6.0 Appendix

The following 10 papers were part of a one-day workshop; “ “Workshop on International Progress in Dam Breach Evaluation,” held at the Annual Conference of the Association of State Dam Safety Officials 2004 Dam Safety, Phoenix, AZ.

CADAM / IMPACT / FLOODSITE

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Abstract

In the EU, the asset value of dams and flood defence structures amounts to billions of Euro. These structures include, amongst others, concrete and embankment dams, tailing dams, flood banks, dikes, etc. Many large dams in Europe are located close to centres of population and industry and the consequences of catastrophic failure of one of these structures would be far worse than most other types of technological disaster. To manage and minimise risks effectively, it is necessary to be able to identify hazards and vulnerability in a consistent and reliable manner, to have good knowledge of structure behaviour in emergency situations, and to understand the potential consequences of failure in order to allow effective contingency planning for public safety. This has led to concerted long term research in Europe (including the CADAM, IMPACT, and FLOODsite projects) to reduce uncertainty in predicting extreme flood conditions and improve predictions of risk due to these structures. The specific objectives of the research described in this paper and the following sessions are to advance scientific knowledge and understanding, and develop predictive modelling tools and methods in a number of areas including: (1) breaching of embankments, (2) catastrophic inundation, (3) mechanisms of sediment movement and (4) embankment integrity assessment through the use of geophysical techniques.

What are CADAM, IMPACT and FLOODSITE?

The European Commission funds multiple, wide ranging programmes of research and development work aimed at improving the efficiency and quality of life in Europe. Research programmes are typically aimed at addressing European, rather than national issues. Research funding is normally to the extent of 50% for commercial organisations so as to encourage integration and support from industry, which in turn helps to ensure the value of the research. CADAM, IMPACT and FLOODSITE are all projects funded by the Commission that address different aspects of flood risk management.

CADAM (Concerted action on dambreak modelling) was completed in 2000 and, amongst other results, provided prioritised recommendations for research in the field of dambreak analysis to improve the reliability of predictions. CADAM did not fund new research work, but provided a mechanism for researchers and practitioners to meet, exchange information and to some extent, co-ordinate existing national research work. The value of funding for CADAM was ~ €250K. IMPACT (Investigation of extreme flood processes and uncertainty) is a 3-year project under the EC 5th framework programme that finishes in November 2004. IMPACT addresses a number of the key issues that were highlighted by the

CADAM project. The value of work undertaken by IMPACT is ~€2.6M. FLOODsite (Integrated flood risk analysis and management methodologies) is a new project funded under the EC 6th framework programme integrating a wide range of work, undertaken by 36 different partners drawn from 13 countries. The value of work funded is ~€14M. The focus of work under FLOODsite is on flood risk management in general, whilst CADAM and IMPACT specifically address dambreak or extreme flood related issues. Nevertheless, there are aspects of research under the FLOODsite project that are of continuing interest to the dams industry.

The CADAM Concerted Action Project

Members of the CADAM project team comprised researchers and industrialists from across Europe who had an interest in the various aspects of dam-break modelling. The CADAM project team aimed to:

- exchange dam-break modelling information between participants: Universities <=> Research organisations <=> Industry
- promote the comparison of numerical dam-break models and modelling procedures with analytical, experimental and field data.
- promote the comparison and validation of software packages developed or used by the participants.
- define and promote co-operative research.

The project was funded as a concerted action by the European Commission, formally commenced in February 1998 and ran for a period of two years. The principal focus of the Concerted Action was a series of expert meetings and workshops, each of which considered a particular topic (e.g. breach formation, flood routing, risk analysis etc). The performance of various numerical models was assessed throughout by comparison under analytical and physical model tests cases and finally against real dam break data. Detailed information relating to the project may be found at www.hrwallingford.co.uk/projects/CADAM).

Findings and Recommendations

The final report from CADAM (available on the CADAM website) drew some 31 specific conclusions and identified a series of areas where further research and development was needed to improve the reliability and accuracy of dambreak analysis. A number of these priority areas form the basis for the IMPACT project research programme. These included:

Breach Formation Modelling:

Considerable uncertainty related to the modelling of breach formation processes was identified and the accuracy of existing breach models considered very limited. Research was recommended in a number of areas including:

1. *Structure failure mechanisms*
2. *Breach formation mechanisms*
3. *Breach location*

Debris and Sediments:

It was identified that the movement of debris and sediment can significantly affect flood water levels during a dambreak event and may also be the process through which contaminants are dispersed. A clear need to incorporate an assessment of these effects within dambreak analyses was identified in order to reduce uncertainties in water level prediction and to allow the risk posed by contaminants, held for example by tailings dams, to be determined.

Flow modelling:

The following research areas relate to the performance of flow models and the accuracy of predicted results:

1. *Performance of Flow Models*
2. *Modelling Flow Interaction with Valley Infrastructure*
3. *Valley Roughness*
4. *Modelling Flow in Urban Areas*

Other research priorities were identified under the headings of database needs and risk / information management. These are not detailed here, but may be found in the CADAM final report.

The IMPACT Project

The IMPACT project addresses the assessment and reduction of risks from extreme flooding caused by natural events or the failure of dams and flood defence structures. The work programme is divided into five main areas, addressing issues raised by the CADAM project. Research into the various process areas is undertaken by groups within the overall project team. Some work areas interact, but all areas are drawn together through an assessment of modelling uncertainty and a demonstration of modelling capabilities through an overall case study application. The IMPACT project provides support for the dam industry in a number of ways, including:

- Provision of state of the art summaries for capabilities in breach formation modelling, dambreak prediction (flood routing, sediment movement etc)
- Clarification of the uncertainty within existing and new predictive modelling tools (along with implications for end user applications)
- Demonstration of capabilities for impact assessment (in support of risk management and emergency planning)
- Guidance on future and related research work supporting dambreak assessment, risk analysis and emergency planning

The core of this paper provides an introduction to the work being undertaken in each of the IMPACT work packages (WPs). Each of these programmes is also detailed under a separate paper in this workshop and more detailed information on all research may be found via the project website at www.impact-project.net. The WPs comprise:

WP2: Breach formation
WP3: Flood propagation
WP4: Sediment movement
WP5: Uncertainty analysis
WP6: Geophysics and data collection

WP2: Breach Formation

Overview of breach work programme aims and objectives

Existing breach models have significant limitations (Morris & Hassan, 2002). A fundamental problem for improving breach models is a lack of reliable case study data through which failure processes may be understood and model performance assessed. The approach taken under IMPACT was to undertake a programme of field and lab work to collate reliable data. Five field tests were undertaken during 2002 and 2003 using embankments 4-6m high. A series of 22 laboratory tests were undertaken during the same period, the majority at a scale of 1:10 to the field tests. Data collected included detailed photographic records, breach growth rates, flow, water levels etc. In addition, soil parameters such as grading, cohesion, water content, density etc. were taken. Both field and lab data were then used within a programme of numerical modelling to assess existing model performance and to allow development of improved model performance.

Current position of research

All field and laboratory modelling work has now been completed. The tests undertaken comprised:

- Field Test #1 6m homogeneous, cohesive embankment. ($D_{50} \approx$ 0.01mm, <15% sand, ~25% clay); overtopping.
- Field Test #2 5m homogeneous non-cohesive embankment ($D_{50} \approx$ 5 mm, <5 % fines); overtopping.
- Field Test #3 6m composite embankment (rockfill with moraine core); overtopping.
- Field Test #4 6m composite embankment (rockfill with moraine core); piping.
- Field Test #5 4m homogeneous embankment (moraine); piping.
- Lab Series #1 This series of 9 tests was based around Field Test #2 at a scale of 1:10. The test material was non-cohesive with variation in material grading, embankment geometry and breach location (side breach).
- Lab Series #2 This series of 8 tests was based around Field Test #1 at a scale of 1:10. The test material was cohesive, with two different materials used, along with different embankment geometry, compaction effort and moisture content.
- Lab Series #3 This series of 5 tests was based around the initiation of pipe formation for Field Test #5.Test data was used to develop reliable failure mechanisms for the field tests. Tests were also undertaken on 1m^3 samples of embankment taken from flood defences in the UK.

In conjunction with the field and laboratory tests and data collection an extensive programme of numerical model testing has been undertaken. Some core objectives of this component of work included:

- Identification of more reliable modelling approaches for simulating breach formation
- Assessment of the level of uncertainty of current breach modelling techniques
- Incorporation of knowledge gained from the field and laboratory tests into existing modelling tools

Figure 1 Field and laboratory breach tests

Modelling was undertaken by members of the IMPACT Team plus additional organisations internationally. Modelling was first undertaken without access to the field or lab data, and subsequently with access. In this way the performance of models and modellers may be assessed objectively – which more closely matches the conditions under which modellers are typically asked to predict embankment failure.

Analysis of the modelling results highlighted some interesting facts and features. Some of these are listed below. All are explained in more detail in the associated paper on breach formation (Hassan et al).

- The laboratory tests highlighted the effect that variation in soil parameters / embankment condition could have on the breach formation process. For example, varying compaction effort and / or changing moisture content, particularly for cohesive materials, could change the erodibility and hence rate and nature of breach growth by an order of magnitude. It was noticeable that very few breach models included these parameters and hence would struggle to reproduce the true embankment behaviour.
- Whilst some models appeared to predict the flood hydrograph reasonably well for some test conditions, all models either over or under predicted the breach growth rate and dimensions. This suggests that prediction of the basic physical growth processes in conjunction with flow calculation is not undertaken accurately. An observation that supports this is the fact that most models predict a critical flow point within the body of the embankment and hence a flow area based upon breach body width. However, both field and laboratory tests often show the growth of curved flow control sections which move upstream out of the breach body and the flow erodes material from the upstream slope.
- Variation in embankment geometry such as slope from 1:3 to 1:2 or 1:4 appears to have little impact on the breach growth process. However, variation in breach location from the centre of an embankment to the side, where lateral growth is restricted in one direction, does have a noticeable effect. This should be taken into consideration when using data to validate models such as from the Teton failure, which was a breach event adjacent to an abutment. It is also relevant in the case of planning breach growth through a landslide generated embankment. Initiating failure adjacent to an abutment will limit the rate of breach growth and hence the rate of flooding downstream.
- An average accuracy of perhaps ±50% may be attributed to many models (broadly considering timing, peak discharge etc). Models simulating aspects of embankment soil behaviour (e.g. slope stability, failure etc.) appeared to show better performance with an indicative accuracy reaching perhaps ±20%.

Future Direction of Research

At the time of writing, analysis of the field, laboratory and numerical modelling data is still to be completed, along with the consideration of scale effects between field and laboratory data. A further area of work investigating factors affecting breach location in long fluvial flood defence embankments is also underway. Results and conclusions from a breach review workshop held at Wallingford in April 2004 will be made available. Research work will continue in this area beyond the completion of the IMPACT project through work packages in the FLOODsite project. It is anticipated, however, that the emphasis of analysis under FLOODsite will shift from breach formation (IMPACT) to breach initiation, so helping to enhance our overall ability to predict and ultimately prevent breach formation occurring.

WP 3: Flood Propagation

Overview and objectives

The objective of this area of work is to improve our understanding of the dynamics of a catastrophic (extreme) flood and to improve our propagation modelling capability. Four partners are involved in this area, namely the Université Catholique de Louvain (Belgium), CEMAGREF (France), CESI (formerly ENEL) and the University of Zaragoza (Spain). The scope is broadly divided into two areas; urban flooding and flood propagation in natural topographies. General objectives of the work package are to:

- identify dam-break flow behaviour in complex valleys, around infrastructure and in urban areas (i.e. gain insight into flood flow characteristics)
- collect flood propagation and urban flooding data from scaled laboratory experiments that can be used for development and validation of mathematical models
- adapt and develop modelling techniques for the specific features of high intensity floods, like those induced by the failure of man made structures
- perform mathematical model validation and benchmarking, compare different modelling techniques and identify best approaches including the assessment of accuracy
- develop guidelines and appropriate strategies concerning modelling techniques for the reliable prediction of flood effects
- identify, select and document a real flood event affecting an urban area to be used as a case study where modelling techniques and lessons learned can be applied and tested

Current position

To achieve these goals a combination of desk work, laboratory experiments, field work and computer modelling has been undertaken.

The mathematical description of extreme flood flows has been tackled on the basis of the non-linear shallow water equations. Issues like non-linear convective transport, the formation of travelling waves (bores and hydraulic jumps), the forcing due to bottom and bank reaction forces (as included in the source terms of the equations of motion) and wetting and drying problems are key issues in devising the appropriate computer model.

As regards modelling of flood propagation in a city, several strategies have been investigated. A simple one-dimensional model of a city with the streets modelled as water channels has proven effective despite its simplicity. Limitations concern model applicability at wide junctions such as squares etc. Also important two-dimensional features of the flow are often lost, as happens with wave reflections and expansions around building corners. Another technique referred to as bottom elevation, represents buildings and obstacles to flow as abrupt elevations of the bed function within a two-dimensional model. This is easy to set up but the numerical method must be robust enough to accommodate this sort of singular source term forcing. Another simple technique analysed entailed increasing the roughness coefficient of the area where buildings or obstructions were located. Finally, the highest level of detail can be attained, at least in theory, by careful two-dimensional meshing of the streets and other city areas prone to flooding.

The aim of the experimental work was to provide an insight into understanding key flow features and to provide data under controlled, reproducible conditions that can be used for computational model validation and improvement. Two types of laboratory experiment at a scaled down geometry have been conducted for urban flooding; one devoted to the study of the flow structure around a single building (front impingement and reflection, refraction etc.) and the other to overall flood-city interaction in which a model city in a scaled down (1:100) valley was subjected to a simulated flood event. The data obtained have been used to set up two benchmark sessions against which computer models have been tested, firstly in a blind phase and then followed by the release of experimental data to allow for model tuning.

A case study based upon a real life flooding event, including inundation of urban areas, has been documented to enable modelling and validation of model results. Modelling of the catastrophic flooding of the small Spanish town of Sumacárcel after failure of the Tous Dam is currently underway and results will be presented at the project final meeting to be held in Zaragoza in November 2004.

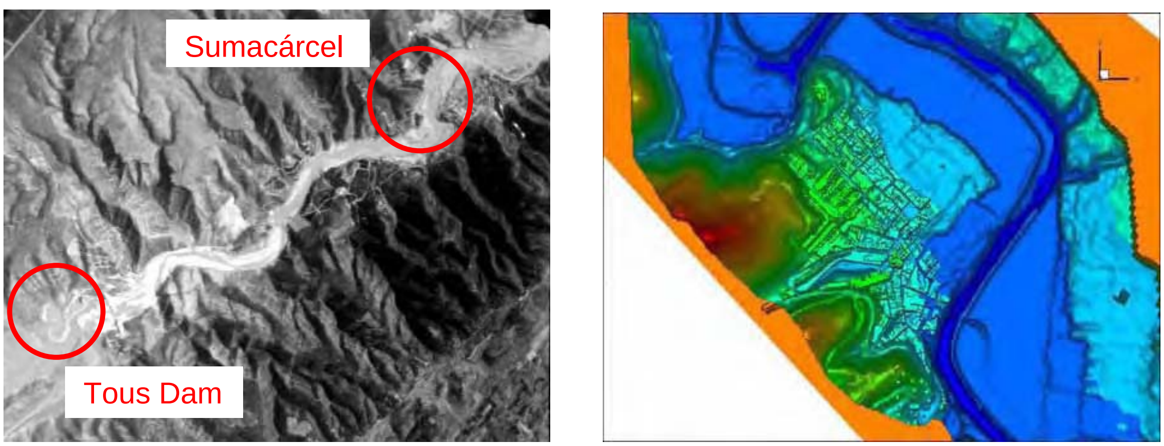


Figure 2: Aerial view of the river reach from Tous Dam to Sumacárcel town about one week after the flood (left); and digital model of the town used for simulations (right).

Results and future trends

Preliminary conclusions from the research can be summarised as follows:

- Catastrophic (high intensity) flooding entails several phenomena that pose difficulties to accurate mathematical modelling. These include highly convective flows, formation of abrupt fronts, wetting and drying of extensive areas and abrupt bathymetries. All these effects are difficult to describe mathematically.
- The most complex mathematical framework currently feasible, based upon the shallow water equations, performs well overall if appropriate integration techniques are used. General trends of the flood as well as some of its details (water depth and velocity evolution at certain locations) can be predicted to within twenty per cent accuracy in most cases

when the flood characteristics (inflow hydrograph and timing etc …) and bathymetry data are well known.

- However important details of the flood may be completely lost when strong deviations from the model equations appear (strong vertical accelerations, high curvature of the streamlines etc…), when the spatial resolution is not enough to precisely describe the geometry or when the flood characteristics are not well known.
  Spatial resolution is likely to be a problem when urban areas are to be treated at the same time as propagation of the flood down a natural valley or open terrain.

As a general conclusion it can be said that careful validation initiatives like the ones represented by the Impact project, in particular involving real life data, are still needed to assess the accuracy and uncertainty of present day models and hopefully improve our modelling capabilities.

WP4: Sediment Movement

Overview of WP4 work-programme aims and objectives

The "Sediment movement" IMPACT work package explores the field of dam-break induced geomorphic flows. In a number of ancient and recent catastrophes, floods from dam or dike failures have induced severe soil movements in various forms. Other natural hazards also induce such phenomena: glacial-lake outburst floods and landslides resulting in an impulse wave in the dam reservoir or in the formation of natural dams subject to major failure risk. In some cases, the volume of entrained material can reach the same order of magnitude (up to millions of cubic meters) as the initial volume of water released from the failed dam.

The main goal of work package is to gain a more complete understanding of geomorphic flows and their consequences on the dam-break wave. Dam-break induced geomorphic flows generate intense erosion and solid transport, resulting in dramatic and rapid evolution of the valley geometry. In counterpart, this change in geometry strongly affects the wave behaviour and thus the arrival time and the maximum water level, which are the main characteristics to evaluate for risk assessment and alert organisation.

Near-field and far-field behaviour

Depending on the distance to the broken dam and on the time elapsed since the dam break, two types of behaviour may be described and have to be understood and modelled.

In the near field, rapid and intense erosion accompanies the development of the dam-break wave. The flow exhibits strong free surface features: wave breaking occurs at the centre (near the location of the dam), and a nearly vertical wall of water and debris overruns the sediment bed at the wave forefront, resulting in an intense transient debris flow. However, at the front of the dam-break wave, the debris flow is surprisingly not so different as a uniform one. An important part of the work program was thus devoted to the characterisation of the debris flow in uniform conditions.

Behind the debris-flow front, the behaviour seems completely different: inertial effects and bulking of the sediments may play a significant role. Surprisingly, such a difficult feature appears to be suitably modelled by a two-layer model based on the shallow-water assumptions and methods. The work package included experiments, modelling and validation of this near-field behaviour.

In the far field, the solid transport remains intense but the dynamic role of the sediments decreases. On the other hand dramatic geomorphic changes occur in the valley due to sediment de-bulking, bank erosion and debris deposition. Experiments, modelling and validation of the far-field behaviour composed he last part of the work package.

Current position of research

It appears that one of the most promising approaches of the near-field modelling is a three-layer description (Fig. 3). Three zones are defined: the upper layer (h_w) is clear water while the lower layers are composed of a mixture of water and sediments, the upper part of this mixture (h_s) being in movement.

In the frame of shallow-water approach, it is possible to express the continuity of both the sediments and the mixture and also the momentum conservation with the additional assumption that the pressure distribution is hydrostatic in the moving layers, which implies that no vertical movement is taken into consideration:

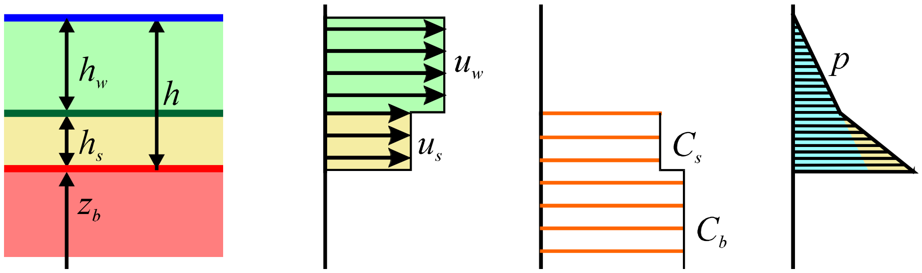


Figure 3. Assumptions for mathematical description of near-field flow

Comparisons and validation were carried out from experimental data on idealised dam-break: typically, horizontal beds composed of cohesionless sediments saturated with water extending on both sides of an idealised "dam", with various sediment and water depths.

For the far field, special attention was paid to the modelling of the bank behaviour. The bank failure mechanism was observed and modelled, taking into account the specific performance of eroded / deposed material in emerged / submerged conditions.

The far-field experiments consisted in a dam-break flow in an initially prismatic valley made of erodible material, evidencing the bank erosion, the transport of the so-deposed material, and the genera widening of the valley.

Results and future direction of work

Experimental results, obtained at the University of Louvain (Belgium) were proposed as a benchmark to various partners: University of Louvain (Belgium), University of Trento, Cemagref (France) and Technical University of Lisbon (Portugal). Fig. 4 presents a comparison between experimental observation and the model presented above.

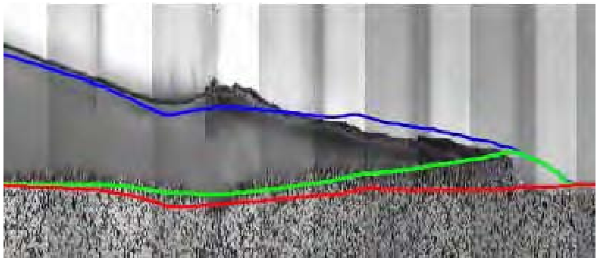


Figure 4. Comparison between experiments and numerical results, 1 s after the dam break

It appears that some characters of the movement are well modelled, such as the jump at the water surface, the scouring at the dam location, the moving layer thickness. The modelled front is ahead but this advance appears constant, which implies that the front celerity is correctly estimated.

Also for the far-field behaviour and the valley widening, the models at this stage can produce valuable results to compare with experimental data from idealised situations.

But it is suspected – and this is general conclusion for the "Sediment movement" work package – that we are far from a completely integrated model able to accurately simulate a complex real case. A tentative answer to this could probably be given from the results of real-case benchmark regarding the Lake Ha!Ha! dike break occurred in Quebec in July 1996. The first results of this benchmark will be available after the last IMPACT meeting to be held in Zaragoza, Spain, in November 2004.

WP 5: Uncertainty Analysis

The objective of this work package was to try and identify the uncertainty associated with the various components of the flood prediction process; namely uncertainty in breach formation, flood routing and sediment transport models. In addition, to demonstrate the effect that uncertainty has on the overall flood prediction process through application to a real or virtual case study and to consider the implications of uncertainty in specific flood conditions (such as water level, time of flood arrival etc.) for end users of the information (such as emergency planners). The scope of work under IMPACT does <u>not</u> allow for an investigation of uncertainty in the impact of flooding or in the assessment and management of flood risk.

The challenge of assessing overall modelling uncertainty is complicated by the need to assess uncertainty within two or more models, to somehow transfer a measure of uncertainty between these models and to develop a system that allows for the different complexities of the various models. Two basic approaches were adopted, namely sensitivity analysis and Monte Carlo analysis. However, whilst a breach formation model may be able to run hundreds or thousands of simulations within a period of hours, it is unrealistic to assume that a complex 2D flood propagation model can undertake a similar process without undertaking weeks or months of analysis. A compromise solution was adopted for IMPACT that combines sensitivity analysis, Monte Carlo simulation and expert judgement. Whilst this approach may provide an estimate of uncertainty which contains a degree of subjectivity (expert judgement) it also provides a mechanism that is achieved relatively simply and provides a quick indication of potential uncertainty.

At the time of writing, the approach had been tested using the HR Breach model only. The steps undertaken included:

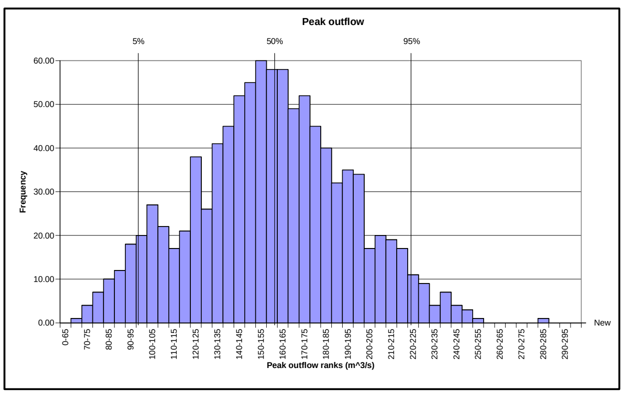
- Sensitivity analysis of the model to a range of model parameters (implicit within this is expert judgement on selection of appropriate and realistic ranges for parameter variation)
- Prioritisation of the modelling parameters to identify those with the greatest effect on modelling results
- Selection of the top three parameters for Monte Carlo analysis (implicit within this is the selection of a probability distribution function for each parameter, again based upon judgement)
- Analysis of results from 1000 model runs; selection of upper, mid and lower scenarios leading to a comparison between the base run (best estimate of model with chosen

modelling parameters) and the uncertainty analysis upper, mid and lower estimates

Table 1 shows analysis of results from 44 model runs to assess model sensitivity to various modelling parameters. This analysis considers only peak discharge. Figure 5 then shows the distribution of model results from the Monte Carlo analysis (based on peak discharge) and specific flood hydrographs representing base, upper, mid and lower scenarios.

