METHODS FOR ASSESSING THE FAILURE PROCESS OF CONCRETE DAMS FOUNDED ON ROCK

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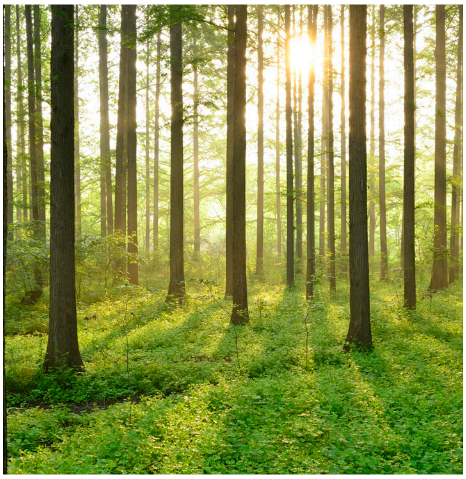

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Methods for Assessing the Failure Process of Concrete Dams Founded on Rock

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Foreword

This study was conducted on behalf of SVC (the Swedish Centre for sustainable hydropower) and focuses on simulating dam failures. The knowledge regarding the failures of concrete dams is relatively limited. Nowadays we have fine methodologies for assessing the ultimate load of a concrete dam but not for simulating the post failure development therefore this report targets on the methodologies for simulating dam failures.

The project was financed by SVC, the Swedish Centre for sustainable hydropower and carried out by Jonas Enzell (KTH the Royal Institute of Technology in Stockholm/Sweco AB), Richard Malmn (KTH/Sweco AB) and Erik Nordström (KTH/Vattenfall AB)

Stockholm, October 2021

Sammanfattning

Kunskapen om betongdammars brottförlopp är relativt begränsad. Idag används mycket förenklade antaganden för faktorer som bräschens storlek och brottförloppets tid, trots att dessa faktorer har en mycket stor inverkan på översvämningssimuleringar och beredskapsplanering. Bra metoder existerar för att avgöra betongdammars brottlast men inte för att simulera förloppet efter att dammen gått till brott. Den här rapporten fokuserar därför på metodologi för att simulera dammbrott.

Numeriska simuleringar har utförts för att undersöka tillämpbara metoder i kommersiella FE-programvaror. Först simulerades ett skaltest där stödskivan från en betongdamm trycktes till brott med hjälp av hydraulcylindrar. Brottmoderna från skaltestet reproducerades på ett bra sätt. En ny metod för att simulera ett deformationsstyrt brottförlopp i betongdammar föreslogs därefter. Metoden implementerades på stödskivan på en betongdamm och resultatet jämfördes med en klassisk deformations- och laststyrd simulering. Den nya metoden gav resultat som överensstämde med de tillgängliga metoderna. Fördelen med den nya metoden är att den kan tillämpas i samtliga kommersiella FE-programvaror utan programmering av subrutiner och att den inte lider av konvergensproblem, vilket innebär att dammens beteende efter det initiala brottet kan simuleras.

Brottsimuleringar genomfördes för en svensk armerad valvdamm. Valvdammen är sprucken på nedströmssidan och sprickorna har orsakats av säsongsberoende temperaturvariationer i dammens omgivning. Simuleringarna utfördes för att undersöka sprickornas inverkan på brottmod och brottlast. Resultaten visade att sprickorna hade liten inverkan på både brottmoden och brottlasten i den undersökta valvdammen. Inverkan blir liten eftersom sprickorna till stor del går längs dammens valv och medför att valvet överför lasten till upplagen på dammens sidor på ett effektivt sätt.

Ett koncept för att numeriskt simulera vattnets dynamiska inverkan på ett dammbrott föreslogs. Simuleringarna, baserade på Euler Lagrange-metodik, utfördes på en generisk betongdamm med en fördefinierad brottyta för att testa metoden vilket visade lovande resultat.

Slutligen föreslogs ett koncept för att simulera dammbrott i en lamelldamm i en fysisk skalmodell. Konceptet inkluderar fem monoliter i skala 1:20, vilka trycks till brott med hjälp av vattentryck. Försöket utförs i en ränna med ett lock för att kunna höja vattentrycket. Betongens materialegenskaper skalas i samma skala som geometrin. Målet med försöket är att undersöka brottförloppet inklusive vattnets effekt på de intilliggande monoliterna efter den första monoliten går till brott och tiden det tar för brottet att utvecklas. Resultaten från experimentet kommer att öka förståelsen för dammbrott och fungera som referens vid framtida numeriska simuleringar.

Summary

The knowledge regarding the failures of concrete dams is relatively limited. However, factors such as the size of a breach and the time of the development of the failure has a large impact on flooding simulations and emergency planning. Today, good methodologies exist for assessing the ultimate load of a concrete dam but not for simulating the post failure development. This report therefore focuses on methodologies for simulating dam failures.

Numerical simulations have been performed to examine different aspects of the available methods in finite element analysis. First, a physical model test of a concrete buttress, which was pushed to failure using hydraulic jacks was reproduced. The failure modes were reproduced in a reliable manner. A new displacement-controlled method for simulating dam failures using nonlinear spring was thereafter proposed and implemented on a case study. The method was compared to classical displacement- and load-controlled simulations. The results from the proposed method corresponded to the existing methods. The proposed method can be introduced without the implementations of subroutines in all commercial FE-software. The method is also stable and does not suffer from convergence issues, which allows for the simulation of the post-peak behavior.

Numerical failure simulations were performed on a Swedish reinforced concrete arch dam with cracking induced by seasonal temperature variations. The simulations examined the effects from the cracking on the failure modes and the ultimate load. The results showed that in this case, the cracking had a limited effect on the failure mode and the ultimate load. This is largely because the cracks were aligned along the arch of the dam, wherefore the compressive stress from the water could still be transferred in an effective manner to the abutments of the dam.

A concept for numerical simulations of dam failures including the dynamic effects of the water using coupled Euler Lagrange-simulations was proposed. Sample simulations on a generic gravity dam with a predefined fracture plane was performed to test the method, which gave promising results.

Finally, a concept for simulating the failure of a concrete buttress dam using physical model tests was proposed. The concept includes pushing five concrete monoliths in scale 1:20 to failure using hydrostatic pressure. The experiment will be performed in a concrete chute with a lid to enable a higher pressure and the material properties of the concrete dam will be scaled to the same degree as the geometry. The goal of the physical model test is to investigate the failure of concrete gravity dams including the dynamic effects from the water. The effect on the adjacent monoliths after the first monolith has failed and the time for developing the failure will also be examined. The results from the test will increase the understanding of dam failures and work as a benchmark for future numerical simulations.

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1 Introduction

The vast majority of the concrete dams around the world are considered to be safe and fulfil the prerequisites to resist all the loads a dam can be exposed to. In spite of this, dam failures have occurred since hydropower was first introduced about 150 years ago. A majority of these occurred during the early 20th century, but failures still occur today but with lower frequency (Nordström et al., 2015). However, compared to other civil structures, relatively few dam failures have occurred, especially in concrete dams. The knowledge regarding failures in concrete dams is therefore relatively limited. The initiation and development of the width of the breach and the time required for a failure to propagate are examples where the current knowledge is limited. The reason for this is lack of observed and documented failures and breach developments.

The assumptions made regarding the size of a breach and the time it takes for a concrete dam to fail are often crucial when evaluating the effects of a dam failures. The assumptions are important for performing realistic flooding simulations and emergency preparedness planning including action plans. The factors discussed (size and time of the breach) are important both in design and assessment of existing structures. The assumptions regarding the breach size varies in different national design codes and guidelines. According to ICOLD (1998), the assumption is usually that one or two monoliths fail. However, it is pointed out that the reported dam failures often are larger, sometimes up to 100 m wide. Also, the time of failure development varied between 10 and 60 minutes for the observed cases in Nordström et al. (2015).

Knowledge regarding the actual failure modes and how the failure develop is also important for the design of monitoring systems where the early signs of changes needs to be captured to be able to take mitigating actions or activate action plans.

Because of the dynamic nature of dam failures and the influence from the water, it is difficult to simulate realistic dam failures. Physical model tests could give additional knowledge of the failure process of concrete dams and provide important benchmarks for future numerical simulations.

1.1 SCOPE AND AIM

The general goal of this project was to examine existing and developing new methodologies for performing failure simulations of concrete dams. The benefits and limitations of performing physical model tests were also to be examined. Finally, the project should give recommendations and present concepts for how physical model tests can be performed in the future. These model tests should be aimed to capture the size of the breach and the time required for a failure to develop after it has initiated. In these tests, water should be used to push the dam to failure in order to create realistic failure progressions in lab environment.

1.2 LIMITATIONS

The project has been performed as a literary review and by performing numerical simulations. Performing the physical model tests are not within the scope of this project.

2 Theory

To ensure function and safety, a concrete dam connected to hydropower must safely retain and discharge water (Nordström et al., 2019). More specifically, the dam must be:

- Watertight against prevailing water pressure.
- Have acceptable stability and act monolithically.
- Meet requirements for load-bearing capacity and stiffness.

This report will primarily focus on the global stability of concrete dams and have less focus on internal failures unless these can lead to decreased stability of parts of the dam.

2.1 CONCEPT OF SAFETY FOR CONCRETE DAMS

In most codes, the stability of concrete gravity dams is determined by simple analytical calculations based on the assumption of rigid body motions. The concrete gravity dam is assumed to fail along a predefined plane in the interface between the foundation and the dam, in lift joints in the dam or along faults in the rock. Two global failure modes are considered; sliding and overturning. The resisting and driving forces are compared and a factor of safety is determined for these two failure modes. For the sliding failure mode, the factor of safety is defined as

$$s_s = \tan\delta_g \frac{R_V}{R_H} \tag{1}$$

where, δ_g is the friction angle, $\tan\delta_g$ is the coefficient of friction, R_V is the resultant of the forces perpendicular to the sliding plane and R_H is the normal force in the sliding plane. The factor of safety for the overturning failure is defined as

$$s_o = \frac{M_R}{M_D} \tag{2}$$

where, M_R is the resisting moment and M_D is the overturning moment.

According to the Swedish guidelines for dams, Ridas (2017), the safety factor for sliding failure of a concrete gravity dam on good quality rock should be larger than $s_s = 1.35$. The safety factor for overturning should be larger than $s_o = 1.5$.

