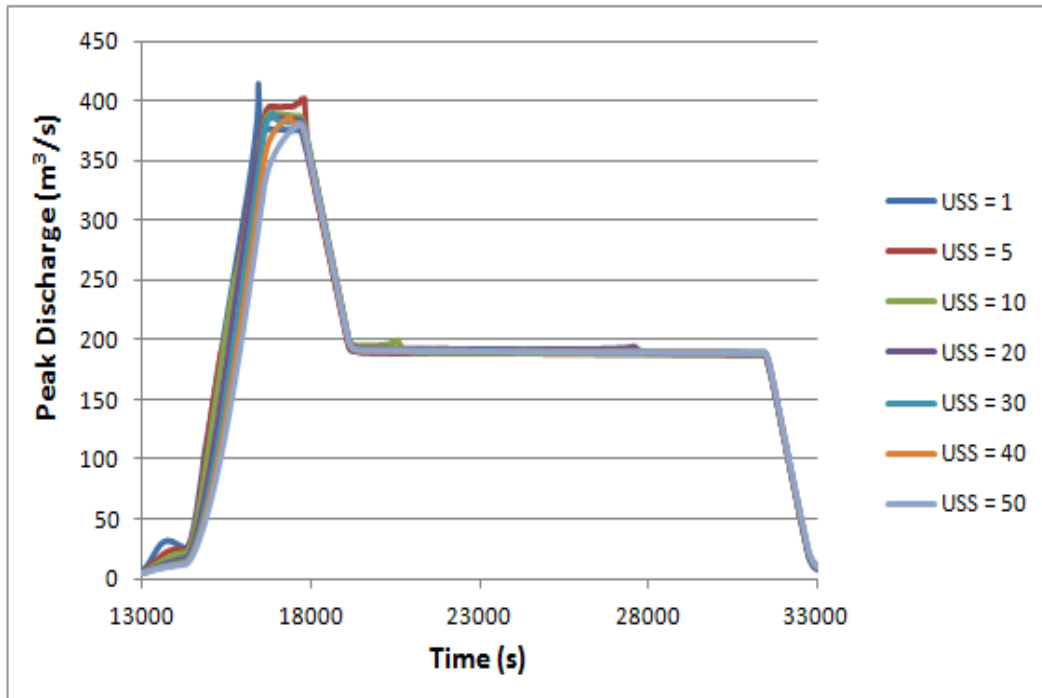


Figure 12.64: MIKE11 sensitivity analysis - upstream side slopes (uss=h:v)

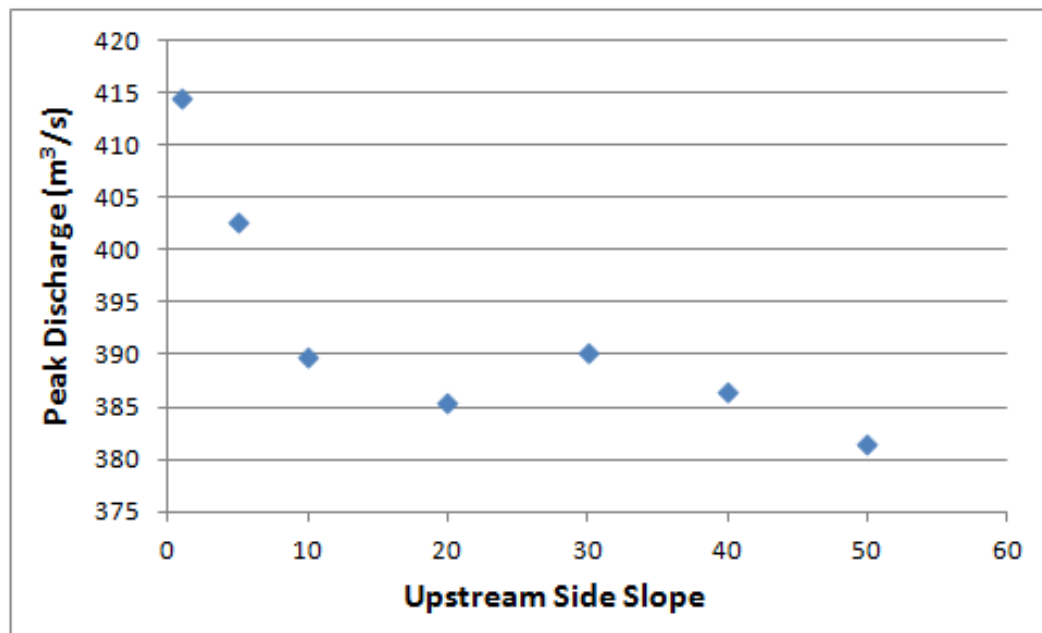


Figure 12.65: MIKE11 sensitivity analysis - upstream side slopes (uss=h:v) - peak discharge

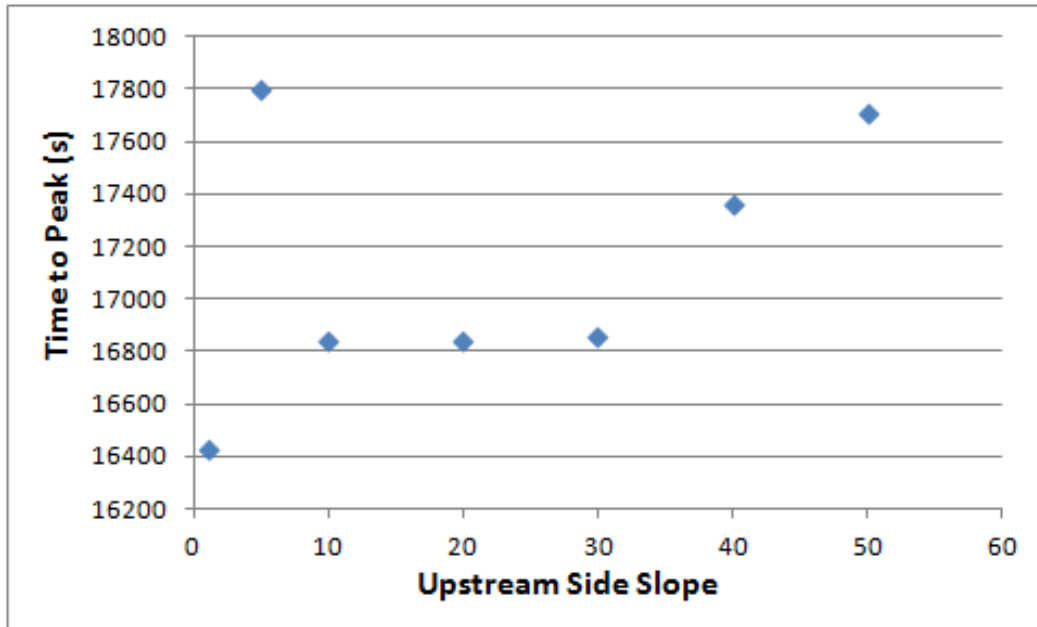


Figure 12.66: MIKE11 sensitivity analysis - upstream side slopes ($uss=h:v$) - time to peak discharge

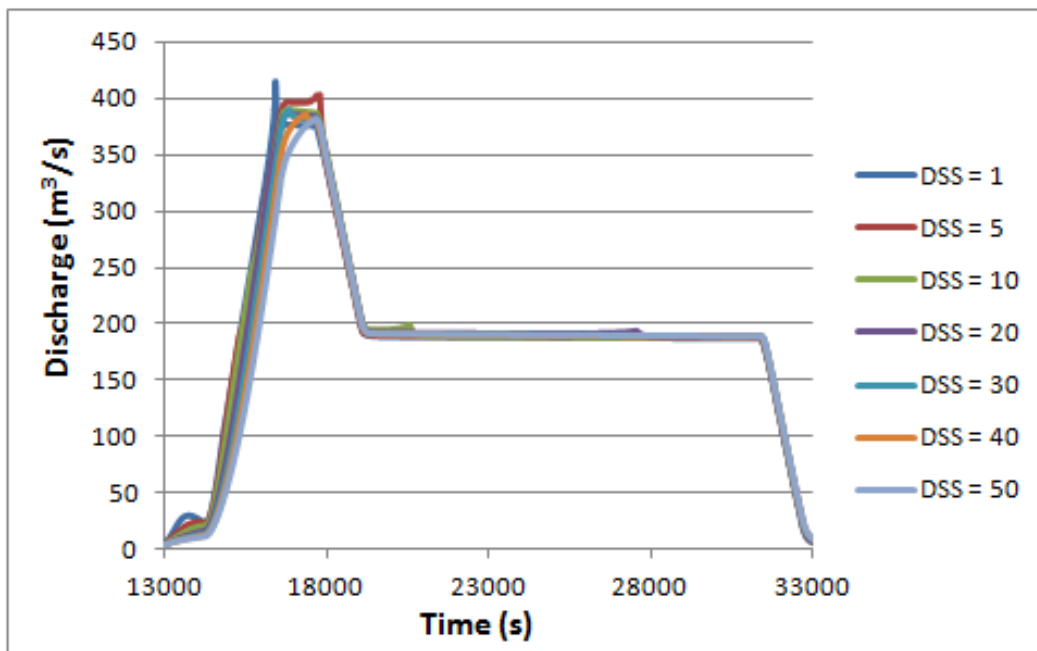


Figure 12.67: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$)