Table 1: Sensitivity of the peak outflow to the different input parameters

Input Parameter	Input Parameter Range		Output Parameter variation analysis									
			Min	Max	Mean	Base	% Var. from the Mean			% Var. from the base		
							Min.	Max.	Range	Min.	Max.	Range
Sediment transport eq.	-	-	50	236	118	131	-57.4	100.4	157.7	-61.6	80.4	142.1
Sediment flow factor	0.5	3.0	61	190	132	131	-53.6	44.6	98.2	-53.4	45.3	98.7
Angle of friction	25	45	67	166	122	131	-44.8	35.9	80.6	-48.6	26.4	75.0
Breach width to depth ratio	0.5	2.5	49	116	93	131	-46.7	24.8	71.5	-62.3	-11.6	73.9
CD	1.5	1.9	115	193	160	131	-28.1	20.6	48.7	-12.1	47.4	59.5
Density (KN/m3)	19	23	67	127	104	131	-35.4	21.7	57.1	-48.6	-3.2	51.8
Manning	0.020	0.045	115	146	132	131	-12.7	10.1	22.7	-11.8	11.1	23.0
D50 (mm)	3	6	124	151	138	131	-9.7	9.8	19.5	-5.2	15.3	20.5
Cohesion (KN/m2)	0	10	120	132	126	131	-4.7	4.4	9.1	-8.2	0.5	8.8



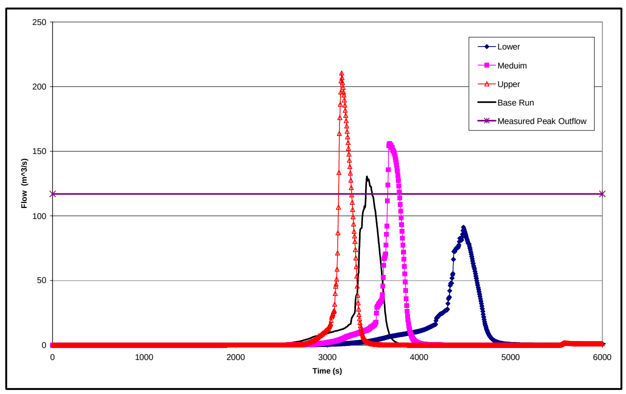


Figure 5: Probability distribution of peak outflow and selected upper, mid and lower scenarios

These results start to give an indication of the uncertainty within the overall flood prediction process. Certainly for breach peak discharge, the suggested realistic range (in this case) is at least ~±30% for peak discharge. However, it is interesting to note that the base run, which represents the experts best judgement in this case, is within 10% of the observed peak discharge. It remains to be seen within the IMPACT project how this band of uncertainty translates through a flood propagation model to uncertainty within the prediction of flood water levels lower in the valley. Whilst attenuation of the flood hydrograph along the valley will tend to reduce the band of uncertainty in water level prediction, the addition of uncertainty within the flood propagation model itself will tend to increase the uncertainty in water level prediction.

Future Direction of Work

During the summer of 2004 further analysis will be undertaken to link propagation of breach and flood routing uncertainty, leading to an overall prediction of uncertainty within the estimation of flood water levels lower in the valley.

WP6: Geophysical Investigation

This 2-year module of work was added to the IMPACT project through a programme to encourage wider research participation with Eastern European countries. The work comprises two components; (1) review and field testing of different geophysical investigation techniques and (2) collation of historic records of breach formation.

The objective of the geophysical work is to develop an approach for the 'rapid' integrity assessment of linear flood defence embankments. This aims to address the need for techniques that offer more information than visual assessment, but are significantly quicker (and cheaper) than detailed site investigation work. Research is being undertaken through a series of field trial applications in the Czech Republic at sites where embankments have already been repaired and at sites where overtopping and potential breach is known to be a high risk.

The objective of collecting breach data is to create a database of events that includes as much information as possible relating to the failure mechanisms, local conditions, embankment material and local surface materials. Analysis may then be undertaken to identify any correlation between failure mode, location and embankment material, surface geology etc.

Geophysics and Data Collection

Geophysical methods may be used in several variants, most commonly in the surface variant (measurement is performed directly on the earth surface), in the underground variant (in the drills, adits, cellars, etc.) and in the variant of remote monitoring from aeroplanes, satellites, etc. (so-called remote sensing).

During a survey for either hydrogeological, engineering-geological, or geotechnical purposes, it is necessary to select an appropriate combination of these approaches and methodology for the field works. Likewise, it is necessary to understand the relationship between measured physical properties of rocks and parameters that have to be measured or indirectly determined. From the principle point of view, geophysical methods are considered to be indirect methods, because they substitute direct field works such as drilling, excavation etc. Thus, they may significantly save both time and money in comparison and / or combination with direct methods. The main contribution of geophysical methods consists, therefore, in getting higher quality, more extensive and more reliable background information for further survey works. During their application, it is necessary to remain flexible in selecting the most appropriate methods for the site, by working in stages the most effective approach, both scientifically and financially, may be developed.

The key to success in utilising results of geophysical methods is in attaining close co-operation of geophysicists with specialists in hydrogeology and engineering geology. The aim of geophysical testing measurements within the IMPACT project is to investigate and confirm the possibility of using these non-destructive methods in assessing the description and state of existing flood defence dikes. In particular, the rapid assessment of long lengths of flood defence embankment. Geophysical measurements have been conducted on a number of pilot sites within the IMPACT study utilising the following geophysical methods:

- ***geoelectric methods***
  resistivity profiling (RP), self potential method (SP), multielectrode method (MEM), electromagnetic frequency method (EFM)
- ***seismic methods***
  shallow seismic method (SSM), seismic tomography (ST), multi-channel analysis of seismic waves (MASW)

- ***microgravimetric methods***
- ***GPR methods***
- ***geomagnetic survey, gamma-ray spectrometric survey***

Current position of research

Geophysical tests and the monitoring of dikes have demonstrated the possibility of developing and subsequent application of specific geophysical technology that could be utilised for a “rapid integrity assessment“ approach. The proposed approach is based upon the use of modified apparatus GEM-2. To finalise this approach it is necessary to collect verification data within the defined catchment, and to verify performance by supplementary methods (multi-electrode method ARS-200, method of spontaneous resistance polarisation – SRP, etc.). If successful, the new geophysical monitoring system should help catchment management and organisation through:

- **quick testing measurement** – its purpose is to provide a basic description of dike materials and structures and to delimit quasi-homogeneous blocks and potentially hazardous segments. Repeated quick testing measurement data will be stored in the database, allowing us to analyse long-term changes of the dike condition.
- **diagnostic measurement** for a detailed description of problematic and disturbed dike segments – it serves for optimal repair planning
- **monitoring measurement** of changes of geotechnical parameters – it serves for repair quality control and for observation of earth structures ageing processes, etc.

Results and conclusions so far

The IMPACT monitoring and tests show that with expert application of geophysical methods we can better describe the real state of a dike. We expect that a proposed combination of geophysical methods would supplement other surveys and activities performed during maintenance (analysis of aerial and satellite photographs, inspection walks). We anticipate proposing a convenient methodology for embankment integrity assessment.

Future direction of work

At the present time, we are striving to prepare a programme of work for the year 2005 and 2006. This programme should cover regular monitoring of dikes in the catchment of Odra or Morava river, and the creation of a database with data measured by modified GEM-2 equipment. In problematic sections (changes of homogeneity of dikes) basic data should be complemented with data obtained by the more precise measurements performed by detail geo-electrical method ARS-200. Assessment of this database will enable us to determine the effectiveness and success of this methodology based on the fast preliminary measurement by GEM-2. This methodology should (together with other supplementary and more precise methods) serve to provide a fast and inexpensive check on flood defence embankments, allowing specific changes in their state with time to be identified (i.e. systematic monitoring of dikes by modified GEM-2 should be used for evaluation of weak places in the dikes, where failure could occur in case of overspill). In future, this fast and non-destructive geophysical method could make operation of water-management bodies within the catchment more effective, and assure early prevention of dike failure. Regular measurements of the dike state could enable estimation of risk of the dike failure, under the various hydrological conditions and plan early and effective repairs of embankments in selected sections. This method can be used for dikes up to about 10 m high of any length.

Where from here?

The IMPACT project has made significant advances in science in a number of areas but work is still to be implemented during the summer of 2004 in order to pull together and demonstrate this new knowledge. The Tous Dam failure (Spain) and Lake Ha! Ha! failure (Canada) will be used as case studies to demonstrate modelling capabilities in breach, propagation, sediment movement and uncertainty analysis. Final reporting of this work will be made through a 4th and final project workshop to be held in Zaragoza, Spain on 4-5 November 2004. Information will also be posted via the project website (www.impact-project.net).

The nature of the work undertaken and the type of funding from the EC (50%) means that much of the research work has also been integrated with existing national or organisation research projects. Within the UK, the research is meshed within a wider national programme of work funded by the government. Consequently, uptake of knowledge occurs through these links and, where appropriate, continuation of the research.

The concept and potential value of integrating research programmes, both nationally and internationally, is now being recognised. Effective integration of work avoids duplication of research effort and allows ideas and concepts from a wider range of sources to be considered. Building on best practice and experience from around the world has to be a more beneficial for all partners than remaining isolated in approach!

Within the last few years, real integration of research programmes can be seen nationally and internationally. This has probably been prompted by the growing use of the Internet as a means of disseminating information. For example, within the UK there are three major programmes of work for which full integration is being attempted. These programmes will run during the coming 3-5 years and comprise:

- Environment Agency / Defra national flood defence programme (applied research in field of flood risk management) [See www.defra.gov.uk/environ/fcd/default.htm]
- EPSRC / EA / Defra Flood Risk Management Research Consortium (FRMRC) programme (academic research programme) [See www.floodrisk.org.uk]
- EC FLOODsite Project (processes through to implementation for flood risk management) [See www.floodsite.net]

The FLOODsite Project

The FLOODsite project addresses a wide range of issues dealing with flood risk management. The research programme structure is shown by Figure 6 and covers activities from research into specific processes, through flood risk management, integration of tools, pilot site application and development, training, dissemination and networking. FLOODsite is the European Commission project addressing flood risk management.

The scope of FLOODsite is such that it is not possible to detail the programme here. However, it should be noted that issues of direct relevance to the dams industry such as breach initiation, flood inundation, integrated modelling and decision support tools, emergency planning tools, vulnerability, social and economic impacts are included within the programme of work. The European Commission has also recognised the value of integrating research from around the world. Under FP6, research projects may include partners from outside of the EC – such as the United States. Such a change in approach has not yet been widely recognised by European researchers, and uptake of funds to date has been limited. Opportunities exist here! For more information on European research initiatives see the Cordis website at www.cordis.lu. For detailed information regarding the FLOODsite project, see www.floodsite.net

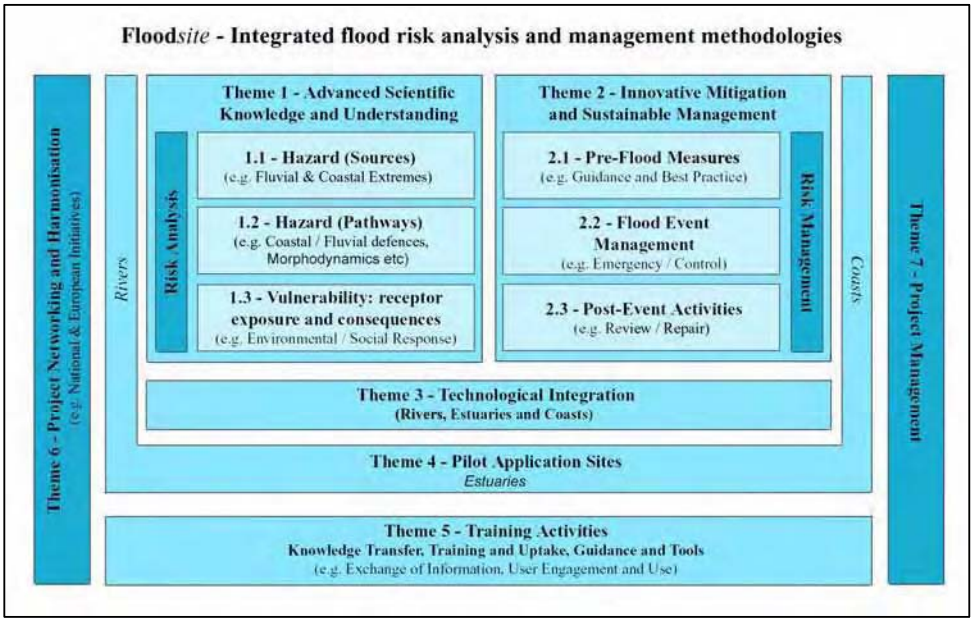


Figure 6 Structure of the FLOODsite project

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The IMPACT project team comprises Universität Der Bundeswehr München (Germany), Université Catholique de Louvain (Belgium), CEMAGREF (France), Università di Trento (Italy), Universidad de Zaragoza (Spain), Enel.Hydro (Italy), Sweco (formerly Statkraft Grøner AS) (Norway), Instituto Superior Technico (Portugal), Geo Group (Czech Republic), H-EURAqua (Hungary) and HR Wallingford Ltd (UK).

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PHYSICAL MODELING OF BREACH FORMATION
Large scale field tests

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Abstract

During extreme flooding and internal erosion failure events, detailed data on flow and formation processes of breaches through embankments/dams are rarely recorded. Consequently, to support numerical model development, testing, and verification, field and laboratory tests of embankment breaches created through overtopping and piping have been conducted. Controlled field tests of rock fill, clay, and glacial moraine embankments, 5-6 m high, have been conducted in Northern Norway. The large-scale field tests are both part of a Norwegian research project called Stability and Breaching of Embankments Dams and an EC project called IMPACT. Laboratory tests of sand, and clay embankments, 0.5 - 0.6 m high (i.e. scale of 1:10) have also been conducted in a large flume at HR Wallingford, UK. This work is presented in a companion paper. An overview and initial observations/conclusions from the large-scale field tests are presented in this paper.

Introduction

There is international focus on the safety evaluation, rehabilitation and strengthening of old embankment dams to resist hydraulic and seismic loads. Updated hydrological information and design criteria often lead to higher anticipated flood levels than the dam was originally designed for. Furthermore, society's increasing awareness of flood risk requires more rigorous analyses of the impact of dam failure, including the assessment of breach formation, flood wave propagation and inundation, and early warning systems (e.g. Høeg, 1998, 2001).The profession and dam regulatory agencies in Norway and abroad realized the need for new and improved technology to develop better guidelines and practice. The Research Council of Norway therefore provided funding to establish a research program in combination with

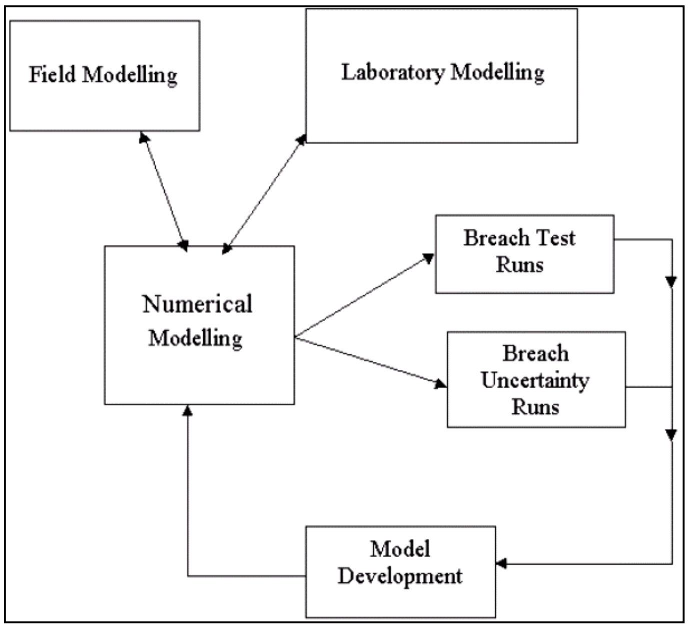

Figure 1 Interaction between modeling approaches

contributions from Norwegian dam owners. The EC and several foreign sponsors also joined the programme. The total budget for the 2001-2004 programme was ca. NOK 19 mill (ca. USD 2.9 mill). In addition to significant financial support, Statkraft SF, Norway's biggest dam owner, allowed the use of the Rössvatn Dam spillway gates and reservoir to supply water to the field tests located downstream, and also provided other services to the research project.. Furthermore, BC Hydro, Canada has sponsored additional testing related to the use of geophysical methods for the possible detection of internal leakage (erosion) in embankment dams. Initiation of the European IMPACT[1] Project was undertaken in parallel with the Norwegian project. An important part of the IMPACT project is the undertaking of field, laboratory and numerical modeling of breach formation through embankments. Objectives for this modeling work are to:

- Establish a better understanding of the embankment breaching process
- Provide data for numerical model validation, calibration and testing, and hence improve modeling tool performance
- Provide information / data to assess the scaling effect between field and laboratory experiments
- Identify best approach /approaches to simulate breach formation through embankments
- Assess and quantify the level of uncertainty of the current breach modeling techniques

Figure 1 shows the interaction between the three modeling approaches undertaken within the IMPACT project. In this paper, details of large-scale field modeling are given. Two additional companion papers detail the laboratory modeling, breach data analysis and uncertainty assessment (Hassan et al, 2004 and Morris et al, 2004)

Description of the embankment breach test site

The large scale embankment test site is located in the middle of Norway in Nordland County and the Hemnes Municipality, near the town of Mo i Rana. The location is shown in Figure 2. On the detailed map the Røssvatn Reservoir and Røssåga River flowing north and into Sørfjorden is seen. Figure 3 shows a picture of the test area with the Røssvassdammen Dam and about 1000 m of the Røssåga River. The test site is located as indicated on the picture, about 600 m downstream of the Røssvassdammen Dam. The location downstream of the Røssvassdammen Dam makes it possible to control the inflow to the reservoir behind the test dam by regulating one or more of the three flood gates. Røssvatn is the intake reservoir for Upper Røssåga power plant which outlets into Stormyr-bassenget reservoir at an elevation of 247.9. The 8500 m reach of the

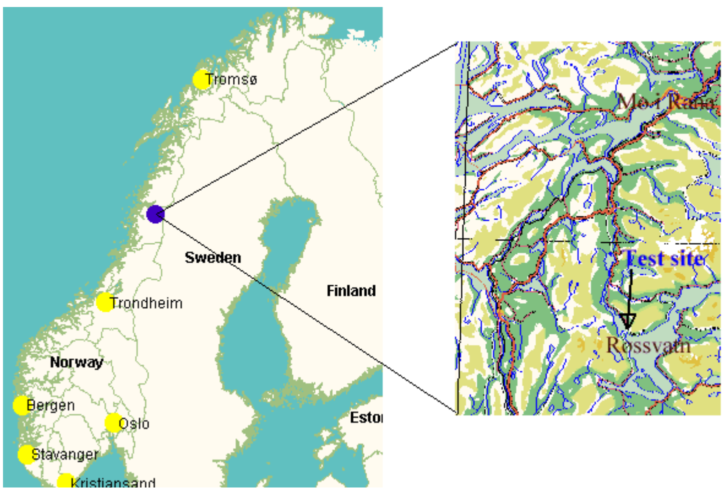

Figure 2 Location of the test-site

1 IMPACT Project: Investigation of Extreme Flood Processes and Uncertainty. EC Contract No: EVG1-CT-2001-00037. www.impact-project.net

River Røssåga in between is normally dry (local inflow only) and floods during spring only occasionally. Stormyrbassenget Reservoir is the intake for Lower Røssåga power plant with outlet in Røssåga River at Korgen. The 12000 m reach of the River Røssåga in between is also normally dry (local inflow only) and only floods occasionally during the spring.

Figure 3 The test-site

Instrumentation

The test site and dams were instrumented and monitored to collect data on inflow and outflow, pore water pressures in the dam body, and detailed information on breach initiation, formation, and progression.

Inflow to the test reservoir was determined by the positioning of the Rossvatn Dam spillway gates. Water level in the reservoir upstream of the test embankment was monitored by two water level gauges, VM1 and VM2, that were constructed and calibrated during the autumn of 2001. Two other gauges, VM3 and VM5, were positioned downstream of the test embankment to measure discharges from the test site. VM3 was a V-notch weir designed to measure discharges less then 100 l/s. VM5 was a tailwater level gauge used to determine discharges greater than 10 m^3/s.

During construction up to eight peizometers were placed inside the dam body for the monitoring of pore pressures. The test dams were equipped with “breach sensors,” for monitoring of the rate of breach development. A breach sensor consists of a tilt sensor and a microprocessor that records the time at which the sensor is displaced. After the dam failure the sensors were picked up in a calm section of the river (small lake) downstream, and the data was retrieved from each sensor. About 100 such sensors were placed in each test dam to map the breach development in space and time. A grid (1x2m) was painted (sprayed) on the dam crest and downstream slope to facilitate the documentation of the breach development. Several digital video cameras were running continuously during the tests as well. A shallow channel or notch was used as a trigger mechanism in overtopping failure tests. This was to ensure that the overtopping failure, when it started, would develop in the centre of the dam and not towards the abutments. Otherwise the presence of the abutments would interfere with the development of the breach opening both vertically and laterally.

Field Test program

A total of 7 field tests (Table 1) have been performed with 5 of these tests as part of the IMPACT project. All the test embankments, with the exception of Test No.4, were tested to complete failure, but the tests were run in stages to gather information for sub-projects a) and b) before failure occurred.

Table 1 Listing of field tests.

Test No.	Type of dam	Objective	EC-IMPACT
1	Homogeneous rockfill dam a) three different test with specially built drainage toe b) Toe removed and the dam brought to failure	Test of stability with high through flow and breaching mechanism of a rock filling.	
2 (1-2002)	Homogeneous clay fill dam	Breaching mechanism of a homogenous cohesive dam	IMPACT
3 (2-2002)	Homogenous gravel dam a) With a rockfill berm up the downstream slope b) With rockfill berm partly removed c) With rockfill berm removed and dam brought to failure	Test of stability of gravel dam and study of the breaching mechanism of a dam of non-cohesive material	3C is part of the IMPACT project
4	Homogeneous rockfill dam, but with a smaller crest width and coarser rockfill than that used in Test # 1.	Test of stability with high through flow and breaching mechanism of a rock filling.	
5 (1-2003)	Rockfill dam with central moraine core. Dam was failed by overtopping	Study of the breaching mechanism	IMPACT
6 (2-2003)	Rockfill dam with central moraine core. Dam was failed by internal erosion	Study of the breaching mechanism	IMPACT
7 (3-2003)	Homogeneous dam made of same moraine as for the core in Tests # 5 and #6. Dam was failed by internal erosion.	Test #7 was run to compare the progression of the breaching process with that observed in test # 6	IMPACT

Test 1-2002 Homogenous clay dam

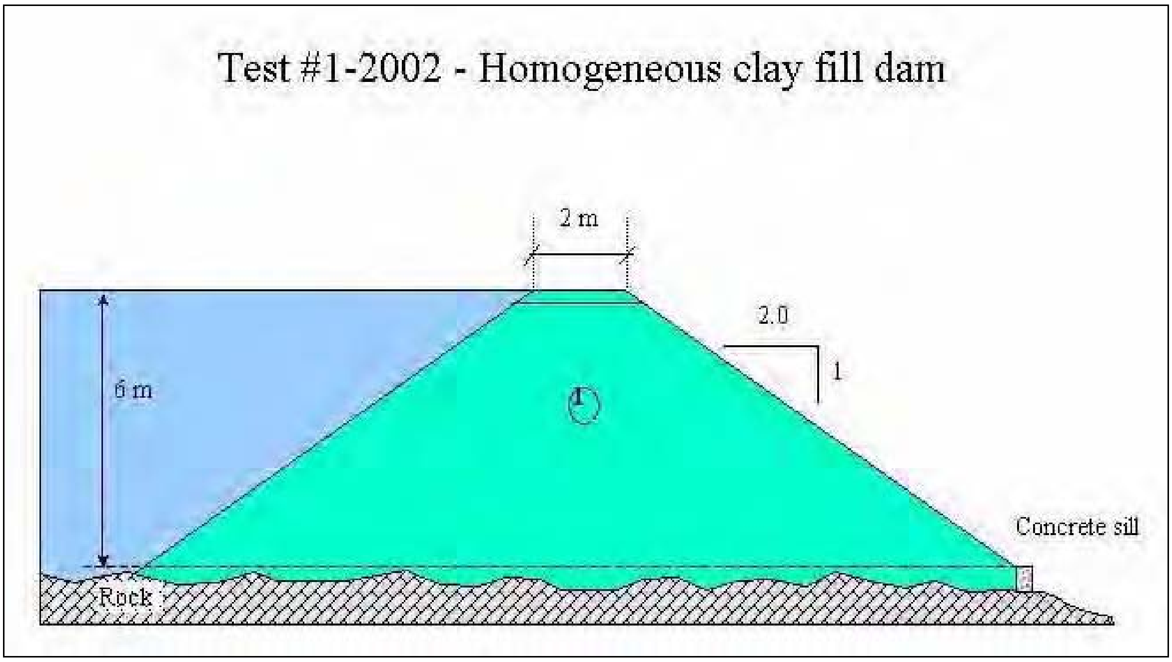

Figure 4 Homogenous clay fill dam (from Höeg et al. 2004)

The layout of the test dam 1-2002 is shown in Figure 4.The sieve curve for the clay(marine clay) is shown in figure 5. The dam was constructed during the period 14 August to 10 September 2002 to the specifications shown in Figure 4. A 0.5 m deep and 3 m wide channel at the top of the dam was made for initiation of the breach. During construction the soil was placed in 15 centimeter layers and mechanically compacted. Due to high water content in the borrow material (w = 28-33%) and extremely wet weather conditions, construction of the dam was difficult. Therefore, construction procedures were altered, placement layer thickness was increased to 0.4 m and the compaction pressure was reduced. Figure 6 shows 4 pictures taken during the test. The outflow hydrograph is shown in Figure 7.

