Numerical modelling of a clay core earth fill dam with and without creep deformation

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ABSTRACT
Clayey soils would tend to display creep deformations. This could be exhibited as slow movements of natural clay slopes and embankment structures founded on clays. An aging earth fill dam with clay core and clay blanket was analyzed with emphasis on the effect of creep behaviour on displacements using a finite element software program. Undisturbed clay samples taken from site were tested in the laboratory to obtain parameters to be used in the numerical modelling. Laboratory tests include: index testing, one-dimensional consolidation tests, consolidated drained direct shear tests, consolidated undrained direct simple shear tests, and isotropically consolidated undrained triaxial compression tests. Results from numerical simulations with and without creep are presented in this paper.

RÉSUMÉ
Les sols argileux auraient tendance à présenter des déformations dues au fluage. Cela pourrait être présenté sous forme de mouvements lents de pentes d'argile naturelles et de structures de remblais fondées sur des argiles. Un barrage en terre vieilli avec noyau en argile et couverture en argile a été analysé en mettant l'accent sur l'effet du comportement de fluage sur les déplacements à l'aide d'un logiciel utilisant des éléments finis. Des échantillons d'argile non perturbés prélevés sur le site ont été testés en laboratoire pour obtenir les paramètres à utiliser dans la modélisation numérique. Les tests de laboratoire incluent: les tests d'indice, les tests de consolidation unidimensionnels, les tests de cisaillement direct drainé consolidé, les tests de cisaillement simple direct consolidé non drainé et les tests de compression triaxiale non drainée consolidée isotrope. Les résultats des simulations numériques avec et sans fluage sont présentés dans cet article.

1 INTRODUCTION

Numerous water-retaining earth fill dams were built in the 1950s. Some of these aging earth dams are exhibiting delayed instabilities after performing satisfactorily for over fifty years. It is possible that time-dependent creep displacements have been slowly occurring for many years until sufficient movements led to instabilities.

Secondary compression, or creep, is defined as time-dependent deformation that occurs under constant effective stress. Soil creep behaviour could depend on several factors such as soil structure, stress history, drainage conditions, and changes in pressure and environment with time (Mitchell and Soga, 2005). Compared with sandy materials, clayey soils under sustained constant pressure or self-weight usually exhibit creep behaviour.

Creep deformation is important in analyzing geotechnical structures in cases where long-term behaviour is of interest. The time-dependence of soil properties is often ignored which could lead to unwarranted deformations leading to failure or collapse with time (Campanella and Vaid, 1974). The predicted movements or instabilities from the long-term analysis considering creep in the design process could urge for the need for more frequent field monitoring as well a plan for remedial solutions, if necessary.

Research is currently ongoing on the stability assessment of aging earth fill dams in hydroelectric generating stations as part of its periodic dam safety review. A calibrated model based on the conditions of an aging dam which experienced irregular movement will be used to evaluate the long term performance of similar aging dams. The development and calibration of a numerical model of an aging earth fill dam with clay core and clay blanket considering creep deformation behaviour with the use of a finite element software program will be presented in this paper. Parameters used in the numerical modelling were determined from laboratory test results performed on undisturbed clay samples taken from field investigations.

2 SITE INVESTIGATION AND SAMPLING

This paper is focused on an aging water-retaining earth fill dam that was constructed in the 1950s. After over fifty years of good performance, irregular movement was observed in the upstream section of the dam. There was no loss of water with this movement. Proactive measures are already in place by the dam operator and owner in order to cease further deformation from occurring.

2.1 Site Description

The location was part of an old creek, which explains the presence of a layer of permeable sandy silt and gravely sand underneath the dam. With the absence of an impermeable natural clay layer, a compacted clay blanket was placed in the upstream side and was tied into the inclined clay core as a means to control seepage. The cross section of the dam can be seen in Figure 1.

The local clay material used in the construction of the dam was classified as high plasticity lacustrine clay, often described as highly fissured in nature with nuggetty or crumbly appearance.

