Realistic numerical simulations of concrete dam failures

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ABSTRACT: Dam failures may have catastrophic consequences, including the release of large amounts of water, significant property damage, and loss of life. However, safety assessments of concrete gravity and buttress dams often rely on simplified methods that do not consider the interaction between monoliths, the shape of the foundation or the presence of stiff abutments. Numerical modeling can be a valuable tool for analyzing the stability of these dams, but it can be difficult to validate these models due to a lack of documented dam failures. This paper presents the results of a numerical study examining the ability of dynamic finite element analyses to simulate dam failures. The study used the results from a series of physical model tests as a case study for validation. It was found that the numerical model was able to accurately reproduce the failure mode and breach development observed in the physical model tests and capture the effect of the loading rate on the failure mode and time for the failure to develop. Simulations were also performed in prototype scale to verify that the model tests were representative of a real dam failure. Further research is needed to determine the reliability of the numerical models under different loading conditions and in realistic geological settings. However, these findings suggest that numerical modeling can be a valuable tool for analyzing the stability of concrete gravity and buttress dams and predicting the development of failures.

1 INTRODUCTION

Dam failures may have catastrophic consequences, including the release of large amounts of water, significant property damage, and loss of life. Knowledge about the failure process is important for emergency preparedness and the dam's safety evaluation. An increased understanding of failure events in concrete dams also allows for establishing better alarm limits for the instrumentation installed in many concrete dams. Safety assessments of concrete gravity and buttress dams often rely on simplified analytical stability analysis, considering sliding and overturning failure modes of a simplified geometry (USACE 1995, FERC 2016, Ridas 2017). However, the failure of a concrete dam is a complex process, impacted by irregular geometries, nonlinear material behavior, dynamic effects from water release, and rock and soil erosion. In the stability analyses, single monoliths are considered, and potential interaction between monoliths is not considered. The foundation is commonly also simplified to a flat surface. The development of a breach is typically based on simplified assumptions for concrete dams (ICOLD 1998). Contrarily, methods have been developed for embankment dams to calculate the size of a breach (ICOLD 1998, Zagonjolli 2007).

Models can be used to evaluate concrete gravity dams' stability and failure process, particularly for complex geometries or when nonlinear behavior is expected. Before the introduction of commercial finite element software, physical model tests were the standard tool for analyzing complex structures within civil and structural engineering (Fumagalli 1973). In physical model tests, a scale model is created by scaling a prototype, e.g., a dam, to a manageable scale for the specific structure. Similitude requirements must be met for the scale to retain a valid representation of the actual structure. These are based on Buckingham's theorem (Buckingham 1914) and are presented by, e.g., Fumagalli (1973) and Harris and Sabnis (1999). However, producing accurate physical models is difficult, especially when material properties require scaling.

As an alternative to physical models, numerical analysis can be used to simulate the behavior of the structure. In such analysis, a numerical model of the full-size structure is used, removing the issues associated with the similitude requirement. Numerical analysis can be used to evaluate the stability of concrete gravity dams, particularly for complex geometries or when nonlinear material behavior is expected (Mirzabozorg et al. 2014, Hellgren and Malm 2017, Hellgren et al. 2020). However, it can be challenging to validate numerical models, especially in the case of dam failures where no sufficiently documented cases exist. An approach to overcome this problem is to use physical model tests and experiments to validate the numerical models. The validated numerical model can then be used for further studies. This approach has been used to validate numerical analyses of concrete dams (Oliveira and Fariab 2006, Hofstetter and Valentini 2013, Enzell et al. 2021).

In this study, physical model tests performed by Enzell et al. (2023) are utilized to examine the ability of dynamic finite element simulations to simulate dam failure. The physical model consists of five scale-model monoliths loaded to failure using hydrostatic pressure. The physical model test is replicated using a finite element model consisting of the five model monoliths, the foundation, and two side supports. All simulations were load-controlled and performed using nonlinear geometry. The model tests were also simulated at full scale to validate whether the results from the physical model test were representative of an actual dam.

