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THEME C

Computational Challenges in Consequence Estimation for Risk Assessment: Numerical Modelling, Uncertainty Quantification, and Communication of Results

Part 1 – Hydraulic Modelling and Simulation

Sponsoring Organizations:

U.S. Army Corps of Engineers (USA)
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1 Introduction

In the last decade, computational capacity has grown dramatically such that multiprocessor computing techniques are now widely available. This increase in resource availability has allowed the development of a vast array of new models for flooding and consequence assessment. Many of these models are computing flood wave propagation at extremely high temporal and spatial resolutions. When these models are coupled to equally complex models of population mobility, infrastructure impact and economic consequence, simulation frameworks are created that can support a paradigm shift from standard approaches to dam risk analysis.

Although computational advances have increased the availability and applicability of novel tools, there is a deficiency in benchmarks on the use of those tools in risk assessments. An obvious application of increased availability and efficiency of computational resources is to conduct probabilistic risk assessments using Monte Carlo techniques, but the application of such approaches entails a wide range of assumptions and technical decisions regarding the management of uncertainty such as variable uncertainty, parameter uncertainty, uncertainty in probabilistic sub-model, measurement error, computational errors, and numerical approximation to name a few.

Universities, engineering companies and regulatory bodies are invited to contribute to the benchmark and take part in the discussion of results gained.

This document is part 1 in a 2-part series for Theme C. Part 1 pertains to the hydraulic modelling and simulation of the dam breach and subsequent flood wave and provides details regarding the available data, dam geometry and failure, and expected modelling and simulation solution requirements. Part 2 primarily focuses on consequence estimation using the modelling and simulation results from Part 1.

1.1 Benchmark Focus

The numerical problem proposed for the workshop consists of estimating the consequences of failure of a dam near populated areas with complex demographics, infrastructure and economic activity. The dam in question will be near the city of Hydropolis, a virtual testbed for flood risk analysis to be built in preparation for the benchmark study.

Theme C participants are free to select the type and sophistication of the simulation engines used to solve the problem, including 1-d, 2-d and 3-d flood simulation tools, Population at Risk (PAR) and Loss of Life (LOL) estimation techniques, and infrastructure and consequence assessment models.

2 Flood Modelling and Simulation

The following sections are intended to provide information regarding the data provided for the dam failure modelling and simulation benchmark. Specifically this information includes the topographic data and dam geometric and construction information. It is not the intent of this benchmark to set requirements as to which modelling and simulation environment should be used. Therefore, the descriptions and data provided are intended to be useful to a wide range of modelling and simulation environments at many levels of fidelity.

2.1 Dam Information

A hypothetical embankment dam was constructed in a mountainous region. The high-hazard dam sits directly above a lightly populated area and 3.5 kilometres away from an urban environment. The front and rear views of this dam are shown in Figure 2-1 and 2-2, respectively.

The primary function of the dam is flood control for heavy snowmelt and strong monsoonal weather patterns. In addition, the reservoir provides some water supply and recreational activities to nearby communities. The following sections provide more detail regarding the geometry and the construction of the hypothetical dam.