These analytical methods give an estimation of the safety that is sufficient for many practical applications. The availability of commercial FE-software has however given a possibility to perform more detailed analyses. Based on the same concept of safety, analyses can be performed that includes irregular geometry in the interface between the rock and the dam, deformations of the material and combined failure modes. In addition, using FE-simulations, nonlinear material behavior and cracking of the concrete and a detailed analysis of the reinforcement in the structure can be included. The numerical calculations give the same results as the analytical if the same assumptions are used. Since additional factors can be considered, such as the irregular rock surface, which gives a higher safety or nonlinear material behavior, which can lower the safety, the dam safety can be considered both higher or lower from a more detailed analysis. Numerical estimations of the safety of concrete dams have for an example been performed by Fu and Hafliðason (2015) and Hellgren and Malm (2017).

The resistance of a structure cannot be calculated directly using FE-simulations. Therefore, the failure of the dam must be simulated. This is done by a pushover analysis, where the operational state of the dam with applied design loads is first simulated. From the operational state, the driving forces (horizontal hydrostatic pressure, ice load, uplift etc.) are increased until a failure is reached.

Because the simulations usually are load controlled, the best practice is to perform static analyses (Malm, 2016). Static analyses do however often lead to convergence issues when using nonlinear material models. Dynamic or quasi-static solvers are therefore often used. Using these solvers does however introduce inertia forces, which can impact the results and it is important to control the amount of kinetic energy in the simulation and discard any results where the kinetic energy is too high.

To reach results corresponding to those from the analytical calculations, it is important to retain the location of the force resultants when simulating the failure. This is done by increasing the hydrostatic pressure triangularly and keeping the pressure at zero at the reservoir surface (Malm, 2016). This is analog to increasing the density of the water. A more likely failure scenario would be to simulate an overtopping of the dam, this does however give results that does not correspond with the analytical calculations.

The pushover method described above is an overload method, where the loads are increased to induce a failure. This approach is widely used in structural engineering to investigate the load capacity for other type of concrete structures such as bridges, buildings, nuclear power plants etc.

In numerical modeling and in physical model tests, the reduced strength method can also be used. This entails reducing the strength of the material or some other factor that induces the failure. The reduced strength approach is commonly used in geotechnicnical engineering where the cohesion and friction angle are successively reduced (c-φ reduction). In cases with concrete dams, of course the density and friction angle could be reduced to capture the global behavior. However, concrete is prone to crack and these cracks may initiate alternative failure modes. In order to capture this, the material strength must also be reduced. However, this limits the use of the reduced strength method for concrete dams, since the material model will not behave correctly if the strength is reduced. In addition, if an element is already fully cracked then further decrease of the strength has no effect since the element is already considered to have zero strength. This can cause unwanted effects possibly unrealistic failure modes. Therefore, it is often preferable to use overload methods for concrete dams.

2.2 NUMERICAL SIMULATIONS

2.2.1 Load-controlled simulations

The methodology for simulations with the overload approach described in the previous section gives results corresponding to the concept of safety defined in the dam safety guidelines. They are however load-controlled, which introduces some issues. For performing failure analysis, it is often preferable to use displacement-controlled simulations. In a displacement-controlled simulation, the displacement of one point, e.g. the center point of a beam, is given a predefined displacement and the equilibrium forces are solved for that displacement. This allows for unloading of the structure when the ultimate load is reached. In a load-controlled simulation, the load usually does not have the possibility of unloading. This means that when the ultimate load is reached, the structure can become unstable. In the case of a dam, the entire concrete structure can leave the foundation and accelerate downstream. The results from the load-controlled simulations can therefore be used to determine the ultimate load of the structure but not to simulate the post failure behavior. Hence, this approach cannot be used to capture the breach size or the time for the failure progression.

The failure could be simulated using a load-controlled method such as the arc length method, which allows for unloading. However, such a method requires a stable static solution and a dam failure is not static. This means that when the dam reaches the point of instability and fails, the arc length method will drop the load to zero or even reverse the load in order to maintain the equilibrium.

2.2.2 Displacement-controlled simulations

When performing displacement-controlled analyses for simulations of dam failures, it is usually suitable to use a control point in the crest of the dam. The displacement-controlled simulations are useful for determining if the dam has a sudden or more ductile failure. However, the displacement-controlled analyses also use a static loading algorithm and can therefore not be used to simulate the actual failure progression. Convergence issues are also common for displacement-controlled simulations and must be countered by the analyst. Displacement-controlled analyses can be performed in most commercial FE-software. However, subroutines must be used in order to implement the procedure of defining a control-point in some of the software, e.g. Abaqus.

A methodology has been developed during this project to simulate dam failures, which uses springs to push the dam to failure. The method uses a dynamic solver but is defined to be quasi-static. It thereby avoids some of the issues of the previously mentioned methods. This methodology is discussed further in Section 4.2.

2.2.3 Fluid-structure interaction

In order to simulate the post-failure behavior of a dam failure more realistically, the dynamic effects from the water must be incorporated. Simulating the flow of water is complicated and computationally demanding, therefore simplified methods are commonly used. For example, in cases with seismic response of concrete dams, acoustic elements are commonly used, which describes the water by pressure wave motions and neglects the flow. To capture the post failure behavior of a concrete dam, however, the flow must be simulated. There are available methods for fluid-structure interaction (FSI), which incorporates the flow of the water. Coupled Euler Lagrange (CEL) is such a method, which has been studied in this project. The method is further described, and simulations are presented in Section 4.4.

2.3 PHYSICAL MODEL TESTS

Reports gathering experience from failures in concrete dams exists, see e.g. Nordström et al. (2015). In total, there have been relatively few failures in concrete dams through modern history. Concrete dams also differ from other civil engineering structures in geometry and loads. The limited practical knowledge of dam failures makes it difficult to make accurate predictions regarding the failures of concrete dams. Numerical simulations are used to a large extent because of the good ability for making predictions and the low cost of performing simulations. However, few or zero cases of actual dam failures exist with sufficient documentation and measurements to be used as validation for the numerical simulations. Physical model tests can be of great value to increase the knowledge regarding dam failures. At the same time, the physical model tests work as validation to increase the predictable ability of numerical analyses such as finite element simulations.

Before the development of the finite element method and modern computers, physical model tests were the common practice when designing arch dams because of the lack of reliable computational models (Fumagalli, 1973). One common method is to induce a failure using hydraulic jacks. This was done for the Zillergründl arch dam as reported by Hofstetter and Valentini (2013) and the Alto Lindoso arch dam as reported by Oliveiraa and Fariab (2006), see Figure 1. Hydraulic jacks have also been used for testing gravity dams. For example, Sas et al. (2021) performed pushover tests on the supporting buttress of an Ambursen concrete dam in scale 1:5 to failure using hydraulic jacks. These model tests are also used in a case study in this report, see Section 3.1.

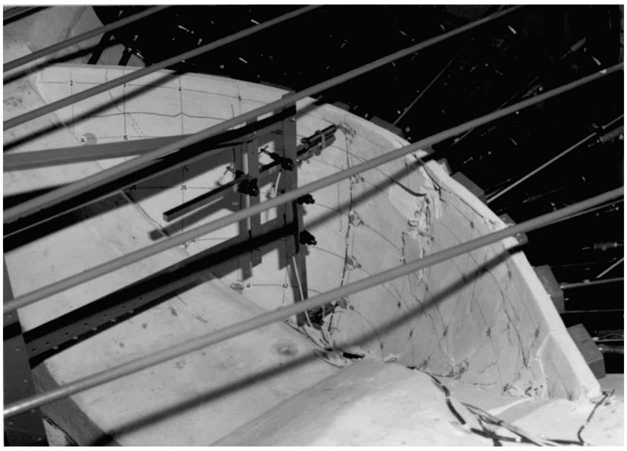


Figure 1. Experimental setup with hydraulic jacks after the failure in the model test of the Zillergründl arch dam (Hofstetter and Valentini, 2013).

Water is often omitted from the physical model tests when simulating dam failures due to increased loads, since it adds complexity and makes it more difficult to increase the load up to failure. However, in physical model tests of the seismic response of concrete dams, the water have been considered by e.g. Harris (2002) to account for the hydrodynamic forces. An example of a seismic model test is presented in Figure 2, where models of the concrete dam, the foundation rock including the water-filled reservoir are placed on a shaking table. As can be seen in the figure, the excitation was increased to induce failure in the concrete dam.

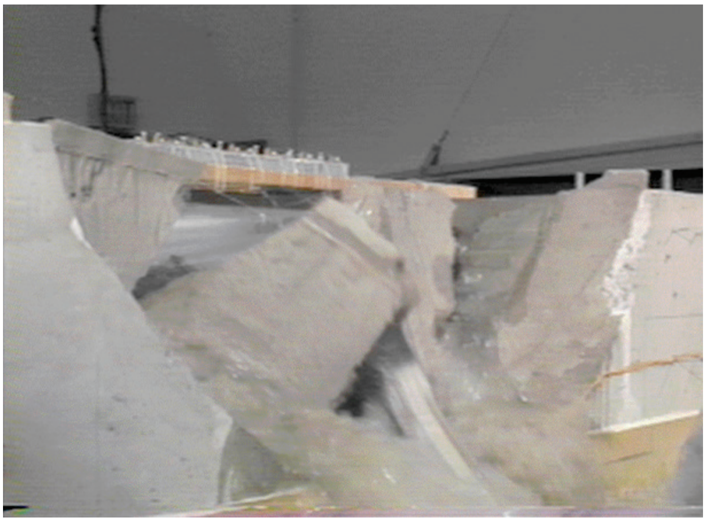


Figure 2. Seismic test using a shake table on a model of a concrete arch dam with a water-filled reservoir (Harris, 2002).

When physical model tests are performed, laws of similitude must be followed in order to maintain the correct relationship between the physical properties of the prototypr (the actual structure) and the model. The scaling laws are for an example described by Fumagalli (1973).

Most physical properties can be described from four main scale factors:

- λ_l, governing the geometry and displacements
- λ_γ, governing the density of the material
- $\lambda_\sigma = \lambda_l \lambda_\gamma$, governing the stress and strain in the model
- $\lambda_\varepsilon = 1$, governing the strains and other dimensionless parameters

The relation between the scale factors must be maintained. The relation between the prototype quantity and the model quantity is $x_p = Sx_m$, where x_p is the quantity in prototype scale, x_m is the model quantity and λ is the scale factor. For an example, the length in model scale is calculated as: $L_m = L_p/S_l$.