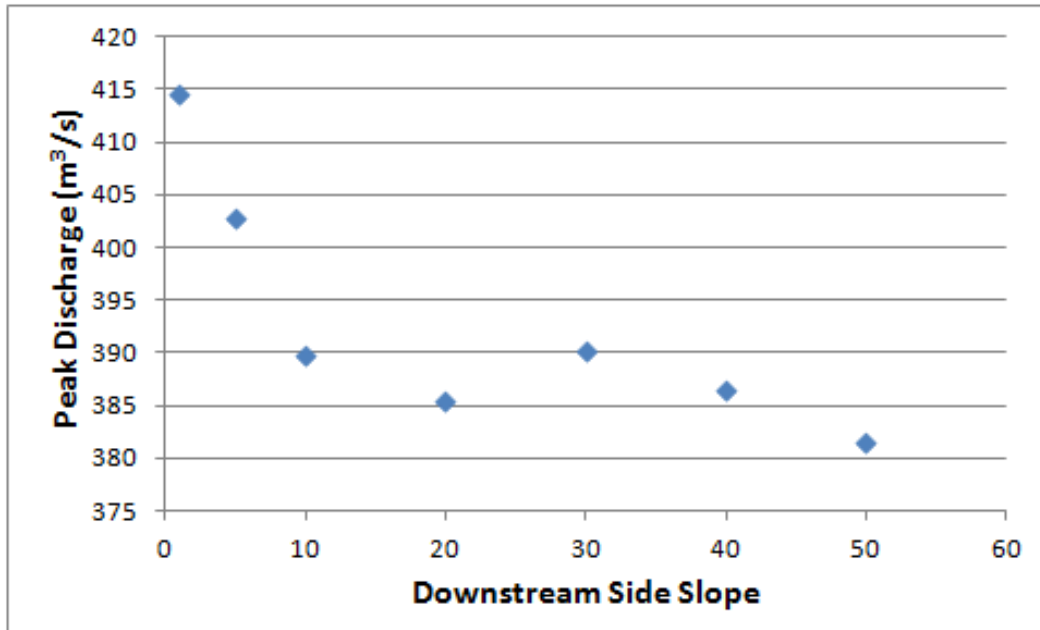


Figure 12.68: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$) - peak discharge

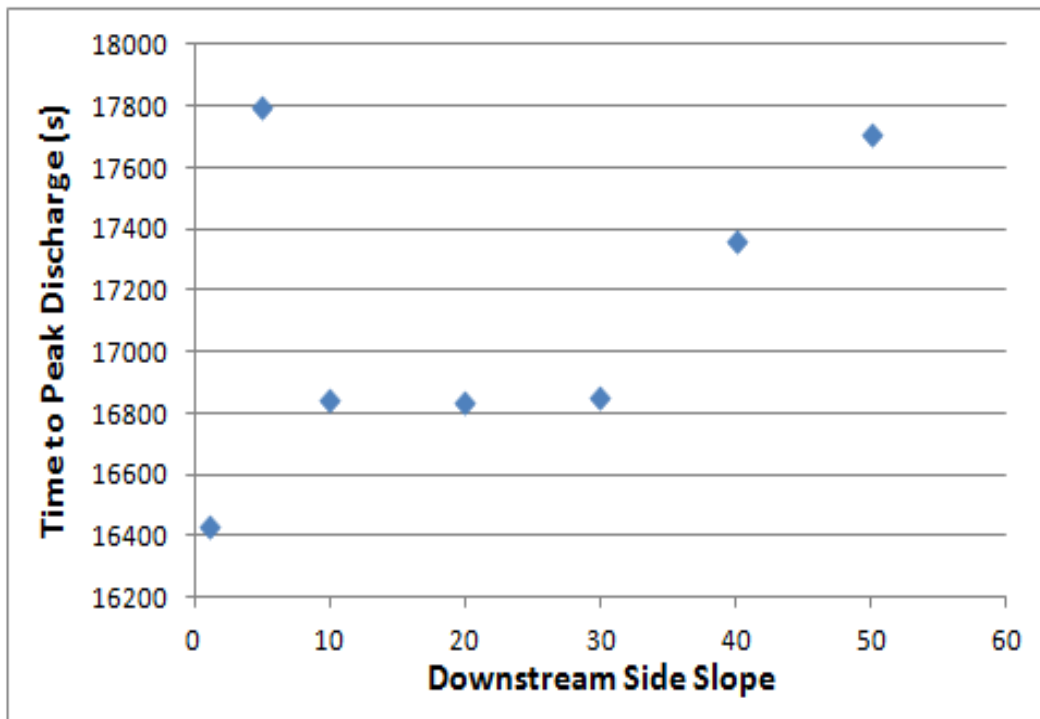


Figure 12.69: MIKE11 sensitivity analysis - downstream side slopes ($dss=h:v$) - time to peak discharge