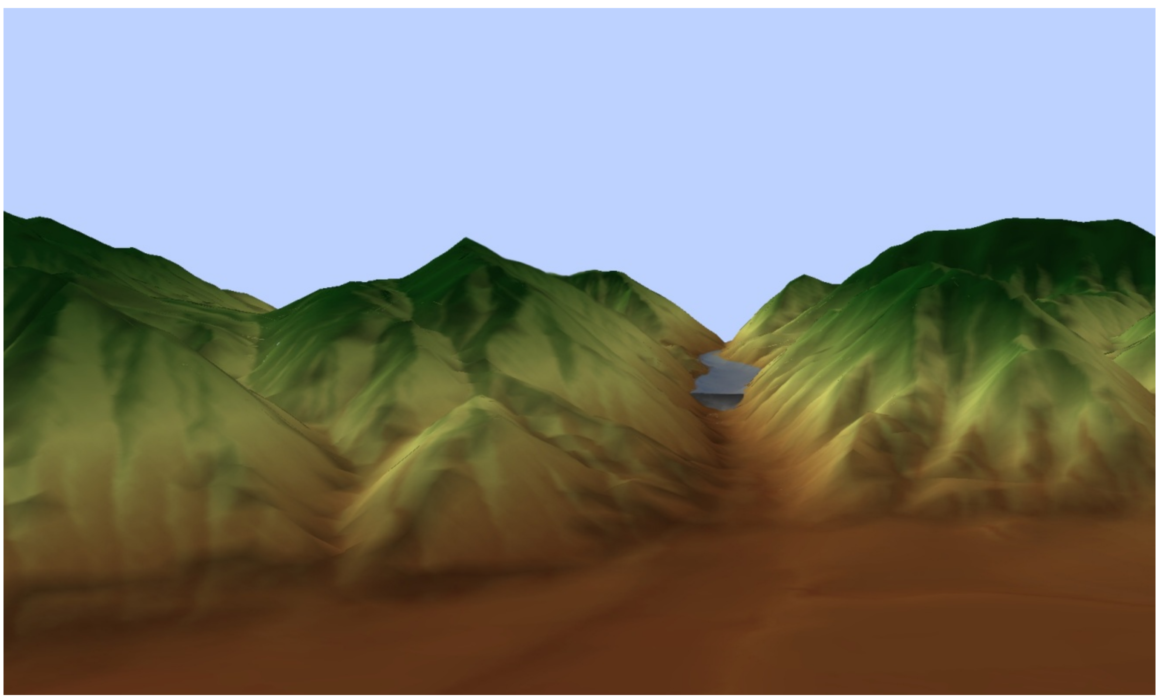

Figure 2-1. Front view of dam and surrounding topography.

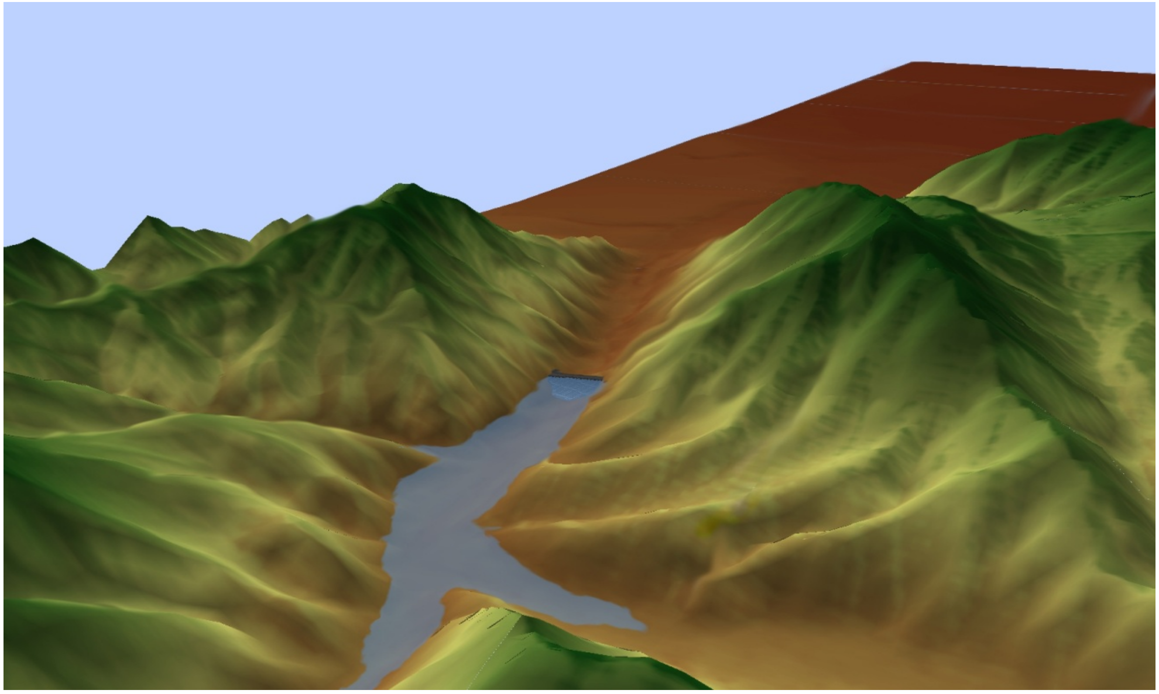

Figure 2-2. Rear view of dam, reservoir, and surrounding topography.