3 Case studies

3.1 PHYSICAL MODEL TEST

In 2019, a collaboration project, *Stable Dams,* was conducted between Norut and Luleå University of Technology where a series of physical model tests of a concrete buttress wall were performed (Sas et al., 2021). The buttress wall from a Norwegian concrete Ambursen dam was recreated in scale 1:5 and an overturning and sliding failure was induced using hydraulic jacks. The goal of the tests was to determine the influence of irregularities in the interface between the buttress and the foundation on the dam safety. In this project, these model tests have been used to validate the numerical method used to simulate failure of concrete dams.

The test setup is illustrated in Figure 3. The model buttress was 1.4 m high and with a thickness of 0.1 m, the original dam was 9 m high. Low strength concrete, with a compressive strength of 8.2 MPa was used for the buttress. The buttress was cast on a foundation made from normal strength concrete. The hydrostatic pressure and the ice load were introduced using hydrostatic jacks and the uplift was considered using a static weight connected to a wire.

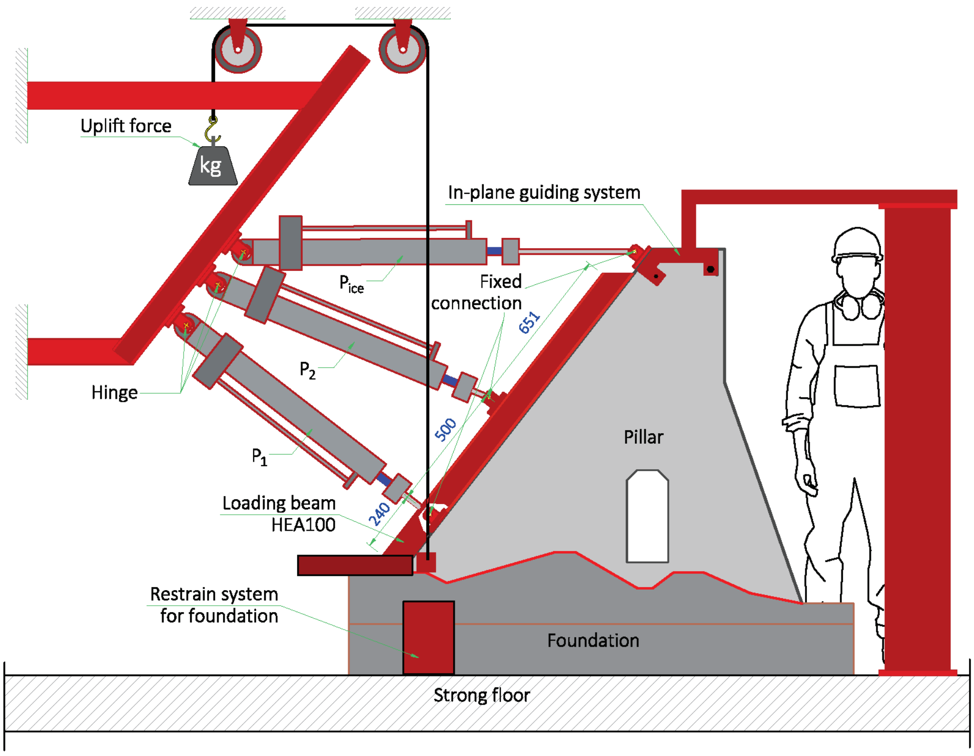


Figure 3. Experimental setup for the Norut model tests (Enzell et al., 2021b).

Rubber paint was used during the casting to break the cohesion in the interface between the buttress and the foundation. After the buttress had cured, it was removed, and the rubber was ground away. The coefficient of friction was experimentally determined to be $\mu = 0.60$. The material properties of the dam are presented in Table 1.

During the experiment, photogrammetry and video was used to monitor the displacements and strains of the concrete. LVDT-sensors were also used to monitor the displacements. The loads in the hydraulic jacks were also recorded.

Two types of failures were captured, overturning failures and sliding failures. During the overturning failure, the jacks representing the hydrostatic pressure and the weights for the uplift were loaded to the design load and the failure was thereafter induced by increasing the ice load. During the simulated sliding failure, the jacks representing the hydrostatic pressure were loaded until the dam failed, no uplift or ice load was induced during the sliding.

Table 1. Material properties for the Norut model tests. Only the material properties relevant for the numerical simulations are presented.

	Buttress	Foundation
Elastic modulus (GPa)	6	30
Poisson's ratio	0.2	0.2
Density (kg/m^3)	2354	2354

3.2 REINFORCED CONCRETE ARCH DAM

A Swedish reinforced concrete arch dam has been used as a case study to investigate the influence of existing cracks on the dam failure. The influence of the cracks was investigated regarding failure mode and ultimate factor of safety. Failure analyses were performed using finite element analyses to determine the factor of safety and failure mode with and without pre-existing cracks. The dam has cracks along the downstream face caused by seasonal variations in the ambient temperature. The geometry of the dam including the simulated crack pattern is illustrated in Figure 4. The dam was previously used as a case study in the 14th ICOLD International benchmark workshop on numerical analysis of dams, https://www.icold-bw2017.conf.kth.se/. In the conference theme, the participants were asked to predict the crack pattern of the dam based on temperature curves provided by the formulators. The proceedings from the conference were published by Malm et al. (2017).

The dam is a single curvature arch dam with a radius of 110 m. It is 40 m high and has a crest length of 170 m, which gives a crest-to-height ratio of 4.25. The ratio is high compared to most arch dams which means that the dam is relatively low and wide. The dam is slender with a thickness of 2.5 m at the crest and 5 m at the base. The dam is reinforced close to its upstream and downstream surfaces with a grid reinforcement consisting of 30 mm reinforcement bars with a spacing of 300 mm. The downstream surface has an additional layer of vertical bars of the same dimension and spacing to reduce cracking. The reinforcement bars are anchored in the abutments and over all construction joints to ensure that the arch dam acts monolithic.

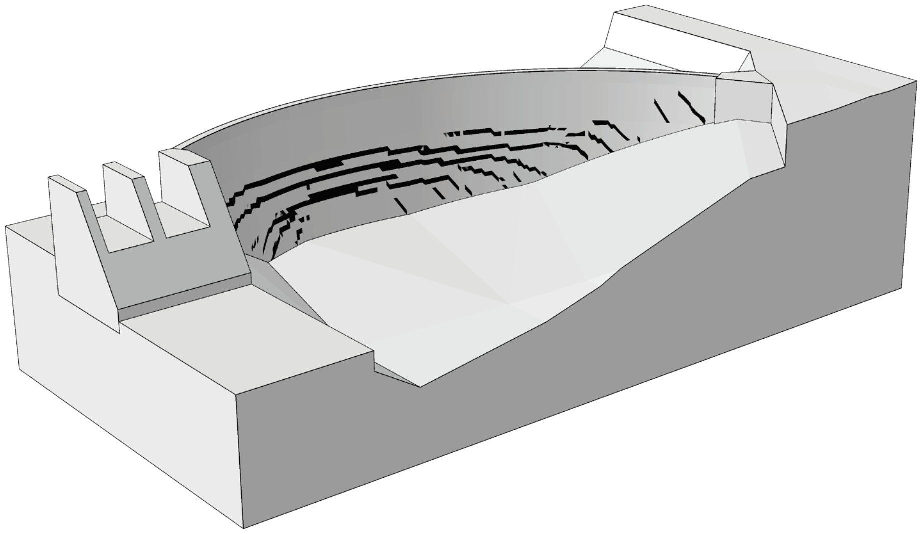


Figure 4. The Swedish arch dam used as a case study including the crack pattern in the downstream face.

The dam is located in northern Sweden, where the temperature variations are severe. This has led to cracking in the downstream face of the dam. To reduce the effect from the ambient temperature, an insulated climate barrier was installed on the downstream face of the dam. This limits the seasonal crest displacements and reduces the effect from cracking.

The dam has control-points installed and the deflection of the arch is measured using a total station. The control points are placed along the crest of the arch dam, on the dam body, and on the abutments and foundation. During the year 2000 to 2020, measurements were performed once or twice every year. During the early life of the dam, measurements were performed more often. The measurements have been used to verify the FE-model, see e.g. Malm et al. (2020), Enzell et al. (2021a) and Enzell and Tollsten (2017).

4 Numerical simulations

4.1 REPRODUCTION OF A PHYSICAL MODEL TEST

Simulations have been performed to recreate the Stable Dams physical model tests presented by Sas et al. (2021), the experiments are described in Section 3.1 The presented simulations were first presented by Enzell et al. (2021b). In order to reproduce the results from the physical model tests, the experimental setup had to be simulated. In the test setup, the hydrostatic pressure was applied using two hydraulic jacks, connected to a steel beam, which was placed at the upstream face of the buttress. The ice load was simulated using a third hydraulic jack and the resulting force from the uplift pressure was applied by hanging weights in a wire connected by pulleys to the upstream toe of the dam.

A numerical model was created using the FE-software Abaqus version 2019 and is illustrated in Figure 5. The foundation and the buttress were created with plane stress elements. They were connected using an interaction, which allowed for infinite normal compressive stress but no tensile stress, i.e. the surfaces could separate. Coulomb friction was used in the tangential direction with a friction coefficient of $\mu = 0.60$. The coefficient of friction was experimentally determined prior to the model tests. A steel beam was introduced at the upstream face of the buttress. The steel beam was connected to the buttress by an interaction. The hydraulic jacks were included in the numerical model by using spring elements.

The hydrostatic pressure and the ice load were applied by introducing a force, which created an expansion in the spring elements simulating the hydraulic jacks. The gravity load was calculated using the built-in function in the software and the uplift was applied as a point load.

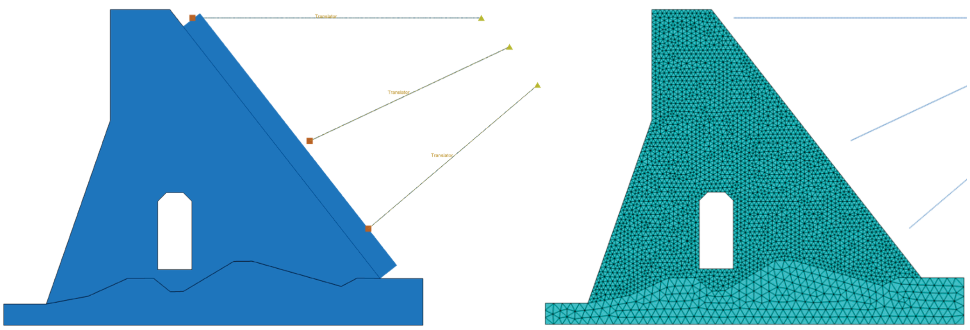


Figure 5. Geometry and spring elements of the numerical model and mesh (Enzell et al., 2021b).