12.6 BREACH Sensitivity Analysis

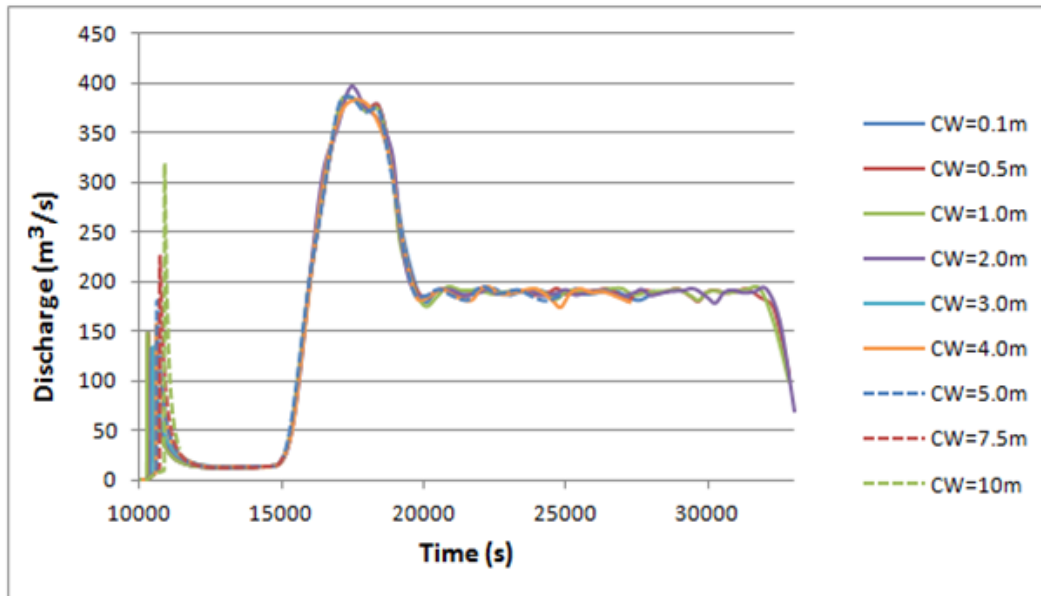


Figure 12.70: BREACH sensitivity analysis – crest width

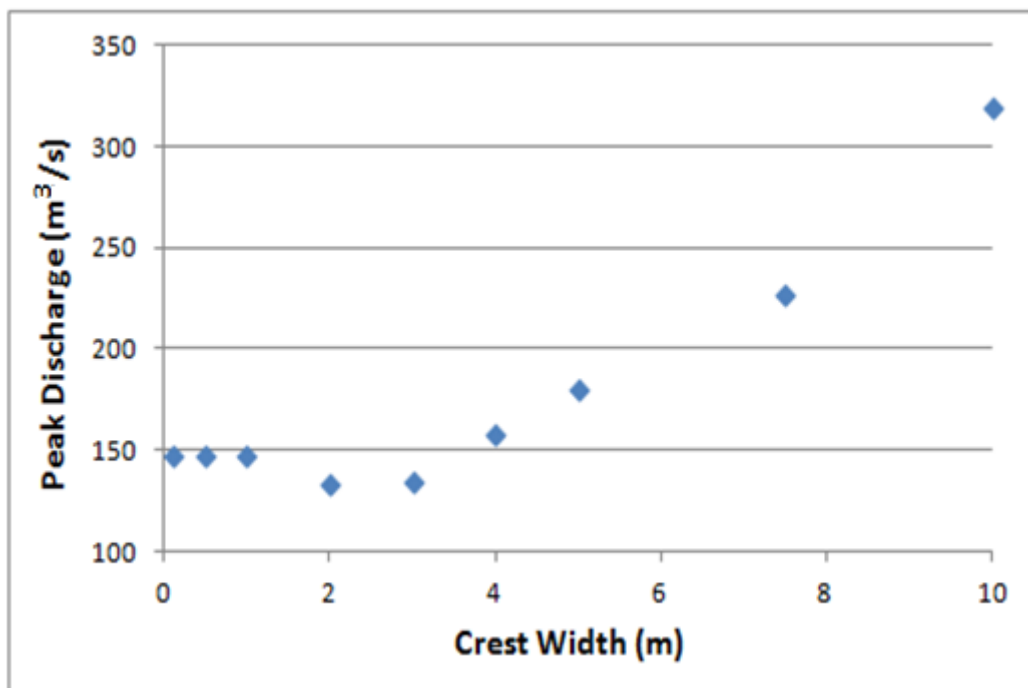


Figure 12.71: BREACH sensitivity analysis – crest width - first peak discharge

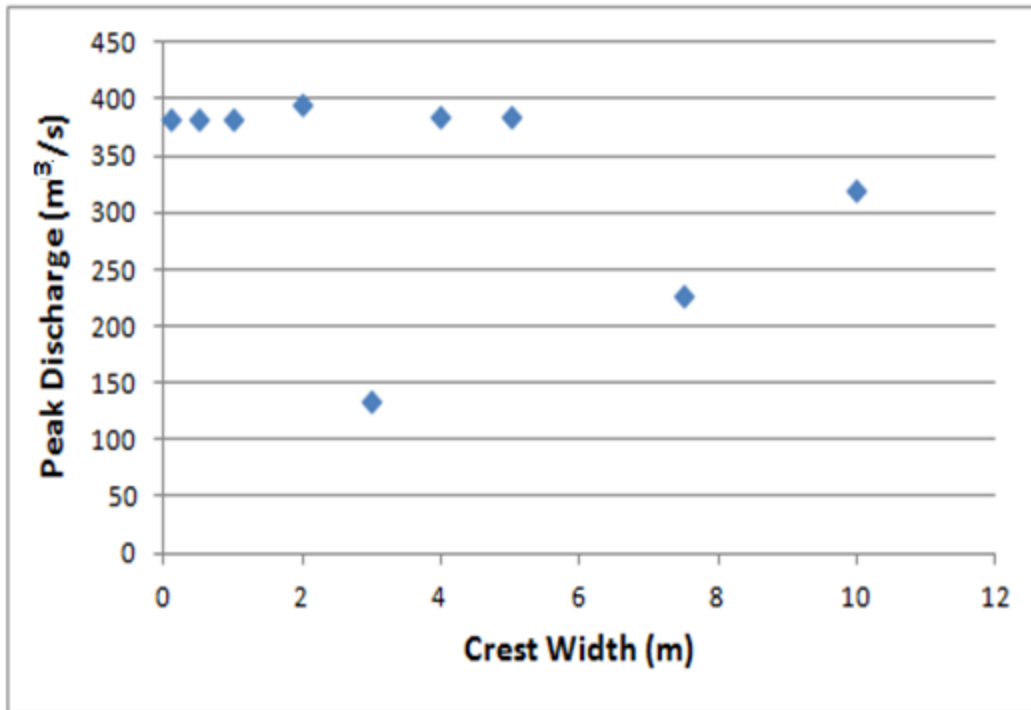


Figure 12.72: BREACH sensitivity analysis – crest width –second peak discharge

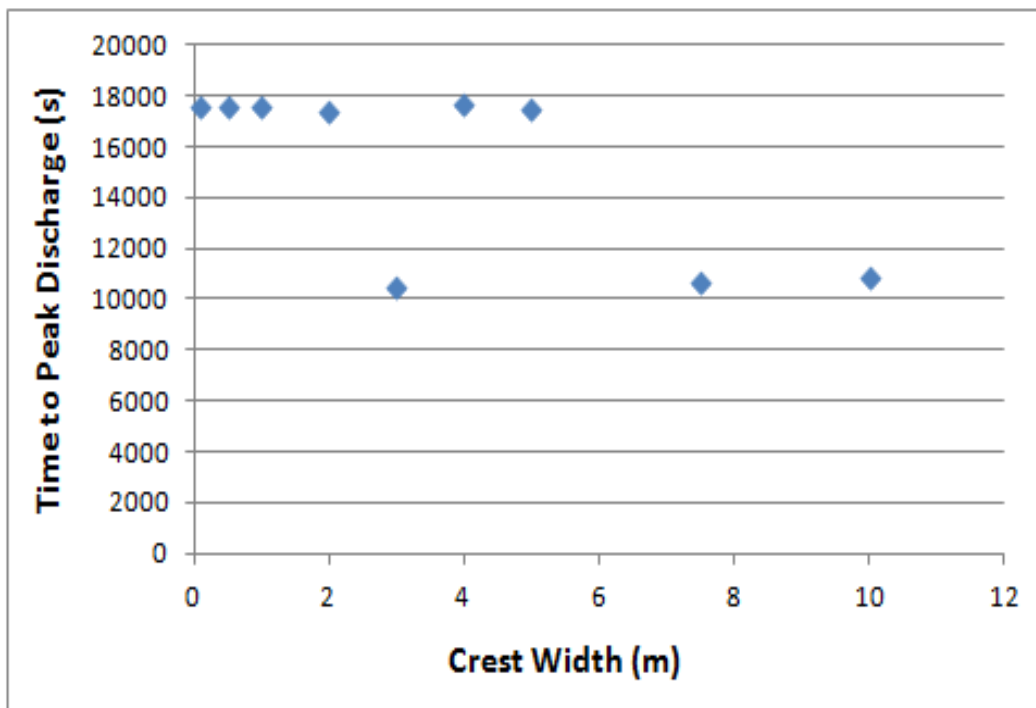


Figure 12.73: BREACH sensitivity analysis – crest width - time to peak second peak discharge

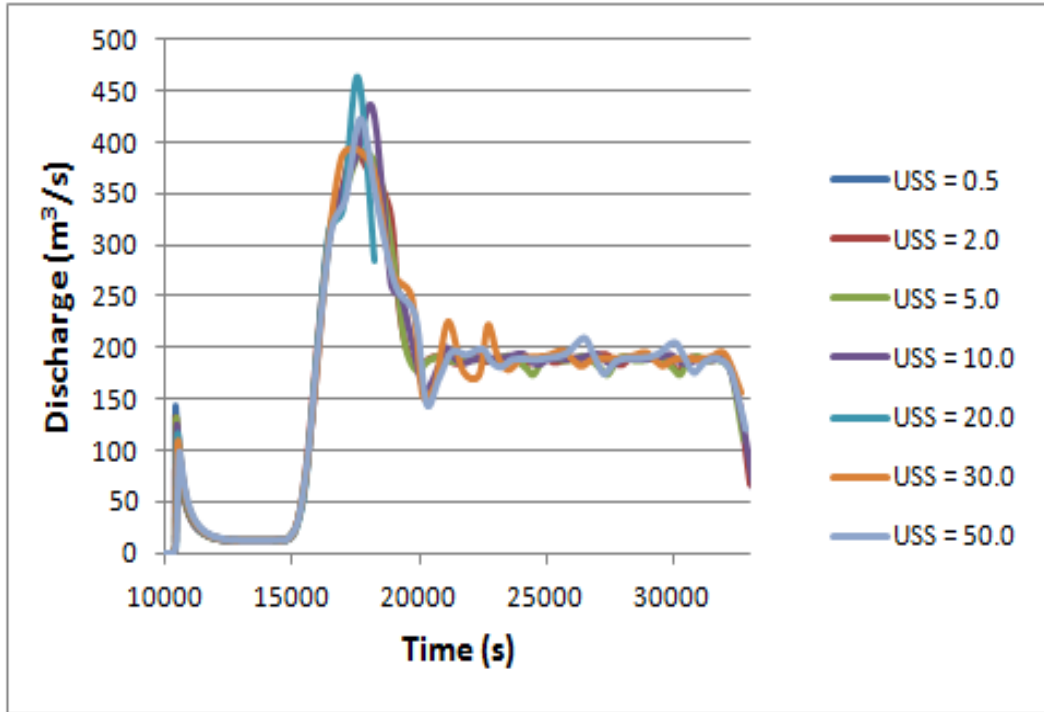


Figure 12.74: BREACH sensitivity analysis – upstream side slope ($uss=h:v$)

