

## Construction, Rapid Drawdown, and Earthquake Simulation of an Earthen Dam

### Summary

Construction of earthen dams entails sequential placement and compaction of soil layers and the subsequent fill-up of the embanked reservoir. In the design of earthen dams, two potentially critical events must be considered: the rapid emptying (or drawdown) of the reservoir and the dynamic loading of an earthquake. The possibility of dam failure in these situations depends on the respective build-up and dissipation of the fluid pore pressure in the soil.

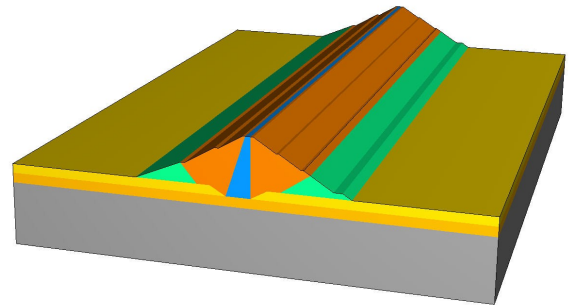
For the purposes of a macroscopic analysis, the soil in an earthen structure can be considered as a three-phase continuum consisting of solid particles, a wetting liquid, and a gas. Abaqus provides several material models and a coupled diffusion-displacement analysis procedure for modeling such problems. These capabilities can, thus, be utilized for assessing the safety and reliability of earthen dams.

### Background

Earthen dam structures rely on the ability of a soil mass to remain stable when the slopes of the soil mass are less than a certain value. Under certain loading conditions, however, the stability of the mass can be compromised and these loading conditions must be considered in the design process.

Sandy soils derive their shear stiffness from the frictional resistance between the solid grains. This frictional resistance depends on the normal compressive contact forces between the soil particles. In a fully saturated soil the voids are filled with water. If the pore water is prevented from draining at a sufficiently rapid pace during deformation, more of the compressive load may be transferred to the pore water. Such a condition may arise during earthquakes when the solid particles in the soil move and take up a new configuration with a smaller void ratio, resulting in build-up of excess pore water. When this excess water is not able to drain out rapidly, the pore water pressure increases and the normal compressive contact forces between the soil grains reduce. This reduction in the contact forces between the grains results in a loss of shear strength of the soil. During an earthquake the deformation is quite rapid, occurring on a time scale that is much shorter than the characteristic diffusion time for pore fluid flow. The pore water, hence, may not be able to drain at a sufficiently rapid pace from regions of high pore pressure, resulting in a loss of shear strength of the soil. This phenomenon is known as liquefaction.

In a soil that is or becomes fully saturated or submerged, the pore water exerts a buoyancy force on the soil grains,



### Key Abaqus Features and Benefits

- Ability to model transient or steady-state distribution of fluid pore pressure in soils.
- Ability to model consolidation; i.e., the deformation in soils associated with the redistribution of pore water and pressure.
- A variety of constitutive models for soils, including Mohr-Coulomb, Drucker-Prager, Cam-Clay, and Cap plasticity models. Creep behavior can be included with the Drucker-Prager and Cap plasticity models.

reducing the forces between the soil grains. The buoyancy effect can reduce the load on the soil grains, requiring less shear resistance between the soil particles to resist significant deformation. When the water level of the reservoir formed by an earthen dam is suddenly reduced, in regions where the pore fluid can drain rapidly leading to a reduction in the pore pressure, the forces acting on the soil particles suddenly increase due to the reduction in buoyancy. The frictional resistance between the soil particles may not be sufficient to resist this increased shear stress to which they are subjected. The increased shear stress is due to the increase in vertical load as the overlying soil may still be saturated or almost saturated. Typically, the problem will occur between a region that is draining relatively rapidly and lies on top of a region of soil that is draining more slowly. Situations such as these can occur during the rapid drawdown of a water reservoir, possibly due to irrigation or safety concerns.

In this technology brief we present an example of an earthen dam first being constructed and then subjected to a rapid drawdown condition and to a dynamic excitation due to an earthquake resulting in liquefaction of the soil.