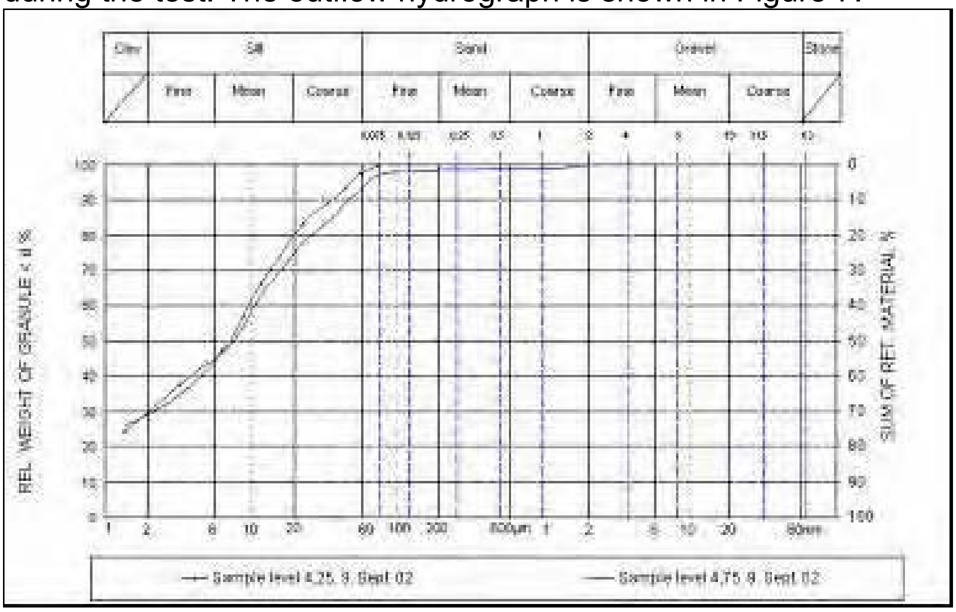

Figure 5 Sieve curve for clay.

Figure 6 Pictures taken during the test of the clay dam

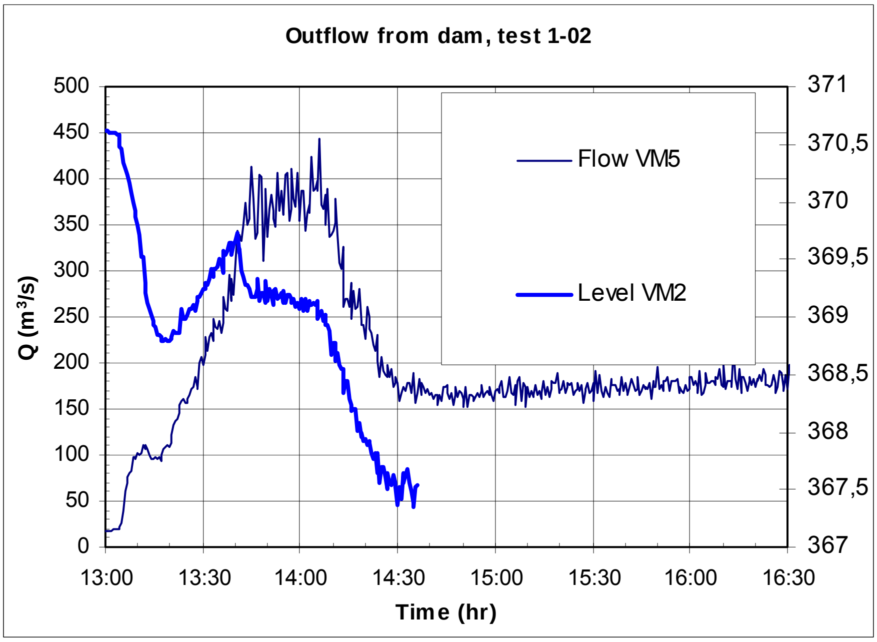

Figure 7 Outflow hydrograph

The initiation phase of the test was long. During this phase headcut development was observed. The width of the headcut remained equal to the width of the initial notch. When the head-cut had moved back to the upstream side of the dam, the breach developed rapidly. The time for the breach can be seen as the sudden drop in the water level at VM2.

Test 2-2002 Homogenous gravel dam

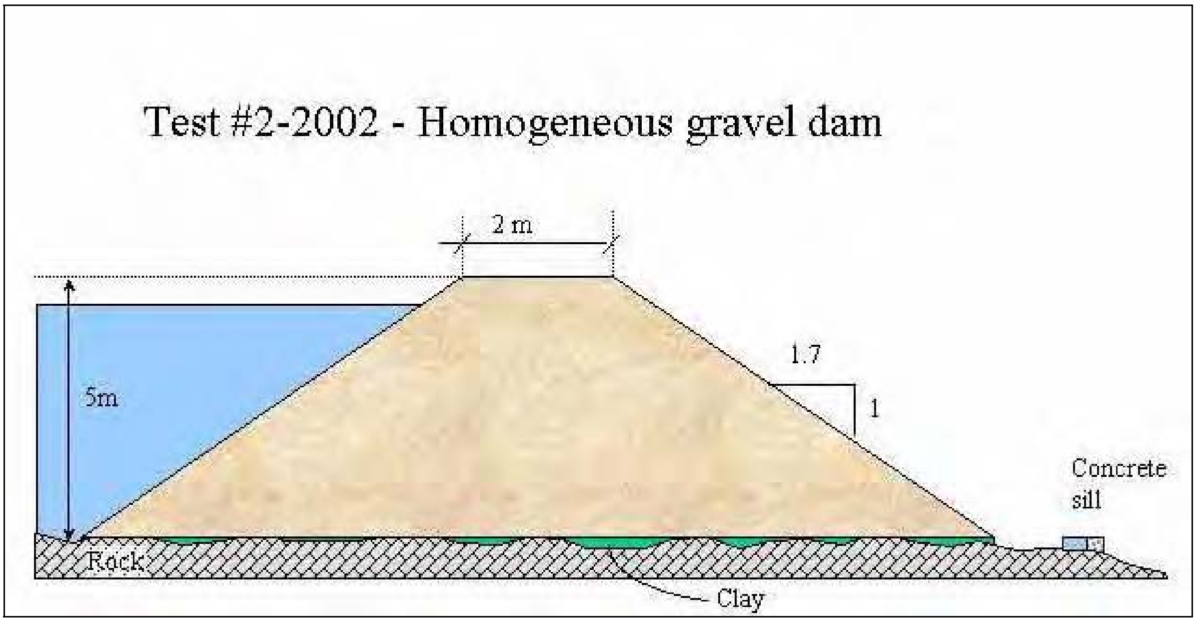

Figure 8a Test of homogenous gravel dam. (from Höeg et al. 2004)

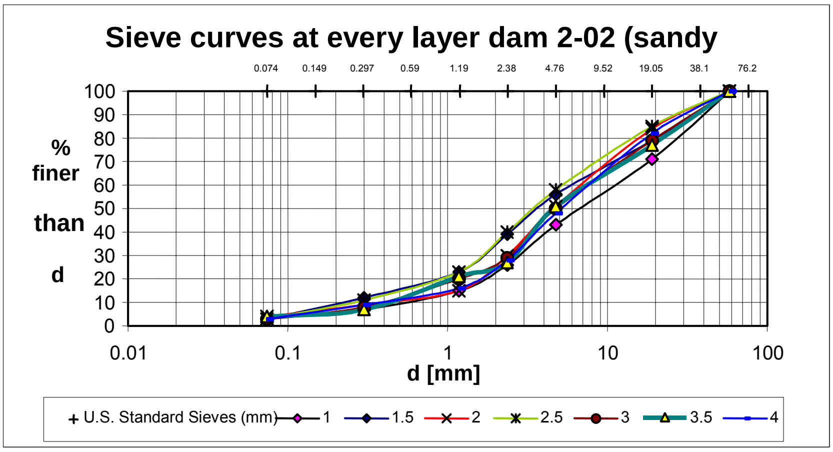

Figure 8b Sieve curves for test 2-2002

The layout of the test dam 2-2002 is shown in Figure 8a. The sieve curve for the gravel used in the dam is shown in Figure 8b. Figure 9 shows 4 pictures taken during the test. The outflow hydrograph is shown in Figure 10.The test was conducted late in the autumn of 2002. The air temperature was zero degree Celsius (freezing point) the night before the test. The upper layer of the dam (a few centimeters) was frozen. Before we could start the test we had to melt that layer. There was no release of water from the gates at Røssvassdammen during the test. Consequently the level of the test dam was lowered rapidly during failure.

The initial phase of the failure process was not as expected. The overtopping discharge was steadily increased from 30 to 50 l/s in a 2-meter wide notch for 45 minutes. During this phase there was headcut development more or less the same way as in the clay dam. Following the breach initiation the vertical erosion finished after 5 minutes and the horizontal erosion after 5-10 minutes

Figure 9 Pictures taken during the test of the gravel dam

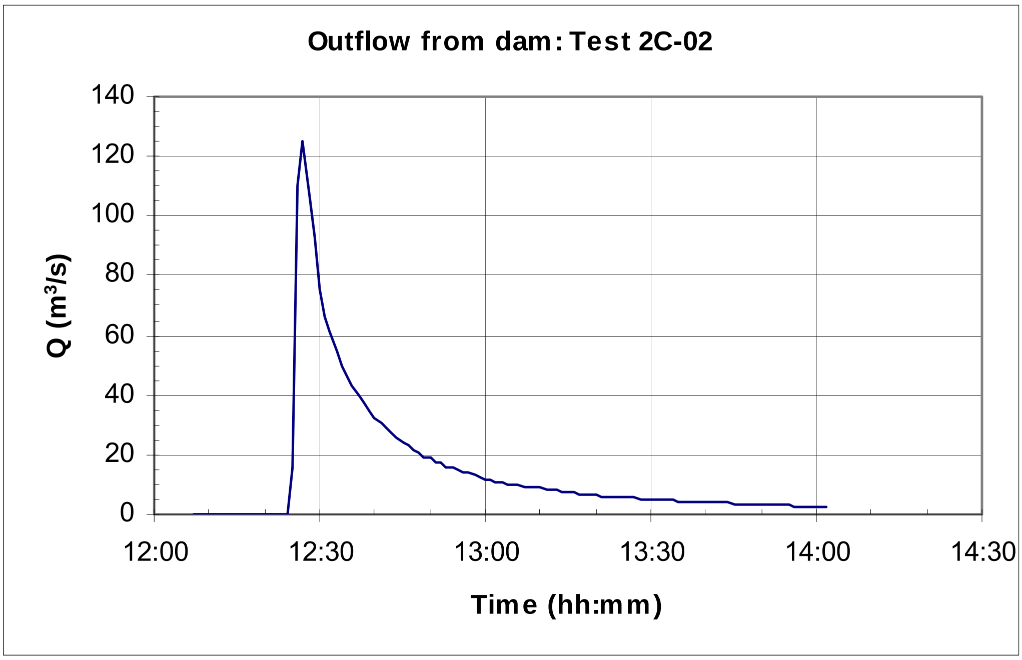

Figure 10 Reservoir level and outflow hydrograph during the gravel dam test

Test 1-2003 Rockfill dam with central moraine core

The layout of the test dam 1-2003 is shown in Figure 11. The dam consists of a moraine core supported by rockfill on the upstream and downstream sides. The grading curves for the moraine (1) and rockfill on the downstream side (2) are shown in Figure 12. On the upstream side the rock material (3) was 300 - 400 mm.

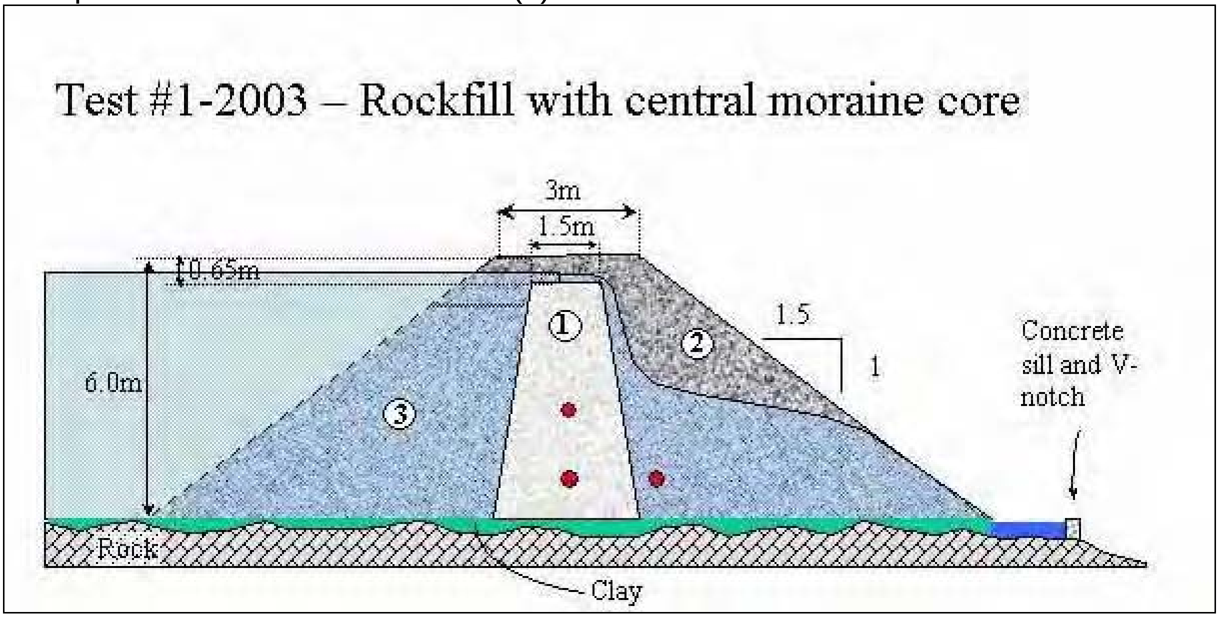

Figure 11 Composite dam: Rockfill with moraine core. (from Höeg et al. 2004)

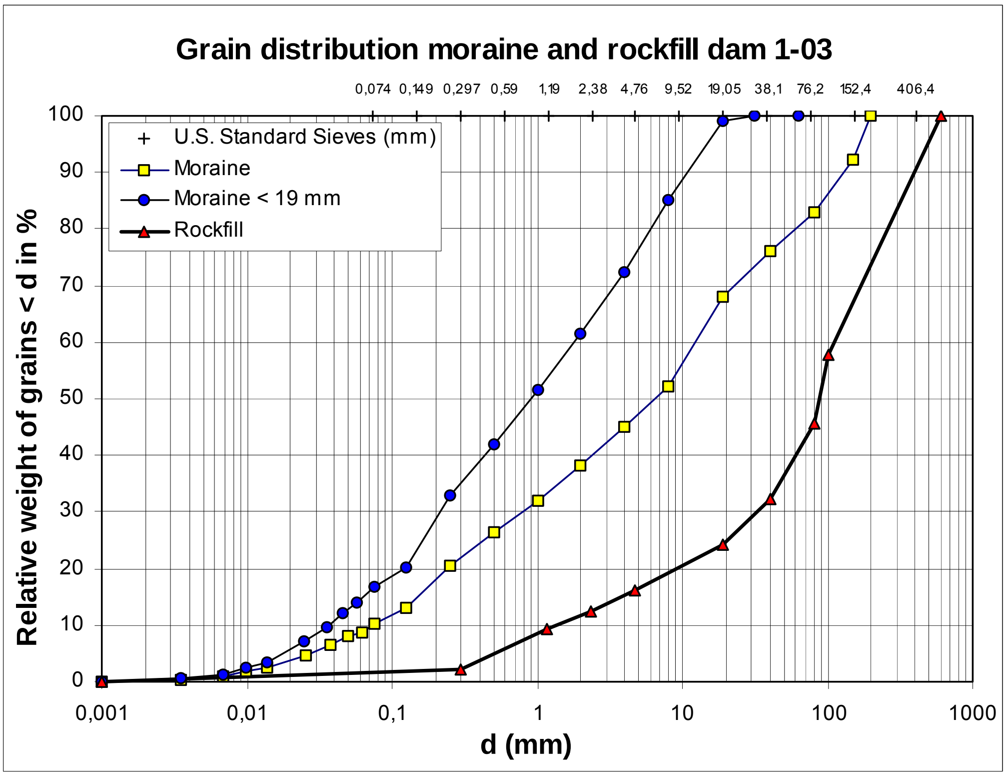

Figure 12 Sieve curves for the materials in the composite dam

Figure 13 Pictures from test with rockfill dam with moraine core.

The outflow hydrograph is shown in Figure 15. Figures 11 and 15 show the elevations of the core, the bottom of the dam crest depression, and the dam crest. From 09:30 to 11:30 hrs the core was overtopped, and the corresponding discharge was 60 and 100l/s for the two water levels shown on Figure 11. From 11:30 to 13:00 hrs the dam crest depression, 0.25m deep and 8m wide, was also overtopped. From 13:00 to 14:00 hrs the dam crest was overtopped, and the combined discharge in the depression and over the crest is shown. Maximum discharge before the gradual downstream erosion (scour) reached the upstream edge of the crest, was 8 m^3/s of which 4 m^3/s was discharged in the dam crest depression, giving a unit discharge of 0.5 m^3/s downstream. A few minutes before 14:00 hrs the dam breach initiated, and the breach developed during the subsequent 10 minutes. The outflow hydrograph is shown in Figure 15. An attempt was made to maintain the maximum reservoir level, but in this case the breach developed too fast compared to the Rössvatn Dam spillway gate operation. The peak breach discharge was about 220 m^3/s.

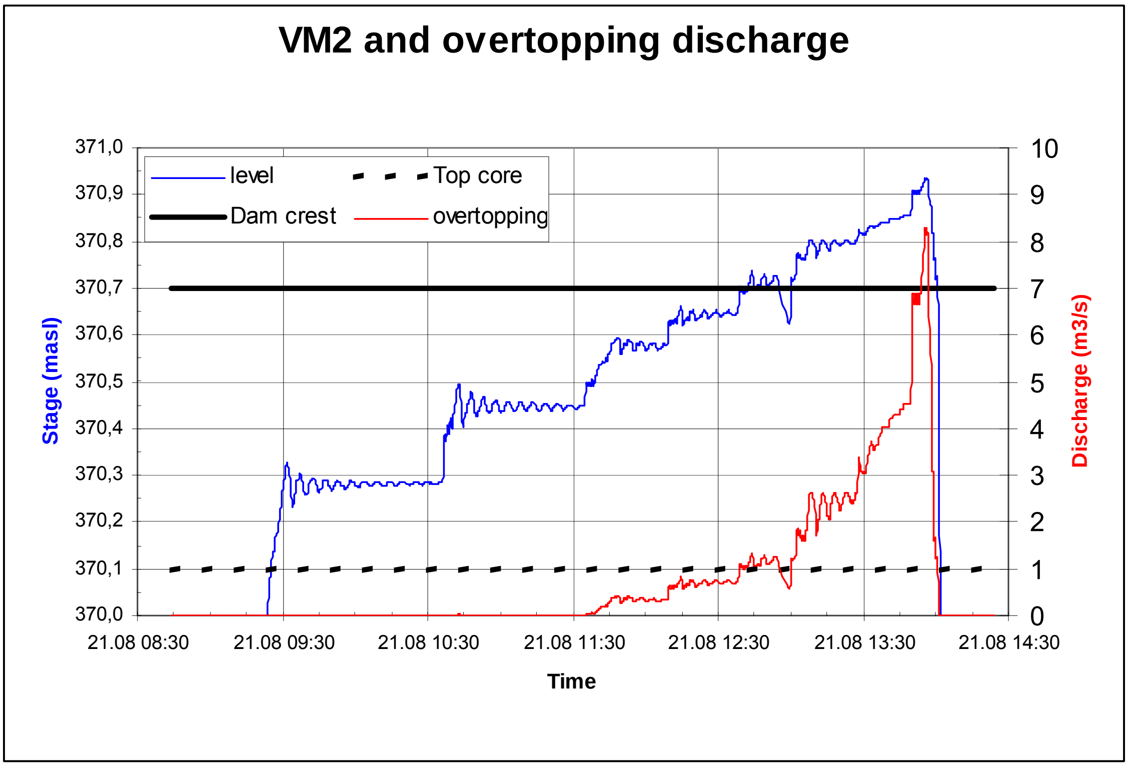

Figure 14 Water elevation in the testdam prior to the test

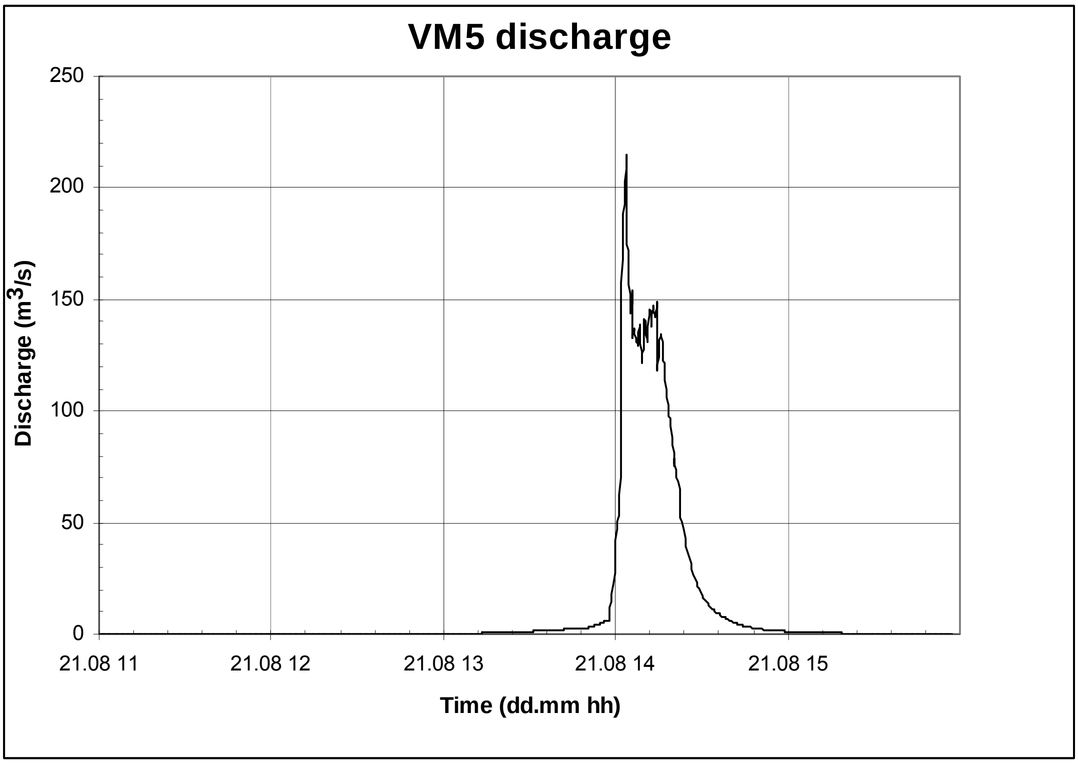

Figure 15 Outflow hydrograph from the test with rockfill dam

Test 2-2003 Rockfill dam with central moraine core breaching by piping

The layout of the test dam 2-2003 is shown in Figure 16. This dam was made of the same material, as the dam in test 1-2003. Two different trigger mechanisms to initiate internal erosion were built into the dam. Two pipes, diameter of 200 mm, with openings on the top were used, as triggers (Figure 17). These pipes were covered with homogenous sand. Trigger device number one was covered with a sand layer of 1 by 1 meter. The sand layer around trigger number 2 was extended to the top of the dam. At the start of the test the pipes were closed at the downstream end. By opening of the valve at the downstream end of the devices the sand was flushed out and the internal erosion started. Trigger device number one was opened first and was kept open for 4 days, but we had no failure of the dam. After opening trigger device number 2 a sinkhole rapidly formed on top of the dam. This can be seen in Figure 18. The sinkhole formed a notch through the dam and the dam failed the same way as by overtopping. The outflow hydrograph is shown in Figure 19.