2.1.1 Dam Geometry

The hypothetical embankment dam is considered high head (61 m) with moderate storage (38 million cubic meters). An overview of the dam geometry is provided in Table 2-1 and a cross-sectional profile is shown in Figure 2-1.

Table 2-1. Dam geometric parameters

Parameter Description	Value
Dam Location (x, y)	4499.66, 6681.57
Crest Length (m)	360
Crest Width (m)	24
Crest Elevation (m)	272
River Bed Elevation (m)	211
Upstream Embankment Slope (?H:1V)	3
Downstream Embankment Slope (?H:1V)	3

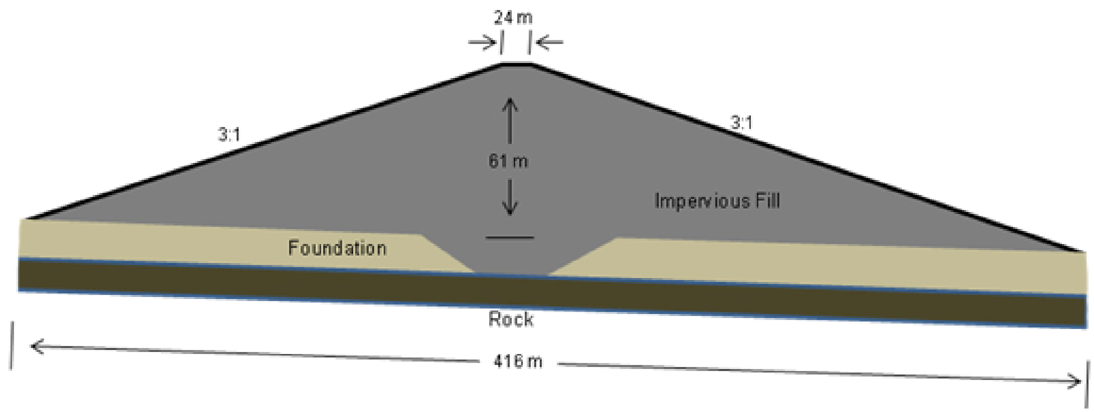

Figure 2-3. Cross-sectional view of the hypothetical dam.

The storage capacity at crest elevation is more than 38 million cubic meters. The stage-volume curve for the reservoir is shown in Table 2-2.

Table 2-2. Reservoir stage-volume curve.

Elevation (m)	Surface Area (m²)	Volume (m³)	Elevation (m)	Surface Area (m²)	Volume (m³)
211	0	0	243	605,559	6,214,463
213	898	266	245	682,169	7,500,447
215	7,812	7,432	247	762,461	8,944,612
217	25,502	39,871	249	848,052	10,553,044
219	43,821	109,250	251	916,938	12,322,332
221	62,409	216,078	253	978,640	14,218,482
223	85,038	364,294	255	1,035,581	16,233,223
225	117,544	565,695	257	1,094,197	18,365,080
227	153,732	836,860	259	1,155,316	20,614,116
229	190,728	1,181,477	261	1,216,975	22,987,820
231	222,068	1,594,314	263	1,278,543	25,483,100
233	252,239	2,067,481	265	1,338,586	28,100,382
235	294,797	2,613,155	267	1,398,359	30,837,356
237	351,808	3,254,198	269	1,467,826	33,701,532
239	461,611	4,084,756	271	1,543,126	36,712,416
241	531,463	5,077,507	272	1,584,052	38,276,344

2.1.2 Dam Construction Information

2.1.2.1 Homogenous Dam

The dam is a rolled earth fill structure composed of predominantly sandy clays and clayey sands. Compaction was achieved using 4 passes of a 50-ton pneumatic-tired roller on 0.3-meter loose lifts.

Strength properties for the dam were developed from results from undrained triaxial strength tests. The drained strength parameters were based on 5% axial strain and were selected to represent initial confining stresses up to 478 kPa. The undrained strengths were interpreted from an approximate evaluation of S_u / σ'_{mc} ratios estimated from the reported undrained tests. The undrained strength S_u was taken as one-half the maximum deviator stress for axial strains up to 10%. The estimated strengths are summarized in Table 1.