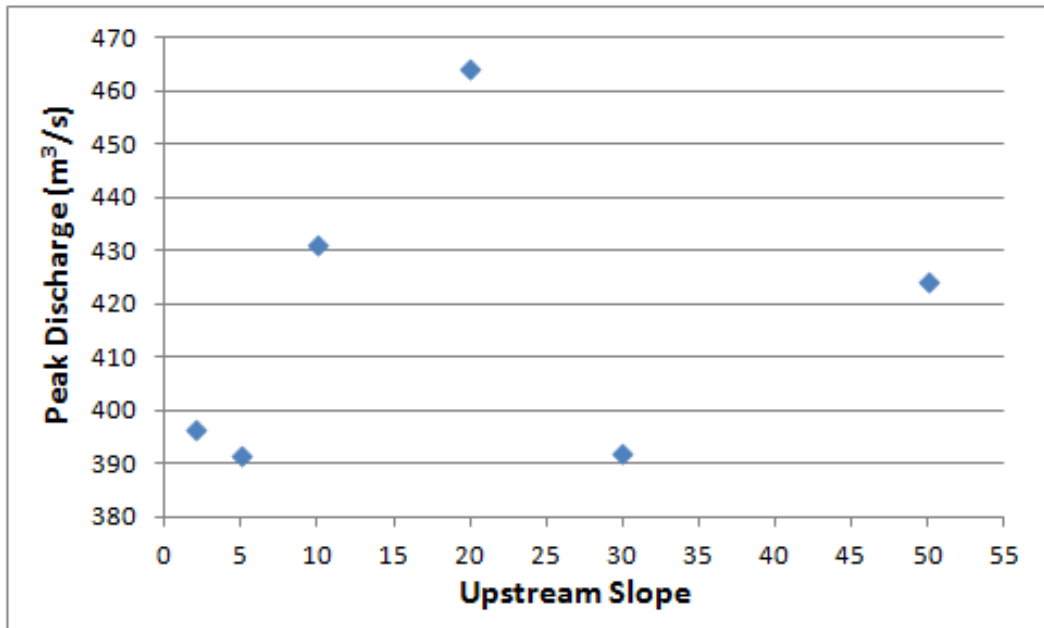


Figure 12.75: BREACH sensitivity analysis – upstream side slope ($uss=h:v$) - peak discharge

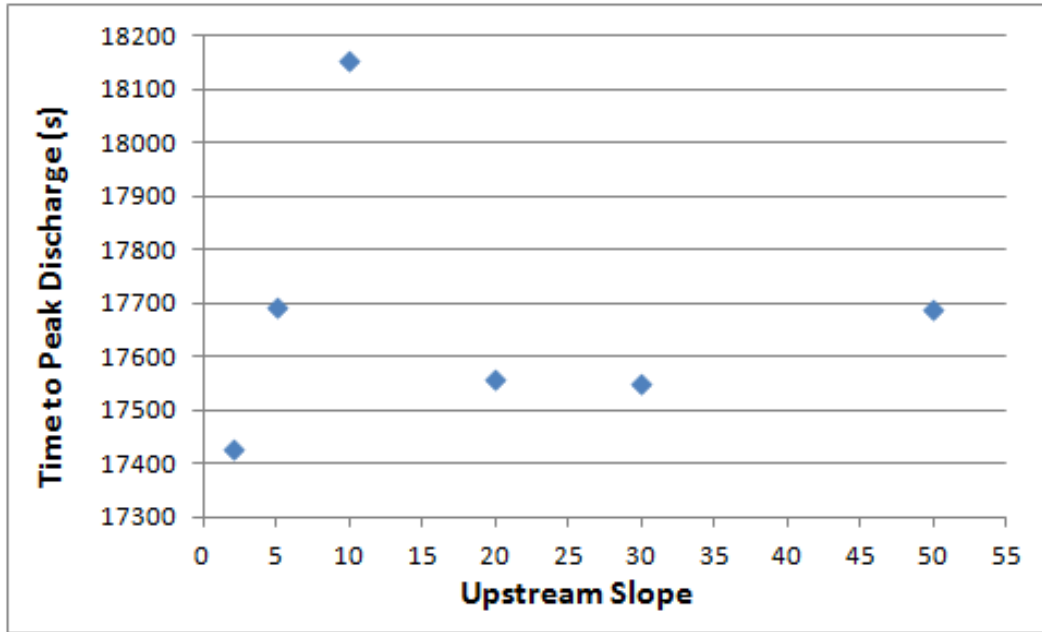


Figure 12.76: BREACH sensitivity analysis – upstream side slope ($uss=h:v$) - time to peak discharge

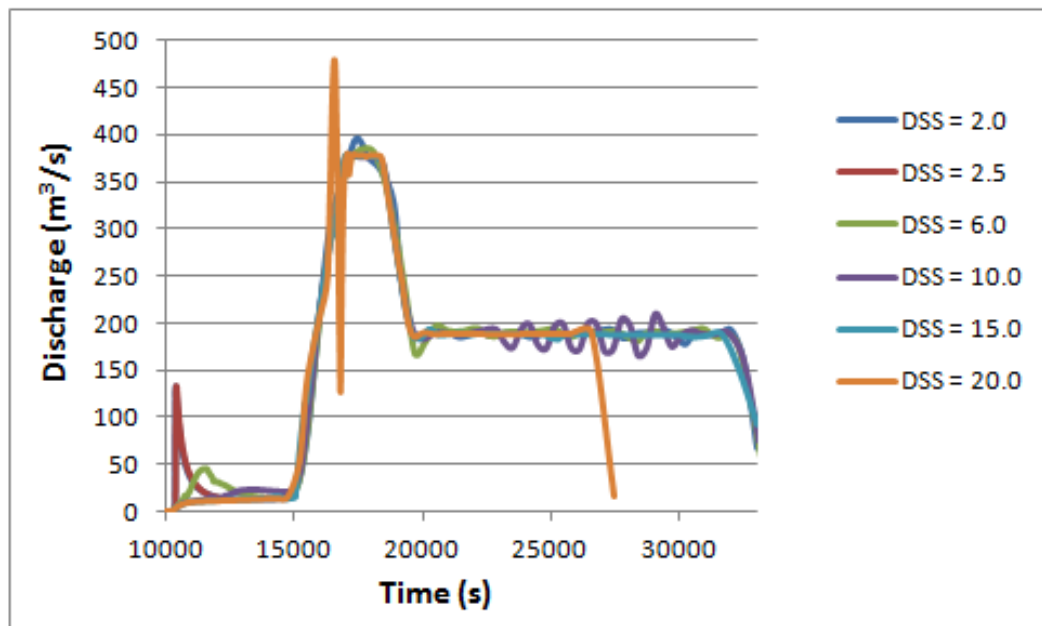


Figure 12.77: BREACH sensitivity analysis – downstream side slope ($dss=h:v$)