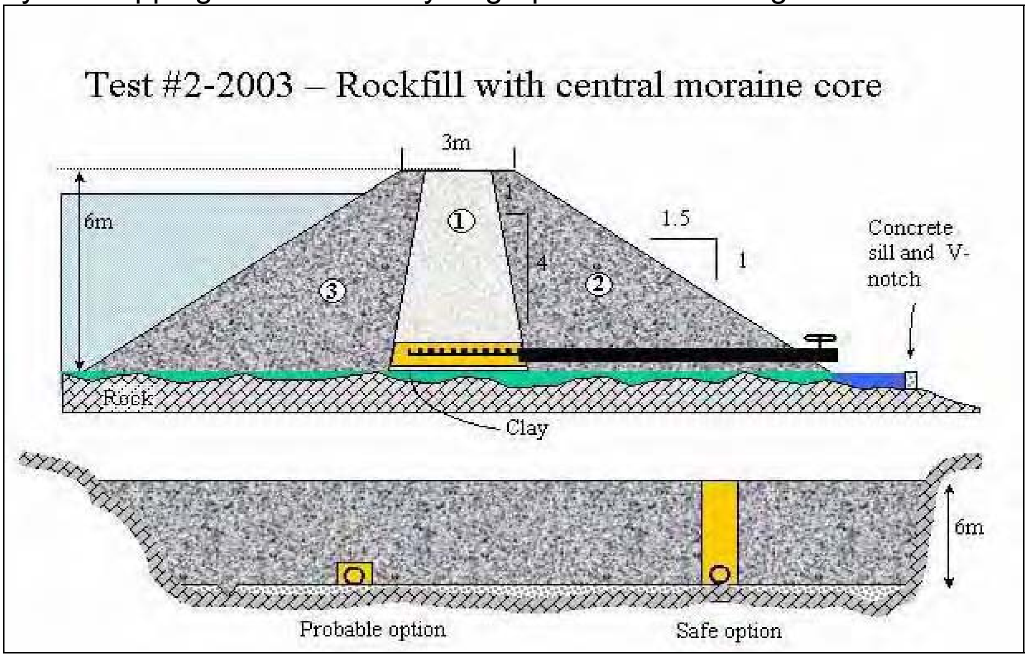

Figure 16 Composite dam with moraine core breaching by overtopping. (from Höeg et al. 2004)

Figure 17 Construction of trigger devices

Figure 18 Picture from test 2-2003

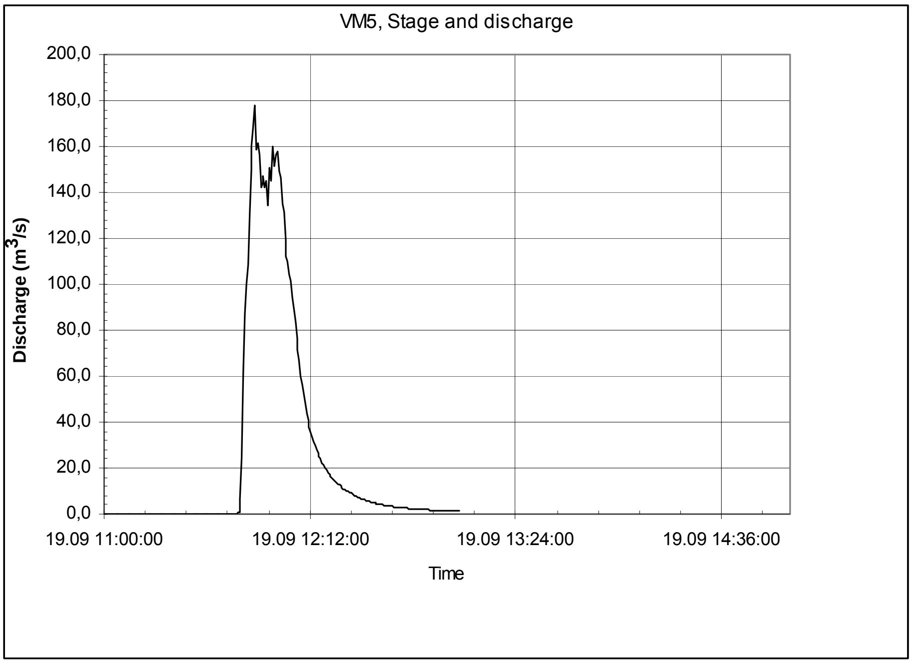

Figure 19 Outflow hydrograph from test 2-2003

Test 3-2003 Homogenous moraine dam

Test 3-2003 was run to compare the progression of the breaching process with that observed in 2-2003 where the moraine was protected by the rockfill upstream and downstream. The layout of the test dam is shown in Figure 20

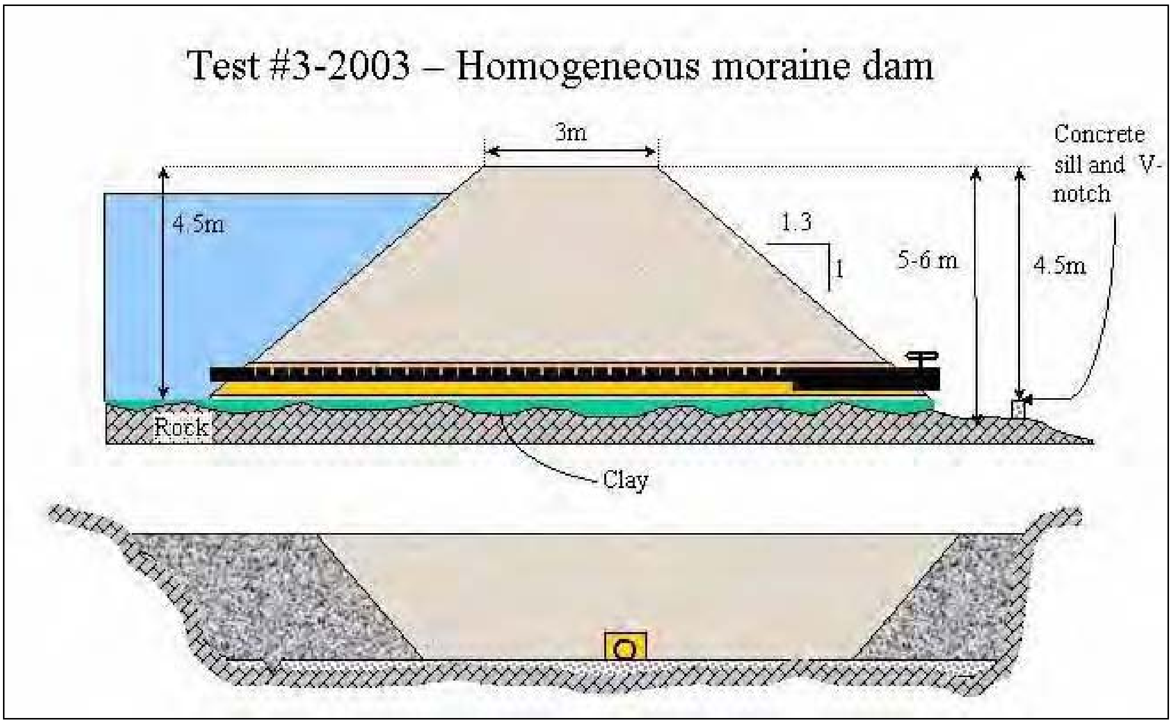

Figure 20 Homogenous clay fill dam. **(from Höeg et al. 2004)**

The sieve curve for the moraine is the same as the sieve curve shown in Figure 12. The trigger mechanism was exactly like the trigger mechanism 1 in test 2-2003. Figure 22 shows 4 pictures taken during the test. The failure of this test was very rapid. It took only about 20 minutes from opening of the trigger mechanism until the dam was breached. The outflow hydrograph is shown in Figure 21. It is interesting to note that the piping failure tests 2-2003 and 3-2003 resulted in virtually identical peak discharges.

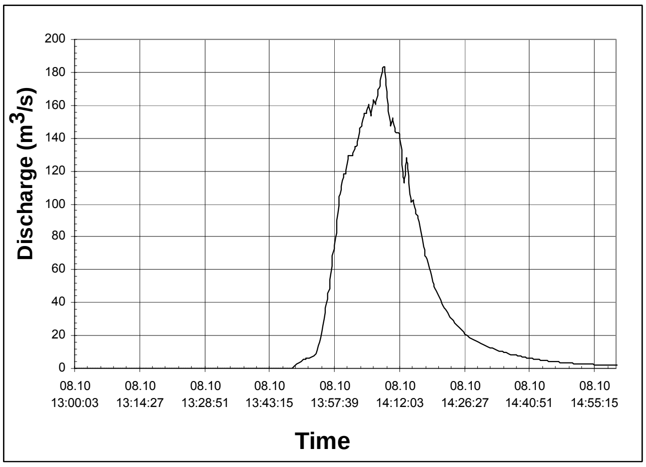

Figure 21 Outflow hydrograph in test 3-2003

Figure 22 Pictures taken during the test

Conclusions

This paper has provided an introduction to field work undertaken in Norway during the last few years, aimed at collecting reliable information and data sets detailing the failure mechanisms of a range of different embankment dams. Reliable data sets now exist for the failure of a range of different large-scale embankment geometries and material types. Analysis of this data has started and is likely to continue for some years. The data will assist in the development of understanding and validation of predictive models. Prior to this analysis, some initial, broad observations may be made based upon field observations and data analysis to date. These include the following:

- The data collected for each test comprises a mixture of flows, levels, breach growth dimensions, video and still photo footage. The failure processes of the different embankments may be observed. Features such as cracking, arching (pipe formation), headcut formation and progression were all observed. Existing breach models does not accurately simulate many of these features.
- The first phase in the external erosion of the downstream slope due to overtopping is slow and very gradual. However, when the scour and unraveling finally reaches the upstream edge of the dam crest, the breaching is rapid and dramatic. The same general observations were made for the rockfill, gravel and clay dams. The opening of the breach first progresses

down to base of the dam, before it expands laterally. The sides of the breach were very steep, almost vertical, in all three materials.

- The rate of breach growth for the homogeneous clay and gravel dams was not as expected. The clay dam failure more quickly, whilst the gravel dam more slowly than expected. It is likely that this was due to the condition of material and nature of construction / compaction. This demonstrates the significant impact that material condition and construction method may have on breach formation and hence the need to consider these aspects within predictive models.
- The internal erosion process, initiated at the defects built into the moraine core of the rockfill dam (Test 2-2003), took a very long time to develop, even in this dam with no filters between the moraine core and the downstream rockfill. Breaching of the dam did not take place until the erosion had proceeded up to the dam crest, and then the dam failed by overtopping as in Test 1-2003, but the breach opening was not so wide.
- The difference in rate of embankment failure for the homogeneous moraine embankment and the composite moraine / rockfill embankment was significant. This demonstrates the importance of the interaction between layers of material within a composite structure. This has implications for overall dam stability and in the development of predictive breach models.
- Many of the field test scenarios simulated typical rockfill embankment dams. As such, there was surprise that the rate and mechanisms of failure observed were typically more resistant than existing analyses and guidelines prescribe.

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BREACH FORMATION:
Laboratory and Numerical Modelling of Breach Formation

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Abstract

Our ability to predict the flow and rate of development of a breach through a flood embankment or dam to date has been limited. Lack of data and understanding of the breach processes are probably the main reasons for this. A program of field and laboratory experiments has been undertaken under the IMPACT project to improve the understanding of the breach processes. In conjunction with this, a programme of numerical modelling comparison and development has been conducted using the data from field and laboratory experiment undertaken under the project. This paper presents details of the undertaken laboratory experiments and numerical modelling. Details of the field experiments are given in a companion paper.

Introduction

An important part of the IMPACT[1]project is the undertaking of field, laboratory and numerical modelling of breach formation through embankments. Objectives for this modelling work are to:

- Establish a better understanding of the embankment breaching process
- Provide data for numerical model validation, calibration and testing, and hence improve modelling tools performance
- Provide information / data to assess the scaling effect between field and laboratory experiments
- Identify best approach /approaches to simulate breach formation through embankments
- Assess and quantify the level of uncertainty of the current breach modelling techniques

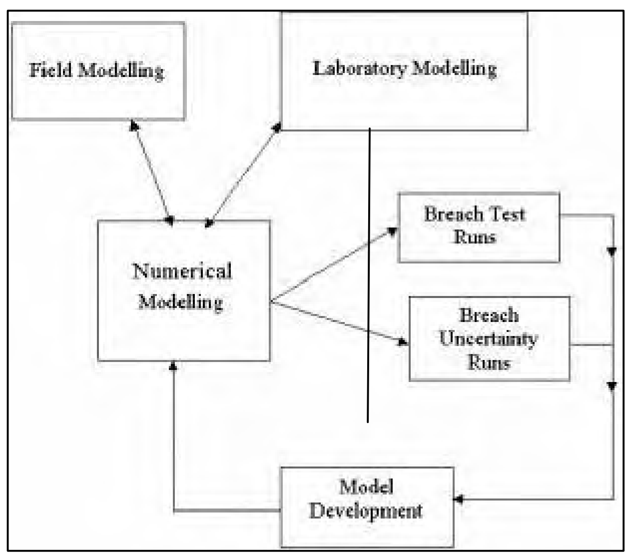


Figure 1: Interaction between modelling approaches

Figure 1 shows the interaction between the three modelling approaches undertaken under the IMPACT project. In this paper, details of the laboratory modelling**,** the breach test

[1] For details on the IMPACT project visit www.impact-project.net

runs, and part of the numerical modelling are given. In another two companion papers[2] details of the field modelling and breach uncertainty runs are given.

Laboratory Modelling

A total of 22 laboratory experiments have been undertaken at HR Wallingford in the UK. The overall objective of these tests was to better understand the breach processes in embankments failed by overtopping or piping and identify the important parameters that influence these processes. These tests were divided into 3 series. Table 1 shows the details of each series of tests. The focus, in this paper, is on the analysis of series #1 and #2.

Table 1: Laboratory tests description

Laboratory Test Description		Laboratory Test Objective
Series # 1 (9 tests)	This series of tests was based around the homogeneous non-cohesive field test at scale of 1:10. Each embankment was built from non-cohesive material, however, more than one grading of sediment were used along with different embankment geometry, breach location and time before failure (seepage effect).	To better understand breach formation processes and to identify the effect of a variety of parameters on these processes in homogeneous non-cohesive embankments failed by overtopping
Series # 2 (8 tests)	This series of tests was based around the homogeneous cohesive field test at scale of 1:10. Each embankment was built from cohesive material, however, two different grading of sediment were used along with different embankment geometry, compaction effort and moisture content.	To better understand breach formation processes and to identify the effect of a variety of parameters on these processes in homogeneous cohesive embankments failed by overtopping
Series # 3 (5 tests)	Assess initiation of the piping mechanism and dimensions for the homogeneous field test	Provide information about the pipe formation to assist in development of the field test failure mechanism
	Material brought from a UK flood embankment. Samples were 1m (W) x 1m (L) x 0.8m (D)	Monitor piping initiation and development

Series #1 - breach processes

The following processes were observed during the breach formation for this series of tests:

1. Water erodes the downstream slope and the slope becomes milder. Head cutting was not observed in this series of tests
2. The crest of the embankment retreats and erodes downward
3. Once the breach is fully developed (i.e. material is nearly eroded to the base), the material below the water level is eroded. This undermines the slopes and leads to block failure
4. The above processes continue until there is not enough water to erode more material
5. Upstream slope erosion was also observed leading to a curved ‘bell mouth’ entrance to the breach. This ‘bell mouth’ weir controlled flow through the breach. Slumping was also

[2] See special workshop #1: “International Progress in Dam Breach Evaluation”

observed on the upstream face due to this erosion.

Figure 2 shows the above processes.

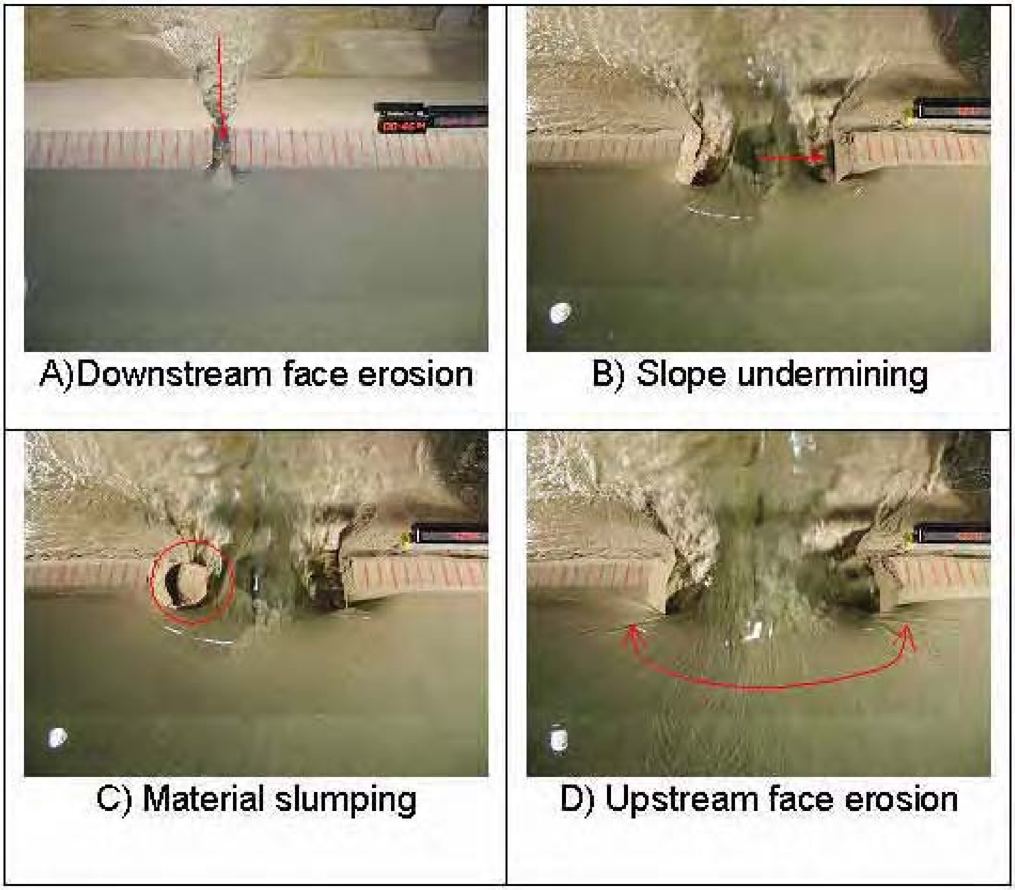


Figure 2: Series #1 - breach processes

Series #1 - effect of various parameters on the breach processes

The effect of various parameters such as grading and geometry was examined in this series of tests. In the following sections, the effect of these parameters is presented.

Effect of D_{50} and Grading

The following three different gradings have been used to examine the effect of material grading on the breach processes:

1. Uniform coarse grading with D_{50} = 0.70-0.90 mm
2. Uniform fine grading with D_{50} = 0.25 mm
3. Wide grading (4 types of sand were used) with D_{50} = 0.25 mm

Figure 3 shows the outflow and inflow hydrograph and breach top width growth with time for grading 2 and 3. It can be seen that little effect is shown in this figure in terms of peak outflow value, time to peak and breach growth rates and final breach width. These two runs show that the effect of grading is insignificant at this laboratory scale.

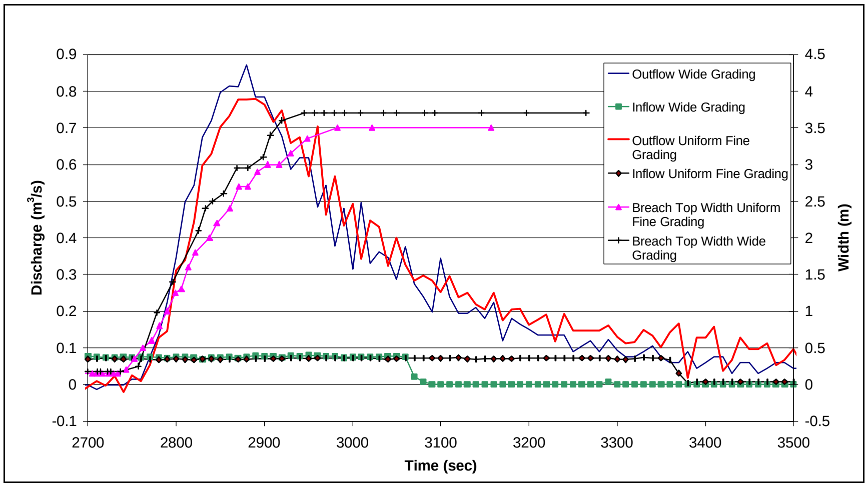


Figure 3[3]: Grading variation results

Effect of Breach Location

To examine the effect of the breach location, the initial breach notch was placed once on the centre and once on the side of two different embankments with similar properties and same geometry. Figure 4 shows the outflow and inflow hydrograph and breach top width growth with time for these two tests.

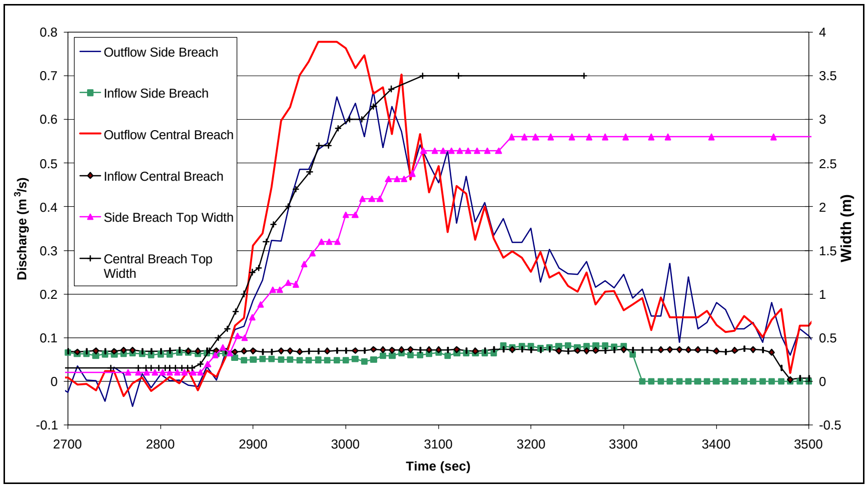


Figure 4: Breach location variation results

It is noticeable that a side breach has a lower peak outflow, erosion rate and final breach width. These two runs show that the effect of breach location is significant at this laboratory scale.

[3] The authors acknowledge that it is difficult to interpret the figures unless they are in colour. If this not the case, the reader can obtain a digital coloured copy from www.impact-project.net or the conference CD.

Effect of geometry changes

The following two geometry variations were tested in this series:

1. Upstream and downstream slopes were increased to 1:2 instead of 1:1.7
2. Crest with was increased to 0.3 m instead of 0.20 m

Figure 5 shows the outflow and inflow hydrograph and breach top width growth with time for the slope variation tests. It can be seen that increasing the slope has delayed slightly the erosion and the time to peak outflow, but, peak outflow value and final breach width were very similar.

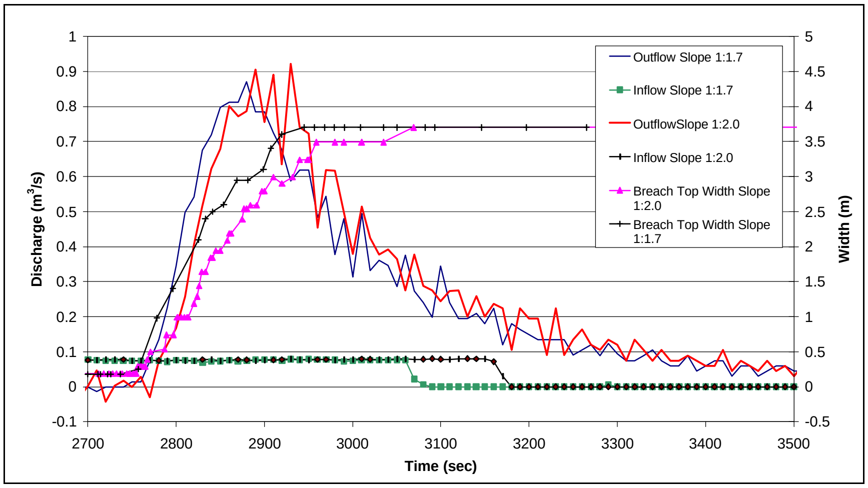


Figure 5: Slope variation results

Figure 6 shows the outflow and inflow hydrograph and breach top width growth with time for the crest width variation tests.

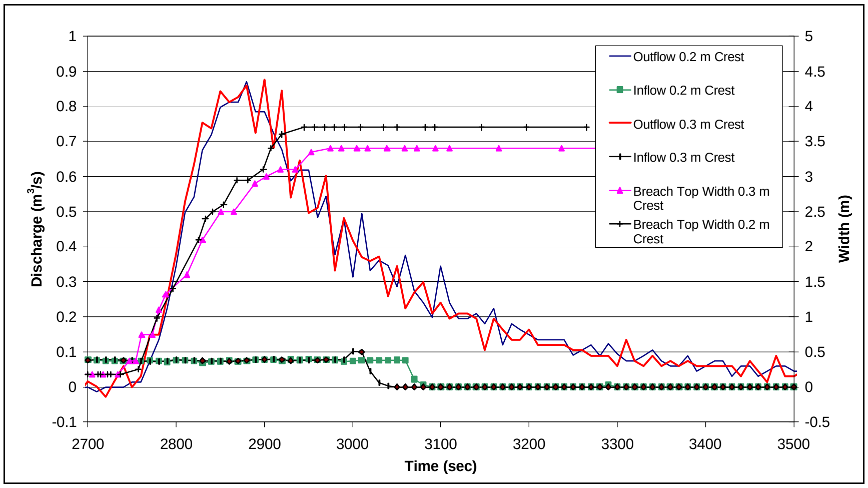


Figure 6: Crest width variation results

It can be seen that increasing the crest width had nearly no effect on the peak outflow, time to peak, and erosion rates for these two runs. In general, the effect of geometry changes was insignificant at this laboratory scale for this series of tests.