2 CASE STUDY

2.1 *Physical model tests*

The results from the physical model tests performed by Enzell et al. (2023) were used to validate the numerical simulations performed in this study. In the model tests, a concrete buttress dam was loaded to failure using hydrostatic pressure. The dam failed in a sliding mode, and the water was allowed to act on the dam throughout the failure, see Figure 1. The model was created in the scale 1:15 and consisted of five monoliths. It was 1.2 m high and 4.0 m wide. The foundation of the dam was a flat concrete slab. Since a sliding failure mode is close to a rigid body motion and no cracking or crushing was expected, the model was created from regular strength concrete, i.e., the properties material were not scaled.

a) b)

Figure 1. Presentation of the physical model a) setup of the physical model and b) an example of a dam failure (Enzell et al. 2023).

The model was built with modular shear keys and side supports. The side supports added a stiff boundary condition, which controlled the failure mode. The test series used in this study are presented in Table 1. The failure load is given in mm of hydraulic head.

The foundation was built with a flat concrete slab for a foundation. The tests did not consider cohesion in the foundation-dam interface. The friction angle between the monoliths and the foundation and the monoliths and the side supports was 29.2°, representing a friction coefficient of 0.56.

Table 1. Test series in this study.

Test series	Acronym	Number of tests	Avg. failure load [mm]	Std. Dev. [mm]
Failure of a single Monolith	SM	3	1083	81
Full dam, Side Supports, no shear keys	SP	6	1309	94
Full dam, Side Supports and shear keys	SP+SK	6	1586	56

2.2 *Finite Element Models*

The physical model tests presented in Table 1 were first simulated in the model scale. Thereafter, the dam failure was simulated in the prototype scale, and the results were compared between the two scales. A nonlinear material model was added for the concrete to analyze the impact of the material behavior on the failure mode in the prototype scale. The nonlinear material behavior turned out to have a significant impact on the results in the case SP+SK. Reinforcement was therefore added to the monoliths in a separate analysis.

The finite element model consisted of the five model monoliths, the foundation slab, the two side supports, and the stop bar. The geometry and mesh of the model are shown in Figure 2. The foundation slab was defined as a 1 mm thick shell fixed in all degrees of freedom (DOF). Abaqus 2019 was used for all simulations. Linear hexahedral elements, denoted C3D8R, were used for the monoliths, side supports and stop bar. The foundation and the reinforcement in the nonlinear simulations were defined using 4-node shell elements, denoted S4R in Abaqus. The number of elements and nodes is presented in Table 2. The shear keys are small, so these models require more elements.

Table 2. Summary of mesh for the presented numerical models.

Model	Scale	Elements	Nodes
Single monolith	Model scale	4210	5352
Single monolith	Prototype scale	5380	6478
Full dam, supports	Model scale	12,532	17,654
Full dam, supports	Prototype scale	14,720	20,240
Full dam, SP+SK	Model scale	15,297	21,568
Full dam, SP+SK	Prototype scale	17,400	23,600
Full dam, SP+SK, reinforced	Prototype scale	59,026	70,982

The material properties are presented in Table 3. The concrete density was modified in the numerical model so that the total weight matched the monolith's measured weight in the physical model tests. In the nonlinear simulations, all parts except the dam and the reinforcement were linear elastic. For the nonlinear behavior of the concrete, a coupled damage and plasticity material model (Lubliner et al. 1989, Lee and Fenves 1998) was used, denoted Concrete Damaged Plasticity in Abaqus. The nonlinear compressive behavior was defined according to the uniaxial curve given in Eurocode 2 (2013). The exponential crack opening curve defined by Reinhardt et al. (1986) was used for the tensile behavior. A dilation angle of 35° was used, and the remaining parameters in the material model were assigned default values for concrete according to Dassault Systèmes (2014). See Malm (2016) for further information.