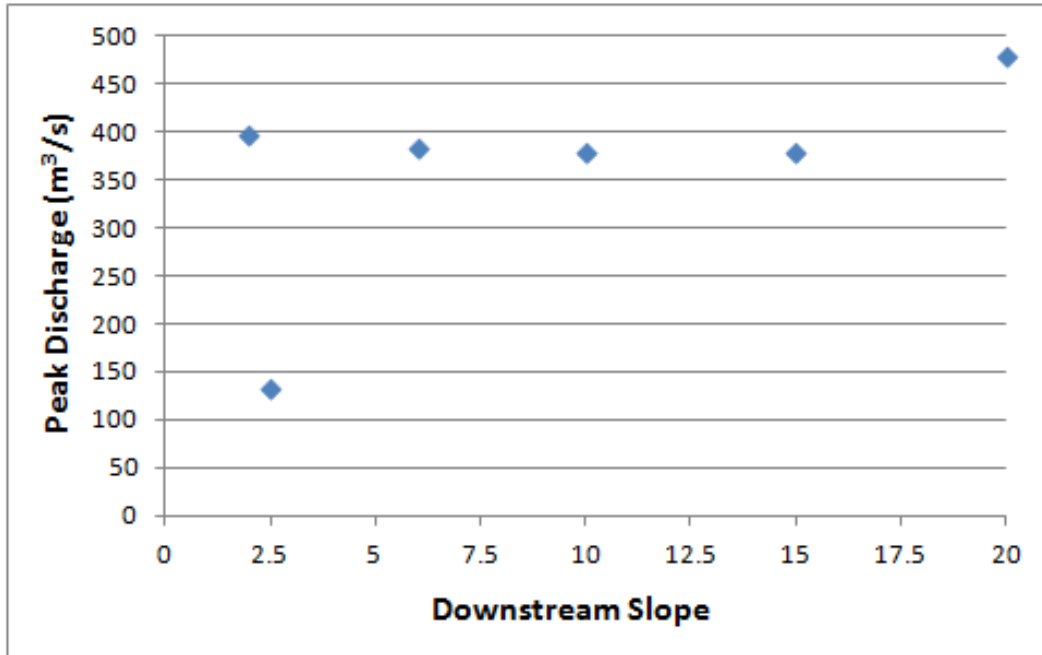


Figure 12.78: BREACH sensitivity analysis – downstream side slope ($dss=h:v$) - peak discharge

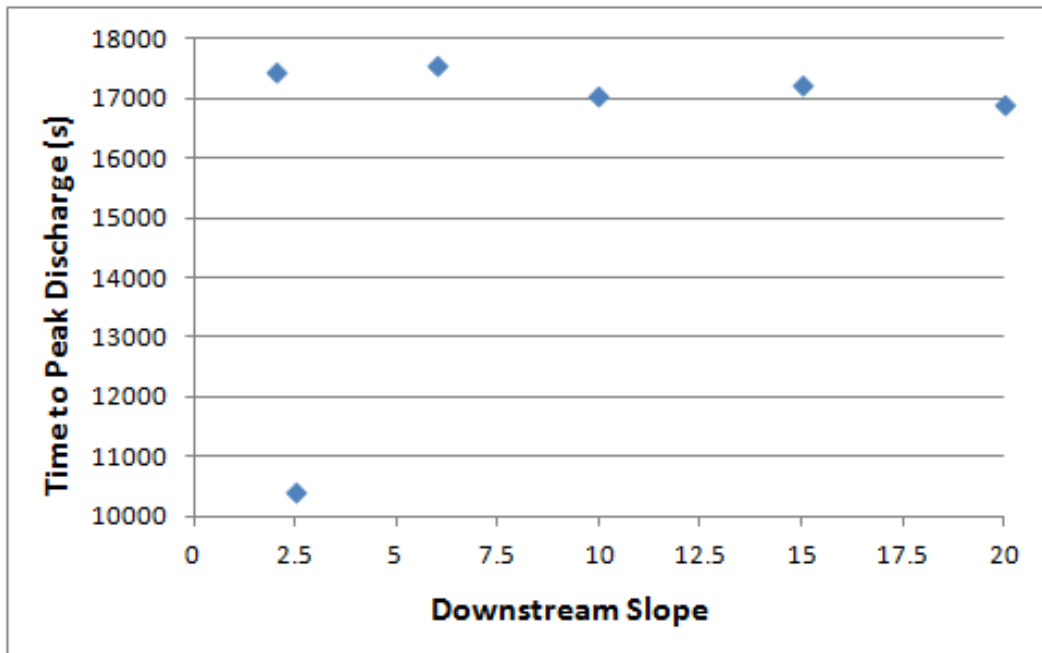


Figure 12.79: BREACH sensitivity analysis – downstream side slope ($dss=h:v$) - time to peak discharge

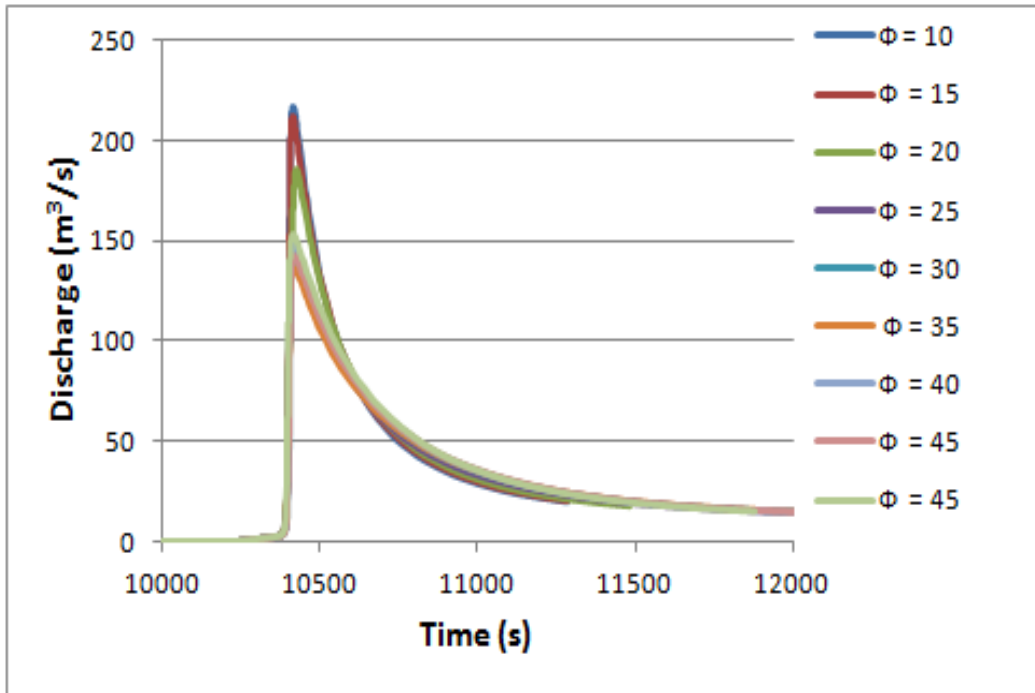


Figure 12.80: BREACH sensitivity analysis – internal angle of friction

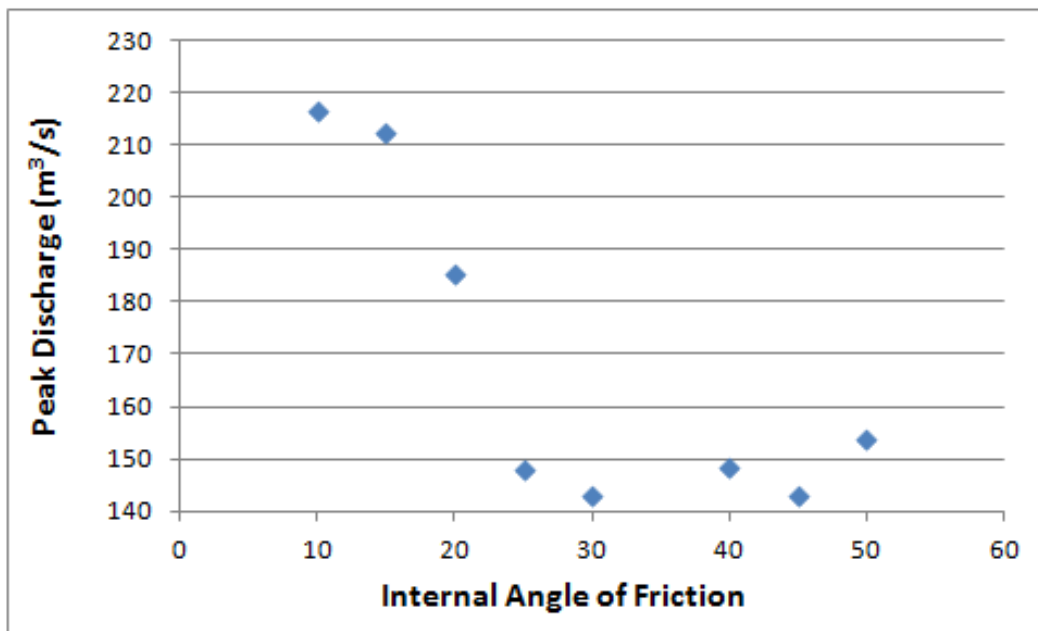


Figure 12.81: BREACH sensitivity analysis – internal angle of friction - peak discharge