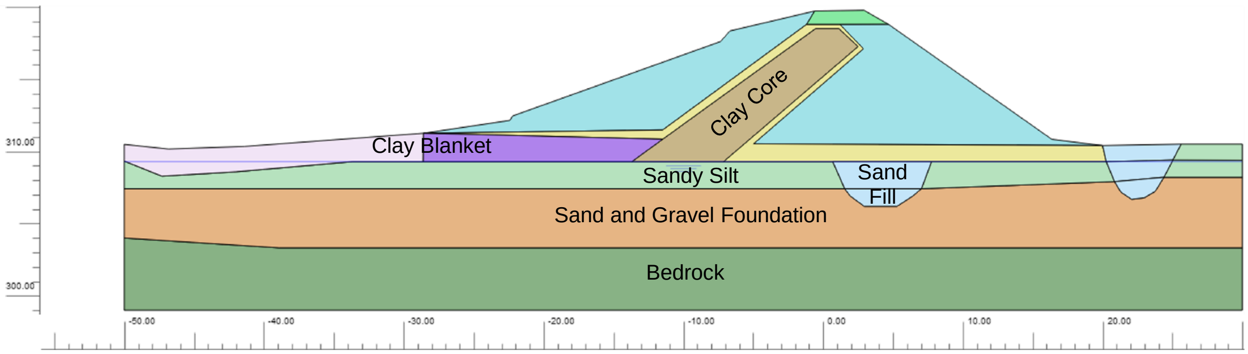

Figure 1. Cross – section of dam

2.2 Undisturbed Soil Sampling

After completing a preliminary assessment of the dam, drilling and undisturbed soil sampling of the clay core was performed during the fall season when the ground was not frozen. Continuous soil sampling by means of a Hollow Stem Auger (HSA) using Shelby tubes was used to collect clay core samples from predetermined borehole locations. The use of Shelby tubes with a diameter of 102 mm (4 in) was recommended in order to obtain a better representation of fissures within the clay samples.

Undisturbed soil sampling of the clay blanket was done during the winter season. The frozen reservoir allowed the drilling equipment to be appropriately positioned over borehole locations and for soil sampling to take place without inducing further disturbance onto the placed clay. The clay blanket was thought to have less fissures having been under water for over fifty years. Because of this, typical 76 mm (3 in) diameter Shelby tubes on a track-mounted piston sampler was used to collect clay blanket samples.

3 LABORATORY TESTING

Several tests were performed to determine the index properties, strength, deformation, and creep characteristics of the collected clay core and clay blanket samples. These laboratory tests were done in accordance with ASTM standards.

3.1 Index Properties

The index properties of the tested samples are shown in Table 1. Extruded clay core samples were a mottled mix of brown and grey with random silt lenses and silt pockets. It had a matted texture with medium to stiff consistency. Clay blanket samples were brown in color with soft to medium consistency in the upper layer; and mottled mix of brown and grey with stiff to very stiff consistency from 0.6 m and below. Clay core and blanket samples were classified as "fat clay" of high plasticity (CH) according to the Unified Soil Classification System (USCS).

Table 1. Index properties

Properties	Clay Core	Clay Blanket
Moisture Content (%)[1]	39	41
Liquid Limit (%)	85	82
Plasticity Index (%)	60	57
Specific Gravity, *Gs*	2.72	2.68
Minus #200, <0.075mm (%)	100	100
Clay Fraction, <0.002mm (%)	74	71
Activity	0.81	0.80

[1]moisture content of specimens after preparation

3.2 Deformation and creep characteristics

Deformation characteristics were determined from one-dimensional consolidation tests. Creep in compression parameters were determined from the same test provided that the duration of the load application was long enough for secondary compression to occur. The results of these tests are summarized in Table 2.

Table 2. Deformation characteristics

Properties	Clay Core	Clay Blanket
Apparent preconsolidation pressure (kPa)	150	170
Compression index, C_c	0.113	0.121
Recompression index, C_r	0.031	0.039
Secondary compression index, C_α	0.035	0.044

3.3 Strength parameters

Based on the shape of the postulated slip surface, the strength parameters for the clay core and clay blanket were determined from isotropically consolidated undrained triaxial compressions tests and consolidated undrained direct simple shear tests, respectively.