## Finite Element Analysis Approach

Earthen dams are constructed by laying an embankment of compacted earth to block a natural stream and create an impounded reservoir. To minimize water seepage through the dam, a central clay core with orders of magnitude smaller permeability than the surrounding dam material can be placed within the dam. Rock-fills can also be placed in the downstream and upstream toe regions for providing stability and increased permeability to help in dissipating pore pressure.

Earthen dams are constructed by first excavating the site and constructing the foundation. Layers of earth are then placed on the foundation and one on top of each other along with portions of clay forming the core region of the dam. Each of these layers is about 10 cm to 20 cm thick and is compacted individually after placement. After the dam is constructed to the full height, impounding of water in the embanked region is allowed to take place.

The geometry of the dam that is being analyzed is depicted in Figure 1. We use a two-dimensional model for the dam, which is assumed to be in a plane strain condition. The dam has a height of 55 m from the ground level and is 245.2 m wide at its base. It tapers to a width of 12 m at the top. The dam is made up of a central clay core that has a width of 28 m at the base and tapers to a width of 6 m at the top of the dam. The foundation of the dam is laid from ground level to a depth of 10 m below the ground level, as shown in Figure 1. The foundation lies on top of a layer of weathered rock of thickness 10.667 m overlaying a solid rock base. For analysis purposes the solid rock is considered to extend up to a depth of 90 m from the ground level and appropriate boundary conditions are applied to the model at this depth. In addition, the total width of the foundation, weathered rock, and solid rock is considered to be 550 m, as shown in Figure 1, and appropriate boundary conditions are applied on the model's vertical edges at these locations. Upstream and downstream rock-fills are also present in the model, as shown in Figure 1.

Although an earthen dam may be constructed using a large number of layers, for simplification we have assumed that only three layers are placed in the dam model being presented here. The first layer is 21 m thick, and the placement of this layer is completed in 90 days. The second layer is 19 m thick, and it is laid in 60 days. The third layer is 15 m thick, and it is constructed in 30 days.

The finite element mesh for the dam is completely defined at the beginning of the analysis. We model the construction of the dam in four construction stages. In the first step the dam elements are removed and the deformation of the rock layers under geostatic or gravity loading is modeled, as shown in Figure 2a. Each additional construction layer is added in a separate step, as shown in Figure 2b, Figure 2c, and Figure 2d. To simplify the analysis, it is assumed that the deformation of each layer under gravity

loading is small and that the next layer can be added back into the analysis using its original undeformed geometry, as defined at the beginning of the analysis. A transient consolidation analysis is carried out over the time period of each layer placement.

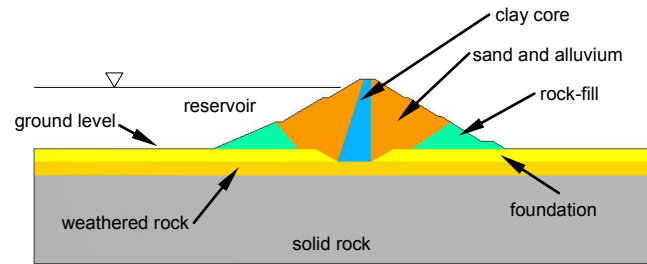


Figure 1: A sketch of the cross-section of the earthen dam

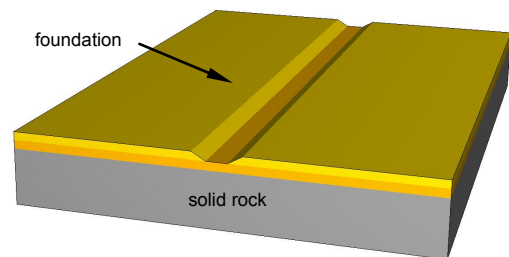


Figure 2a: The first stage of construction of the dam. The site is excavated, and the foundation is laid on the weathered rock layer.