Series #2 - breach processes

The following processes were observed during the breach formation for these tests:

1. Head cutting was observed on the downstream face contrary to the smoothing process observed in series #1. More than one head cut was formed (See Figure 7A)
2. The head cuts combine into one deep head cut and this migrates upstream and then erodes downward
3. Once the breach is fully developed (i.e. material is nearly eroded to the base), the material below the water level is eroded. This undermines the slopes and lead to block failure
4. The above processes continues until there is not enough water to erode more material
5. Upstream slope erosion was also observed producing a similar bell mouth shape to Series #1. Again, slumping was also observed on the upstream face due to this erosion.

Processes 3,4, and 5 were very similar to series #1 except that the frequency of material slumping was lower in this series. Breach widening erosion rates and final width were also smaller than those observed in series #1. Figure 7 shows the above processes.

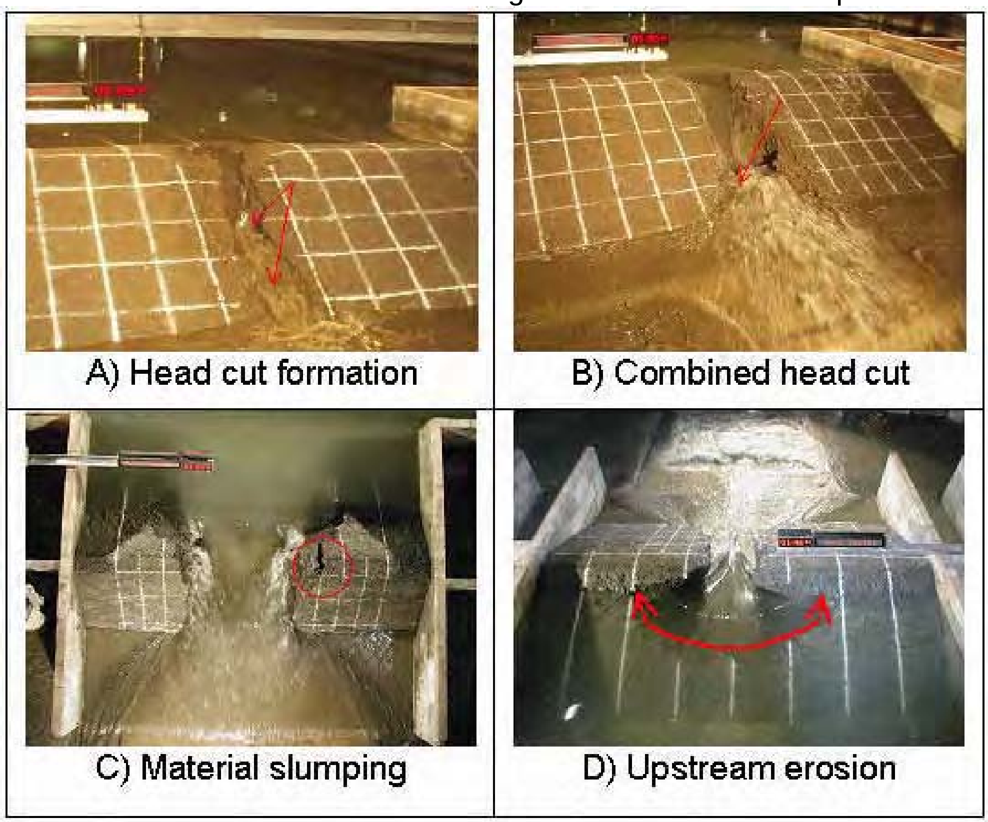


Figure 7: Series #2 - breach processes

Series #2 - effect of various parameters on the breach processes

The effect of various parameters such as grading, compaction, water content, and geometry was examined in this series of tests. In the following sections, the effect of these parameters is presented.

Effect of material type and grading

The following two material grades were used to examine the effect of material type and grading on the breach processes:

1. Fine-grained clay material with D_{50} = 0.005 mm with 24-43 % of clay (This was used for all the tests except one where the moraine material was used)
2. Moraine material with D_{50} = 0.715 mm with less than 10 % fines.

Figure 8 shows the outflow and inflow hydrograph and breach top width growth with time for the material variation tests. It is quite clear that the moraine material was more erodible than the clay material. This has accelerated the erosion process and led to a higher peak outflow and larger final breach width.

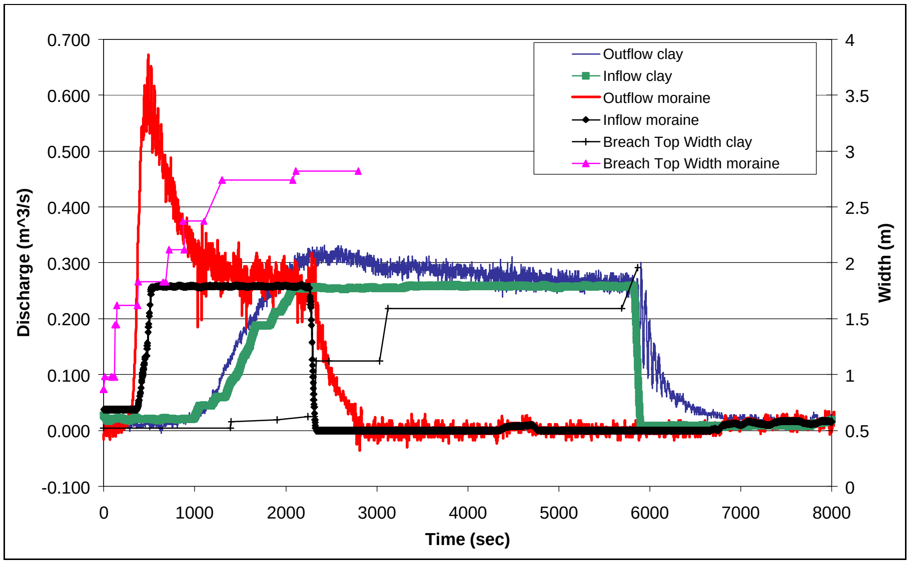


Figure 8: Material variation results

Effect of compaction

Two compaction efforts were used, to examine the effect of compaction on the clay material, with one compaction effort half of the other. Figure 9 shows the outflow and inflow hydrograph and breach top width growth with time for the compaction variation tests. Halving the compaction had an impact on the breaching processes for these two test cases but that impact is clouded by the effects of the compaction water content which is discussed in the next section. The decrease in compaction effort has accelerated the erosion process and led to a higher peak outflow and final breach width at this laboratory scale. The compaction water content for the higher compaction effort case was 25% and the half compaction effort case was 22%. This decrease in water content, as discussed in the next section, also accelerates erosion rates.

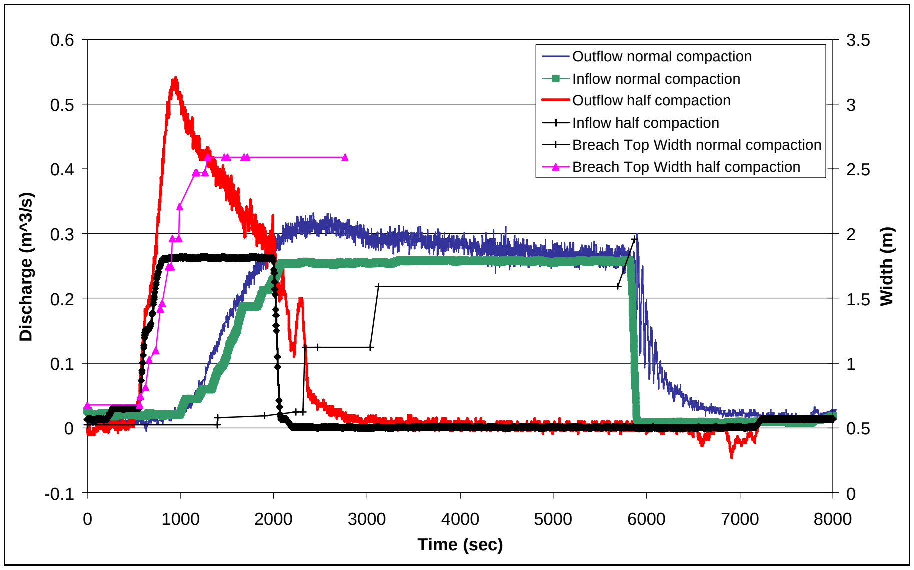


Figure 9: Compaction variation results

Effect of water content

To check the effect of the compaction water content, two different values of water content were used. The first was very near to the optimum water content (30 %) for the material used in this series of tests. The other was the natural water content of this material (24 %) which is lower than the optimum water content value.

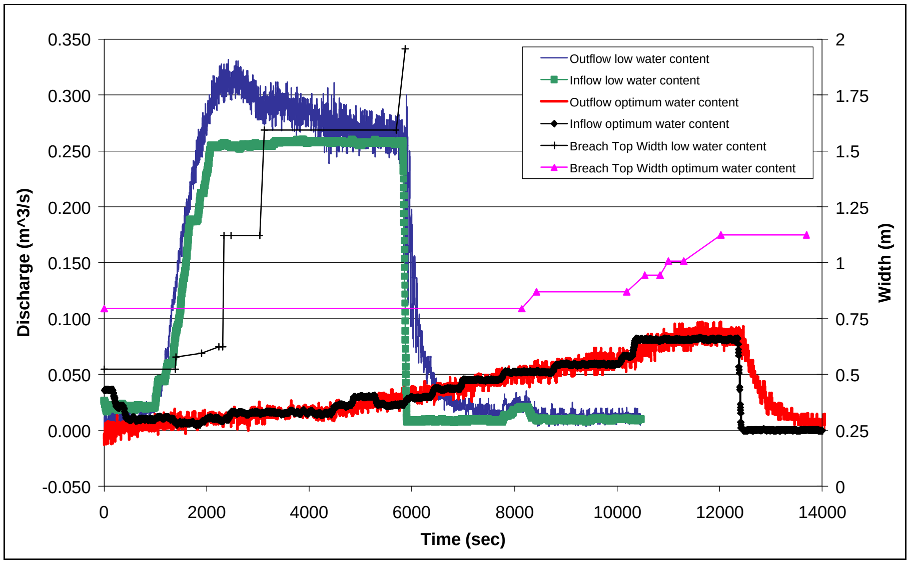


Figure 10: Water content variation results

The compaction effort for these two tests was basically the same therefore, the effect on erosion and outflow can be attributed to changes in the compaction water content. Figure 10 shows the outflow and inflow hydrograph and breach top width growth with time for the water content variation tests. Changing the water content to the optimum has significantly change the erosion properties of the material used. The embankment with optimum water content resisted failure and at the end of the test only partially failed with a smaller breach width and a lower peak outflow value compared to the other embankment. Laboratory tests, undertaken using the Jet-Test apparatus (ASTM, 1996), showed a difference in the erodibility of about 93 % between the two embankments.

Effect of geometry changes

The following two geometry variations were tested in this series:

1. Downstream slope was changed to 1V:1H instead of 1V:2H
2. Downstream slope was changed to 1V:3H instead of 1V:2H

Figure 11 shows the outflow and inflow hydrograph and breach top width growth with time for the slope variation No. 1. Both tests, the test with 1V:1H and the test with 1V:3H slopes, sped up failure and led to a higher peak outflow. This was not expected for the 1V:3H slope embankment as it had more material that the other two slopes (i.e. longer failure time). This could be due to the fact that both tests were at a lower bulk density than the 1V:2H slope and also at lower water content. These issues clouded the outcome of both tests and made the results of these two tests inconclusive.

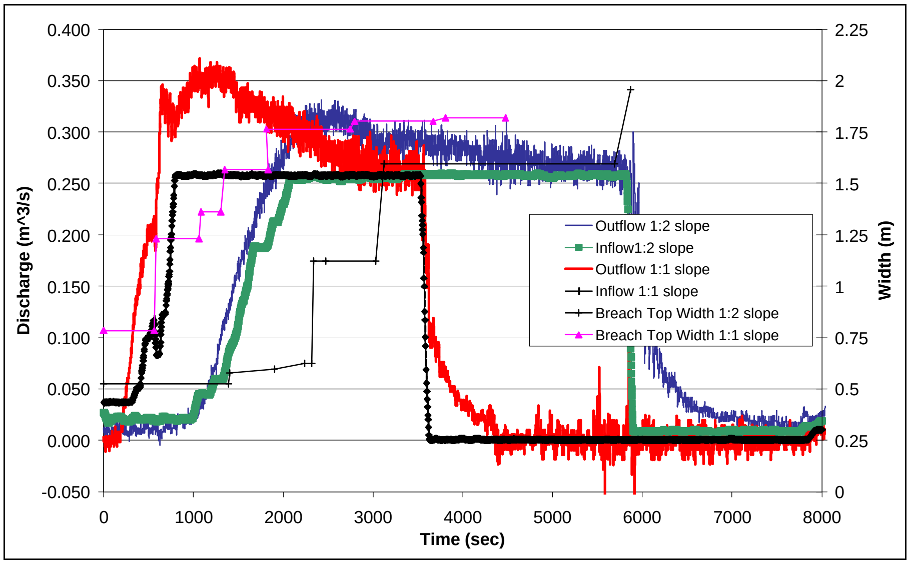


Figure 11: Geometry variation results

Numerical Modelling - Breach Test Runs

Extensive numerical modelling has been undertaken by selected members of the IMPACT project team and the value of model comparison was enhanced by additional participation from modellers world-wide (See Table 2 for details). A significant number of numerical model runs has been undertaken as blind tests to ensure complete objectivity. Blind means that numerical modellers were asked to undertake their work and submit their results before the results from the field and laboratory tests are released. Modellers were then invited to submit further (revised) modelling results after receiving the field or lab test results (Aware testing). Results presented in this paper are blind except for laboratory series #1 where only aware testing was undertaken due to data processing errors.

Table 2: Researchers who participated in the numerical modelling programme

No	Organisation	Country	Modeller	Model(s)
1.	HR Wallingford	UK	Mohamed Hassan	HR Breach NWS BREACH
2.	Cemagref	France	Andre Paquier	Simple model
3.	UniBW	Germany	Karl Broich	Deich_P
4.	ARS-USDA	USA	Greg Hanson	SIMBA model
5.	Delft Hydraulics	Holland	Henk Verheij	SOBEK Rural Overland Flow
6.	Ecole Polytechnique de Montreal	Canada	Rene Kahawita	Firebird model

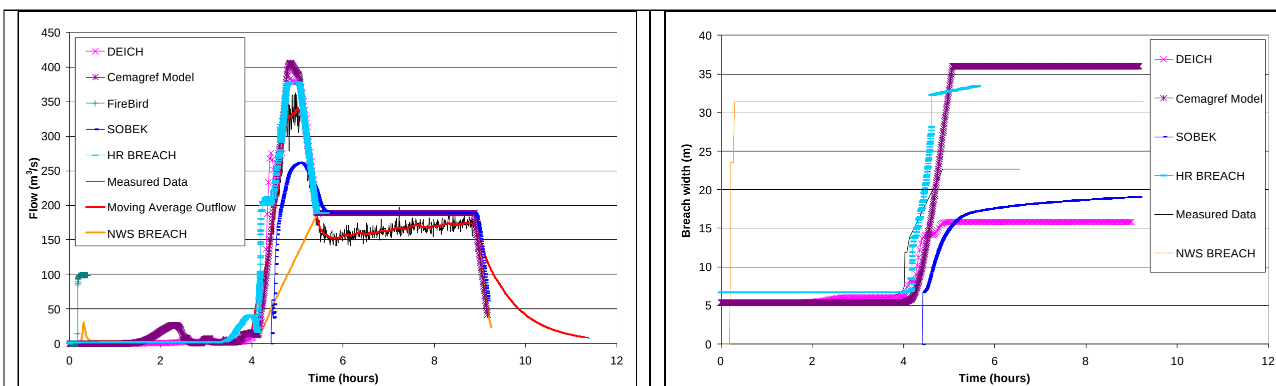
Numerical modelling of field test cases

In the following sections, results of the numerical modelling of the field and laboratory test cases are presented and followed by the conclusions drawn from this numerical work.

Overtopping of the homogeneous cohesive embankment (Field test #1)

This embankment was built mainly from clay and silt (D_{50} = 0.01 mm) with less than 15% sand and 25% of clay. The purpose of this test was to better understand breach formation in homogeneous cohesive embankments failed by overtopping.

Figure 12 shows the numerical modelling results of this field test vs measured data. It can be seen that most of the models have predicted well the peak outflow value, time to peak, and hydrograph shape. This is somewhat surprising as these models are mainly developed for modelling failure in non-cohesive embankments rather than cohesive ones. Breach growth was also simulated reasonably well. However, as seen in Figure 12B, final breach width was either over or under predicted.

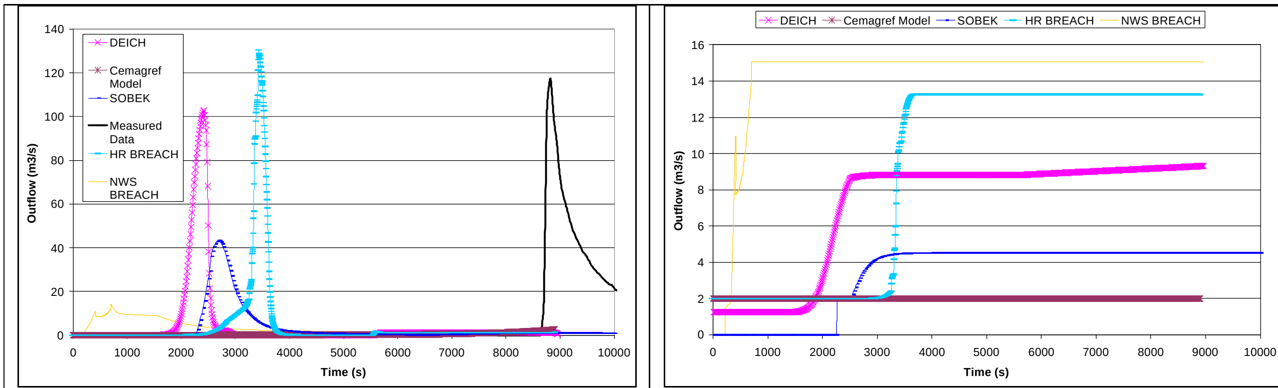


A) Outflow hydrograph B) Breach width

Figure 12: Modelling results for Field test #1

Overtopping of the homogeneous non-cohesive embankment (Field test #2)

This embankment was built mainly from non-cohesive materials ($D_{50} \approx 5$ mm) with less than 5% fines. The purpose of this test was to better understand breach formation in homogeneous non-cohesive embankments failed by overtopping.



A) Outflow hydrograph B) Breach width

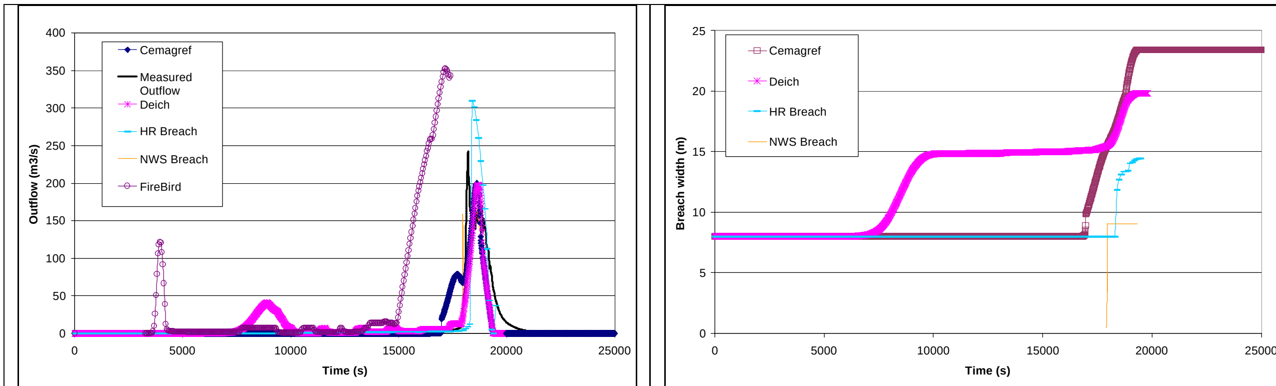
Figure 13: Modelling results for Field test #2

Figure 13 shows the numerical modelling results of this field test vs measured data (If available). It is quite clear that all of the models have underestimated the time to peak (See Figure 13A). Investigation of this issue revealed that the test conditions given to modellers were different, in terms of the initiation of the breach, from those actually occurred in the field. This resulted in underestimation of the time to peak. Despite that, results of the peak outflow still represent what would be the output of the model if field test conditions were used. It can be noted from Figure 13A that the scatter of the results is wider than that of Field test #1. This is again surprising as these models are mainly developed for modelling failure in non-cohesive embankments. Models have either over or under predicted the final breach width of the breach compared with the average final measured breach width that was about 12 m.

Overtopping of homogeneous composite embankment (Field test #3)

The upstream and downstream shoulders of this embankment were built from rock fill with a central moraine core. The purpose of this test was to better understand breach formation in composite embankments failed by overtopping

Figure 14 shows the numerical modelling results of this field test vs measured data (If available). It can be seen that most of the models have predicted well the peak outflow value, time to peak, and to a certain extent the hydrograph shape. This was unexpected, as most the models used to model this case do not model the complex processes due to the existence of the central core. This poses an important question of how simple could we make models without being far from predicting what really happens. Models have either over or under predicted the final breach width of the breach compared with the final measured width that was about 18 m.



A) Outflow hydrograph

B) Breach width

Figure 14: Modelling results for Field test #3

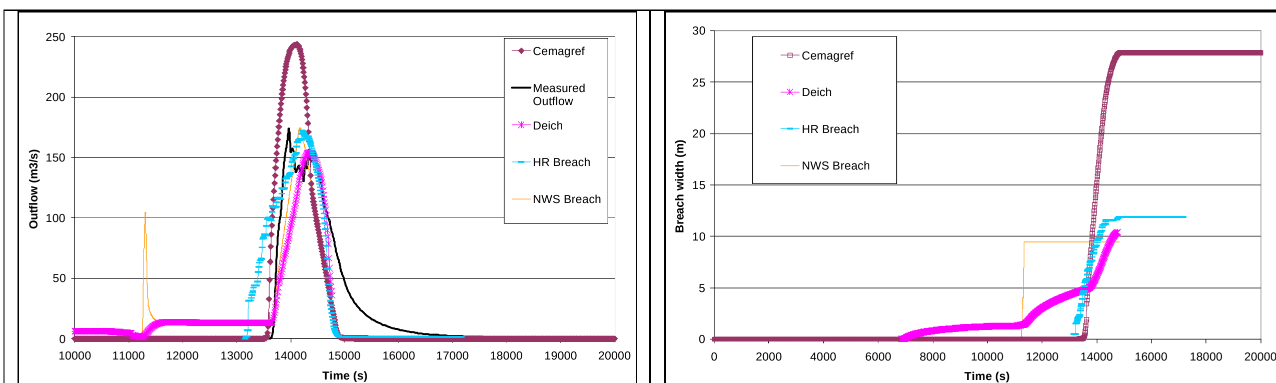
Piping of composite embankment (Field test #4)

This embankment was built as field test #3 with two mechanisms to trigger piping failure. The purpose of this test was to better understand breach formation in composite embankments failed by piping.

Figure 15 shows the numerical modelling results of this field test vs measured data (If available). Results are very similar to those of Field test # 3 and pose the same question on how far we could simply model complex processes without affecting the accuracy of models significantly. Models have consistently either over or under predicted the final breach width of the breach compared with the final measured width that was about 16 m.

Piping of homogeneous moraine embankment (Field test #5)

This embankment was built from moraine material (D_{50}= 7 mm). The purpose of this test was to better understand breach formation homogeneous embankments failed by piping.

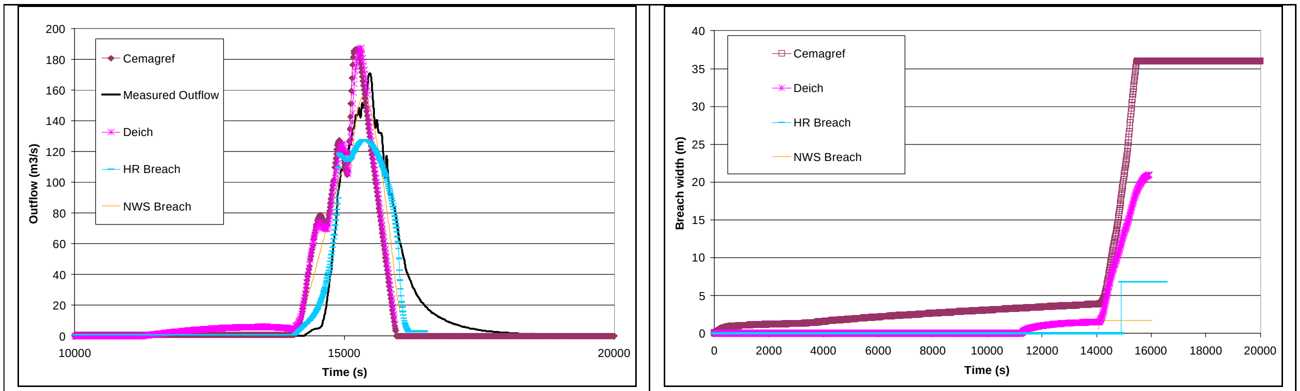


A) Outflow hydrograph

B) Breach width

Figure 15: Modelling results for Field test #4

Figure 16 shows the numerical modelling results of this field test vs measured data (If available). It can be seen that most of the models have predicted well the peak outflow value, time to peak, and the hydrograph shape. Despite that breach growth and breach final width scatter is wide (See Figure 16B) and final breach width was either over or under predicted compared with the final measured width that was about 15 m.