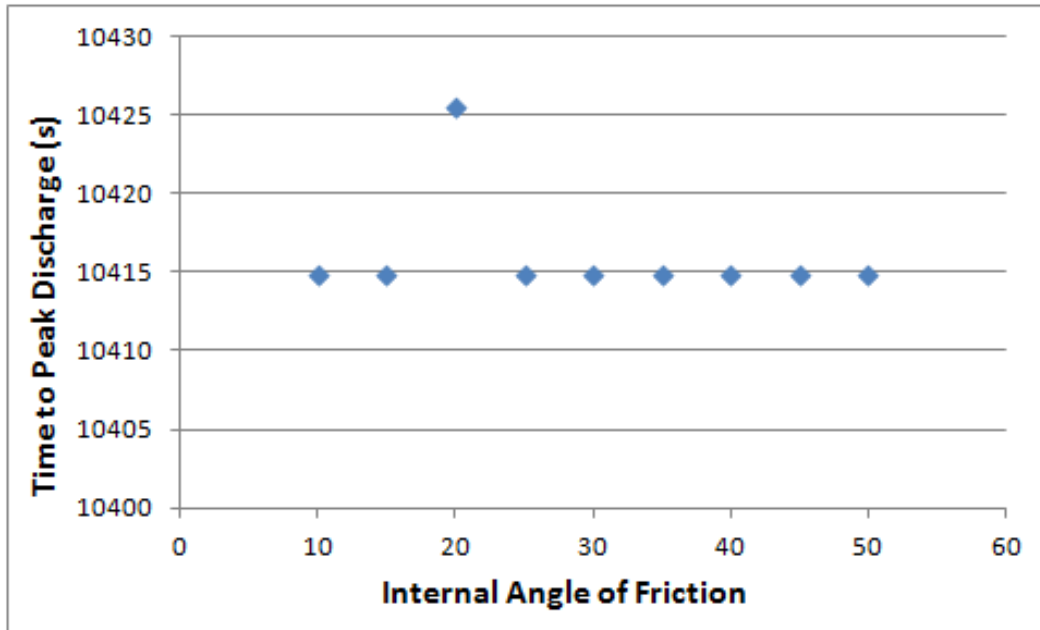


Figure 12.82: BREACH sensitivity analysis – internal angle of friction - time to peak discharge

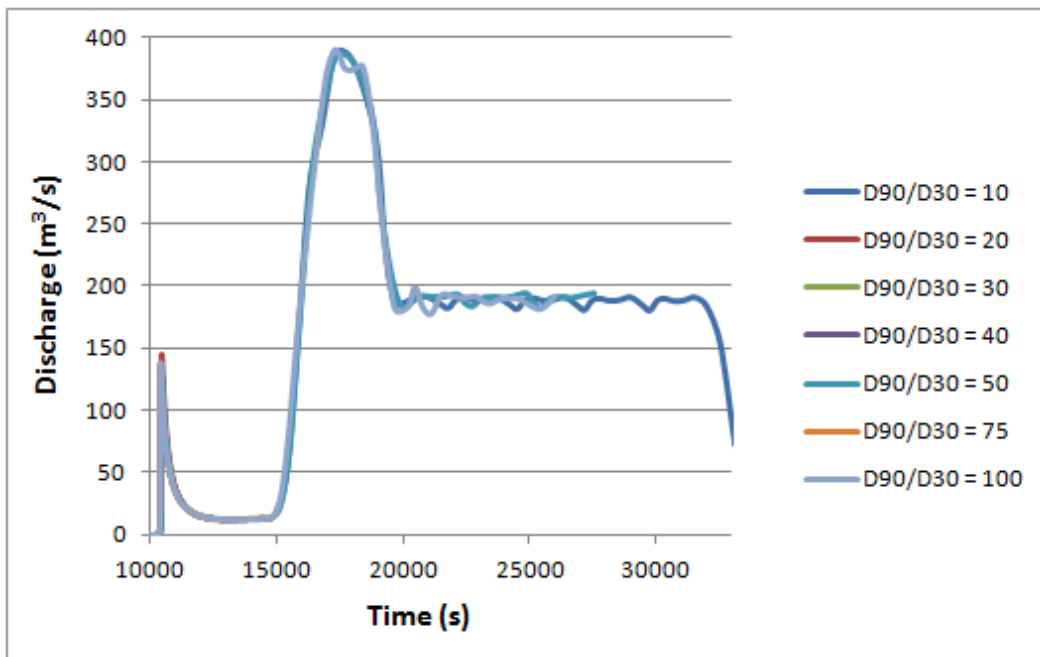


Figure 12.83: BREACH sensitivity analysis – gradation (D90/D30)

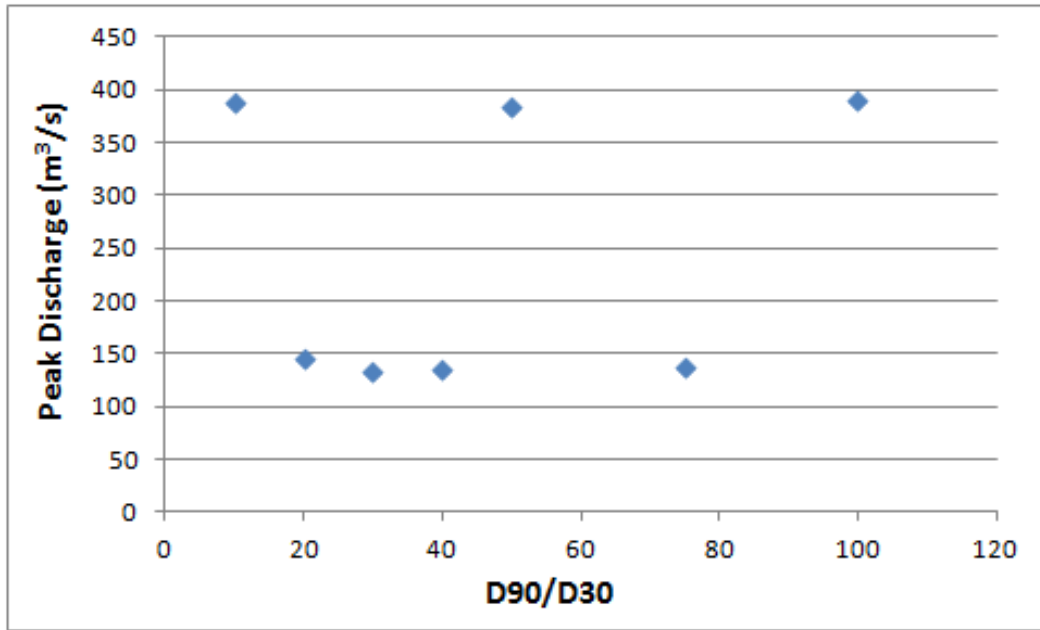


Figure 12.84: BREACH sensitivity analysis – gradation (D90/D30) - peak discharge

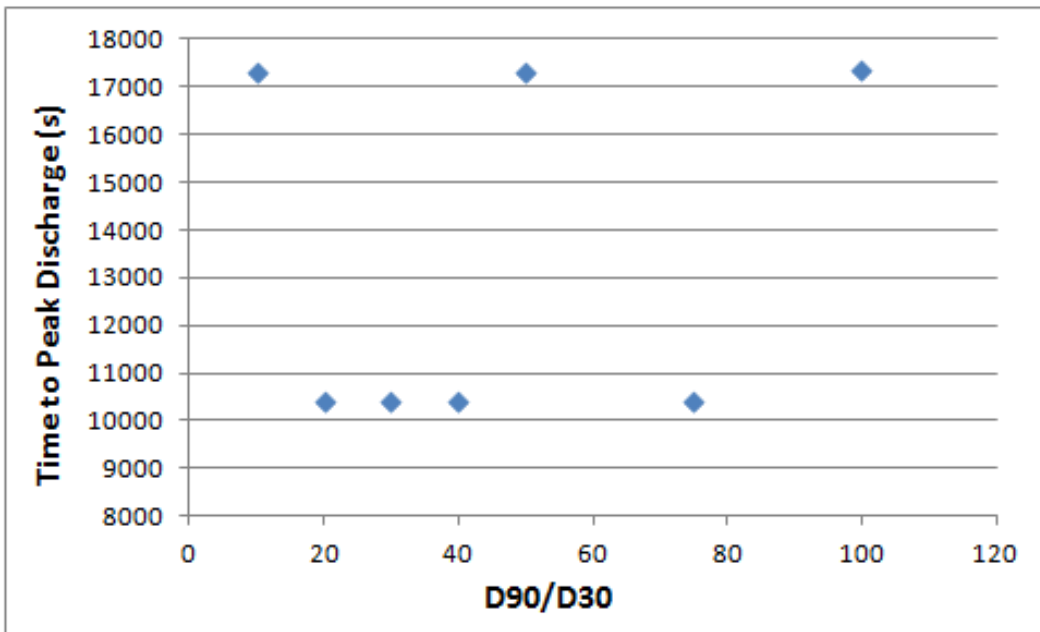


Figure 12.85: BREACH sensitivity analysis – Gradation (D90/D30) - time to peak discharge