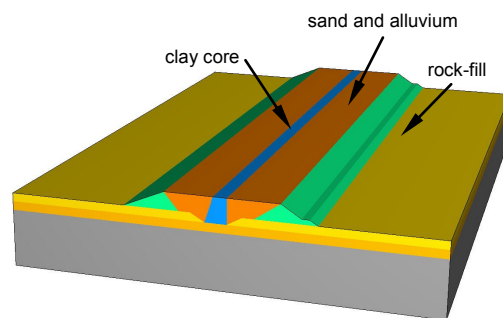


Figure 2b: The second stage of construction of the dam. The first construction layer is laid on the foundation. Sand, alluvium, clay, and rock-fill are placed at the locations shown.

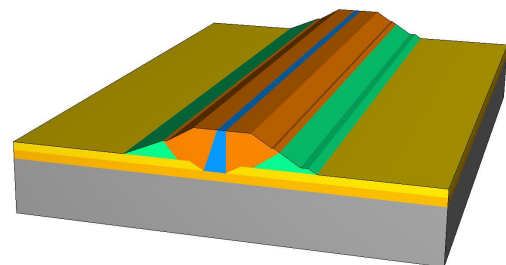


Figure 2c: The third stage of construction of the dam. The second construction layer is added.

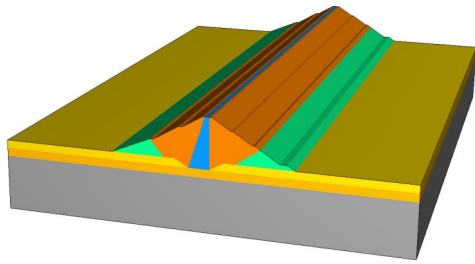


Figure 2d: The final stage of construction of the dam. The topmost construction layer is added.

After all three construction layers are laid, the dam reservoir is slowly filled up in three stages. In the first stage water is impounded to a depth of 21 m above the ground level. In the second and third stages of fill-up the water rises to a depth of 40 m and 50 m above the ground level, respectively. Each of these stages takes place over a period of 30 days. All these stages are modeled using a transient consolidation analysis, which computes the soil deformation as well as the pore pressure distribution. Figure 3a, Figure 3b, and Figure 3c depict the pore pressure distribution at the end of each of the three reservoir filling stages.

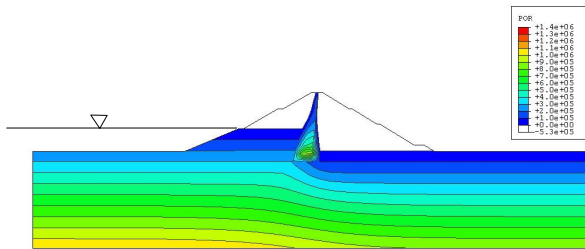


Figure 3a: Pore pressure distribution in the dam at the completion of the first stage of the fill-up of the reservoir.

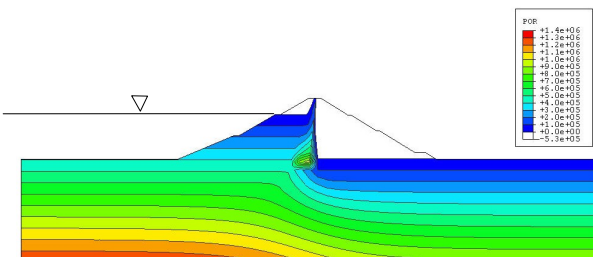


Figure 3b: Pore pressure distribution in the dam at the completion of the second stage of the fill-up of the reservoir.

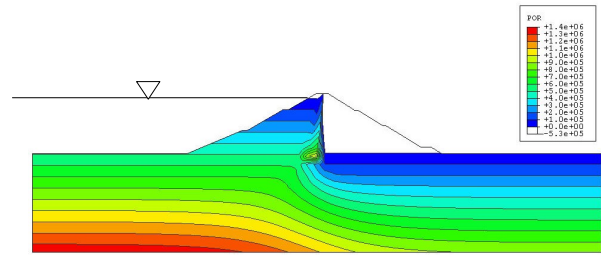


Figure 3c: Pore pressure distribution in the dam at the completion of the third and final stage of the fill-up of the reservoir.