Table 2-3. Selected parameters for dam.

Parameter	Description	Value
Strength Parameters:		
c'	Effective (drained) cohesion in kPa	19.15 kPa
ϕ'	Effective (drained) friction angle	14°
S_u	Undrained strength in kPa	$43.09 + 0.175 \cdot \sigma'_{mc}$
Stiffness Parameters:		
V_{s1}	Shear wave velocity at $\sigma'_v = 1$ atm	152.4 m/s
$G_{max,1}$	Maximum shear modulus at $\sigma'_v = 1$ atm	46443 kPa
Others:		
γ_{sat}	Saturated unit weight	2002 kg/m³
k	Permeability	$1.9 \cdot 10^{-6}$ cm/s

2.1.2.2 Foundation

Foundation stiffness parameters were based on shear wave velocity values without reduction for strain level or changes in effective stress. A simplified stiffness distribution was used to reflect the gradual increase in stiffness with depth. The selected properties are summarized in Table 2.

Table 2-4. Selected parameters for foundation

Parameter Description		Value: Base of dam to depth of 3.6 m	Value: Foundation below 3.6 m
Stiffness Parameters:			
V_s	Shear wave velocity	167 m/s	185 m/s
G	Shear modulus	$6.75 \cdot 10^5$ kPa	$8.33 \cdot 10^6$ kPa
ν	Poisson's ratio	0.25	0.25
Others:			
γ	Unit weight	2,242 kg/m³	2,402 kg/m³
K	Permeability	$9.5 \cdot 10^{-7}$ cm/s	$9.5 \cdot 10^{-7}$ cm/s

2.1.3 Dam Failure Guidance

The dam failure for this benchmark takes place when the pool elevation is at crest elevation. The mode of failure will be assumed as an overtopping failure. While it is open for participants to conduct detailed (physically-based) modelling of the breaching process as part of the benchmark, several breach parameter estimation approaches available in the literature are summarized in the appendix of this document.

2.2 Data Provided

In addition to the information provided in this document, gridded data representing the topography and the land use classification are provided. All gridded datasets conform to the same domain and cellsize. The resolution of data provided is considered a base dataset and may be altered if required by the modelling and simulation environment used by the participant.

2.2.1 Digital Elevation Data

Two digital elevation models (DEM) are provided to benchmark participants:

- DEM representing pre-dam construction.
- DEM representing post-dam construction.

Participants in the benchmark may decide which DEM is more appropriate for use based on individual requirements for modelling and simulation.

2.2.2 Land Use/Cover

Benchmark participants are provided a gridded dataset representing the hypothetical land use/cover for the simulation region. These data follow the classification guidance and values provided in the National Land Cover Dataset (NLCD). For completeness, a description of land use classifications is provided in Table 2-5.

Table 2-5. Land use/cover classifications and descriptions

NLCD Class	Description
11	Open Water
12	Perennial Ice/Snow
21	Developed-Open Space
22	Developed- Low Intensity
23	Developed- Med. Intensity
24	Developed- High Intensity
31	Barren Land
41	Deciduous Forest
42	Evergreen Forest
43	Mixed Forest
52	Shrub/Scrub
71	Grassland/Herbaceous
81	Pasture/Hay
82	Cultivated Cropland
90	Woody Wetlands
95	Herbaceous Wetlands

Correlation of these classifications to surface roughness for modelling and simulation should be reported by the participant.

2.3 Flood Modelling and Simulation Reporting Requirements

For consistency between benchmark participants, each participant is requested to generate information described in Table 2-6 from the results of the flood simulation. These data focus on the hydraulics of the simulation.

Table 2-6. Reporting requirements for flood modelling and simulation

Required Data	Data Description
Breach Discharge	This hydrograph should show the discharge rate from pre-failure to empty reservoir, in units of m^3/s
Cross-Section Discharge	These shall consist of a complete hydrograph for unsteady simulation environments and peak discharges for steady-state simulation environments
Peak Flood Depths	Gridded dataset representing the peak flood depth, units of meters
Flood Wave Arrival Time	Gridded dataset with flood wave arrival time in 5-minute intervals
Peak Unit Flow Rate	Gridded dataset with a value representing the peak unit flow rate in units of m^2/s
Flooded Area	Summation of the total flooded area, units of m^2. In addition, participants will provide flooded area categorized by range of flood depths at .5 meter intervals

The locations in which participants should provide hydrographs are shown in Table 2-7.