The simulations were performed for a monolith with two main irregularities in the interface between the foundation and the buttress, see Sas et al. (2021). Two different loading procedures were considered in these experiments to ensure that the failure development was either governed by an overturning failure or a sliding failure. In the model test, the same model of the buttress wall was used for both the overturning and sliding failure tests. The buttress wall was placed back in the original position after the first experiment, which was the overturning test.

Numerical simulations were performed to recreate both tests. In the overturning test, the hydraulic jacks were initially loaded to produce the design value of the hydrostatic pressure. After this, the ice load was increased until an overturning failure was induced. A comparison between the displacements for the numerical simulation and the experiment is presented in Figure 6. The displacements agree well, however the displacement in the base is somewhat lower in the numerical simulation. In Figure 7, the load-displacement curve for the numerical simulation and the physical model test are presented. A second overturning test, using a separate model with identical geometry and material properties, was also tested in the lab. In the second experiment, the monolith was unloaded quickly after achieving the ultimate load. This is because the experimenters were primarily interested in the ultimate load, not the post failure behavior. The results show high correlation between the physical model tests and the numerical simulations.

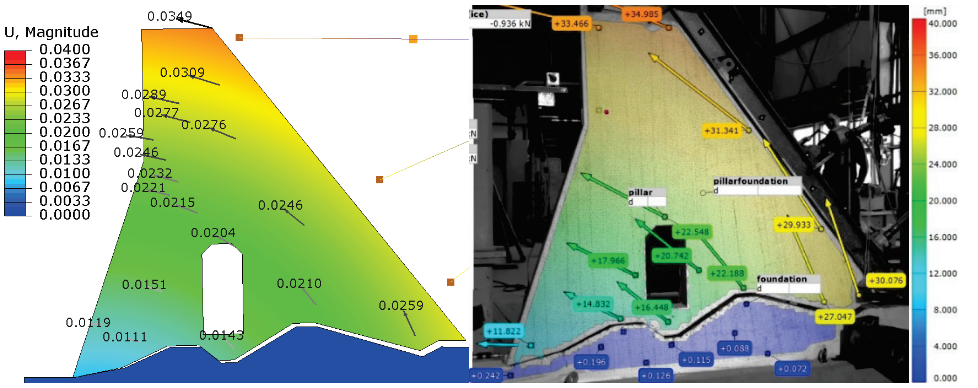


Figure 6. Comparison of the displacements between the numerical simulations and the physical model test for the overturning failure (Enzell et al., 2021b).

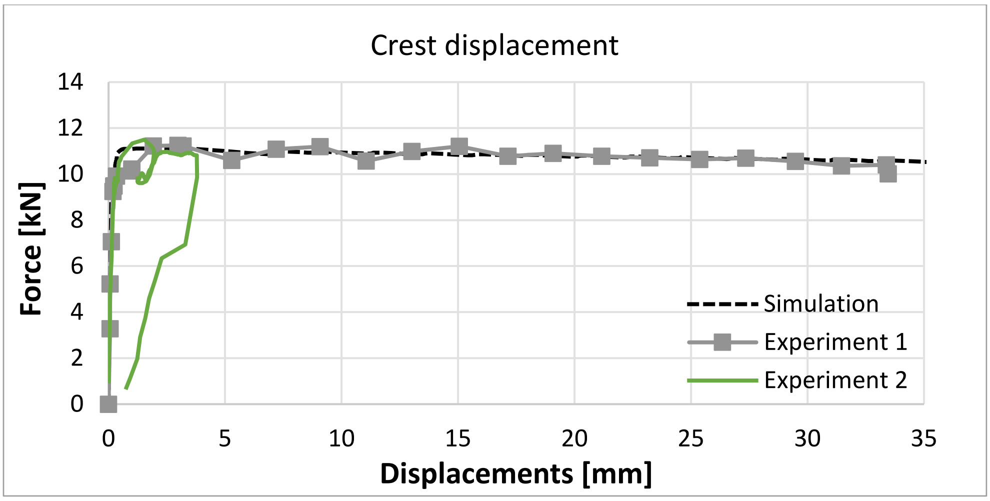


Figure 7. Failure curve for the numerical simulation and the physical model test for the overturning failure (Enzell et al., 2021b).

In the physical model test with sliding failure, the buttress was pushed to failure using only the hydraulic jacks representing the hydrostatic pressure. The buttress failed due to cracking in the dam body. The failure mode is illustrated in Figure 8a. In Figure 8b, the results from the numerical simulation of the sliding experiment is shown. The obtained crack pattern from the simulation shows good agreement with the crack pattern found in the experiment. In the numerical model, there is a zone below the inspection gallery, where a large amount of cracking occurs. This cracking is due to crushing of the concrete and the cracks run perpendicular to the compressive stress. No cracks can be seen in this area in the model test. Some damage could however have occurred which is not visible on the photogrammetry presented in Figure 8.

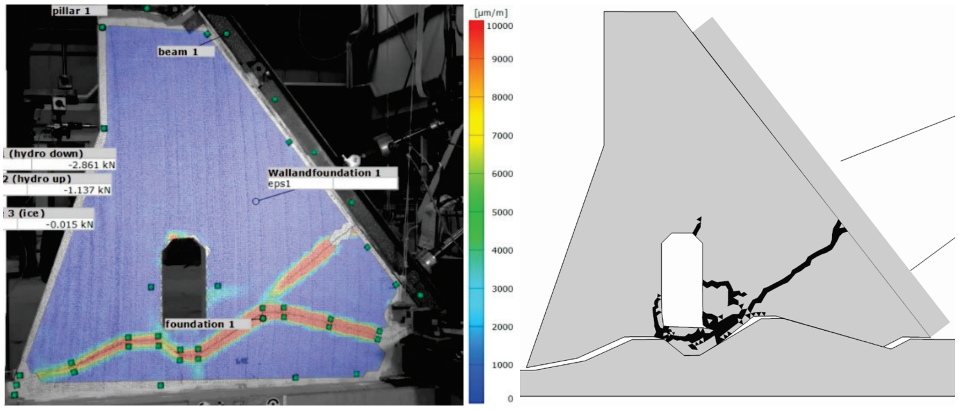


Figure 8. Failure mode of the sliding experiment a) strains from the physical model test b) Crack pattern from the numerical simulations.

In Figure 9, the failure curves for the physical model test and the numerical simulation are presented. In the physical model test, the buttress behaves nonlinearly during the loading, and a sudden failure occur at a total load of 38 kN. The numerical simulation has a slightly stiffer behavior during the loading phase and the failure occurs at a load of about 42 kN.

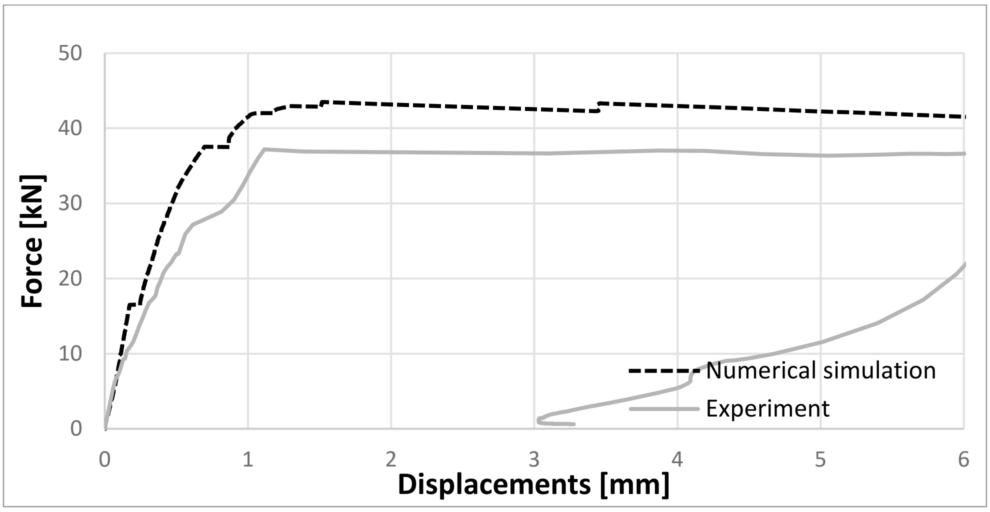


Figure 9. Failure curve for the numerical simulation and the physical model test for the sliding experiment (Enzell et al., 2021b).

4.2 METHODS FOR SIMULATING DISPLACEMENT-CONTROLLED FAILURES

In Section 2.2, the issues of using load-controlled simulations when simulating dam failures was presented. A method for performing displacement-controlled simulations on concrete dams have been developed during this project. The simulations presented in this section were first presented by Enzell et al. (2021b). The method uses spring elements to apply the hydrostatic pressure and push the dam downstream. The numerical model used to simulate the Stable Dams physical model test (Sas et al., 2021) was used for these simulations.

The simulations presented in this section are performed according to the concept of safety defined in the Swedish Hydropower Companies Guidelines for Dam Safety (Ridas, 2017). I.e. they are not intended to simulate the physical model tests presented in the previous section.

Three FE-models were created and are compared in the following:

- A load-controlled simulation
- A classic displacement-controlled simulation
- A displacement-controlled using nonlinear springs to apply the hydrostatic pressure

The models were created using the same geometry, material properties and mesh as the Stable Dams model test, see Section 3.1. The load-controlled and the displacement-controlled simulations were created using Abaqus ver. 2019 and the classical displacement-controlled simulation was created using Comsol Multiphysics 5.5. Comsol was used for the classical deformation-controlled analysis since it was easier to edit the constitutive equations of the model and defined an auxiliary control point for the deformation-controlled load procedure. In this analysis, the crest displacement was defined to continuously increase throughout the simulation. In order to define the same procedure in Abaqus, it would have required to develop suitable user-subroutines which was considered to be more cumbersome than using Comsol.

One difference between these simulations and the previously presented in Section 4.1 was that the beam along the upstream face and the spring elements representing the hydraulic jacks were removed. In the load-controlled and the classical displacement-controlled simulation, the hydrostatic and ice load were applied as pressure loads. The uplift was applied as a point load, see Figure 10.