After the filling of the reservoir is complete, there will be a transient phase until the pore fluid flow through the dam reaches a steady state. Because this transient phase will occur over a very long period and the changes are very gradual, Abaqus allows the performance of a steady-state analysis using the conditions at the end of the previous step, which is the final filling of the reservoir, as the initial estimate of the steady-state conditions. The long-term steady-state solution for the pore pressure is shown in Figure 4. The surface connecting points where the pore pressure is zero defines the phreatic surface within the dam, as shown in Figure 5.

The steady-state condition of the dam is used as the starting point for modeling the two critical design conditions: rapid drawdown of the water in the reservoir and high frequency shaking of the dam foundations due to an earthquake. The rapid drawdown condition is achieved by lowering the reservoir water level to the ground level. This drawdown condition takes place over seven days, and is simulated using a transient consolidation analysis similar to that used to simulate the filling of the reservoir.

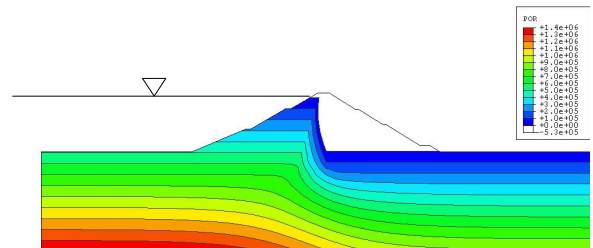


Figure 4: Long-term pore pressure distribution in the dam after a steady-state analysis.

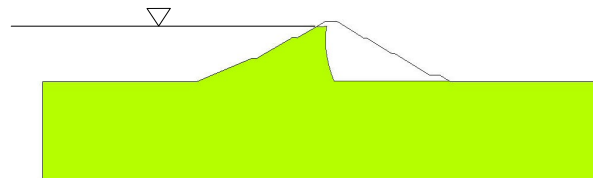


Figure 5: The position of the phreatic surface connecting points where the pore pressure is zero.

The earthquake event occurs over a time period that is very short compared to the characteristic time scale for pore fluid diffusion. Therefore, redistribution of pore pressure during the time period of the earthquake will be neglected. Since the loading due to the earthquake is dynamic, all the inertial forces need to be taken into account, including the effect of the pore fluid in the dam as well as the water in the reservoir. In this analysis we have taken into account the inertial forces due to the pore water in the dam; however, the inertial forces from the reservoir water are not considered in the analysis. For the pore fluid in the dam, only the fluid below the phreatic line is included in the inertial forces. To include these inertial forces, we compute point masses for user-defined elements located at the nodes within the soil and include the contributions from these elements in the dynamic analysis.

## Results and Conclusions

The distributions of pore water pressure during the reservoir filling stages are shown in Figure 3a, Figure 3b, and Figure 3c. The development of pore pressure within the core during the construction and filling phases (Ref. 1) is evident from the contour plots. The position of the phreatic surface in a steady-state condition of full reservoir level can be visualized in Figure 5. This steady-state condition has been used to conduct two analyses: dynamic excitation due to earthquake motion and a rapid drawdown condition.

Usually, the tendency of a soil material to liquefy in a dynamic event is assessed indirectly by comparing the computed stress results to the experimentally determined liquefaction resistance of the soil. The resistance value of the soil against liquefaction is termed as the Cyclic Resistance Ratio (CRR) (Ref. 2). This quantity typically is based on the results of the Standard Penetration Test (SPT) or Cone Penetration Test (CPT). For this analysis we have assumed that the dam material has a CRR of 0.5. In addition, the core and the rock-fills on the upstream and downstream regions of the dam are assumed not to liquefy.

For the case of the dynamic excitation of the dam due to an earthquake, the dynamic stress of interest is the shear stress in the horizontal plane. The ratio of the dynamically induced shear stress to the effective vertical normal stress at the beginning of the dynamic excitation is termed as the Cyclic Stress Ratio (CSR). As suggested in Ref. 3, we have used a factor of 0.65 to scale the maximum value of the dynamically induced shear stress.