A) Outflow hydrograph

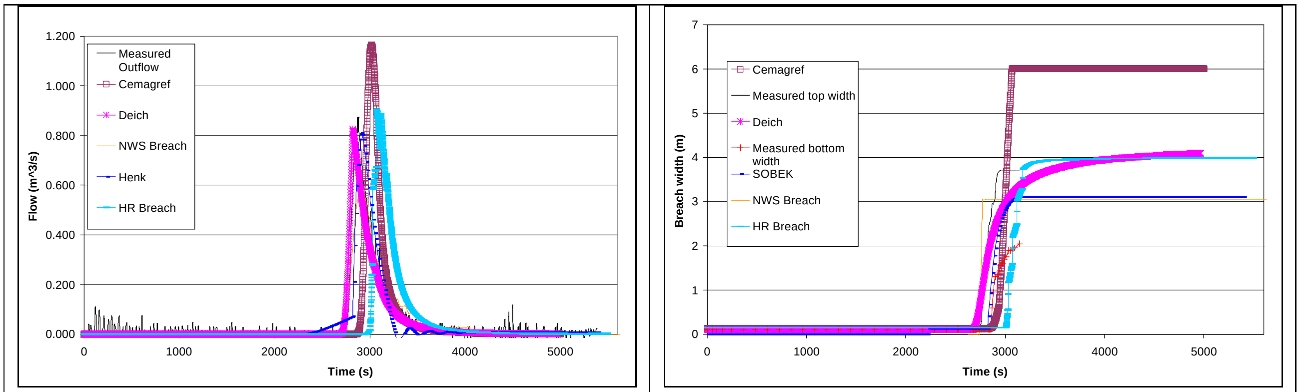
B) Breach width

Figure 16: Modelling results for Field test #5

Numerical modelling of laboratory test cases

Replication of Field test #2 – Laboratory Series #1

This test case was a direct replication of Field test # 2. Geometry and material size was scaled down to 1:10 scale. Figure 17 shows the numerical modelling results of this test vs measured data. It can be seen that most of the models have predicted well the peak outflow value, time to peak, and the hydrograph shape. Breach growth rate and final breach width were also reasonably predicted by most models in this case, contrary to the rest of the test cases.



A) Outflow hydrograph

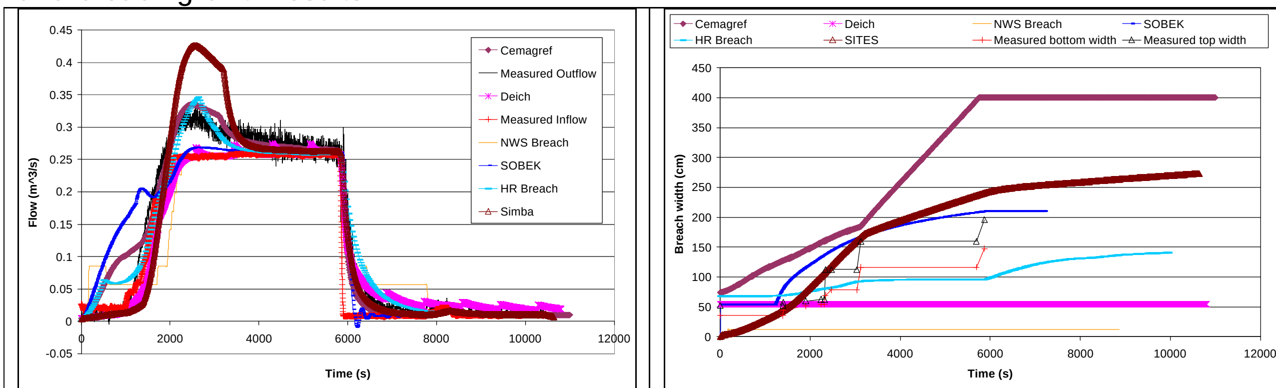
B) Breach width

Figure 17: Modelling results of replication of Field test #2 - Laboratory Series #1

Replication of Field Test #1- Series #2

This test case was a direct replication of Field test # 1. Geometry was scaled down to 1:10 scale. Material was scaled down based on the erodibility rather than geometrically (i.e.

Material used in the laboratory was ten times more erodible to compensate for the reduction in shear stresses). Figure 18 shows the numerical modelling results of this test vs measured data. Most of the models have predicted well the time to peak, and to a certain extent the hydrograph shape. This is may be because the test was driven mainly by the inflow as can be seen in Figure 18A. A wide scatter of results was observed in the peak outflow, Breach growth and breach growth results.



A) Outflow hydrograph

B) Breach width

Figure 18: Modelling results of replication of Field test #1 - Laboratory Series #2

Conclusions - Numerical Modelling

1. Based on the methodology proposed by Mohamed[4] (2002), the following indicative ranking was obtained for the models participated in this programme (See Table 3). This score was obtained by combining the accuracy of the predictions of the peak outflow, water level at peak outflow, time to peak, and final breach width. Based on the overall performance score, It can be seen that the HR BREACH and DEICH models have scored the highest scores in the laboratory and field runs. This suggests that approaches used in these models are better than those used in other models.

Table 3: Overall models performance scores

Model	Field Tests Average Score	Field Tests Modelled	Lab. Tests Average Score	Lab. Tests Modelled	Overall Score
HR BREACH	7.9	5	8.3	14	8.1
DEICH	8.7	5	6.3	14	7.5
Cemagref	7.2	5	7.0	14	7.1
SIMBA	---	None	6.4	8	6.4
SOBEK	4.6	2	8.2	6	6.4
NWS BREACH	6.1	5	5.6	14	5.8
Firebird	4.1	2	---	None	4.1

2. Although most of the models used in the above analysis were developed mainly to predict the failure of non-cohesive embankments, they predicted the failure of cohesive test cases reasonably well and sometimes even better than the non-cohesive test cases. This could be due to the inflow influence on the cohesive tests rather than the erosion processes.

[4] Details of the methodology are given in appendix 1.

3. All models, for field and laboratory tests, have either overestimated or underestimated the breach width. This might be because most of these models were not calibrated or verified with breach growth data or this is the effect of using sediment transport equations not suitable or valid for breach test conditions.

4. The test conditions of Field Test # 2 delayed the failure of the dam by about 4 times the time predicted by most of the models. This highlights the importance of including site specific properties when modelling the failure of embankments.

References

1. ASTM (1996). Designation: D5852-95 Standard Test Method for Erodibility Determination of Soil in the Field or in the Laboratory by the Jet Index Method. Annual Book of ASTM (American Society for Testing and Materials) Standards, Section 4 Construction, Volume 04.09 Soil and Rock.

2. Mohamed, M. (2002). Embankment Breach Formation and Modelling Methods. PhD. Thesis, The Open University.

Appendix 1: Numerical modelling scoring system

Model performance is judged by a category and score. Based on the difference in percentage between the measured and predicted values, the following categories are suggested:

1. Very Good Performance. 2. Good Performance 3. Reasonable Performance
4. Satisfactory Performance 5. Unsatisfactory Performance 6. Inadequate Performance

These categories overlap as shown in Figure 20. The score is a number represents the model performance and it is computed based on these categories. If the difference between the measured and the predicted data is more than ±50 %, the model performance is considered unacceptable and the score is assumed to be zero. Parameters can also be weighted according to their importance. The overall performance of the models can be calculated according to the following formula (Other terms can also be added to the above formula to take into consideration other parameters such as breach dimensions, growth rate, etc..):

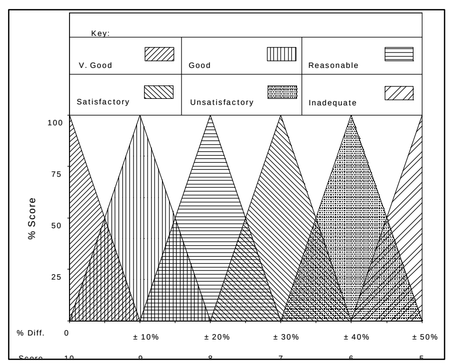


Figure 19: Model performance criteria

$$T_s = \frac{S_Q * C_{sQ} + S_T * C_{sT} + ...}{C_{sQ} + C_{sT} + ...} \qquad \textbf{(1)}$$

where: T_s : Overall score of the model performance. S_Q : Outflow of the model score.
S_T : Time to peak score C_{sQ} : Outflow weighting factor.
C_{sT} : Time to peak weighting factor.

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CASE STUDIES AND GEOPHYSICAL METHODS

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Introduction

In recent years, due to more and more frequently occurring weather effects of extreme nature which cause disastrous floods, increased attention has been paid to inspection and maintenance of dikes. This contribution is aimed at drawing attention to the possibilities brought about by the application of geophysical methods in these activities. Analyses of databases and compiliation of existing data on the history of dike breaches and failures during particular floods (case studies) may also be of significant help in the field of flood prevention.

Results of experimental geophysical measurements at selected locations in the Czech Republic are presented in this paper. These studies were performed within the framework of the EU **IMPACT** project (see website www.impact-project.net). This project also facilitated development of databases containing information on historical dike breaches and failures in the selected catchment areas (river basins) in Hungary and the Czech Republic.

Case Studies

Experience gained by us in the Czech Republic show that inadequate attention has so far been paid to the documentation of dike breaches and failures after extensive floods. Basic data on the reasons for, and the extent and course of dike breaches are missing in the majority of the cases. Exact data are seldom known, even from the recent disastrous floods in central Europe that occurred in 1997 and 2002. The data are often incomplete and of insufficient authenticity.

However, it is evident that analyses of such information, followed by appropriate adjustments and repairs of the dikes, may significantly reduce the risk of occurrence of new dike breaches and failures. We particularly talk about those dike segments where the reasons for destruction were, for example, inappropriate dike structure, inappropriate material or reduced stream channel capacity due to clogging. Furthermore, after analyzing a database, it often turned out that dike breaches in these sections had occurred repeatedly.

Statistical analysis of dike breach parameters may also allow some important generalizations related to the causes and characteristics of breach in specific river basins (catchment areas). For example, it turns out that the prevailing reason for dike breach occurrences in Slovakia is liquefaction caused by seepages in the underlying beds. The main reason for dike failures in Hungary is overtopping. Entirely different mechanisms of dike breach occurrences of course require different types of preventive dike modifications.

Within the framework of IMPACT project, a database of dike breach parameters has been designed and prepared for Hungarian river-basin agencies by colleagues from H-EURAqua Ltd. (contact: Eur Ing. Sándor Tóth M.Sc.E., Eur Ing. László Nagy M.Sc.E.). A database heading, presenting the monitored dike breach parameters, is shown on Fig. 1.

Fig. 1 Breach parameters database heading

Date	Country	Location of the breach			Breach data				Origin of flood			
		river										
dd/mm/yyy-hrs		name	bank	stationing of dike	pieces	final length	final depth below crest	scour pit depth	snowmelt	rainfall	ice jam	other
				km+000m		m	m	m				

Failure mechanism																	Cause of breach					
overtopping	stability loss of embankment due to					wave erosion	scouring from water side	structure failure	deliberate cut	stability loss of foundation due to				other	not well identified		bad material	bad design	bad construction	bad maintenance	lack of appropriate emergency operation	no information
	seepage		leakage		saturation					sandboil (piping)		liquefaction	hydraulic failure									
	dike body	at the base	dike body	at the base						slow	rapid											

Flood parameters								Data on damages				
H_b above base	The breach was		H_b below peak	River flow rate	Return period of flood	River flow velocity	Flow through breach (estimated max.)	inundated area	victims	houses destroyed	other losses	estimated total loss
	before peak	after peak										
m			m	m^3/s	yr	m/s	m^3/s	ha	capita	pcs		M€

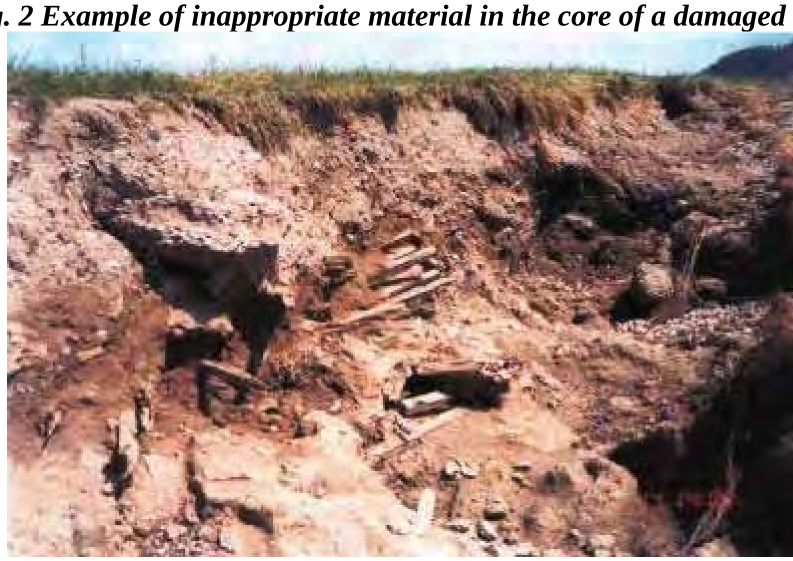
<table>
<tr><th colspan="5">Embankment parameters</th><th colspan="3">Soiltypes</th><th rowspan="4">River morphology description</th><th rowspan="4">Remarks</th><th rowspan="4">Literature</th></tr>
<tr><td rowspan="2">height of dike</td><td rowspan="2">crest width</td><td rowspan="2">water side slope</td><td rowspan="2">air side slope</td><td rowspan="2">base width of dike</td><td colspan="3">according to BS categorisation</td></tr>
<tr><td rowspan="2">dike body</td><td colspan="2">foundation soil</td></tr>
<tr><td>h</td><td>b</td><td>$\tilde{n}_{w}$</td><td>$\tilde{n}_{pr}$</td><td>B</td><td>depth</td><td>soil</td></tr>
<tr><td>m</td><td>m</td><td>v/h</td><td>v/h</td><td>m</td><td></td><td>m</td><td>nomination</td><td></td><td></td><td></td></tr>
</table>

Geophysical Methods in Dike Maintenance and Monitoring

At present, dike maintenance and preventive repairs are based on a system of visual inspection complemented by analyses of airborne or satellite photographs. Only rarely is the project documentation of complemented by detailed information on dike structures and material properties, i.e. information acquired by engineering-geological investigation, drilling, laboratory tests of soils, etc.

The reason for this is the considerable cost of such investigation and the large extent of the dikes. However, we believe that information on the nature of materials and basic dike structure is essential for efficient failure prevention. This particularly applies to old dikes for which construction documentation is missing. Furthermore, in some countries (for example, developing countries or countries of former East Europe) we may expect low quality of construction work that may contribute to dike breach when stressed (see Fig. 2).

Fig. 2 Example of inappropriate material in the core of a damaged dike



It is in this area that a package of geophysical methods can be of particular value. Geophysical methods provide a continuous image of physical properties of a dike body and, furthermore, this type of investigation is relatively inexpensive. Within the framework of IMPACT project, we concentrated on testing the possibilities of application of the following geophysical methods:

- ***geoelectric methods***
  resistivity profiling (RP), self potential method (SP), multielectrode method (MEM), electromagnetic frequency method (EFM)

- ***seismic methods***
  shallow seismic method (SSM), seismic tomography (ST), multi-channel analysis of seismic waves (MASW)

- ***microgravimetric method***

- ***GPR method***

- ***geomagnetic survey, gamma-ray spectrometric survey***

In order to incorporate the geophysical methods into a complex of dike prevention and maintenance, we first have to identify the effects that can be monitored by these methods. Figure 3 illustrates an approach to incorporation of geophysical methods into a dike maintenance program. From the viewpoint of **dike maintenance – dike breach**, timing of the action is of central importance.

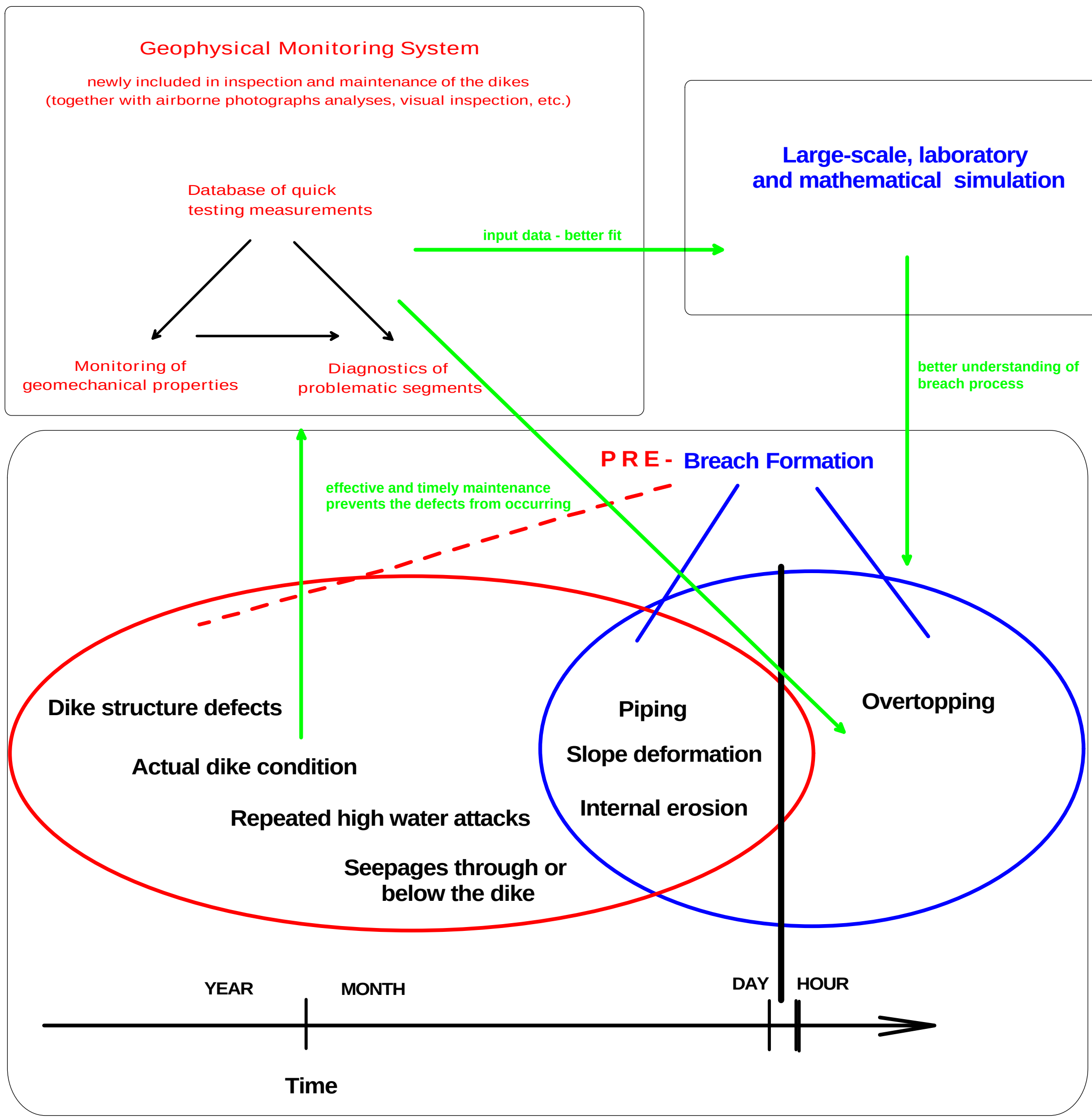
- **breach formation** itself takes place at a time scale of hours, max. a few days. A hazardous segment is evident, application of geophysical measurements is not assumed here.

- however, except for overtopping, the remaining defects mostly show somewhat hidden **PRE-breach formation** stage (for example, seepage through the underlying beds, repeated seepage at an increased water level, structure defects, etc.) which predisposes the point of future dike breach. This stage often lasts for even tens of years.

The above mentioned PRE-breach formation stage is our area of interest for the application of geophysical methods. Based on the needs of dike administrators we recommend the application of geophysical methods in three basic fields that are included in package of geophysical measurements we call a **geophysical monitoring system** (Fig. 3, top left). A detailed description of geophysical monitoring system methodology will be included in Final Deliverable of the IMPACT project.

The basic component of the monitoring system is a **database of quick testing measurements**. We particularly tested the application of electromagnetic frequency method (EFM) of measurement of conductivity. It is a very promising method for assessment of properties of materials used for dike construction. Conductivity (resistivity) is closely related to changes in clay content, porosity and permeability of soils. So far, it has been mostly applied for military purposes (detection of ammunition, subsurface distribution systems, cavities). Its application in the fields of geological or geotechnical surveys brings about certain difficulties, however, we manage to eliminate them step by step. Further then, the GPR method may be applied. All types of measurement should be linked to GPS.

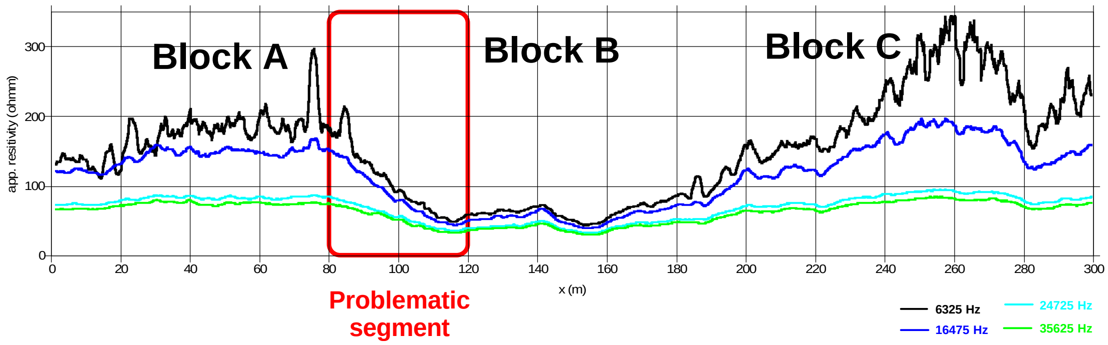
Fig. 3 A diagram of incorporation of geophysical methods into dike maintenance



The database of quick testing measurements provides a basic description of dike materials and structures, division of dikes into quasi-homogeneous blocks (i.e. dike segments showing similar geotechnical and physical properties). Productivity of measurement is rather high, based on the dike character ranging between 10 and 20 km of a dike per day. From the viewpoint of dike maintenance, these data are an appropriate complement to a visual inspection, allowing us to assess relative permeability of the dike material and its homogeneity and to detect subsurface distribution systems reaching a dike, etc. This allows us to more precisely identify problematic dike segments that are disturbed and weakened inside.

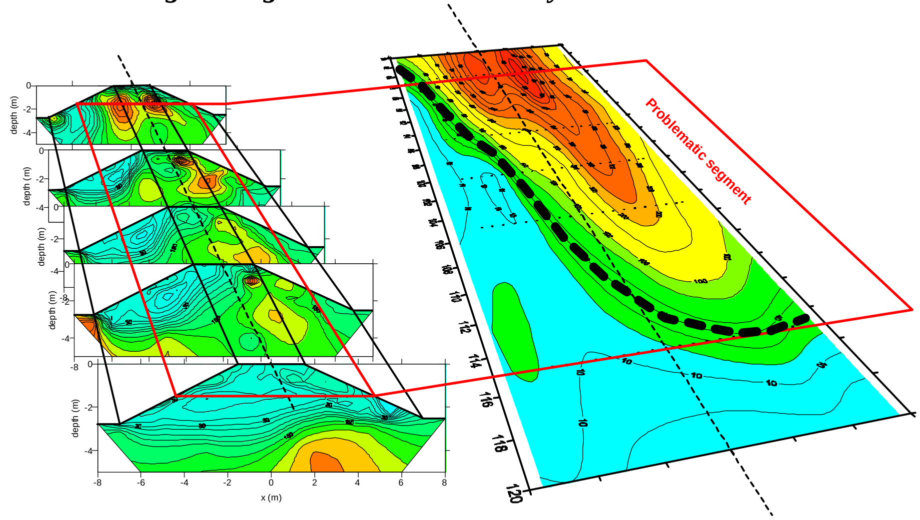
On Fig. 4 we see a demonstration of quick testing measurement by the EFM method at the point of old breach. The breach extent (block B) is entirely evident from the resistivity curves. The curves bring resistivity information from different dike depth levels depending on the frequency applied. We see that the material used for repair shows entirely different properties (lower apparent resistivity indicating higher clay content) in comparison with the original material. For this reason, the contact (keying) of a repaired segment with the original dike can be considered hazardous. Different materials showing different levels of compressibility and absorbability may fail to tie together well, thus allowing water penetration into the gaps. Therefore, the contact between the A and B blocks has been delimitated as a hazardous/problematic dike segment.