Results from isotropically consolidated undrained (CIU) triaxial compressions tests on clay core samples are shown in Figure 2, Figure 3, and Figure 4. It could be observed in Figure 2 that clay core samples show an isotropic elastic behaviour ($m\Delta p \approx \Delta u$) when sheared at a mean effective stress between 100 kPa to 300 kPa. Specimens behaved typically for a normally consolidated specimen when sheared higher than 300 kPa. Samples sheared at 400 kPa were normally consolidated whereas the rest were over-consolidated samples.

Consolidated undrained (CU) direct simple shear tests were performed to determine the shear strength of the clay blanket samples along the horizontal plane. Results from these tests are shown in Figures 5 to 7.

Residual strengths of both the clay core and clay blanket were determined from consolidated drained (CD) direct shear tests. Multiple reversal method was used wherein clay samples were repeatedly sheared until 100 % strain was reached, which corresponded to about 63 mm of displacement. Figure 8 shows the normalized stress-strain behaviour of the clay core and clay blanket samples.

Both samples exhibited strain softening as shown in Figure 8. This indicated that the clay samples from the core and the blanket were behaving in a brittle manner under an applied normal stress lower than 100 kPa and behaved in a more ductile manner as the normal stress increased. The degree of softening is more evident in clay blanket samples compared to the clay core. This could be attributed to the clay blanket being under water for more than 50 years in addition to its fissured nature. The presence of silt lenses and fissures in the clay core also aided the softening behaviour.

The summary of effective shear strength parameters from the performed strength tests on clay core and clay blanket samples is shown in Table 3. The fully softened or post-peak strength values assumed zero cohesion as recommended by Rivard and Lu (1978) for fissured clays.

Table 3. Effective shear strength parameters

	Clay Core		Clay Blanket	
Shear Strength Parameters	c' (kPa)	*Φ'* (°)	c' (kPa)	*Φ'* (°)
CIU Triaxial Test (Peak)	33	24	-	-
CIU Triaxial Test (Post-peak)	0	18	-	-
CU Direct Simple Shear Test (Peak)	-	-	6	17
CU Direct Simple Shear Test (Post-peak)	-	-	0	21
CD Direct Shear Test (Residual)	0	8	0	9

4 NUMERICAL MODELLING

The high clay content in the core and blanket possibly increased the effect of creep under sustained constant pressure during its service years as there were little or no changes in the dam geometry since its completion in the 1950s. Very slow creep movement could have occurred for many years before the accelerated movement (Tavenas and Leroueil, 1981) at the upstream section of the dam. Due to this, there is a need to incorporate time-dependent creep deformation analysis in the model.