Table 2-7. Cross-section locations

Cross Section ID	X Location (m)	Y Location (m)
1	4737.18	6755.39
2	5971.79	7053.10
3	7486.60	7582.85
4	9100.84	7773.91
5	10716.96	7397.67

2.3.1 Solution Metadata

As described above, Participants are expected to provide gridded data for some of the reporting requirements. These gridded data files must conform to the metadata of the original data provided in the benchmark. These are summarized in Table 2-8.

Table 2-8. Gridded data metadata requirements

Metadata Parameter	Value
Left Extent	0
Bottom Extent	0
Right Extent	25831.81905
Top Extent	9930.941439
Cellsize	9.4760892
Columns	2726
Rows	1048

Acronyms and Abbreviations

DEM	Digital Elevation Model
NLCD	National Land Cover Dataset
kPa	Kilopascal

Appendix A: Summary of Breach Formation Methods

MacDonald and Langridge – Monopolis (1984):

MacDonald and Langridge -Monopolis utilized 42 data sets (predominantly earthfill, earthfill with a clay core, and rockfill) to develop a relationship for what they call the "Breach Formation Factor." The Breach Formation Factor is a product of the volume of water coming out of the dam and the height of water above the dam. They then related the breach formation factor to the volume of material eroded from the dam's embankment. The data that MacDonald and Langridge-Monopolis used for their regression analysis had the following ranges:

- **Height of the dams:** 4.27 – 92.96 m (14 – 305 ft)
  - with 76% < 30 m, and 57% < 15 m
- **Breach Outflow Volume:** 0.0037 – 660.0 $m^3 \times 10^6$ (3 - 535,000 acre-ft)
  - with 79% < 25.0 $m^3 \times 10^6$, and 69% < 15.0 $m^3 \times 10^6$

For earthfill dams:

$$V_{eroded} = 0.0261 \left(V_{out} * h_w\right)^{0.769}$$

$$t_f = 0.0179 \left(V_{eroded}\right)^{0.364}$$

For earthfill with clay core or rockfill dams:

$$V_{eroded} = 0.00348 \left(V_{out} * h_w\right)^{0.852}$$

Where:

V_{eroded}	=	Volume of material eroded from the dam embankment (m^3)
V_{out}	=	Volume of water that passes through the breach (m^3). i.e. storage volume at time of breach plus volume of inflow after breach begins, minus any spillway and gate flow after breach begins.
h_w	=	Depth of water above the bottom of the breach (m).
t_f	=	Breach formation time (hrs).

$$W_b = \frac{V_{eroded} - h_b^2\ (CZ_b + h_b Z_b Z_3 / 3)}{h_b(C + h_b Z_3 / 2)}$$

Where:	W_b	=	Bottom width of the breach (m)
	h_b	=	Height from the top of the dam to bottom of breach (m)
	C	=	Crest width of the top of dam (m)
	Z_3	=	$Z_1 + Z_2$
	Z_1	=	Average slope (Z_1:1) of the upstream face of dam.
	Z_2	=	Average slope (Z_2:1) of the upstream face of dam.
	Z_b	=	Side slopes of the breach (Z_b:1), 0.5 for the MacDonald method.

Note: the MacDonald and Langridge-Monopolis paper states that the equation for the breach formation time is an envelope of the data from the earthfill dams. An envelope equation implies that the equation will tend to give high estimates (too long) of the actual breach time (for homogenous earthfill dams). Wahl's study states this method will over predict times in some cases, while many equations will under predict.