The ratio of CSR to CRR serves as a measure of the potential for liquefaction during the dynamic event. Figure 6 shows a quilt plot of the ratio of CSR/CRR. Based on the assumed material properties and the earthquake excitation, the material at locations where the ratio is more than one has the most likelihood of liquefying during the dynamic event.

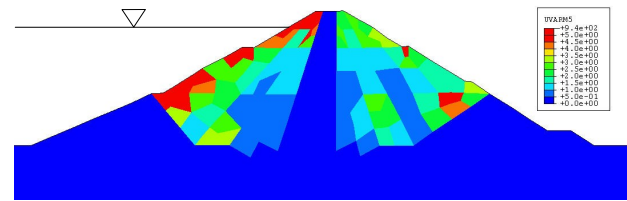


Figure 6: A plot of the ratio of CSR/CRR. Red zones indicate that the soil at these locations has the most likelihood of liquefying due to the earthquake excitation.

The pore pressure distribution after rapid drawdown is plotted in Figure 7. There may be a tendency for the slopes of a dam to fail during rapid drawdown. However, elaborate methods such as first assuming a failure surface and then computing the factor of safety against failure in that mode are required to assess the safety of dams during rapid drawdown reliably. In this analysis, as a preliminary assessment of the likelihood of failure for the rapid drawdown condition, we have used the same failure criteria as that used for the earthquake analysis; that is, of computing the ratio of CSR/CRR.

This ratio is plotted in Figure 8 for the rapid drawdown condition. In actual practice the dam should be checked for safety against a circular arc or linear failure mode.

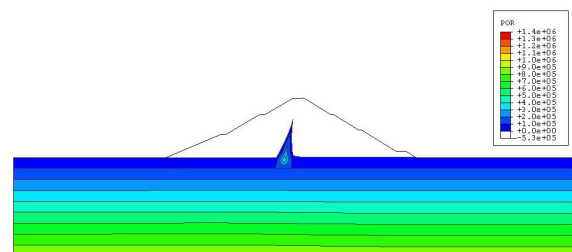


Figure 7: Pore pressure distribution in the dam at the end of the rapid drawdown

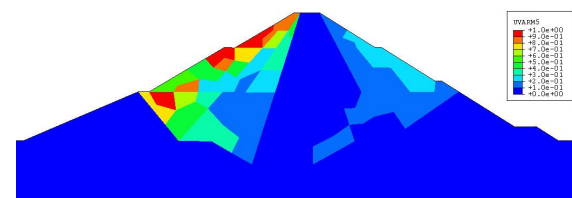


Figure 8: Ratio of CSR/CRR as a preliminary assessment of the likelihood of failure during rapid drawdown.

## Acknowledgements

Dassault Systèmes SIMULIA Corp. would like to acknowledge Finite Element Analysis Services (Pty) Ltd. for providing the finite element model of the earthen dam.

## References

1. Hosseini, M. M, R. A. Fard, "Pore Pressure Development in the Core of Earth Dams During Simultaneous Construction and Impounding," The Electronic Journal of Geotechnical Engineering, vol. 8, 2003.
2. Youd, L., "Updating Assessment Procedures and Developing a Screening Guide for Liquefaction," Research Progress and Accomplishments, 1997–1999; Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, State University of New York.
3. Idriss, I. M., "An Update of the Seed-Idriss Simplified Procedure for Evaluating Liquefaction Potential, New Approaches to Liquefaction Analysis," Lecture Notes of the 1999 Workshop of the Transportation Research Board, Jan. 10, 1999, Washington, D.C.

## Abaqus References

For additional information on the capabilities and techniques outlined above, see the following references to the Abaqus 6.11 documentation:

- Analysis User's Manual
  - "Implicit dynamic analysis using direct integration," Section 6.3.2
  - "Coupled pore fluid diffusion and stress analysis," Section 6.8.1
- User Subroutines Reference Manual
  - "**UEL**: User subroutine to define an element," Section 1.1.27
- Example Problems Manual
  - "Plane strain consolidation," Section 10.1.1
  - "Calculation of phreatic surface in an earth dam," Section 10.1.2

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