4.2.1 Load definition for classical load- and displacement-controlled simulation

In the load-controlled simulation, a quasi-static analysis based on a dynamic implicit solver was used. First, the dam was loaded to its design load. During the failure step, the hydrostatic pressure and the ice load were increased until the failure occurred. The load was increased slowly in order to minimize the effect from the dynamic forces. In this analysis, the driving loads were doubled over a time period of 20 s. This was considered to be sufficiently slow to achieve a quasi-static solution up to the point where the failure occurred.

The classical displacement-controlled simulation was simulated using Comsol Multiphysics 5.5. The gravity load and the vertical component of the hydrostatic pressure was applied first. In the failure step, a displacement was defined in the control point in the crest from 0 mm to 100 mm with a 0.1 mm increment. The displacements and the horizontal component of the hydrostatic pressure, the ice load and the uplift were thereafter scaled to achieve equilibrium.

4.2.2 Load definition for displacement-controlled simulation using springs

In the displacement-controlled simulation using nonlinear springs, the springs were configured according to Figure 10b with boundary condition in their upstream end. Different spring stiffnesses were assigned to represent the distribution of the hydrostatic pressure with stiffnesses according to Table 2. To load the hydrostatic pressure, the boundary conditions connected to the springs were pushed toward the dam. The design load represented a displacement of 10 mm in the boundary conditions. In the step simulating the design load, the boundary conditions were therefore displaced 10 mm toward the dam. In the failure step, the springs were pushed an additional 100 mm toward the dam.

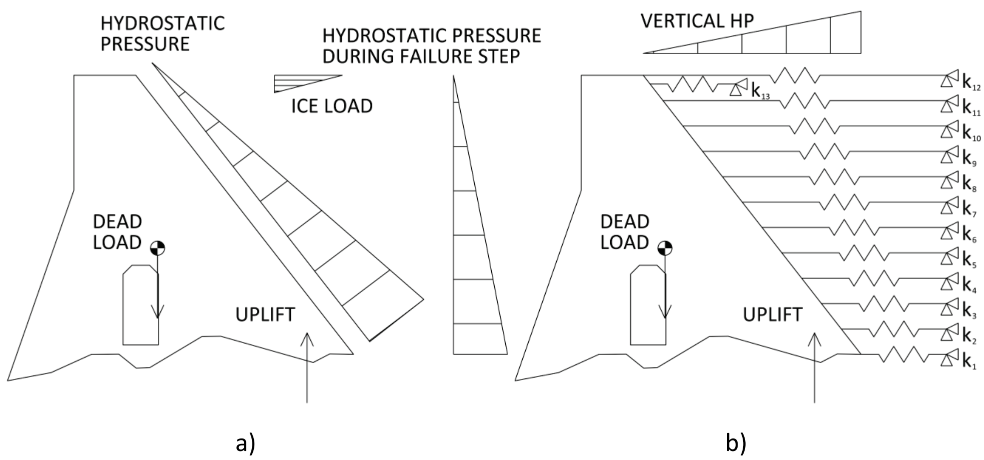


Figure 10. Loads for a) the load-controlled and the classical displacement-controlled simulation, and b) the displacement-controlled simulation using nonlinear springs (Enzell et al., 2021b).

Table 2. Spring stiffnesses and cut off values used for the displacement-controlled model (Enzell et al., 2021b).

Spring	k [kN/m]	Limit [N]	Spring	k [kN/m]	Limit [N]
k_1	70	-857	k_8	51	-623
k_2	127	-1558	k_9	38	-467
k_3	114	-1402	k_{10}	25	-312
k_4	101	-1246	k_{11}	13	-156
k_5	89	-1091	k_{12}	1	-10
k_6	76	-935	k_{13}	170	-2074
k_7	63	-779			

The failure was initiated around the load factor of $1 + \lambda = 1.22$ after which the dam was pushed downstream. The dam fails in a combined overturning and sliding failure-mode, where the crest of the dam moves more than the heel of the dam. This means that the spring closest to the heel is compressed more than the spring closest to the crest and the spring controlling the ice load. Therefore, when linear springs are used, the distribution of the hydrostatic pressure becomes nonlinear with significant pressure increase at the bottom. The springs were given a cut-off value, see Table 2, which represented the 122 % of the hydrostatic pressure at the specific level and 122 % of the ice load for the spring controlling the ice load. The cut-off level was decided because this is where the failure occur. The uplift pressure was also limited to 122 % of the design-value and the cut off time was matched to the time of failure so that the uplift increased at the same rate as the hydrostatic pressure.

With the nonlinear springs, the distribution of the hydrostatic pressure became linear up to the failure and during the post-failure analysis. The distribution of the hydrostatic pressure before and after the failure is presented in Figure 11. In the figure, the colored lines represent the distribution of the hydrostatic at different times during the progression of the failure. The ice pressure is represented with a square of the same color as the hydrostatic pressure of the relevant time. The simulation of the failure lasted for 10 s and at the start of the simulation ($t = 0\ s$), the design loads were already applied. The failure occurred after about $t = 0.7\ s$. The distribution of the hydrostatic and ice pressure is better with the non-linear springs. The total load was also higher in the simulations using linear springs. This is because the resultant is moved downwards toward the heel of the dam and thus preventing the overturning failure.

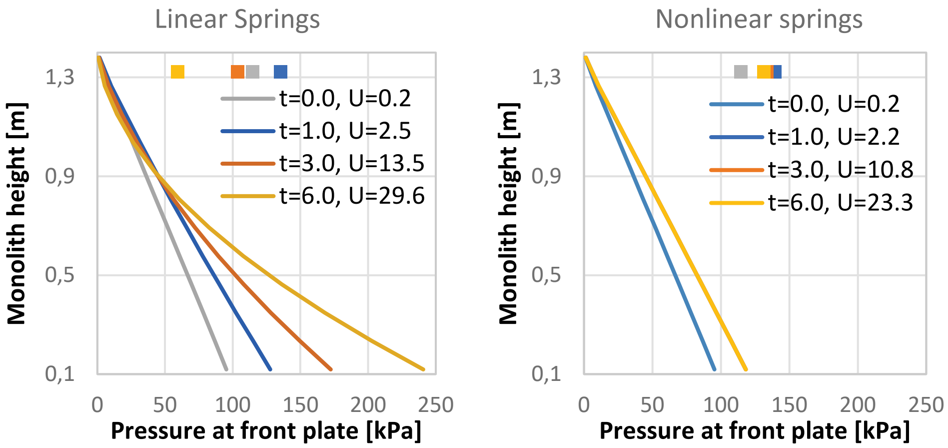


Figure 11. Distribution of hydrostatic and ice pressure during the displacement-controlled failure analysis. The pressure from the ice load has been plotted as a part of the same graph as the hydrostatic pressure. In the legend, t is the analysis time in s and U is the crest displacement in mm.

4.2.3 Results

In Figure 12, the failure curves for the three simulations are presented. In all three simulations, the failure is a combined overturning and sliding failure. The dam starts by overturning and the contact is lost at the upstream heel of the dam. When the contact is lost, the footprint of the dam is reduced, and it starts sliding. No cracking occurs in any of the simulations. The three simulations give similar results up to the point of failure.

As the failure starts to progress, the load-controlled analysis gives increasing displacements for an almost constant load and it not easy to assess if the behavior after the failure is realistic or not. For a sliding failure, this type of behavior is expected, while if material failure occurs then unloading has to occur. The obtained combined failure mode is expected to have similar ductile behavior as a sliding failure considering that the dams global deformation is governed by sliding after the failure load has been reached. The contact between the concrete and rock at the points of irregularities are however expected to cause deviations from the horizontal load and deformation curve.

The two displacement-controlled simulations gives similar results. The classical displacement-controlled simulation does however have convergence issues and stops at a crest displacement of about 18 mm. This is likely because it is difficult to find a static equilibrium when the dam starts sliding. The displacement-controlled simulation using nonlinear springs is considerably more stable and, in this example, the dam is displaced 110 mm without convergence issues.

In conclusion, the load-controlled simulation can be used to find the failure load of concrete dams. The classical displacement-controlled simulation can be used to find the ultimate load of the structure and to determine if the failure is ductile of brittle. The displacement-controlled simulation using nonlinear springs is easier to use when determining the post-failure behavior and experiences less convergence issues. The definition of the springs does however require more work than the classical displacement-controlled simulations. This is especially true if the failure of more complex geometries such as an arch dam is simulated.

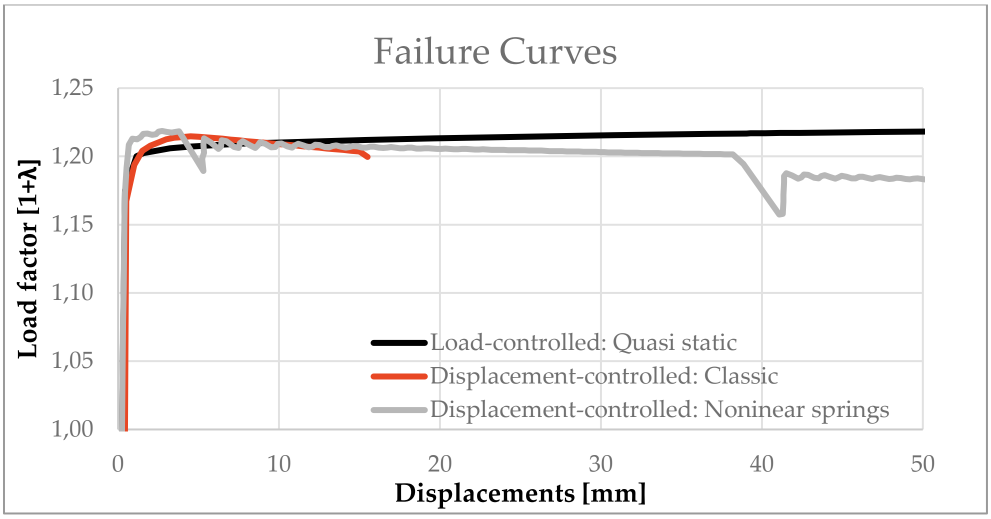


Figure 12. Failure curves for the simulations using load-controlled vs displacement-controlled procedure are presented. (Enzell et al., 2021b).

4.3 FAILURE SIMULATION OF A CRACKED DAM

Failure simulations have been performed for a reinforced concrete arch dam with pre-existing cracks, the case was presented in Section 3.2. This case was studied to show that it is possible to perform failure simulations on cracked concrete structures using the methods described in this report. The simulations were first presented by Enzell et al. (2021a).