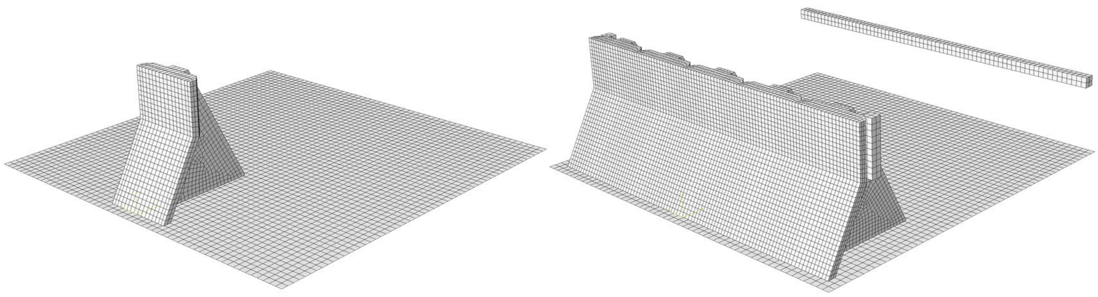
The reinforcement for the dam was not known. Therefore, a reinforcement grid of 20 mm bars at a 200 mm distance was assumed, which is a typical reinforcement amount in buttress dams. All surfaces were reinforced except for the joints, the top and the foundation surface. The nonlinear behavior for the reinforcement was defined according to Eurocode 2 (2013). The failure strain was 4.5 %, and the quote between the ultimate stress and the yield stress was $f_u/f_y = 1.08$. The built-in function Rebar in Abaqus was used to define the correct direction and stiffness of the reinforcement.

Table 3. Material properties for numerical simulations.

		Concrete	Steel
Young's modulus [GPa]	E	33	210
Poisson's ratio	ν	0.2	0.3
Density [kg/m^3]	ρ	2432	7750
Tensile strength [MPa]	f_t	2.9	500
Compressive strength [MPa]	f_c	38	500
Strain at f_c [‰]	ε_{c1}	2.5	
Fracture energy [Nm/m^2]	G_f	200	

The side supports, foundation slab, and stop bar were fixed using boundary conditions. All parts were connected using interactions, defined to transmit compressive forces in the normal direction but not carry tensile forces, resulting in a joint opening. Coulomb friction was defined for the tangential behavior, with a friction coefficient of 0.56, see Section 2.1. The reinforcement was restrained to the surrounding concrete in all DOF.

All simulations were load-controlled and performed using nonlinear geometry. An implicit dynamic solver was used for all simulations except for the nonlinear case with shear keys and supports (SP+SK), where an explicit dynamic solver was used. The water load was applied using pressure loads. To match the loading in the model tests, the water pressure up to the crest level was applied as linearly increasing, and the added pressure from the overtopping was uniformly distributed. The joints were sealed in the model tests. Therefore, the uplift pressure was assumed to be small and not considered in the simulations.



a) b)

Figure 2. Geometry and mesh of the numerical model a) single monolith and b) full dam.

3 RESULTS

A comparison between the simulations and each test series is presented in Figure 3. In the case of the single monolith (SM), the failure corresponded well between the analytical calculation, the numerical simulation and the model tests. In the numerical simulation, some elastic displacements occur, which are not present in the model tests.

The average failure load was accurately predicted in the numerical simulation in the case with supports (SP) and the case with supports and shear keys (SP+SK). However, the pre-failure displacement was not captured well, likely due to the difficulty in accurately representing the imperfect geometry of the physical model numerically. An initial displacement was observed in the numerical simulation but was smaller and had a more even distribution than in the physical model tests.