Fig. 4 Quick testing measurement by the EFM method



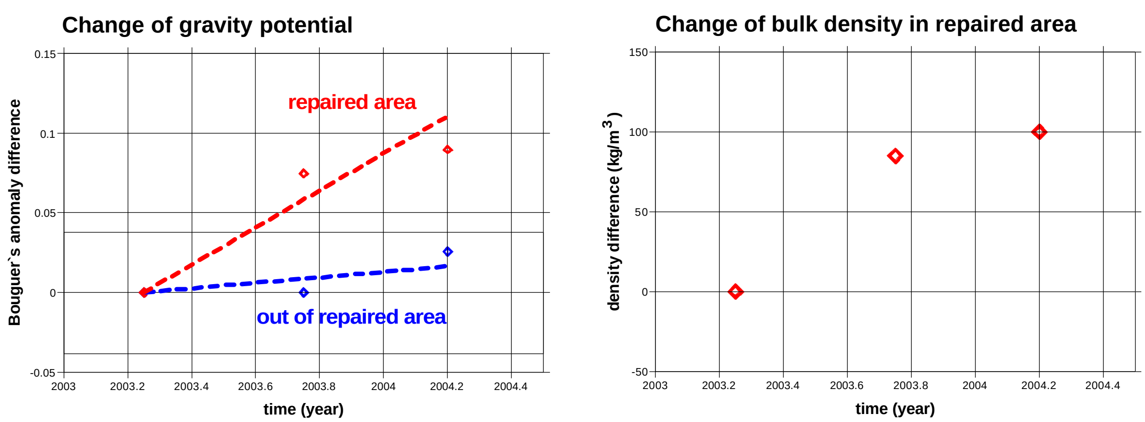
For a detailed description of problematic segments we recommend application of a package of diagnostic methods. These are particularly MEM, SP, microgravimetric method, seismic methods and thermometry. Using a package of these methods allows us to construct a physico-geotechnical model of a problematic segment. This allows us to precisely determine the extent and intensity of dike disturbance and to draw up an optimal way of potential repair. On Fig. 5, a resistivity model of dike breach repair keying at a boundary between A and B blocks is shown. High-level material inhomogeneity of both parts has been confirmed. In addition, microgravimetric and GPR measurements has confirmed the existence of open joints between both parts (significantly increased porosity, tiny cavities). At high water level this means a risk of water penetration into the joints. Dike seepages cannot be excluded.

Fig. 5 Diagnostic measurement by the MEM method



A special group of geophysical measurements is formed by the methods allowing us to monitor the changes of geotechnical properties of selected dike segments (**monitoring measurements**). This approach can be employed, for example, in repair quality control, new dike consolidation monitoring, long-term internal erosion monitoring, etc. Here, particularly seismic methods (ST, MASW) and microgravimetry can be applied. We believe that special applications of these methods might help to better describe breach formation processes for large-scale physical models. As an example of long-term monitoring of a repaired dike segment consolidation we present on Fig. 6 the results of repeated gravity measurements. In the block A area (original, consolidated dike segment), no measurable change in material bulk density occurred during the measurement, on the other hand, block B (repaired dike segment) shows gradual consolidation.

Fig. 6 Gravimetric monitoring of consolidation



Organization of Geophysical Monitoring System

Within the framework of the geophysical monitoring system we assume immediate repeated quick testing measurement in the event of **dike flooding.** Otherwise, we assume the dike measurements will be performed in **intervals of 3 years**. The purpose of quick testing measurements will be to evaluate shape similarities of the measured resistivity curves, and/or to analyze differences in GPR records. Repeated measurement is aimed at drawing attention to new anomalous segments developed as the result of flood action or groundwater action in the dike foundation. Evaluation of repeated quick testing measurements will, of course, be followed by targeted diagnostic or monitoring measurement.

The database of geophysical measurements will be administered by a specialized geophysical company cooperating with the river-basin agency. Their task will be, if necessary, to carry out a new round of monitoring measurements, to archive the data and make comparative evaluation of such data.

With regard to limited duration of the IMPACT project, we could not fully prove expedience of repeated measurements at the dikes. We had an opportunity to monitor an effect of „dike flooding“ at a reservoir which is step-by-step being filled (Velký Bìlèický pond, the Czech Republic). The dam was reconstructed with a segment close to a bottom outlet completely replaced. By means of repeated measurements we monitored the process of dam material saturation with water. The main objective of the monitoring was to delimitate potential points showing increased permeability (more rapidly progressing moisture content) which might be a source of hazardous seepages in the future.

The results of monitoring measurements are shown on Fig‘s. 7a and 7b. Fig. 7a shows resistivity curves acquired by the EFM method applied in the pond dam axis during the pond filling (October 2003 – 0 % of water level, December 2003 – 50 % of water level, April 2004 – 100 % of water level). Resistivity curves are standardized to the resistivity values corresponding to 25 % and 75 % of Min / Max ratio in the analyzed segment. In this way we limit the effect of weather conditions at the locality on the measured data. In other words, the purpose of interpretation of repeated measurements of conductivity is not to compare the absolute values, but the shapes of measured curves (relative anomalies). We see that the curves from October 2003 and December 2003 do not show significant deviations. However, a difference is evident for the curve from April 2004 (100 % of water level in the reservoir), where we can see a local decline of relative resistivities close to the interval between m 60 and m 90. This is a part of the dam that was fully reconstructed. We see a showing of new material saturation with water in the process of the pond water level increasing. We would probably record a similar effect if long-term seepages through a dike were monitored.

A more precise description of where seepage occurs in a dike is provided by diagnostic measurement by the MEM method (Fig. 7b). The left part of Figure 7b represents changes of resistivities in a longitudinal section. The right part shows changes of resistivities in a cross-section. We can see that the most distinctive change was identified at the dam base at the points of contact with the underlying bed. In the cross-section which has a limited depth penetration due to a limited space for placing the electrodes, a showing of an anomaly corresponding to a peak of the seepage curve can be observed.

Fig. 7a Relative resistivity anomalies detected by the EFM method at the Velký Bì lŁický pond dam during filling

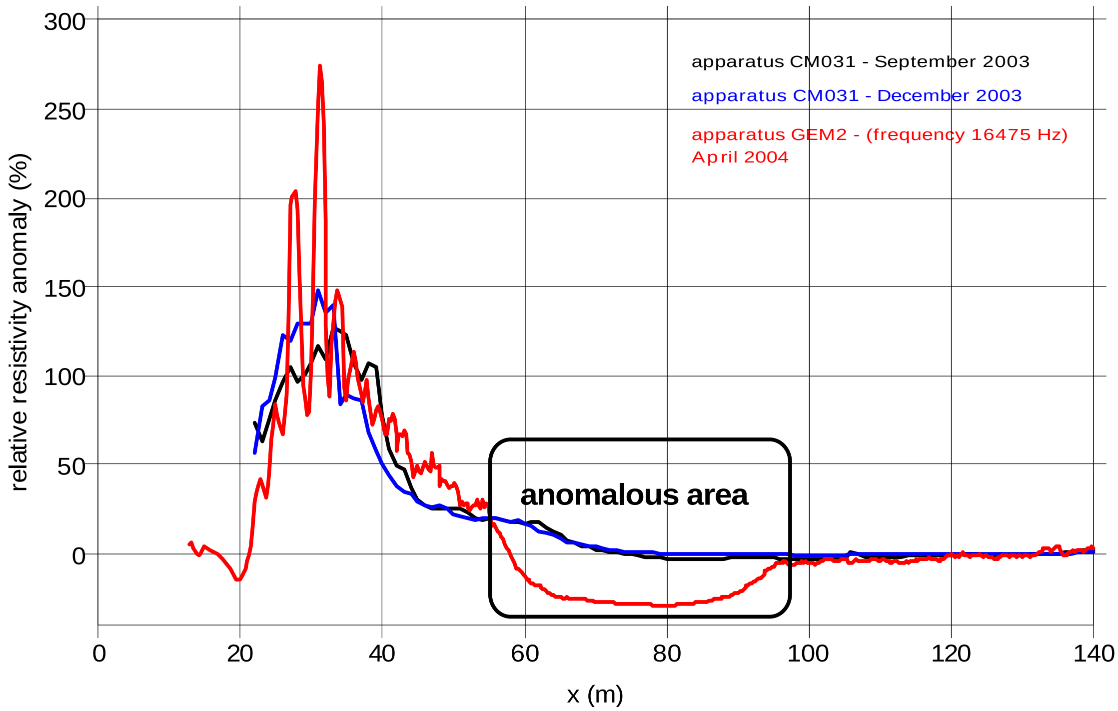
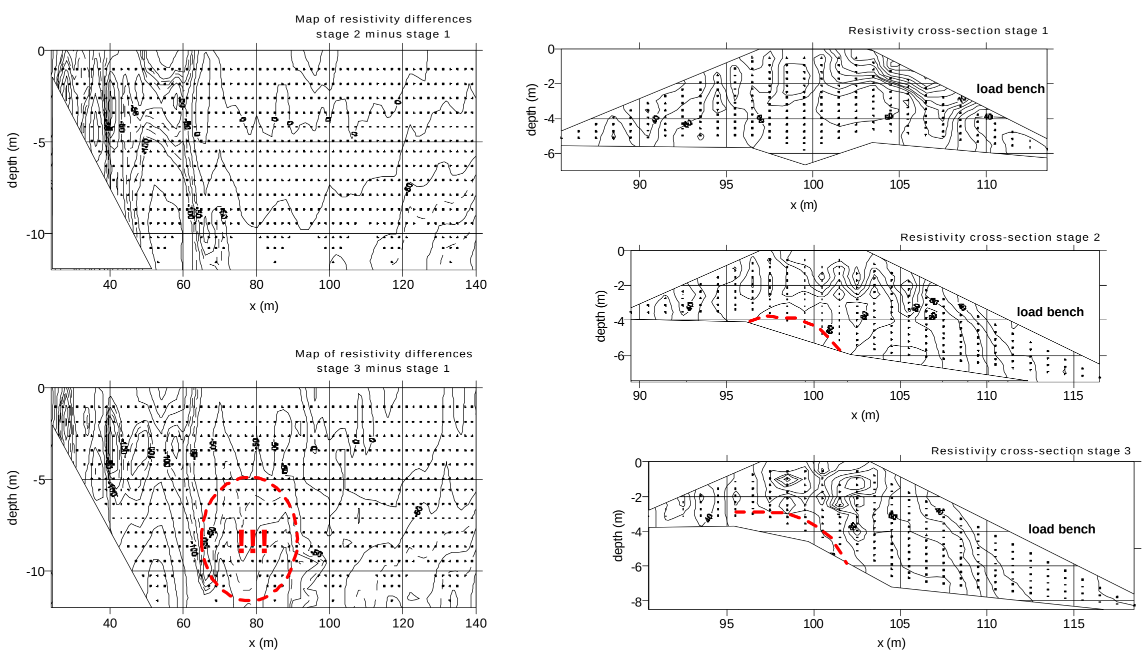


Fig. 7b Map of resistivity differences detected by the MEM method at the Velký Bì lŁký pond dam during filling



Summary

Geophysical testing measurements within the framework of the IMPACT project have proven that combination of appropriate methods brings valuable grounds for preventive repairs and maintenance of the dikes.

A suggested package of geophysical measurements for the purpose of dike prevention and maintenance we call a **Geophysical Monitoring System.** It is comprised of three basic fields:

A) **quick testing measurement** – its purpose is to provide a basic description of dike materials and structures and to delimitate quasihomogeneous blocks and potentially hazardous segments. Repeated quick testing measurement data will be stored in the database, allowing us to analyze long-term changes of the dike condition.

B) **diagnostic measurement** for a detailed description of problematic and disturbed dike segments – it serves for optimal repair planning

C) **monitoring measurement** of changes of geotechnical parameters – it serves for repair quality control and for observation of earth structures ageing processes, etc.

Valuable information for the dike prevention from damage is also provided by analyses of historical dike breaches in a given river-basin area. Keeping a database of dike breaches should be a routine part of dike control and maintenance.

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FLOOD PROPAGATION MODEL DEVELOPMENT

Francisco Alcrudo, Sandra Soares-Frazão, Yves Zech, Guido Testa, André Paquier, Jonatan Mulet, David Zuccala, Karl Broich

Abstract

The prediction of the effects of the uncontrolled release of water from a structure failure requires the ability to simulate rapidly varying flow conditions of much greater magnitude than are normally associated with natural flood propagation. Such conditions often pose difficulties for many modeling methods in commercial use and often adopt one-dimensional approximations that can limit the applicable range and description level of the flood. This often places a greater emphasis on modeler expertise in order to achieve realistic modeling results. This paper describes methods for practical computations in this area, their capabilities and limitations as well as guidelines for use. Presently, an increased interest in predicting flood characteristics in populated areas is perceived as a need within the industry. According to this, strategies adopted for the simulation of flood propagation in urban areas are described. The methods are verified against data from dedicated laboratory experiments and real flood cases, including past flooding of European urban areas.

1. Introduction

Flood propagation modeling can be defined as the art of quantitatively describing the characteristics and evolution of the flow that is set up when a large amount of water moves along the earth surface in an uncontrolled way. Usually computers play a capital role in this art. The sizes and scales of terrestrial floods can span several orders of magnitude as the affected surface areas do. Their nature and origin can also vary, ranging from slow, reservoir-filling like inundations due to long lasting rains, to extreme, short, violent floods that can follow the failure of a dam or other control structure.

The use of computational models in flood propagation dates back to the sixties; although it has been during the last decade that thanks to the availability of high performance computers, an explosion of publications reporting the development and use of flood propagation models has occurred (Alcrudo 2002). Stringent regulations with regards to hazard and risk mitigation and management affecting dam owners, basin authorities, land use planning bodies, etc… have been slowly but constantly enforced by states worldwide for many years now. This fact may have something to do with present interest in flood propagation models.

Progress in this area is directly related to the following issues:

- Understanding the flow processes relative to the problem.
- Formulation of appropriate mathematical laws describing it.
- Development and tuning numerical techniques to solve them.
- Validation of model output against representative experimental data and real life observations.

Very important to every day engineering practice are model interfacing with other engineering tools, such as topography and visualization software, data acquisition, and geographical information systems (GIS), that can considerably simplify and speed up problem set up and analysis time. Most of the models commercially in use are particularly strong on these issues; however they will not be discussed here.

Validation of the output obtained from computer simulations is one of the most important steps in model development in order to quantify the uncertainty associated with its predictions. Every effort should be made to confront model results with real life data. Since it is difficult to retrieve well-documented extreme flood events, resorting to laboratory scaled down experiments is a useful means to gain knowledge in model performance. The amount of output provided by present day models can be overwhelming and is usually analyzed by means of sophisticated graphical software that can easily lead the engineer to think that displayed information is inherently correct. A strong knowledge of a given model basic assumptions and limitations, sound judgment as well as a critical attitude is always advisable. In the following sections these issues are discussed in more detail within the particular framework of the IMPACT Research Project.

2. Mathematical models of flood propagation

2.1 Model equations

From the fluid dynamics point of view, flood propagation constitutes a formidable problem that has to be simplified by means of sensible judgment and sound approximations in order to make it mathematically tractable. Pioneering work can be traced back to the late fifties and the sixties (Isaacson et al. 1958, Cunge et al. 1964, Martin et al. 1969). Some of the models that are yet in use today appeared to the general public already in the seventies like the most well known Fread's DAMBRK (later followed by its sequel FLDWAV, Fread 1993).

The fundamental mathematical laws that govern the phenomenon, the Navier-Stokes equations (NS), are well known. However their solution is practically impossible for the spatial and temporal scales of any real situation (see Alcrudo 2002 for a survey of potential applications of NS and simpler equations to the problem of flood propagation). This leads to the need for simplified descriptions such as the Shallow Water equations (SWE) which is presently the most widely used, despite its many shortcomings. Up to some years ago the vast majority of models available were one-dimensional (1-D) with all the limitations inherent to such approximation. A higher dimensional approach was reserved to research and academic institutions. Presently, two-dimensional (2-D) models have come to a high degree of maturity, and there are now several on the market. A two-dimensional description can provide much more information; and hence, it is quickly becoming the standard save for particular cases where the topography is very markedly 1-D.

The SWE can be derived from the NS system by a depth averaging process, or alternatively from a mass and momentum balance in the plane of motion (tangent to the earth surface) after making several assumptions:

i) Vertical velocities are neglected/not considered (therefore vertical accelerations are identically zero).
ii) The pressure field is hydrostatic.
iii) The bottom slope is assumed small (such that the sinus of the slope angle can be approximated to the angle itself).
iv) A uniform horizontal velocity field is assumed across the water layer.

Since the mathematical expression of the SWE can be easily found elsewhere, it will not be included here for the sake of brevity.

A real flood event implies the movement of water in the vertical direction too and restrictions i) to iii) altogether make the SWE lack fundamental physical effects. It is likely that situations in which vertical movement is substantial be poorly represented in a SWE simulation. An ingenious idea within the SWE context that has been put to work recently (Zhou et al. 2002) considers the movement of the layer of fluid in a horizontal plane over a succession of a piecewise flat discontinuous bed.

It is clear that a boundary layer must extend from the bed to the free surface, and the fact that the horizontal velocity field is considered uniform across the depth of the water layer (point iv) may also be responsible for important deviations between model predictions and real observations. The effect can be shown to be analogous to shear stresses in the plane of motion, but the equivalent kinematic viscosity may be more than an order of magnitude larger than the kinematic viscosity of the fluid, thus competing with turbulence terms. Krüger et al. (1998) have derived an extended form of the SWE including a linear horizontal velocity profile plus a quadratic vertical velocity distribution and pressure correction to the hydrostatic law. Tests run by the authors on supercritical spillway flows with hydraulic jumps show consistent improvement of the modified SWE with respect to the standard model when compared with experimental data. No applications of this idea to flood propagation have been found.

The turbulent contribution to the momentum equations has not been traditionally considered an important matter. In some cases a constant eddy viscosity coefficient is used, which seems to play more the role of a tuning parameter rather than a characterization of the turbulence characteristics. As regards bed friction, empirical laws of the Manning or Chézy type, which scale with the square of the depth averaged velocities, are usually assumed.

2.2 Numerical solution of the model equations

It can be said that the numerical solution of the governing equations is the best addressed issue in latest years. This is due to a well-founded theoretical work on the solution of partial differential equations during the last two decades. The fact is that there are now plenty of numerical methods to accurately solve the equations of motion, what does not prevent ongoing research in this area.

The SWE constitute a system of partial differential equations of the hyperbolic type with two space coordinates (running roughly along the earth surface) and time as independent variables. The dependent variables are water depth and the velocity vector in the surface plane. In order to obtain a numerical solution the domain of integration must be first discretized. This is usually done separately for the space and time variables. Considering the spatial discretization, the great majority of methods fall in one of the following three categories: finite difference, finite volume and finite element. All three are being used in current models, but it seems that the finite volume approach is favored for combining the conceptual simplicity of the first category, and the flexibility of the last one.

Most numerical schemes in use today perform a separate spatial-temporal discretization, whereby the spatial derivative terms are firstly discretized, and then the resulting ordinary differential equation is integrated in time. It is now common use of formally second order accurate operators both in space and time.

Among the important issues in flood propagation stands the numerical location and propagation of wave fronts while conserving water mass. The so called shock capturing methods are capable of automatically locate and propagate fronts. This feature has made them very popular, and most models in use today include some sort of shock capturing operator. It can be said that this is now a solved problem (see Toro, 2001, for a good overview of different methods).

In practical applications, flows governed by the SWE are dominated by the source terms arising from bed slope and, in 1-D, lateral reactions. This has had a profound influence on the development and application of numerical methods for flood propagation: The flux discretization must be performed in a way compatible with the source term contribution. Otherwise the simulation of a mass of water initially at rest contained in a reservoir with abrupt bottom will result in the generation of unphysical movement instead of preserving the water body at rest. Appropriate source term discretizations to fix this problem were firstly developed by Bermudez and Vazquez (1994) for upwind schemes by upwinding also the source term. A collation of other strategies developed by several authors can be found in Soares-Frazão (2002).

Intrinsic to the propagation of a flood is the wetting and drying process of affected areas. Many numerical methods present an unstable behavior in the wetting-drying edge where a transition from zero to a finite depth must occur. Sometimes negative water depths are generated. The effect is severely aggravated for irregular topographies, which is usually the case in practical applications and some methods fail to propagate a flood over a dry bottom. For the sake of brevity, the reader is referred to the review by Alcrudo (2002) where original works on the subject are cited.

3. Modeling urban flooding

The modeling of flood propagation in urban areas has been perceived within the Impact project as a source of increasing interest. The term urban flooding is understood in the context of this paper as the extreme surface inundation of areas with high density of buildings. The reason for this interest is mainly two-fold: On the one hand potential damage in urban areas is several times that in open field due to the intensity of land use; it is therefore unquestionable that industry be eager for modeling capability in that area. On the other hand present day modeling technology is capable of providing some sort of inundation prediction in urban areas and hence the induced interest. Impact project work in this area covers:

- Development of suitable strategies for urban flood modeling.
- Laboratory work to learn about urban flood characteristics and gathering of experimental data.
- Field work to locate past urban flooding scenarios and collect flood data.
- Model validation against laboratory and real life data.

There are several modeling techniques that can be embedded within open field flood propagation models to deal with urban inundation. The simplest approach consists of representing urban environments as areas of reduced conveyance by simply ascribing a high bed friction coefficient. Manning’s roughnesses as high as 0.5 have been reportedly used to account for the presence of a city. During Impact project work evidence suggests that friction coefficient figures are subject to scale effects that are not yet clear: They depend on building density, scale of flooded area, and ratio of flow depth to building height.

A step further is brought by the concepts of *Urban Porosity* and *Transmissivity* that are used to represent the effect that the area subject to flooding is only a fraction of the total surface area, hence affecting the mass conservation equation. This approach has been successfully used by Braschi and Gallatti (1989), who proposed the method. A disadvantage is the lack of momentum interchange between the flow and the buildings. This can be arranged via the friction term in the momentum equation by artificially increasing the roughness in the area where buildings are present as in the previous approach (Testa et al. 1998).

The increased resistance and the urban porosity approaches both provide an averaged view of the city-flood interaction; and, hence no local effects can be told from the output of such type of simulation. The fluid dynamics information provided can have the appearance of being local because the flow variables (water depth and velocity vector) are given for every grid point of the domain. However, they represent averaged or smeared values of those quantities over a certain surface area somehow related to the grid spacing and the building density.

These techniques are devised for two-dimensional models and are best suited for modeling large areas where it is impractical or plainly impossible to seek high resolution of topographic and flow features due to problem size. Although they could eventually be applied within one-dimensional models this does not seem natural.

There is frequently a need to know not only whether a given area is going to be flooded or not but also what local inundation conditions are likely to occur. Within Impact project, work has focused around techniques that can provide insight into local flow conditions, at the cost of higher computational cost or problem size reduction. In particular four strategies have been considered:

- A one-dimensional (1-D) treatment of the city area whereby it can be represented as a channel network (see for instance Tanguy et al. 2001).
- Friction based local representation of buildings and obstacles to flow in a two-dimensional (2-D) approach.
- 2-D topography building representation
- Detailed 2-D meshing and solution of the streets and city areas, incorporating buildings as solid walls.

The first alternative is capable of providing local flow information at low computational cost, although problem set up may require considerable work and expertise for network layout and data management. Problems may arise at junctions, in particular if these are numerous, or in wide areas where some flow features can be lost because the flow is markedly 2-D. Another difficulty of 1-D channel models lies in its interaction with larger area models. If the urban flood is the result of flooding of surrounding terrain, appropriate coupling between the urban network and the outer flood plain model is needed.

Representation of buildings as *local* areas with increased friction coefficient within a 2-D simulation can provide the needed resolution to capture local flow effects with little extra cost. This approach is easy to set up because local friction can be treated as another field variable, and provides reasonably accurate results. The problem lies in that buildings turn practically into local water storage tanks which is not the case in reality. Some sort of urban porosity treatment would be needed to offset this effect. Coupling the urban area with surrounding terrain subject to flooding is straightforward.

The topography based approach involves placing buildings upon the bottom representation within a 2-D calculation scheme. This can be easily done after the meshing stage of problem set up by rising the grid points that fall within a building area to the roof top elevation. Often some sort of mesh adaptation will be needed to accurately represent the city. Although buildings protrude vertically from the surface up, the discrete slope representation is not infinite but is extremely steep and depends on cell size. When flooding water reaches such adverse bed slope, its momentum is abruptly reduced; and if flow head is lower than building elevation, then it stagnates. In case flow head is large enough, water can overtop the building and it is submerged. This simple idea must be implemented with care because many numerical integration methods will not accommodate extremely steep bottom slopes as those generated with such building representation. This has also implications with the wetting and drying treatment. Further, it must be pointed out that the assumptions on which the SWE are based are systematically violated at the building borders. Coupling with surrounding terrain is straightforward as in previously described strategy.

Finally, the most accurate city representation can be obtained from a careful 2-D meshing of the area subject to flooding, excluding buildings from the computational domain. This can be done by blocking grid cells occupied by buildings or by meshing them around so that buildings are treated as impervious zones. The flood propagation model is then run in the void area. This method can theoretically provide the highest accuracy because the assumptions of the underlying model equations (SWE) are less likely to be violated and the topography is more accurately represented. However, the meshing procedure can be extremely complex, particularly if structured grids are used.

The techniques described above have been run and tested by Impact partners participating in flood propagation work. Their performance with respect to experimental data will be discussed in the next section.

4. Experimental data and model validation

Model validation is an unavoidable task that should be conducted in as much as possible under controlled conditions in order that influences from different parameters are properly attributable to their true sources. This is rarely the case when validation is performed with data obtained from real life events because these seldom occur under controlled conditions; rather the opposite. During the Impact research project dedicated laboratory experiments have been conducted to get insight into flow characteristics, and complete data sets usable for model validation that are practically impossible to obtain from a real flood.