The cracking caused by the seasonal temperature variations were calculated for the complete temperature history of the entire life span of the dam. The temperature gradient was calculated in a separate simulation based on transient temperature propagation and was introduced in the mechanical model with a one-way coupling. The calculation of the crack pattern and the seasonal displacements in the dam is presented in more detail in Enzell et al. (2021a). The crack pattern, previously shown in Figure 4, was considered initially on the dam after which the loads were increased until failure occurred.

The failure was introduced using a load-controlled procedure, according to Section 2.1. The design load was first introduced, the destabilizing loads were thereafter increased by a factor λ. The final safety factor is calculated as $1 + \lambda$ for the failure load. In this case, only the hydrostatic pressure was increased since the ice load has a small effect of the behavior of such a large dam. The uplift was also neglected because the dam is slender, and the uplift therefore has a small effect on the stability. The numerical model is further described in Enzell et al. (2021a).

The failure was simulated for three cases:

- Uncracked dam without consideration of temperature effects
- Cracked dam with $T_a = 0\ °C$
- Cracked dam with $T_a = -20\ °C$

where T_a is the ambient temperature. The first case represents the status of the dam at the first impounding, i.e. before the cracking from the seasonal temperature variations had occurred. This is used as a reference for the influence on the dam safety from the cracks. In the second and third case, the crack pattern (seen in Figure 4) is fully developed, and failure the simulation is performed for two different ambient temperatures.

The failure curves are presented in Figure 14. All three cases resulted in similar behavior and failure mode, where the dam behaved linearly up to a load factor of $1 + \lambda = 2$, i.e. twice the normal loads. After this load level, non-linear behavior occurred due to joint opening and cracking which resulted in a ductile behavior. Vertical cracks appeared along the crest of the dam and after this, the dam failed suddenly as tensile stresses occurred in the crest. The development of the crack pattern is illustrated in Figure 14. The uncracked dam fails at a higher load level and the ultimate failure occur around $1 + \lambda = 3.0$. In the case with a cracked dam and an ambient temperature of $T_a = 0\ °C$, the dam has a slightly lower strength and fails around $1 + \lambda = 2.9$. In the winter case the dam fails around $1 + \lambda = 2.6$. In all cases, the arch dams could withstand load levels of at least $1 + \lambda = 2.5$. Large deformations of 100-200 mm do however occur at these load levels. Deformations of this size could lead to uncontrolled release of water.

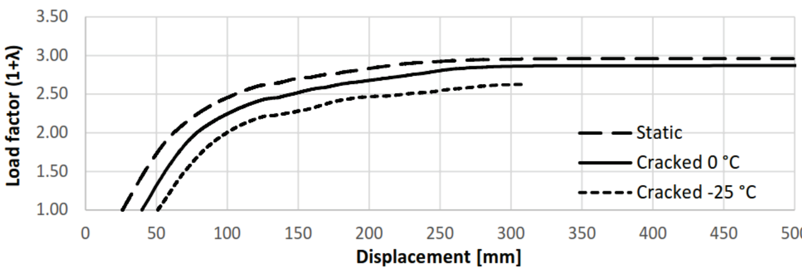


Figure 13. Load-displacement failure curves for the three simulated failure scenarios (Enzell et al., 2021a).

The results showed that the pre-existing cracks does not have a large effect on the dam safety for the studied arch dam. In this case, the difference between the uncracked and the pre-cracked case is primarily that the cracked has larger initial displacements, due to weaker behavior caused by the crack openings. The difference in safety factor between the uncracked and cracked analyses is in this case less than 5 %. This is probably because the arch is still intact, and the dam can redistribute compressive stresses to the abutments in an effective way. It should however be noted that this analysis can only capture failure modes in the concrete body. It is likely that failures in the foundation of the dam or the abutments can occur before the concrete arch fails, especially if there are weaknesses in the rock mass such as rock wedges at the abutments, rock with poor quality in some parts of the foundation etc.

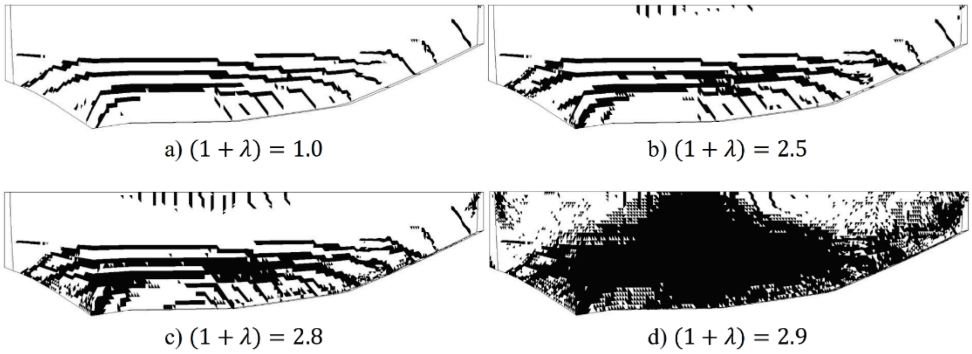


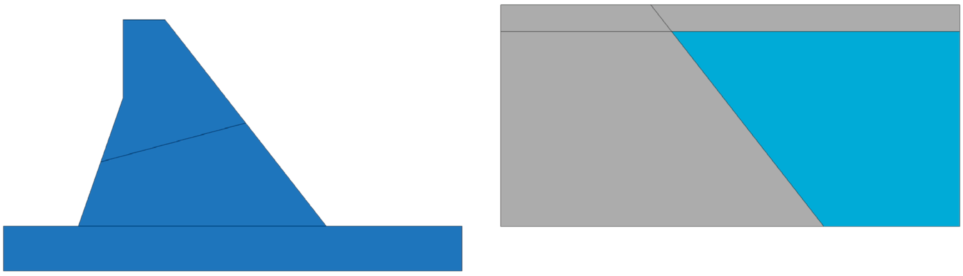
Figure 14. Development of the cracks in the downstream face of the dam during the failure simulation (Enzell et al., 2021a).

4.4 SIMULATIONS INCLUDING THE DYNAMIC EFFECT FROM WATER

The failure of a concrete dam is a highly dynamic process. As explained in Section 2.2, load-controlled simulations can be used to calculate the ultimate load of a concrete dam. Displacement-controlled simulations can be used to calculate the ultimate load and in addition determine if the failure ductile or not. However, to simulate the actual failure of a concrete dam, determine the effects on adjacent monoliths and the rock etc. the reservoir water also must be simulated.

To simulate the structure, the water and the interaction between them, a method for fluid-structure interaction is required, see e.g. Gasch et al. (2013) for a comparison of methods. In this report, simulations have been performed to investigate the capabilities of using the FSI-method Coupled Euler-Lagrange (CEL). CEL incorporates a combination of two kinds of elements, Lagrange-elements which are the usual elements used in finite element analysis, and Euler-elements which are common in CFD-simulations (computational fluid dynamics). Euler-elements consists of a predefined mesh, where the material is free to move within the mesh as mass densities. This allows for large deformations and is suitable for simulating water (Gasch et al., 2013).

A generic concrete gravity dam, 7 m high with a predefined inclined crack, was used for this simulation, see Figure 15. This model should be considered as an academic example and is only intended to show the capabilities of fully coupled fluid-structure-interaction analyses for dam safety analyses.



a) b)

Figure 15. a) Geometry of the dam and foundation. b) Geometry of the Euler mesh used for simulation the water in the CEL-simulations. The horizontal line is the surface of the reservoir and the inclined line coincides with the upstream face of the dam. The initial reservoir is marked with blue color. The volume of the Euler-mesh not filled with water is "empty" i.e. represents air. When the dam starts moving downstream, the water will start moving with the dam and fill some of the empty space of the Euler-mesh.

The model was created using 3D-elements in the software Abaqus ver. 2019 using an explicit dynamic solver. The model had to be created in 3D because the software can only define 3D-Euler elements. The model was however made with a thickness that was only one 3D-element thick and all motion out-of-plane was restricted, that way a plane strain simulation was achieved. The mesh is presented in Figure 16. As can be seen in the figure, the Euler-mesh used for the water overlaps the Lagrange-mesh used for the solids, so that the water can flow past the dam when the monolith starts sliding. In total, 823 cube-elements, denoted C3D8R, were used to simulate the solid parts, i.e. concrete and rock. About 14 500 elements were used to simulate the water, the Euler elements are denoted EC3D8R. The Euler elements were given a size of 50 mm and the Lagrange elements a size of 300 mm.

The foundation and the dam were created using a linear elastic material model with typical concrete material properties: $E = 33$ GPa, $\nu = 0.2$ and $\rho = 2300\ \mathrm{kg/m^3}$. The water was given a density $\rho = 1000\ kg/m^3$ and a dynamic viscosity of $\mu = 0.0013$ Pa/s. The compressibility of the water is calculated from the speed of sound, which is $c_0 = 1500$ m/s for water.

A tie connection was created between the dam and the foundation, which means that no relative displacement may occur. An interaction was created along the predefined crack with Coulomb friction using a friction coefficient of $\mu = 0.3$ in the tangential direction. In the normal direction, compressive forces were allowed but not tensile forces i.e. the surfaces could separate. Boundary conditions restricting translations in all directions were placed along the bottom of the foundation. A gravity load was applied for both the dam and the water. The hydrostatic pressure on the dam was calculated automatically from the gravity load applied on the water.

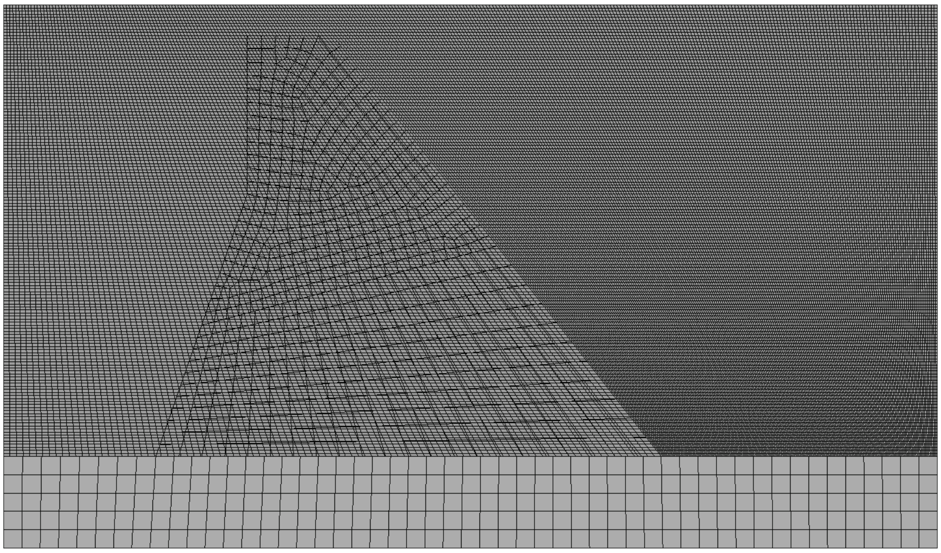


Figure 16. Mesh used for the CEL-simulations.