The comparison between the model-scale simulations and the full-scale prototype simulations is presented in Figure 3d. In the figure, the displacement and hydrostatic pressure are divided by the dam height (18 m in the prototype scale and 1.2 m in the model scale). The agreement between the two simulations is good. Because of the difference in inertia, comparing the time signals for the simulations in prototype and model scale becomes difficult. The difference in inertia is reflected in the similitude requirements, where the time is scaled $1{:}\sqrt{15}$ for a linear elastic, dynamic model (Harris and Sabnis 1999). The time must therefore be multiplied by $\sqrt{15}$, while the compared unit, e.g., displacement, is multiplied by 15.

The stress was high in both models using side supports, see Figure 4. The stress increased as the monoliths had some initial displacements and peaked before the failure of the dam. The peak tensile stress was 4.4 MPa for SP and 153 MPa for SP+SK. The peak compressive stress was 8.0 MPa for SP and 282 MPa for SP+SK. The stress values for SP indicate that some concrete cracking will occur. However, the stress values for SP+SK are unrealistic, and large-scale cracking and crushing of the concrete would have occurred before this point.

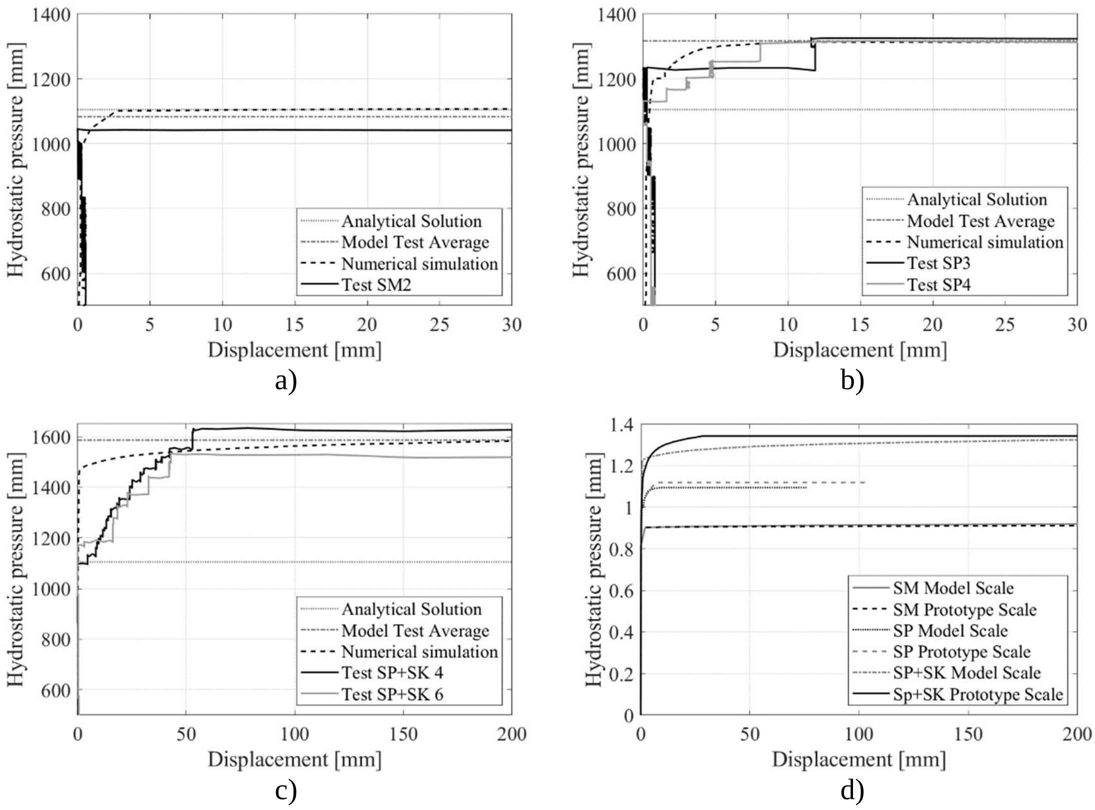


Figure 3. Comparison between model tests and numerical simulations a) single monolith (SM), b) supports, no shear keys (SP), c) supports and shear keys (SP+SK) and d) comparison between the simulations in model scale and prototype scale.