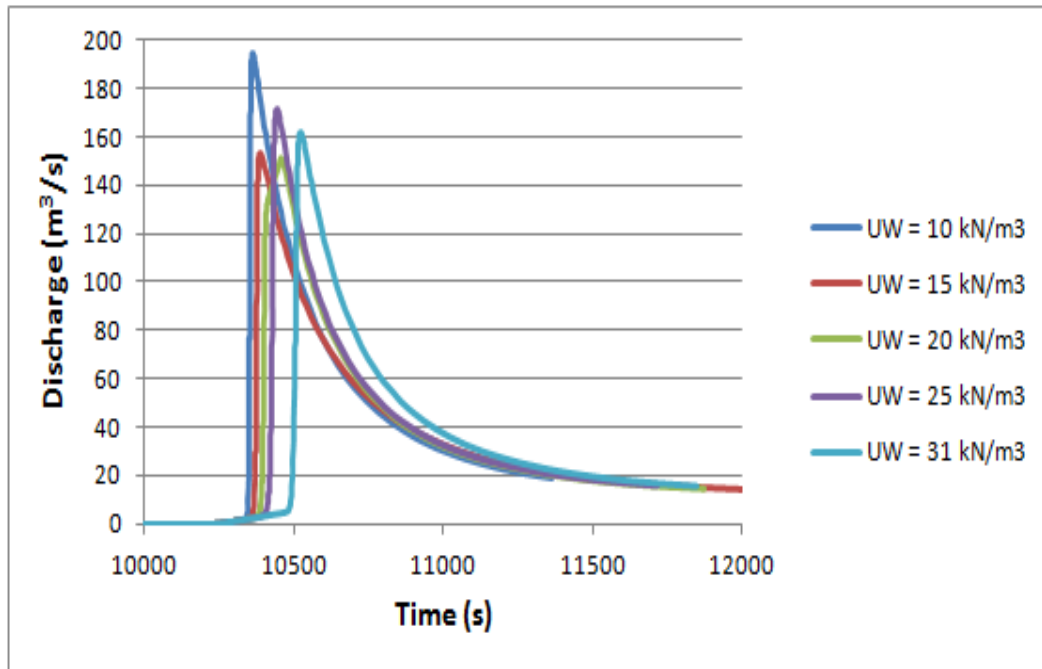


Figure 12.86: BREACH sensitivity analysis – unit weight

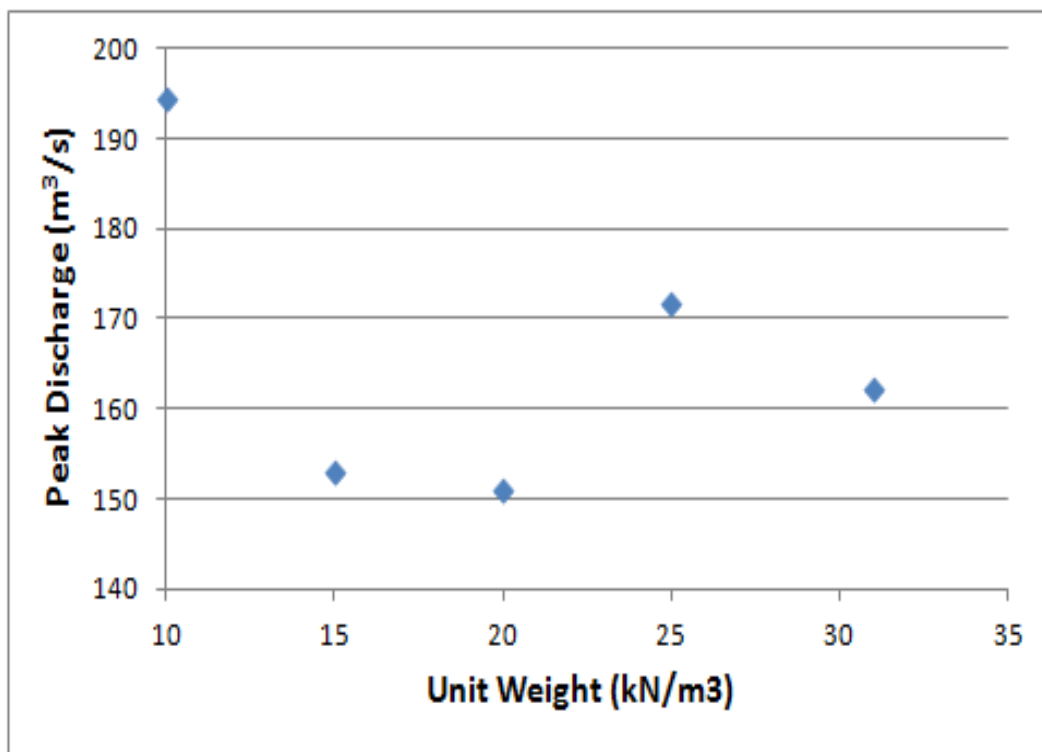


Figure 12.87: BREACH sensitivity analysis – unit weight - peak discharge

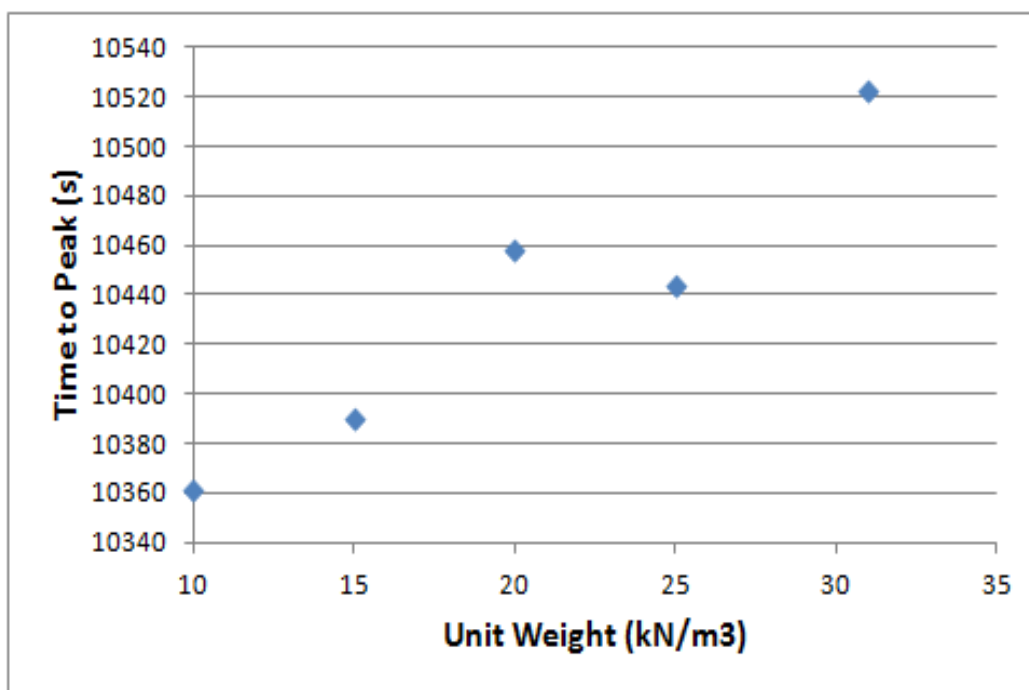


Figure 12.88: BREACH sensitivity analysis – unit weight - time to peak discharge

Table 12.16: Estimated parameter values for different sediment types

Difference In Parameters				
	Gravel	Sand	Silt	Clay
D50 (mm)	13.0	0.4	0.013	0.002
Porosity, n	35	28	42	46
Internal Angle of Friction, Φ	35	35	34	20
Cohesive Strength (kPa)	0	0	75.0	144.0