Froehlich (1995a):

Froehlich utilized 63 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rockfill data sets to develop as set of equations to predict average breach width, side slopes, and failure time. The data that Froehlich used for his regression analysis had the following ranges:

- **Height of the dams:** 3.66 – 92.96 m (12 – 305 ft)
  - with 90% < 30 m, and 76% < 15 m

- **Volume of water at breach time:** 0.0130 – 660.0 $m^3 \times 10^6$ (11 - 535,000 acre-ft)
  - (with 87% < 25.0 $m^3 \times 10^6$, and 76% < 15.0 $m^3 \times 10^6$)

$$B_{ave} = 0.1803\ K_o\ V_w^{0.32}\ h_b^{0.19}$$

$$t_f = 0.00254\ V_w^{0.53}\ h_b^{-0.90}$$

Where:	B_{ave}	=	Average Breach Width (m)
	K_o	=	Constant (1.4 for overtopping failures, 1.0 for piping)
	V_w	=	Reservoir volume at time of failure (m^3)
	h_b	=	Height of the final breach (m)
	t_f	=	Breach formation time (hrs).

Froehlich states that the average side slopes should be:

1.4H:1V	Overtopping failures
0.9H:1V	Otherwise (i.e. piping/seepage)

Froehlich (2008):

In 2008 Dr. Froehlich updated his breach equations based on the addition of new data. Dr. Froehlich utilized 74 earthen, zoned earthen, earthen with a core wall (i.e. clay), and rockfill data sets to develop as set of equations to predict average breach width, side slopes, and failure time. The data that Froehlich used for his regression analysis had the following ranges:

- **Height of the dams:** 3.05 – 92.96 m (10 – 305 ft)
    - with 93% < 30 m, and 81% < 15 m
- **Volume of water at breach time:** 0.0139 – 660.0 $m^3 \times 10^6$ (11.3 - 535,000 acre-ft)
    - (with 86% < 25.0 $m^3 \times 10^6$, and 82% < 15.0 $m^3 \times 10^6$)

$$B_{ave} = 0.27\,K_o\,V_w^{0.32}\,h_b^{0.04}$$

$$t_f = 63.2\sqrt{\frac{V_w}{g h_b^2}}$$

Where:

B_{ave}	=	Average Breach Width (m)
K_o	=	Constant (1.3 for overtopping failures, 1.0 for piping)
V_w	=	Reservoir volume at time of failure (m^3)
h_b	=	Height of the final breach (m)
g	=	Gravitational acceleration (9.80665 m/s^2)
t_f	=	Breach formation time (Seconds).

1.0 H:1V	Overtopping failures
0.7 H:1V	Otherwise (i.e. piping/seepage)

Von Thun and Gillette (1990):

Von Thun and Gillette used 57 dams from both the Froehlich (1987) paper and the MacDonald and Langridge-Monopolis (1984) paper to develop their methodology. The method proposes to use breach side slopes of 1.0H:1.0V, except for dams with cohesive soils, where side slopes should be on the order of 0.5H:1V to 0.33H:1V. The data that Von Thun and Gillette used for their regression analysis had the following ranges:

- **Height of the dams:** 3.66 – 92.96 m (12 – 305 ft)
    - with 89% < 30 m, and 75% < 15 m

- **Volume of water at breach time:** 0.027 – 660.0 m^3 x 10^6 (22 - 535,000 acre-ft)
    - with 89% < 25.0 m^3 x 10^6, and 84% < 15.0 m^3 x 10^6

Note that Von Thun and Gillette's breach formation time equations are presented for both "erosion resistant" and "easily erodible" dams. The paper states: "It is suggested that these limits be viewed as upper and lower bounds corresponding respectively to well-constructed dams of erosion resistant materials and poorly-constructed dams of easily eroded materials".

$$B_{ave} = 2.5\, h_w + C_b$$

Where: B_{ave} = Average breach width (m)
h_w = Depth of water above the bottom of the breach (m)
C_b = Coefficient, which is a function of reservoir size, see below.

Reservoir Size, m^3	C_b, meters	Reservoir Size, acre-feet	C_b, feet
< $1.23 \cdot 10^6$	6.1	< 1,000	20
$1.23 \cdot 10^6$ - $6.17 \cdot 10^6$	18.3	1,000-5,000	60
$6.17 \cdot 10^6$ - $1.23 \cdot 10^7$	42.7	5,000-10,000	140
> $1.23 \cdot 10^7$	54.9	>10,000	180

$$t_f = 0.02\, h_w + 0.25 \quad \text{(erosion resistant)}$$

$$t_f = 0.015\, h_w \quad \text{(easily erodible)}$$

Where: t_f = Breach formation time (hrs).
h_w = Depth of water above the bottom of the breach (m).