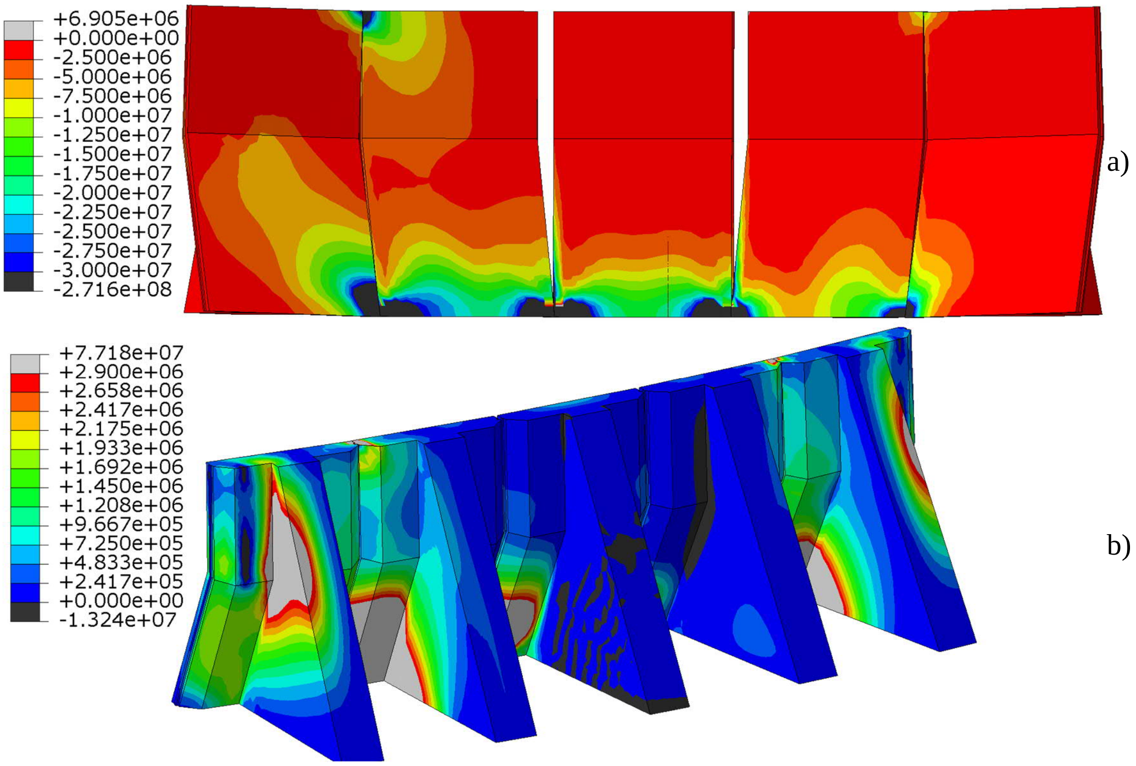


Figure 4. Presentation of the stress distribution in the linear elastic numerical simulation of SP+SK in the prototype scale a) compressive stress and b) tensile stress.

A nonlinear material model was defined for the concrete to assess the impact of the stress on the failure load. The nonlinear simulations were performed in the prototype scale to ensure correct material behavior. The single monolith failure was not simulated using the nonlinear material model since the linear elastic simulations did not indicate cracking. The model without shear keys (SP) experienced some cracking in the joints, as indicated by the linear elastic simulation, see Figure 5. There was large-scale cracking in the model with shear keys (SP+SK). The edge monoliths cracked severely in both the front plate and the support buttress, resulting in large deformations. The three central monoliths had severe cracking in the front plate and some cracking in the buttress. For the case with supports but no shear keys (SP), the failure load was similar for the linear elastic and nonlinear simulations. This means that friction was the governing material property and the material strength had a limited impact on the results. However, when the nonlinear material model was used in the case with shear keys (SP+SK), the failure load decreased by 15 %, see Figure 6. The failure mode also changes from a pure sliding failure to a combined material and sliding failure. Because of the large nonlinear deformation of the monoliths, reinforcement was added to the simulation. The dam still had a combined material and sliding failure with the reinforcement, but the failure load was slightly larger. The failure load was 6 % lower than the linear elastic case.