The experiments performed can be classified into two types: a) very simple geometric configurations in which the flooding is extremely idealized and b) scale physical models of actual topographies with some simplified assumptions.

Idealized situations allow focusing only on a limited number of parameters and providing interesting information on specific features. Scaled down physical models allow a more realistic representation of the flood processes but still under controlled conditions. The first type of experiments has been performed at Université Catholique de Louvain (UCL), in Louvain-La-Neuve (Belgium), and the second type at CESI (formerly ENEL-CRIS) at its Milano (Italy) facilities. Both will be more thoroughly described below.

Since laboratory data are not truly representative of actual flood events due to unavoidable scale effects and experimental simplifications, data from the extreme flood caused by Tous Dam break (Spain 1982) including the catastrophic inundation of the town of Sumacárcel have been collected and used as a case study for model validation. It is needless to say however, that real life data are not as comprehensive as experimental ones.

4.1 Experiments performed at UCL

Experimental work performed at Université Catholique de Louvain (UCL) by the team of Prof. Y. Zech comprises two idealized configurations: (i) a dam-break flow in a prismatic flume with an isolated building (Soares-Frazão et al. 2003 and 2004) hereafter called *The isolated building experiment*, and (ii) a dam-break flow in a prismatic flume with a submersible hill-shaped obstacle (Soares-Frazão et al. 2002), named *The bump case*.

The isolated building experiment was designed with the aims of firstly investigating near-field effects and secondly assessing the consequences of the presence of a building on the downstream flow. The set-up is as sketched in Figure 1, but more detailed descriptions of the experiment can be found in the cited references. Near-field effects around the building consist mainly in the formation of hydraulic jumps and a wake zone behind the building. Figure 2 shows a picture taken with the high-speed cameras placed above the channel, where the main flow features can be identified. After the rapid opening of the gate, the strong dam-break wave reflects against the building, almost submerging it, and the flow separates, forming a series of shock waves crossing each other. A wake zone can be identified just downstream from the building, surrounded by cross waves. The flow rapidly reaches an almost steady state with a decreasing discharge due to the emptying of the reservoir. Also, recirculation zones can be identified between the building and the walls. A flow simulation is shown in Figure 3.

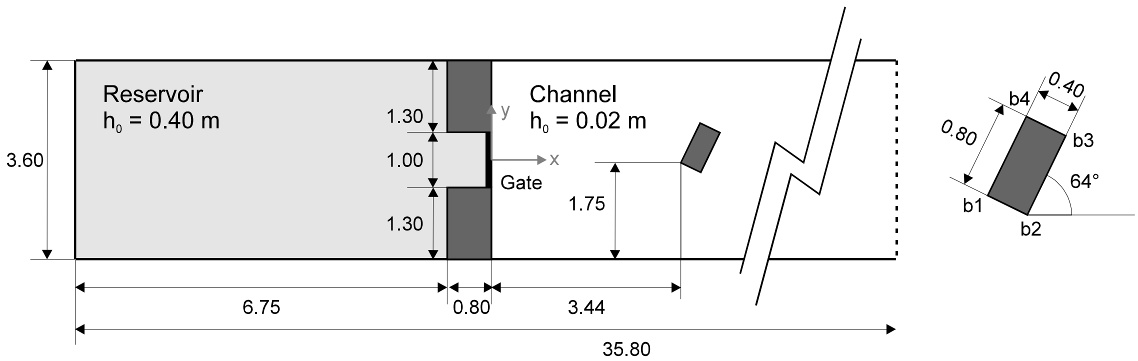

Figure 1: Experimental set-up for the isolated building test case (all units are meters).

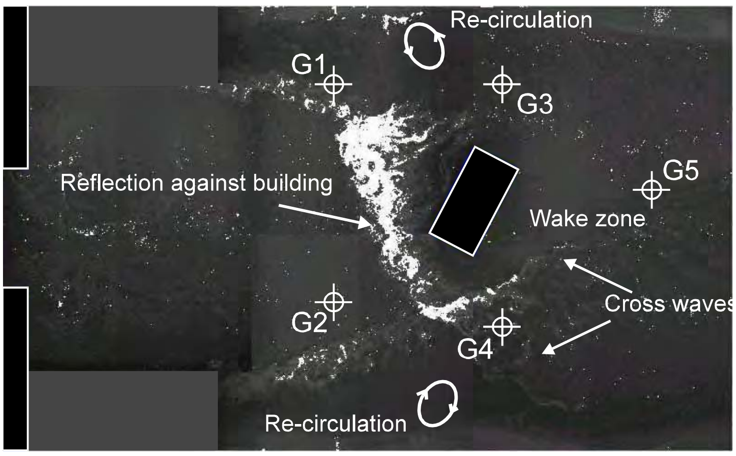

Figure 2: Picture of the flow at time t=5s after gate opening.

Experimental data recorded comprises water level and velocity vector evolution during the runs at some 6 gauging points, plus stills of the surface velocity vectors at several times.

The isolated building experiment was used to set up a benchmark named *The isolated building test case*, (Soares-Frazao et al. 2002) made up of two phases: The first one was of the blind type where experimental data were withheld until model results had been handed over by the participants. In the second phase, experimental data were disclosed to allow model calibration.

The blind test was run by eight different modelers from six different institutions. These modelers were both members and non-members of the IMPACT project. The participant members were (i) Noël, Soares-Frazão and Zech from the Catholic University of Louvain (Belgium), (ii) Mignot and Paquier from CEMAGREF (France), (iii) Mulet and Alcrudo from the

University of Zaragoza (Spain), and (iv) Murillo, Garcia-Navarro and Brufau from the University of Zaragoza (Spain). The non-member participants were (i) Capart from the National Taiwan University (Taiwan), (ii) Aureli, Maranzoni and Mignosa from the University of Parma (Italy), and (iii) Petaccia and Savi from the University of Pavia (Italy). All modelers use a finite-volume method. The detailed results are presented in Soares-Frazão et al. 2003.

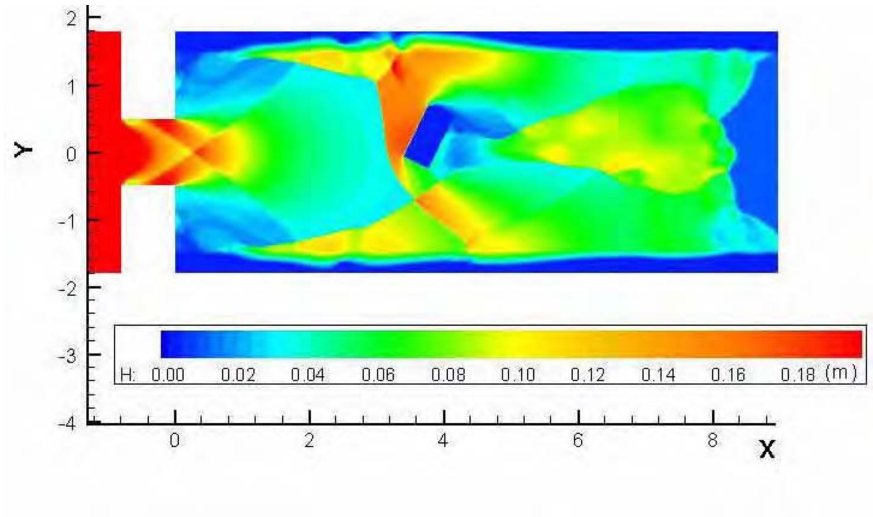

Figure 3: Flow simulation with the bottom elevation technique.

Results at gauge G2 are presented in Figure 4. This gauge is located as indicated in the sketch on the top left corner of the figure. It clearly shows the reflection of the wave against the building, around t=15s. Some numerical models seem to be completely missing the formation of the hydraulic jump. In fact, there is just an error in the position of the hydraulic jump, which is located downstream of the gauge position in these simulations.

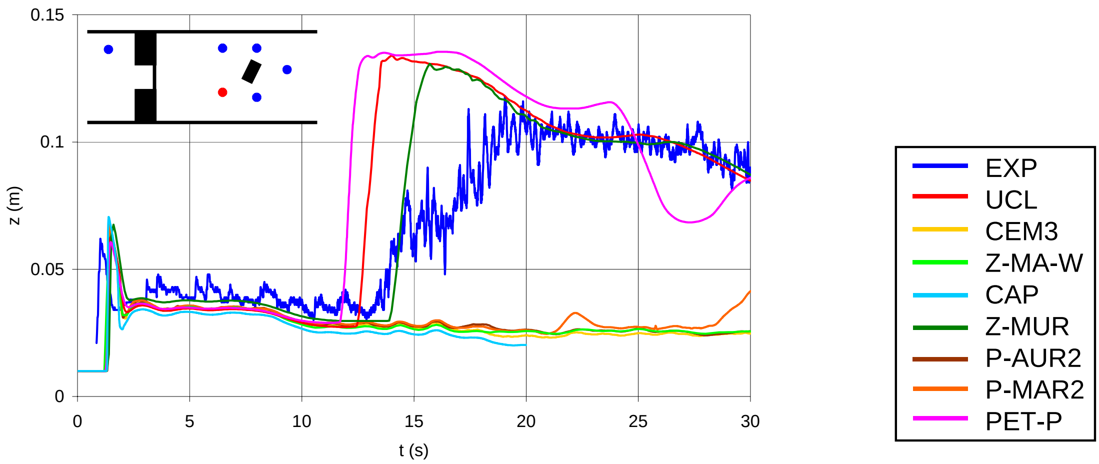

Figure 4: Experimental vs. numerical results (water level) at gauge G2.

Figure 5 presents the results at gauge G3, located downstream from the lateral hydraulic jump formed by the reflection of the front wave against the side-walls of the channel.

The agreement with the experimental results is good and all models give a similar evolution of the water level.

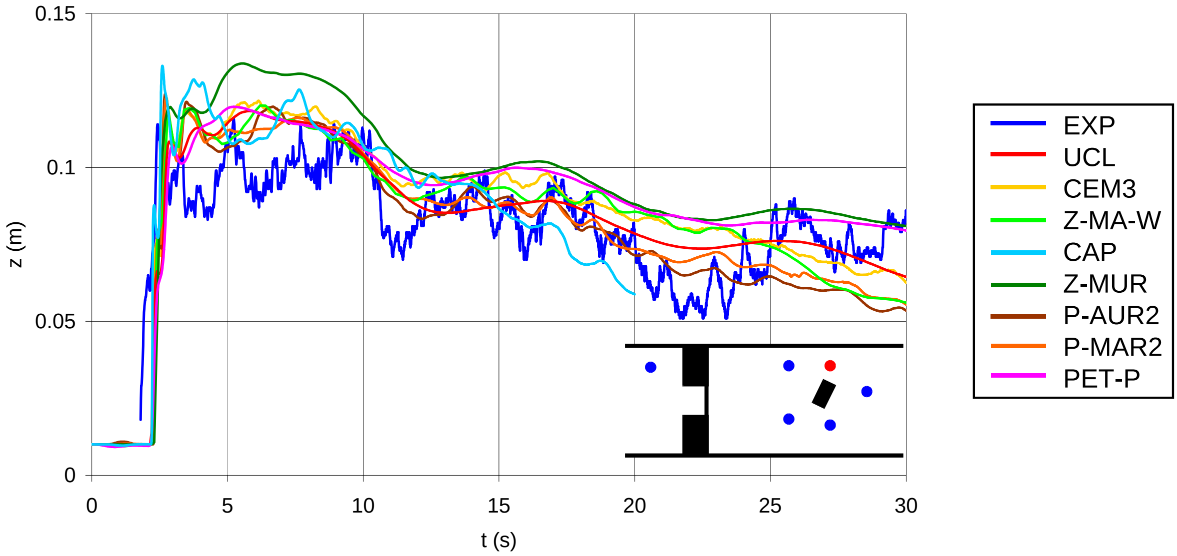

Figure 5: Experimental vs. numerical results (water level) at gauge G3.

The second laboratory experiment, the so-called *bump* test case, consists of a dam-break flow in a prismatic flume with a submersible hill-shaped obstacle. The aim was to investigate the effects of the topography on the dam-break wave, as well as the propagation of the wave front over the hill. A complete description of the experiment can be found in Soares-Frazão et al. (2002). After gate opening, water flows into the dry channel and once reaching the bump, part of the wave is reflected and forms a negative bore traveling back in the upstream direction, while the other part moves up the bump, resulting in a wave propagation on an upward dry slope. Then, after passing the top of the bump, the water flows on the downward dry slope until arriving to the pool of water at rest. The rapid front wave is slowed down abruptly, and a positive bore forms. This bore reflects against the downstream wall and travels back to the bump, but the water is unable to pass the crest. A second reflection against the downstream wall is needed to enable the wave to pass the bump and to travel back into the upstream direction. Multiple reflections of the flow occur both against the bump and the channel ends.

4.2 Experiments performed at CESI

The Impact CESI team (Italy) presently led by G. Testa has been performing a series of experiments at their Milano facilities that comprise a 1:100 scale physical model of a reach of the Alpine river Toce. The model is equipped with more than twenty water depth gauges of the conductivity and pressure transducer types at several locations. Flooding is accomplished by rapidly raising the level of a feeding tank at the upstream end of the model by means of an electric pump. Pump discharge and hence inflow hydrograph (or flood intensity) can be electronically controlled and monitored.

A general view of the model can be seen in Figure 6. Several types of experiments have been conducted in last few years: some concentrating in flood propagation along the reach, others on urban flooding. For the latter, a model city made up of some 20 concrete dice of 15cm side was placed on the model bed. A set of comprehensive test runs was undertaken with many different configurations. Two block (building) layouts were considered: one with streets aligned with the valley axis and the other one with a staggered structure. Two valley bathymetries: The original and a simpler one where the bed was flattened and concrete sidewalls placed along the valley. The latter was an attempt to isolate effects due to the presence of the city from those coming from the valley bathymetry. Finally, different flood intensities without reaching building overtopping were tested. Water depth history was recorded at some 10 gauging points intermingled with the model buildings.

Figure 6: General view of CESI physical model.

A subset of the tests accomplished was used within Impact project to set up a benchmark for model validation under the name of *The model city flooding experiment* (Alcrudo et al. 2002). In all, seven different configurations were included in the benchmark, varying model city layout, valley bathymetry, flood intensity and mathematical representation of buildings within the models as explained in the previous section. As with the isolated building experiment, the benchmarking campaign was of the blind type whereby experimental data were not available to modelers. After simulation results were handed over, experimental data were released to modelers to allow calibration. Most modelers performed a sensitivity analysis regarding mesh convergence, bed friction coefficient, building representation strategy and particular numerical parameters. Benchmark results were presented and discussed at the 3rd Impact Workshop held at Louvain-La-Neuve (Belgium) in November 2003, and a report is to be issued after a more in depth analysis is performed. Some preliminary conclusions that can be drawn are:

- All of the methods used for building representation produced comparable results.
- The one-dimensional approach fails in predicting water depth evolution at some gauges for the staggered city lay out as could be expected.
- Most models predict a shorter front propagation time.
- The friction-based method tends to under predict water depth probably due to the storage effect of building area.
- Overall accuracy regarding water depth history can be estimated to within 20 percent of experimental data for most gauges.

Figure 7: A model city flooding test run.

A general view of one of the urban flooding test runs with a staggered city lay out and the original valley bathymetry can be seen in Figure 7, and a synthetic image representing flood wave arrival produced with a mathematical model by means of the bottom elevation technique described in previous section is shown in Figure 8.

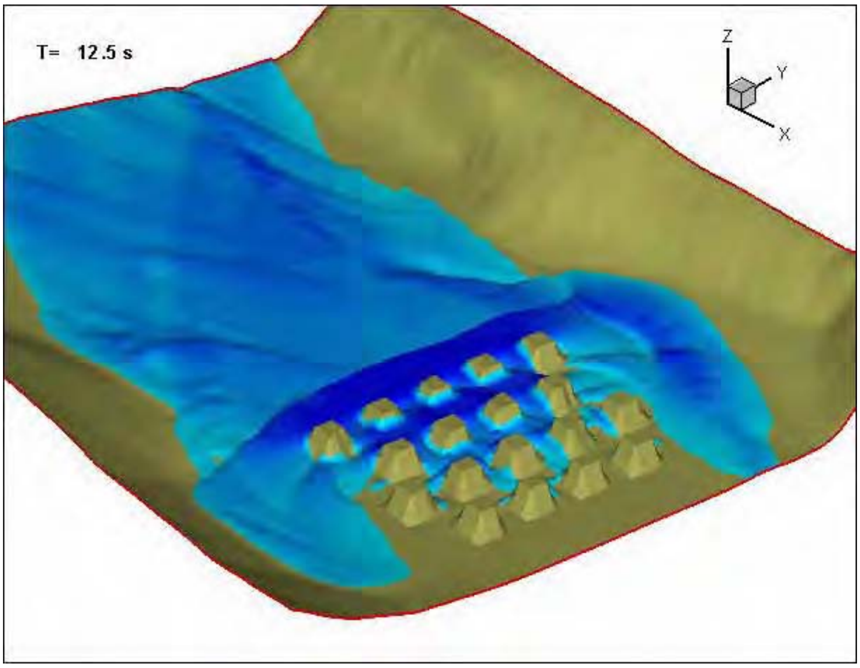

Figure 8: Simulation of the model city flooding with the bottom elevation strategy.

Finally, Figure 9 presents a comparison of the results obtained at gauges No. 5 and No. 6 located after the first and second row of buildings respectively, computed with the different building representation techniques as implemented in one particular model.

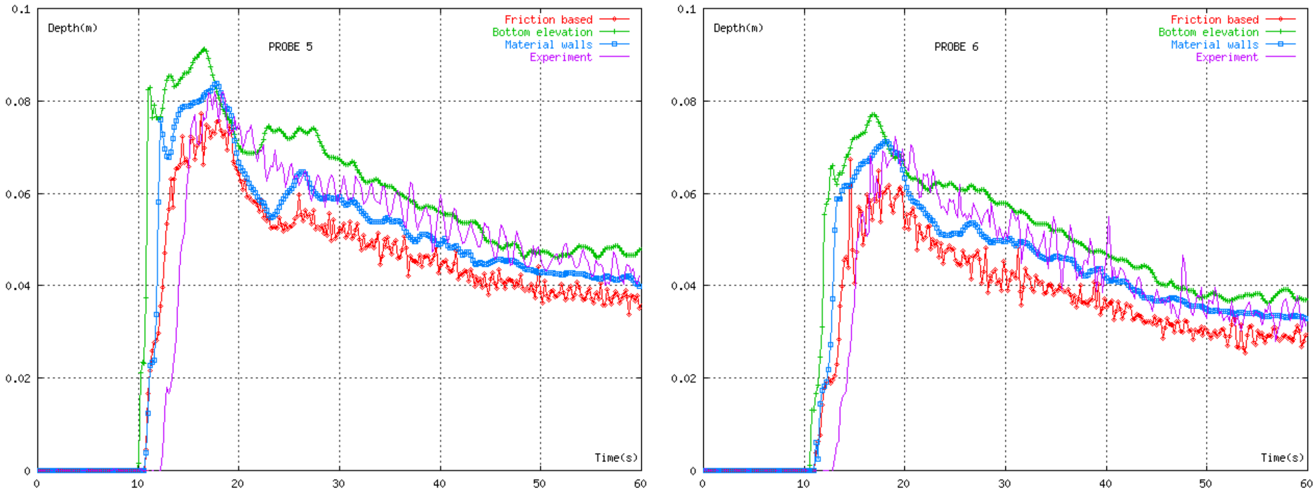

Figure 9: Comparison of experimental data with model predictions at two gauge locations.

4.3 The Tous Dam break case study

In order to compare model predictions with real life data an effort has been made within Impact Project to locate and document an actual extreme flood that could be used for model validation. The main requirements imposed were that the flood were extreme (i.e. catastrophic), such that enough data were available to allow a reasonable modeling scenario and such that urban areas were affected in accordance with the project objectives. Several floods involving cities or towns were considered (Nîmes, France, 1988 flash flood, Florence, Italy 1966, Estes Park after Cascade Dam break, USA 1982, Sumacárcel after Tous Dam break 1982, Badajoz flash flood, Spain 1997). Tous Dam break was finally chosen representing a compromise between required characteristics and data availability.

Tous Dam, located in the Southeast of Spain, close to the Mediterranean Sea, failed in October 1982 after extremely heavy rainfall in the basin totaling 600Hm^3 in three days (largely exceeding reservoir capacity) caused overtopping. The dam was of the rock fill type and stood only a few hours of overtopping. The effects of the flood downstream of Tous Dam were catastrophic: 300 km^2 of inhabited land, including many towns and villages were severely flooded, affecting some 200,000 people of which 100,000 had to be evacuated, totaling 8 casualties. The first affected town is Sumacárcel (population 2000), about 5 km downstream Tous Dam, lying at the foot of a hill on the right bank of the river. The ancient part of the village, located closer to the river course was completely flooded for only a few hours, with high water marks reaching between 6m and 7m above the ground (see Figure 10).

An Impact project team gathered the available information and data from the competent bodies, local authorities, and from several field visits whereby eyewitnesses were interviewed. The collected information was indexed and described in several Impact project reports (Alcrudo and Mulet 2004) and used as a case study for the project. Modeling work is currently underway by involved partners and results and conclusions will be presented at the Impact project closing workshop to be held in Zaragoza (Spain) on the first week of November 2004.

It is interesting to note that information on the breaching process itself could also be retrieved and collated in order to make up a case study for breach formation work within the project (Mulet and Alcrudo 2004, 2004b).

Data available for model validation comprise two digital terrain models (DTM) of the area with 5m spacing: one obtained from CEDEX (Ministerio de Medio Ambiente, Spain) and the other prepared by CESI after ancient cartographic maps provided; a series of digitized cartographic plates to 1:10000 scale of the area one week after the flood, where inundated areas are clearly visible. Hydraulic data of the Tous Dam and Reservoir included the outflow hydrograph and high water marks with some rough timing at several points inside the town of Sumacárcel. The reader is referred to the cited reports for more thorough information.

Figure 10: A cartographic view of the town shortly after the disaster.

The Tous case study represents quite a challenging situation for several reasons: The flood is very severe and its duration is more than two days. The town, where high resolution is needed, is small in comparison with the rest of the valley stretch. This makes the meshing procedure difficult. The combination of space and time scales makes model runs computationally very expensive. Figure 11 shows a simulated view of the flooded streets near peak flood time.

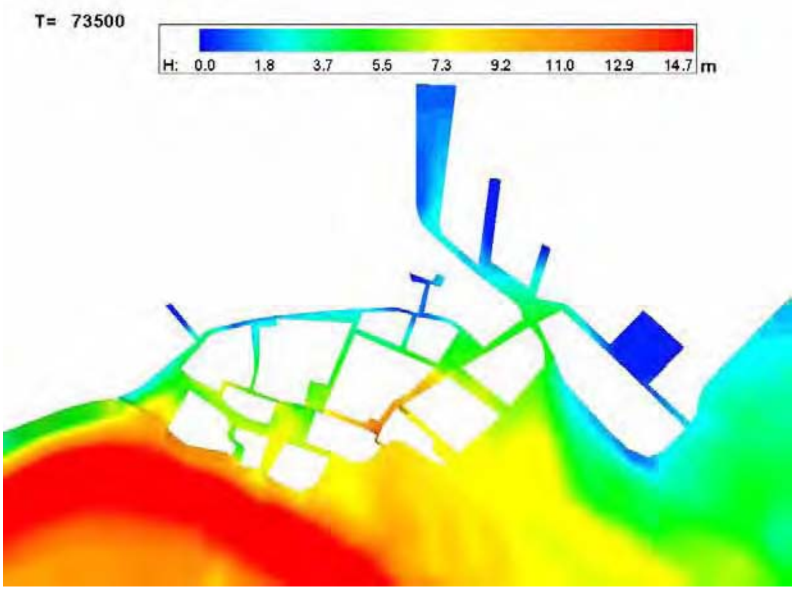

Figure 11: Water depth predictions during flooding of Sumacárcel near peak flood conditions.

5. Conclusions and future trends

A short review of the main issues concerning flood propagation model development has been presented in this paper. Particular emphasis has been put on model validation where the perspective of Impact research project work has been given.

The vast majority of flood propagation models in use today are based upon the Shallow Water equations (SWE). Computational methods developed for their solution provide presently a high degree of accuracy and resolution, and it can be said that all the numerical problems encountered a few years ago are practically solved. The computer power available today makes it possible modeling the inundation of large areas with great topographical detail. This may lead to think that simply by mesh refining more accurate predictions can be made. However, this is only partially true, because the numerical solution so computed will converge to the exact solution of the SWE that is not the solution of the flooding problem itself. It could be said that the main reason for the lack of agreement between experimental measurements and a careful and well resolved model simulation is the inadequacy of the SWE formulation to describe flood flow. Nevertheless, it is the view of the authors that there is yet some room for improvement still within the SWE framework: Vertical slopes should be accommodated within the models and the influence of diffusion terms needs some clarification. In any case more elaborate mathematical descriptions of free surface flow, that is the basis of flood propagation models, are not yet ready for practical use nor is it expected that they will be so in the near future.

In the meanwhile validation is a sure path to gain confidence in model predictions. Routine application of models to real world floods will help sort out strategies and algorithms that perform well in idealized situations or against laboratory experimental data, but may encounter operational problems in real life applications.

The future is challenging because it is perceived that industry interest in flood propagation models grows at the same pace as modeling technology advances, and current models are accurate enough to make useful predictions of the effects of extreme inundations.

6. Acknowledgements

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