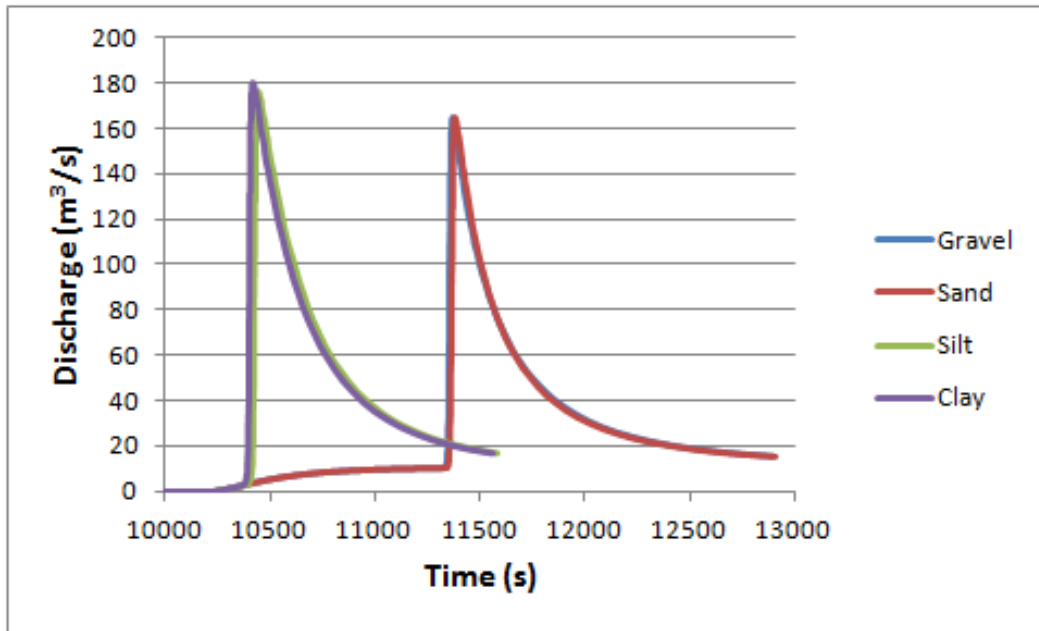


Figure 12.89: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay

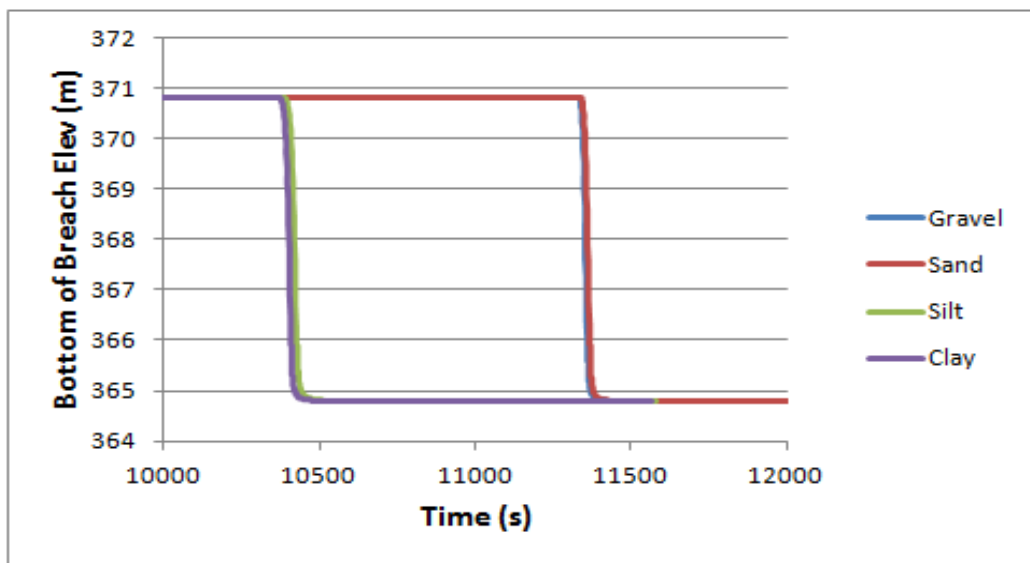


Figure 12.90: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - bottom of breach elevation

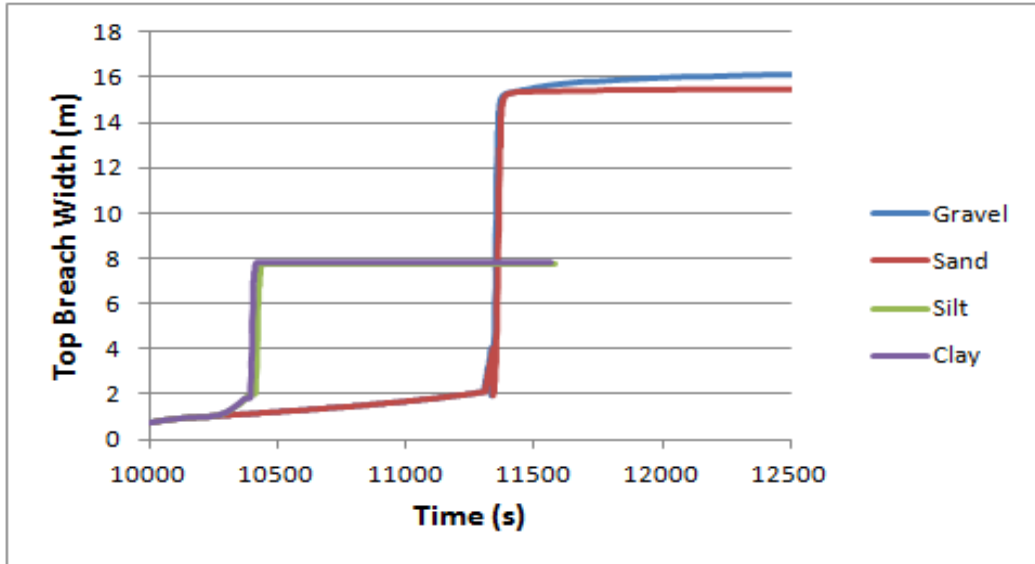


Figure 12.91: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - top breach width

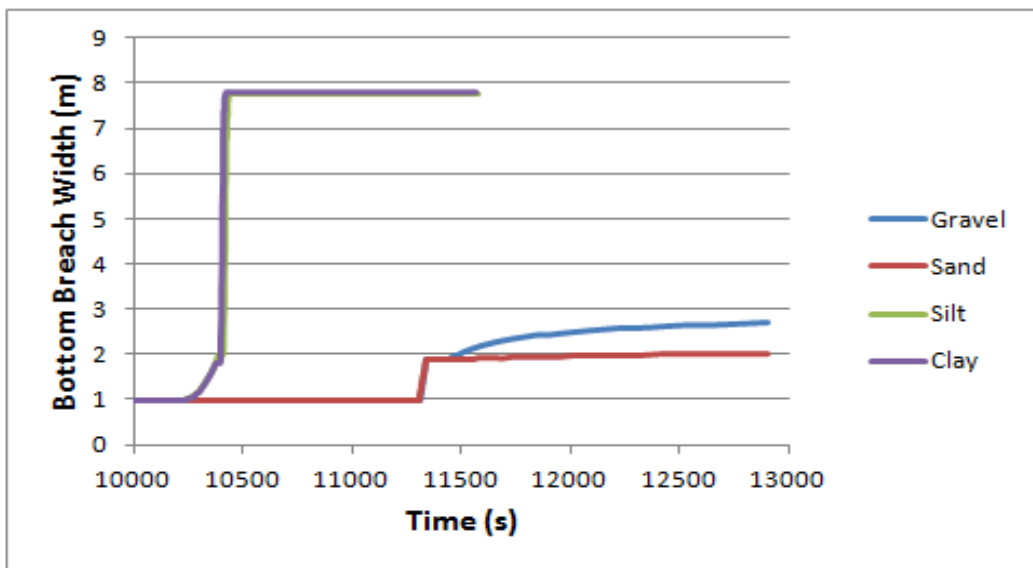


Figure 12.92: BREACH sensitivity analysis - gravel vs. sand vs. silt vs. clay - bottom breach width