4 DISCUSSION

In this numerical study, the physical model tests performed by Enzell et al. (2023) were recreated using load-controlled finite element simulations. The load-controlled simulation gave failure at a correct load level. However, the simulations failed to show the pre-failure displacements obtained in the model tests. The difference is likely due to imperfections in the geometry of the model test. The lack of pre-failure displacement in the numerical simulations might make predicting potential pre-failure displacements challenging in real concrete dams.

The simulations in the prototype scale agreed well with the model-scale simulations. The agreement confirms that the model tests were correctly scaled and representative of the prototype scale. However, the difference in inertia made the comparison difficult. The simulations in the prototype scale required a longer total time with a slower load application. If the load were applied too quickly, the dam would not have time to displace, and the failure load would appear higher than the actual failure load. A step time of 15 seconds seemed sufficient for the simulations in the prototype scale. The issue of inertia in load-controlled simulations was previously discussed by Enzell et al. (2021).

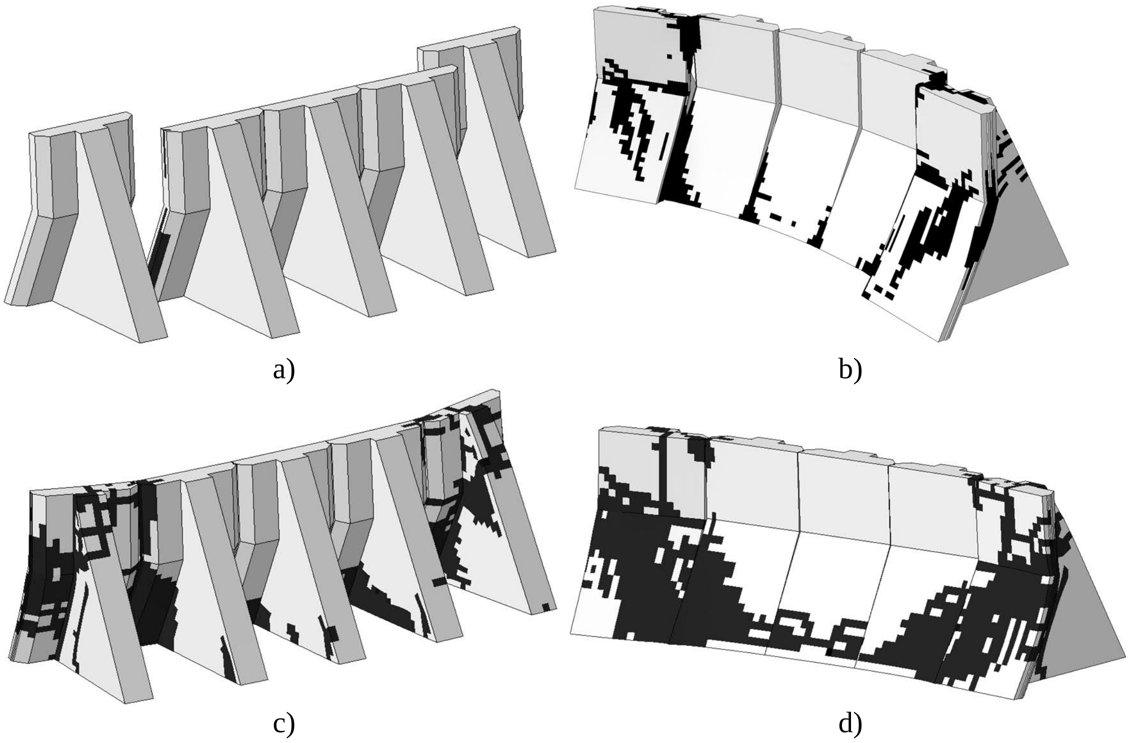


Figure 5. The extent of cracking in the numerical simulation using the nonlinear material model a) cracking in SP simulation, b) crushing in SP+SK simulation using reinforcement, c) cracking in SP+SK simulation using reinforcement and d) cracking in SP+SK simulation using reinforcement.