The results from the simulation are presented in Figure 17. The top of the dam above the crack starts sliding, as soon as the gravity load is applied for the dam and the water. The top part of the dam slides away from the lower section which results in an outflow of water. The water appears to flow in a realistic way.

In this simulation, the failure is induced using the reduced-strength method, see Section 2.1, i.e. the friction coefficient in the crack was defined low enough allow the sliding failure along the crack to occur. The failure could also be induced by having a higher coefficient of friction and increasing the water pressure. The simulation should be viewed as a proof of concept and gives promising results for future failure simulations of concrete dams. The method is expected to have a good ability to predict the behavior of the physical model tests described in Section 5.

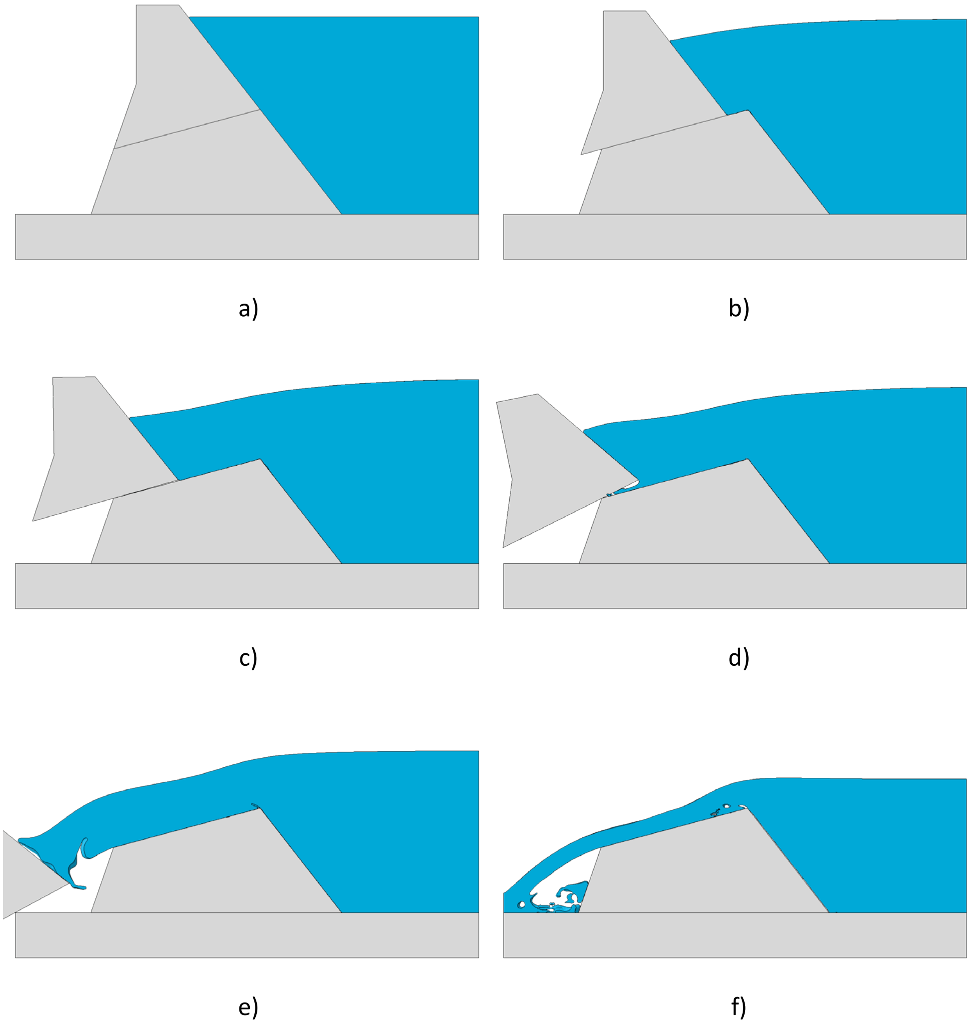


Figure 17. Results from the CEL-simulations at the time: a) $t = 0\ s$, b) $t = 1\ s$, c) $t = 2\ s$, d) $t = 2.4\ s$, e) $t = 3\ s$ and f) $t = 5\ s$.

5 Concept for physical model test

When performing an investigation of a physical structure or phenomenon, the aim of the investigation will determine the kind of model and level of detail required for the study. For an example, when the ultimate strength of a concrete dam is to be determined, it is sufficient to simulate the load from the hydrostatic pressure by an arbitrary load of the same magnitude and distribution. In physical model tests, this can be done by using hydraulic jacks and in FE-modeling by introducing a pressure load.

When the failure process, including the post failure behavior of a concrete gravity dam is to be simulated, including effects on adjacent monoliths, the dynamics of the water must be included in the model. By the knowledge of the authors, this have not been done in physical model tests before. A concept for a physical model test has therefore been developed and is described in this section.

5.1 PLANNED TESTS

A concept for physical model tests of dam failures using water to push the dam to failure have been developed. The goal of the test is to:

- Document the breach development of a buttress dam during failure and study the risk for progressive failure on adjacent monoliths.
- Investigate the time aspect during a failure
- Document a well-designed experiment so the results can be used for validation and calibration of numerical models which can be used for further simulations.

A physical model of a concrete buttress dam consisting of several monoliths will be built. Five monoliths are assumed at this stage. An overload method will then be used, where the dam is pushed to failure by pressurizing a water filled reservoir. Using water to push a dam to failure rather than hydraulic jacks is of obvious benefit because the post failure behavior, dynamic effects and the effect from the water can be examined.

5.2 GEOMETRY AND MATERIALS

For the concept of the physical model tests, a relatively low concrete buttress dam in northern Sweden has been selected. The dam is depicted in Figure 18. It was selected because it is the most common type of concrete dam in the country and the geometry is representative for many Swedish dams. The selected monolith is 11 m high and has an inclined front plate. The front plate is 8 m wide and 1.2 m thick and the buttress is 2 m thick. The crest is rounded to allow for better flow if the dam is overtopped. The monoliths have surface reinforcement and the buttress and the front plate are joined with reinforcement. The front plates are dilated between the monoliths to allow for expansion. The reference dam is 600 m long and consists of about 70 monoliths of varying height.

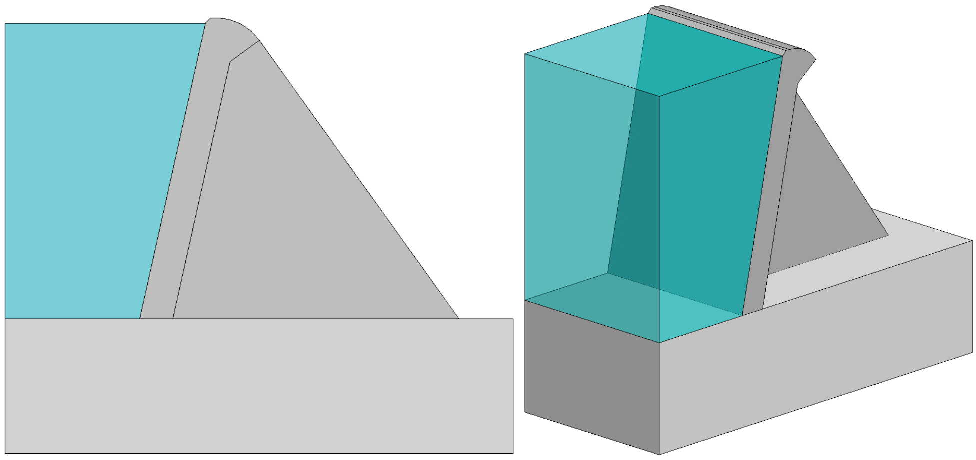


Figure 18. An illustration of the selected dam monolith for the planned physical model test.

The concept proposes creating a physical model including five monoliths with a model scale of 1:20. The geometry is somewhat simplified to make the casting of the monolith easier, see Figure 19. The model monoliths are 550 mm high. The monoliths are casted in place on a predefined rock surface. The monoliths will be created from concrete with the strength reduced to the model scale. The rock will be created from normal concrete. Artificial fracture planes can be introduced in one or several of the monoliths of the concrete dam. Fracture planes can also be introduced between the dam and the foundation or in the foundation.

The monoliths will be placed in a chute and block the entire width. Using the proposed geometry, the chute must be at least 2000 mm wide to fit 5 monoliths and about 1000 mm high to fit the monoliths and the foundation.

The concrete mix will have to be developed and tested before the model tests are performed. The weight of the concrete can be scaled to ensure that the failure occur at a water pressure which is possible to obtain. Scaling of reinforcement is also a challenge in the project. If a good alternative is not fond, the reinforced might be omitted from the experiment.

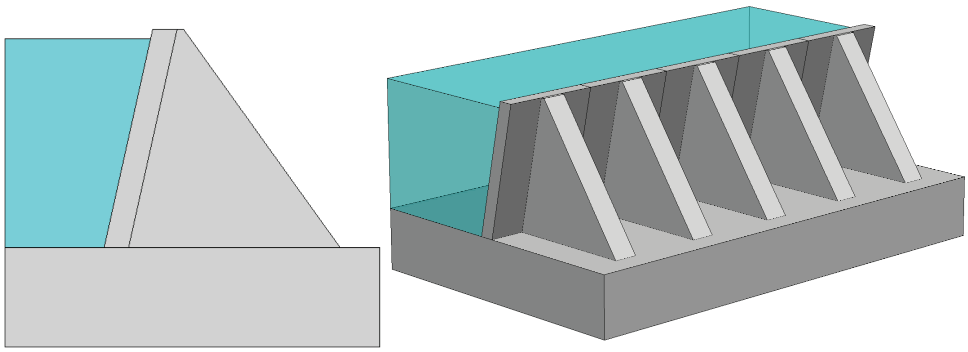


Figure 19. Geometry of the proposed model.