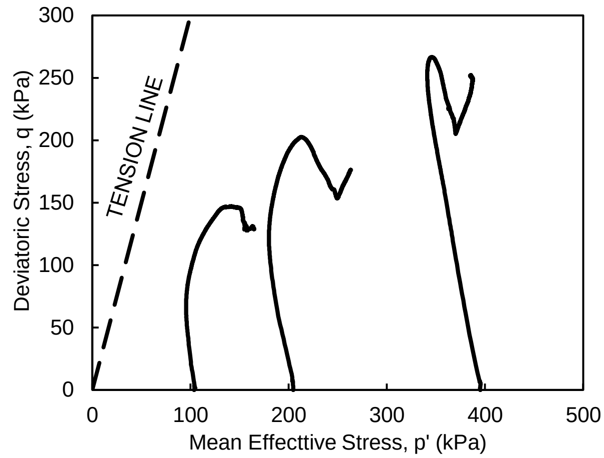

Figure 2. Stress paths in p'-q space from CIU triaxial tests

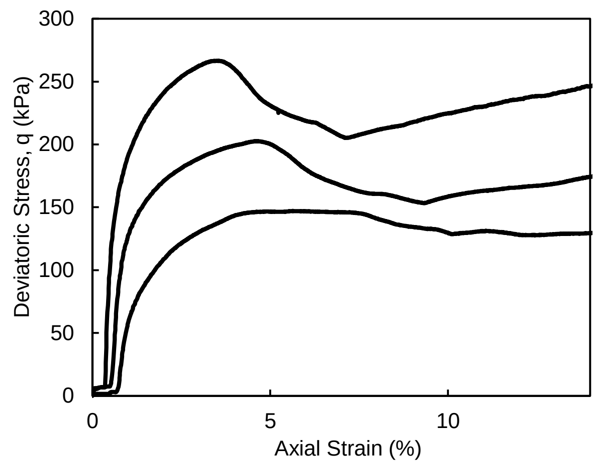

Figure 3. Stress-strain behaviour from CIU triaxial tests

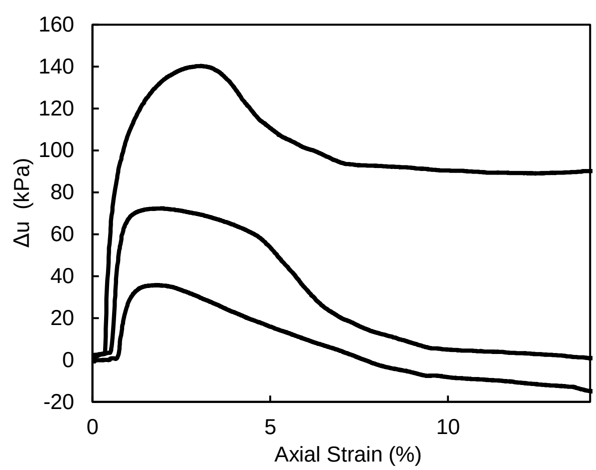

Figure 4. Porewater pressure measurements from CIU triaxial tests

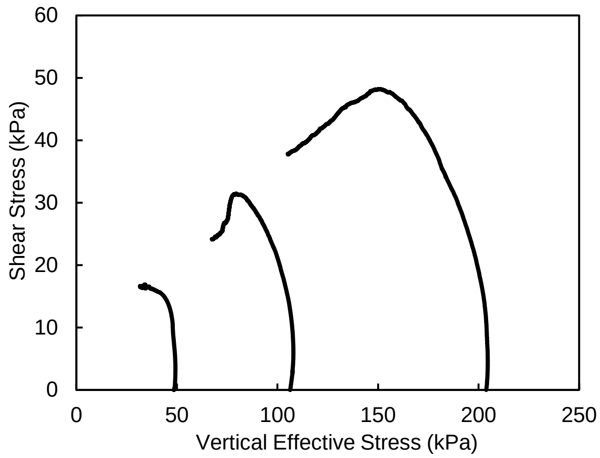

Figure 5. Shear stress versus vertical effective stress plot from CU direct simple shear tests