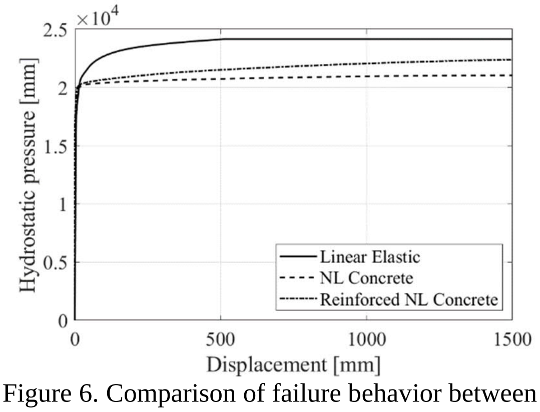


Figure 6. Comparison of failure behavior between the linear elastic and nonlinear simulation.

In the prototype-scale simulations, stresses indicative of crushing and cracking of the concrete was detected. The cracking and crushing were confirmed in the nonlinear simulations. In the case without shear keys (SP), the cracking was limited to the joints, and the failure occurred at the same load level. However, the cracking was extensive in the case with supports and shear keys (SP+SK). A new failure mode was also introduced due to the nonlinear material, and the failure occurred at a failure load 6 % lower. Therefore, the assumption of rigid body motions in the model tests was reasonable except for the case with supports and shear keys (SP+SK). In that case, the material should have been scaled to achieve more accurate results. In traditional dam design and analysis of existing dams, the dam is often assumed to be rigid during the stability analysis. The simulation results indicate that this might be a non-conservative assumption under certain circumstances.

The physical model tests helped to validate the load-controlled simulations. However, in the simple load-controlled simulations, the discharging water and the secondary effects from the flowing water cannot be assessed. A step in developing the numerical routine would be adding fluid-structure interaction (FSI) to the simulations. The water could be simulated by, e.g., Coupled Euler-Lagrange (CEL) or Smoothed Particle Hydrodynamics (SPH) simulations.

The model tests are limited to a specific setup and geometry. As a further study, more numerical simulations should be performed with real dams as case studies. More realistic boundary conditions should also be studied to give a more reasonable range of impact from different types of boundaries, such as rock abutments, embankment dams or massive concrete intake buildings.

5 CONCLUSIONS

This paper presents the results of a numerical study examining the ability of dynamic finite element simulations to replicate dam failures. The results from the physical model tests performed by Enzell et al. (2023) were used as a case study to validate the numerical results. It was found that the numerical model was able to accurately reproduce the failure mode and breach development observed in the physical model tests. However, the pre-failure displacement was not simulated accurately. It could also be confirmed using the numerical simulation that the tests were representative of the dam on the prototype scale. However, the model test with the highest degree of restraint had a lower load-bearing capacity and collapsed in a different failure mode when a nonlinear material model was used. This means that the assumption of rigid bodies was invalid in this model test. The assumption of rigid bodies might also be non-conservative in real dams under certain circumstances.

Load-controlled finite element simulations were sufficient for simulating the behavior of the physical model tests. However, the water discharge and potential effects on adjacent structures could not be simulated. The possibility of implementing FSI methods in the simulation of dam failures should be studied further.

Further research is needed to determine the reliability of numerical models for predicting dam failures under different loading conditions and in more realistic geological settings. However, these findings suggest that load-controlled FE models can be a valuable tool for analyzing the stability of concrete gravity dams. The simulations can also help predict the development of failures for an entire dam, including the interaction between the monoliths and connecting structures, such as abutments.

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