5.3 MEASUREMENTS

Measurements will be performed during the experiment. Displacements and strain will be measured using photogrammetry and video. To validate the video results, strain and displacements will also be measured using strain gauges mounted on the dam and between monoliths and LVDT-sensors mounted on frames downstream of the dam. Another alternative could be to install watertight LVDT-sensors on a frame upstream from the model dam and measure the displacement of the front plate. This way, the risk for damaging sensors during a failure is reduced. Water pressure, volume of water pumped to the reservoir and the velocity of the water will also be monitored.

5.4 PRACTICAL DETAILS

There are several practical issues that the project team must overcome in order to carry out the physical model tests. Initial pilot must be performed in order to resolve some practical issues before the real tests commence.

One of the main difficulties of developing the model test is to design the upstream reservoir and how to pressurize the dam to failure. One could of course design the dam to fail as the water level is increased close to the crest. It is expected to be some scatter in the results, where some tests may fail early, and others may be able to sustain water levels up to the crest. Hence, this was considered risky, since the margin of error in the design may result in no failure if the strength of the dam is too high. Therefore, it was decided to design the model dams to fail at an overtopping of 50-100 %. This requires a pressurizing system and that a lid in mounted over the reservoir. The pump capacity also has to be designed to be able to pump water with the same rate as may occur during dam break in order to simulate a large reservoir. The water will be in a closed system where the water that has passed the dam is sent back to the upstream reservoir. The downstream part of the chute must also be designed to be able to carry the flowing water that may come as a result of a dam failure in the test.

If the dam failure is not sudden, leakage might occur after the initial failure of the monoliths. This can lead to a loss of pressure. An arm connected to a hinge or a similar system that moves with the dam might be required to ensure the pressure. This will be decided during the pilot tests.

Another important aspect is to ensure that the buttress dam consisting of several monoliths are leak tight until a failure is initiated. Three areas have been identified where additional efforts are required to ensure leak tightness of the dam;

- Interface between the first and last monolith and their connection to the chute.
- Interface between the front-plates between the monoliths
- Interface between the monoliths and the foundation

The joints can for an example be sealed using a product such as Synkoflex or a rubber seal. The important aspect here is to design the seal so that it does not contribute with additional strength or stiffness to the dam which may influence the failure modes.

If the practical issues of building and sealing the reservoir are not possible to overcome to a reasonable cost and during a reasonable time span, the model test can be performed as a reduced strength-test. This can be done by introducing fracture planes, reducing friction in joints or cracks or by designing the dam to fail when the reservoir is full.

Preliminarily, ten physical model tests are planned to be performed. Five tests using intact monoliths and five tests including weak planes in the central monolith. If it is possible, additional tests could be performed including e.g. weak planes in the monoliths or the rock. In Figure 20, sample numerical simulations for the model tests are presented.

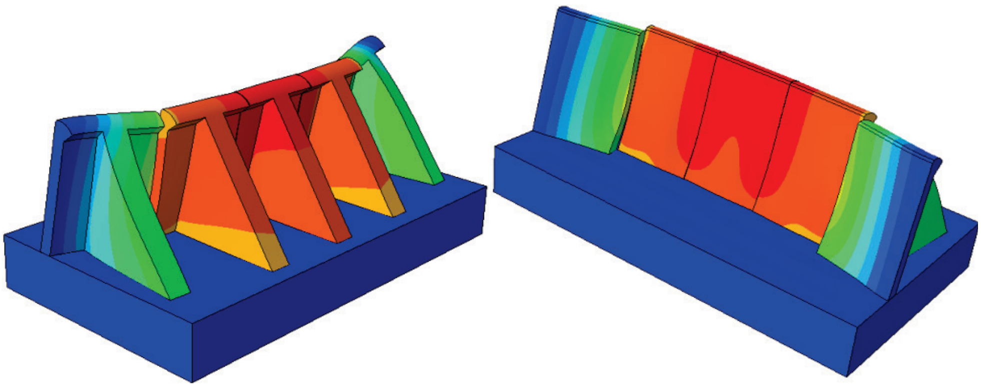


Figure 20. Sample simulations using five monoliths.

6 Discussion and conclusions

When performing stability assessments, the concept of safety is important. In design, the concept of safety must be consistent with the local codes in order to give a coherent assessment of the dam safety. This means that the material properties, loads and that the calculated failure load capacity is obtained in a manner that follows the current code or guideline. It is also very important for the dam owners to comply with the local regulations.

When simulating cracked and damaged structures it is especially important to ensure that the correct concept of safety is obtained, since the classical failure modes used in analytical calculations does not apply. It should be noted that design codes or guidelines typically does not distinguish between assessment of existing structures and new-built structures. This means that the same safety factors should be applied to an old damaged structure and a new-built dam.

Cracks and other types of damage may have a significant influence of the behavior of the dam and on its safety and potential failure modes. New cracks can result in internal failure modes, where part of the dam goes to failure and thereby results in an uncontrolled release of water. In a MSc project performed in connection to this research project, the influence of cracks on concrete buttress dams has been investigated, see Fekadu and Kayastha (2020). As shown in their report, in some cases the cracked dam may have similar safety regarding to failure as the initially intact dam, which also was the case for the arch dam in Section 4.3. In this case, the pre-existing cracks on the downstream side of the arch dam was closed during failure which resulted in small influence on the global safety. In other cases, they showed a significant decrease in safety due to these internal failure modes. It is difficult to assess the mode of action of cracked dams with simplified analytical calculations due to interaction between the different parts of the dam at these cracks. This is especially the case if the dam is reinforced. It should also be noted that degradation such as reinforcement corrosion may change the failure mode and factor of safety as the strength of the reinforcement decreases with time.

When the aim is to replicate an actual failure or a physical model test with numerical analyses, the design codes does not apply, since the concept of safety defined in the codes are not relevant. It is therefore better to simulate the actual conditions as was shown by e.g. Enzell et al. (2021b), see Section 4.1. Thereby, the way that the structure is loaded to failure will influence the ultimate load capacity. It is quite common that a load capacity obtained from a physical model test cannot be directly related to the concept of safety defined in the design code. Most codes build on global safety factors for stability analyses, this means that either all overturning force components should increase at the same rate (overload approach) or that all restraining forces should decrease with the same rate (reduced strength approach). This is difficult to obtain in physical model tests. The difficulty was for instance illustrated in the physical model tests performed within the Stable Dams project (Sas et al., 2021). In the described project, the loading procedure diverged from the optimal loading procedure in a number of ways:

- When the buttress started to deform, hydraulic jacks were inclined which results in a change in inclination of the force. If the deformation is not very small, the change in inclination may have significant influence of the maximum load capacity.
- The uplift pressure was constant and did not increase at the same rate as the hydrostatic force.
- The ice load did not increase at the same rate as the hydrostatic pressure.
- When using hydraulic jacks for a dam with inclined upstream surface, the vertical load component of the water pressure will be increased, which is not desirable and will result in an overestimation of the safety, see Fu and Hafliðason (2015).

Thereby, it should be noted that the results obtained from model tests cannot be translated into a global safety factor as defined in the design guidelines. Physical model tests are, however, good for investigating the influence of different aspects and to develop results that can be used to validate numerical calculation procedures. With the validated numerical model, it can be updated to apply the loads in a manner that is coherent with the specifications of the design codes.

In Section 4.2 and Section 4.3, it was shown that the ultimate load of a dam can be assessed in a good way by using load-controlled simulations. However, the post-failure behavior cannot be simulated using this method. This is because after the failure, the dam will continue to accelerate downstream with continuously increasing displacements for a constant load. Displacement-controlled simulations can be used to some extent to assess the post-failure behavior as was shown in Section 4.2. To capture the actual failure, however, the dynamic effects from the reservoir water must be considered. This can be done by including an FSI-model, such as CEL (Coupled Euler Lagrangian approach), as was shown in Section 4.4.

The failure process of concrete gravity dams is to a large extent unknown regarding the development of the breach and the time scale of the event. If the failure process was better understood, this could be used to improve the assumptions used for flooding simulations and design of instrumentation on concrete dams. To this end, a concept for performing physical model tests on concrete buttress dams has been developed and is presented in Section 5.

7 Further research

In this report, various methods for numerical simulations of dam failures have been tested and compared. A concept for performing physical model tests using water to push a concrete buttress dam to failure have also been developed. The methods for simulating the ultimate load according to the concept of safety defined in the codes using numerical simulations works satisfactorily and are being used by engineers today. Determining the post failure behavior of dams is however considerably more difficult, since it involves dynamics and flowing water. The post failure behavior is important for understanding the process of the failure and potential breach size. More research is required in this field.

It is therefore important that physical model tests are performed where water is used to push the dam to failure to study the failure modes and especially how adjacent monoliths are influenced if one monolith fails. The developed concept for physical model tests is considered feasible. However, it should be noted that many issues remain, particularly practical problems regarding the experimental setup. The test must be designed in detail and simulations performed to verify the concept before the experiments can be performed.

The process of loading models to achieve failure should also be researched further in order to achieve results, which corresponds better with reality. This can potentially lead to more realistic concepts of safety than those used today.

Keywords

Dam safety, physical model tests, numerical simulations, fluid structure interaction

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METHODS FOR ASSESSING THE FAILURE PROCESS OF CONCRETE DAMS FOUNDED ON ROCK

Kunskapen om betongdammars brottförlopp är begränsad. Idag används mycket förenklade antaganden för faktorer som bräschens storlek och brottförloppets tid, trots att de här faktorerna har stor inverkan på simuleringar av översvämningar och beredskapsplaneringen.

Det finns bra metoder för att avgöra betongdammars brottlast men inte för att simulera förloppet efter att dammen gått till brott. Den här rapporten fokuserar därför på metodologi för att simulera dammbrott.

Brottsimuleringar har genomförts för en svensk armerad valvdamm som är sprucken på nedströmssidan. Sprickorna har orsakats av de temperaturvariationer som uppstår i dammens omgivning mellan olika säsonger.

Målet har bland annat varit att undersöka brottförloppet inklusive vattnets effekt på de intilliggande monoliterna efter den första monoliten går till brott och tiden det tar för brottet att utvecklas. Resultaten ökar förståelsen för dammbrott och kan fungera som referens vid framtida numeriska simuleringar.

The Swedish Hydropower Centre SVC, founded in 2005, is a centre of expertise formed by the Swedish Energy Agency, Energiforsk and Svenska Kraftnät together with KTH, Chalmers University of Technology, Uppsala University and Luleå University of Technology. Luleå is also host university for the centre developing new knowledge to contribute to a renewable energy system.