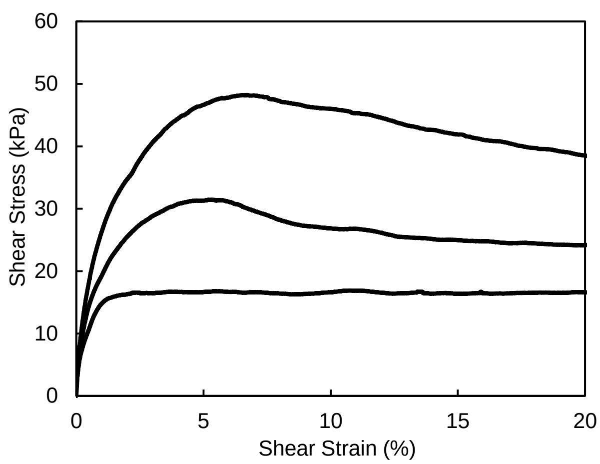

Figure 6. Stress-strain behaviour from CU direct simple shear tests

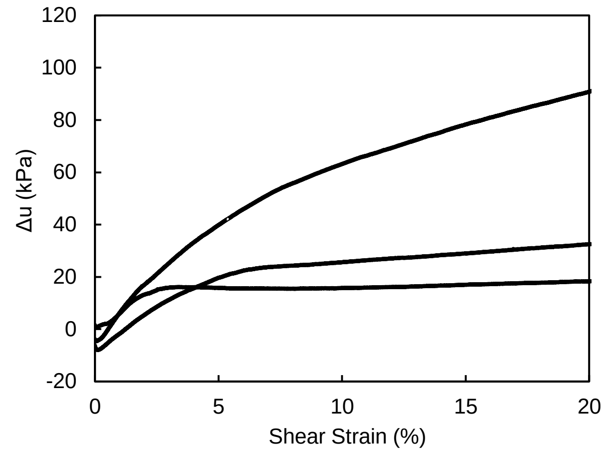

Figure 7. Porewater pressure measurements from CU direct simple shear tests

Numerical modelling was implemented using PLAXIS 2D, a geotechnical engineering software that can perform finite element deformation and stability analyses. PLAXIS 2D was chosen as it has the capability to incorporate time-dependent creep behavior into the soil analysis with the use of its Soft Soil Creep model.

The Soft Soil Creep (SSC) model was developed by transforming the logarithmic creep law of secondary consolidation into a differential form. It was then extended towards general three-dimensional states of stress and strain by incorporating Modified Cam-Clay and viscoplasticity, with a Mohr-Coulomb failure criteria (Vermeer and Neher, 1999). SSC is an extension of the pre-existing Soft Soil (SS) model, with the addition of the modified creep index (μ^*) parameter to the modified compression (λ^*) and modified swelling (κ^*) indices. The SSC model parameters used in the numerical modelling can be seen in Table 4.

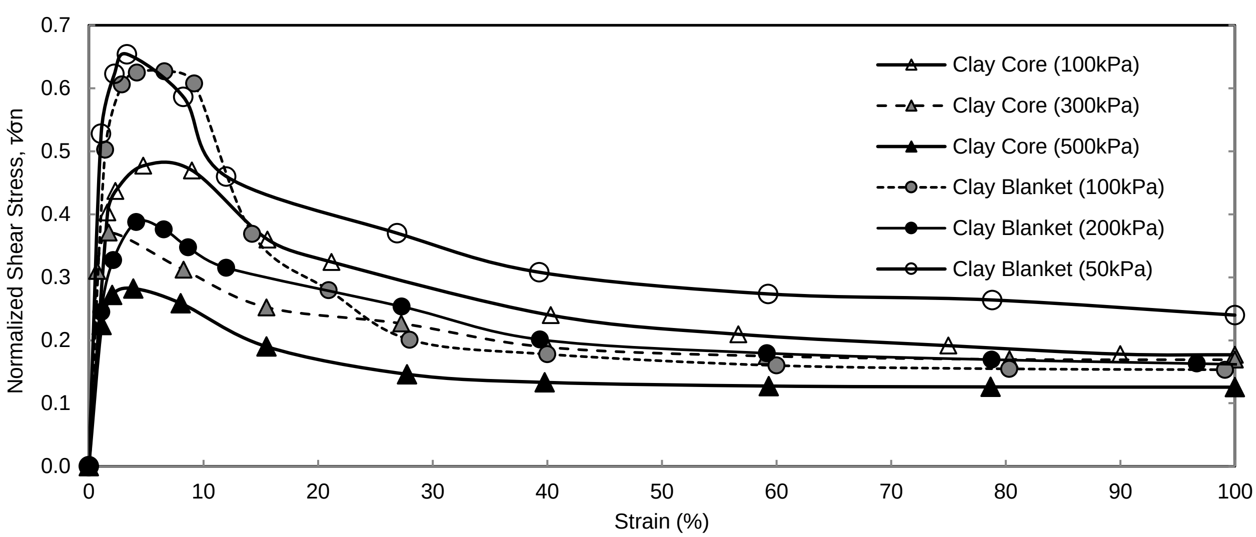

Figure 8. Stress-strain behaviour from CD direct shear tests

Table 4. Parameters for numerical modelling.

	Clay Core	Clay Blanket
Material Model	Soft Soil Creep	Soft Soil Creep
Unit weight, γ (kN/m^3)	17	17
Modified compression index, λ^*	0.023	0.025
Modified swelling index, κ^*	0.013	0.016
Modified creep index, μ^*	0.007	0.009
Post-peak Φ' (°)	18	21
Average[1] Φ' (°)	13	15

[1]average of post peak and residual angles of friction

Two cases were considered in terms of shear strength parameters as indicated in Table 4. Case 1 used fully softened (also known as post-peak and critical state) strengths as the placed compacted clay blanket was assumed to have reached its fully softened state after being underwater for over fifty years. In addition, the clay core was thought to have strain softened due to the presence of silt lenses and its fissured structure. Strain softening was also evident in the consolidated drained direct shear test results. Case 2 used the average value between the post-peak and residual shear strengths, considering that only a portion of the clay core and clay blanket had yielded. This indicated that not all elements reached residual strength along the slip surface during movements (Stark and Eid, 1997).

The numerical model must be able to represent the observed conditions of the dam throughout its service life. Settlement was still observed even after forty years of being in service until the sudden movement in the upstream section after over fifty years. Although the unexplained movement did not cause complete collapse of the dam, it is still seen as a sign of instability. These conditions must be shown in the calibrated model of the dam.

The development of deformation in the dam with and without creep can be seen in Figure 9 using Case 1. When creep was not considered (Figures 9a to 9c), results indicated that displacements became constant forty years after impounding was completed. As investigations noted movement in the dam even after 40 service years, this meant that the model was not able to show observed dam conditions and that creep should be considered.

Figures 9d to 9f display creep analysis results using Case 1 wherein the model exhibited continuous movement even after fifty years, replicating what has been observed over the service life of the dam. However, safety calculations using this case at the time of sudden upstream movement revealed that the dam was stable with a factor of safety of 1.203. Knowing that the dam already showed signs of instability, a factor of safety of less than unity was expected. Using Case 2 considering creep in the model, a factor of safety value of 0.996 was obtained. This implied that the completed calibrated numerical model based on the observed conditions of the dam would include time-dependent creep deformation analysis using clay strength values between the post-peak and residual shear strengths. This calibrated model would be used to evaluate the long term performance of similar aging earth fill dams.

5 SUMMARY

There was a need for a numerical model to be calibrated to the conditions of an aging earth dam that exhibited irregular movement on its upstream section in order to understand the delayed instability despite its good performance for over fifty years. Site investigation and soil sampling were conducted to characterize the ground conditions and collect samples for laboratory testing. Parameters from laboratory test results were used in the development and calibration of the numerical modelling.

Collected clay core and clay blanket samples were classified as “fat clay” of high plasticity (CH) according to the Unified Soil Classification System (USCS). Both clay core and clay blanket exhibited strain softening which was attributed to the fissured nature of the clay in addition to the silt lenses observed in clay core samples.

Investigations indicated that movement was observed in the dam even after forty years leading to the upstream movement after more than fifty years of service. This could be an indication that the dam was exhibiting slow creep movement with respect to time. The high clay content of the soil could have increased the effect of creep under the sustained constant pressure from the dam.

The developed calibrated model signified the need to include time dependent creep in numerical modelling. In doing so, the predicted long-term displacements were included in the analysis. Safety analysis indicated that the shear strength of fissured clays could be as low as the average of post-peak and residual strength of the material (Skempton, 1985) as not all elements along the slip surface would reach residual strength when sheared (Stark and Eid, 1997).

Future research work include the evaluation of similar aging water-retaining earth fill dams with the use of the calibrated model. The long-term performance will be assessed if these aging dams still meet current safety standards.

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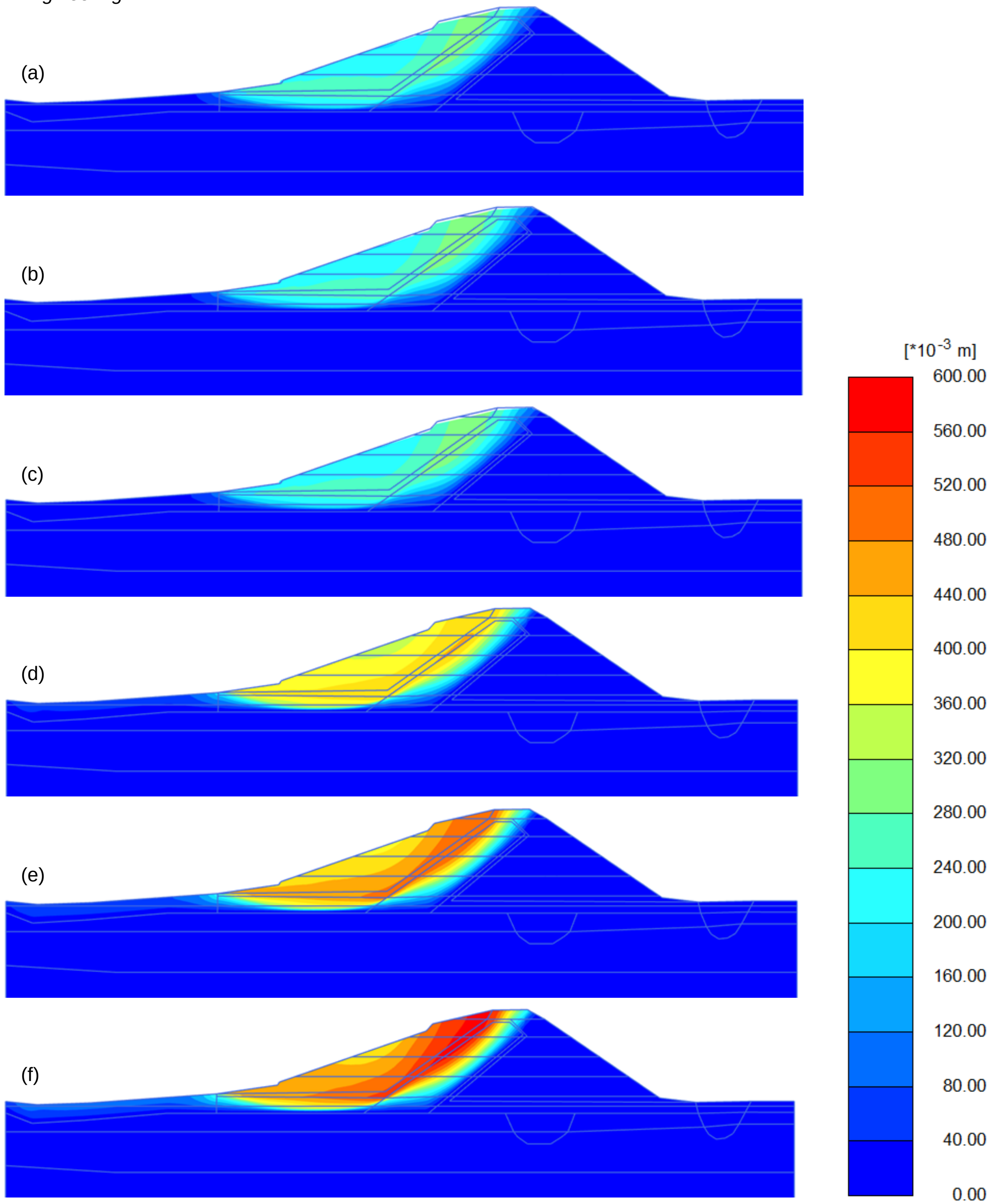

Figure 9. Deformation development at different stages: (a) after construction using Case 1 without creep, (b) 10 years after impounding using Case 1 without creep, (c) 50 years after impounding using Case 1 without creep, (d) after construction using Case 1 with creep, (e) 10 years after impounding using Case 1 with creep, and (f) 50 years after impounding using Case 1 with creep.