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Seismic Deformation Analysis of Rockfill Dam

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Abstract: Core-shell interface is the interface between the core and the fill of the rockfill dam where due to the non-homogeneity of material there is the most likely slippage to occur. This slippage is modeled using ANSYS 10 with contact elements. Among the various parameters in the slippage phenomenon the material non-homogeneity and coefficient of friction is considered in the analysis. Analysis performed is free vibration and initial static deformation. The result from this analysis is then used as input in the time history analysis using Wilson-θ method performed in SAP 14. Then Newmark's chart is used to find the settlement of the dam which is considered as the permanent settlement. The added mass of water for the time history model at reservoir full condition is calculated using Westerguaard's Added Mass formulation.

Keywords: Rockfill dam, core-shell interface, coefficient of friction, slippage, displacement, seismic coefficient, damping coefficient, Westerguaard's Added Mass.

1. Slippage Phenomenon in Core-Shell Interface

Sliding of the shell (rockfill) on the surface of the clay core is likely to occur in reality of clay cored rockfill dam and modeling of such dam without considering this phenomenon causes error in the result. The slippage is modeled using contact element also known as no-tension-gap-friction element. The coefficient of friction between dry concrete and dry clay is 0.4 and wet concrete and wet clay is 0.2. This available data is used as the coefficient of friction for core-shell interface at unsaturated and saturated conditions respectively.

2. Likhu Hydropower Dam

Likhu dam is a rockfill dam with central clay core. The dam is 15.8 m high above the lowest foundation. The crest is 5 m wide with the base width of 60.3m. The core used in the actual project is incline core.

3. Westerguaard's Added Mass

During an earthquake, the interaction between the dam and the reservoir creates additional pressures on the upstream face of the dam. These hydrodynamic pressures may be approximated by Westergaard (1933) formula which uses a parabolic approximation for the additional pressures due to earthquake motion.

According to Westergaard (1933) one can visualize the dynamic action of water on the upstream face of dam, by thinking of a certain body of water in the reservoir as moving with the dam while remainder of the reservoir remains inactive.

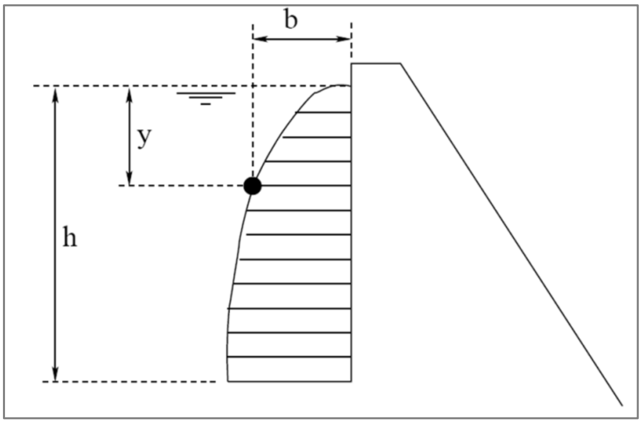

Figure 3.1: Westergaard's added mass concept

γ_{lump}=K(hy)½/g(1-16ρ_wh2/gkT2)½

where,

K is constant defined by Westergaard as K=8011.4 N/m3,

k is the elastic modulus of water,

T is the period of horizontal vibration,

ρ_w is the weight per unit volume of water,

g is the acceleration due to gravity, and

γ_{lump} is the lumped mass per unit area.

4. Newmark's Chart

Analyses of seismic slope stability problems using limit equilibrium methods in which the inertia forces due to earthquake shaking are represented by a constant horizontal force (equal to the weight of the potential sliding mass multiplied by a coefficient) are commonly referred to as pseudo-static analyses.

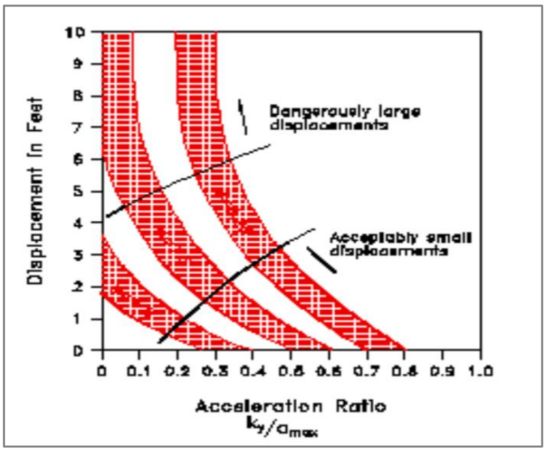

Figure 4.1: Typical Displacements Computed by Newmark Method (Seed, 1979)

The figure shows displacements computed by the Newmark method as a function of the acceleration ratio, k_y/a_{max}, where k_y is the critical seismic coefficient and a_{max} is the expected peak acceleration.

5. Wilson-θ Method

This method was developed by E.L. Wilson which is the modification of the conditionally stable linear acceleration method that makes it unconditionally stable and is available in SAP 2000. This modification is based on the assumption that the acceleration varies linearly over an extended time step $\delta t=\theta.\Delta t$. The accuracy and stability properties of the method depend on the value of the parameter θ, which is always greater than 1. If θ=1, this method reverts to the linear acceleration method, which is stable if $\Delta t<0.551TN$, where TN is the shortest natural period of the system. If θ≥1.37, Wilson's method is unconditionally stable, making it suitable for direct solution of the equation of motion θ=1.42 gives optimal accuracy.

In dynamic analysis of structures and foundations, damping plays an important role. However, due to the limitation in our knowledge about damping, the most effective way to treat damping within modal analysis framework is to treat the damping value of a MDOF system as an equivalent Rayleigh Damping in the form:

$$[C] = \alpha[M]+\beta[K]$$

in which [C], [M] and [K] are the damping, mass and stiffness matrix of the physical system respectively; α and β are the predefined constants called Rayleigh damping coefficients. As such, in most of the practical engineering analysis, the analyst makes the simplifying assumptions in selecting damping ratios (constant for all significant modes). It is a fact that modal mass participation decreases with increase in modes. Based on above, one can infer that, as mass participation decreases with higher modes, the frequency increases and it is indeed an observed phenomenon. With reduction in modal mass for successive modes, critical damping will decrease with increase in mode. Overall damping of a system being a constant (since total mass and stiffness are constant for a system), the damping ratio will increase with increasing modes. To incorporate this reality, one option may be to compute the Rayleigh coefficients using the basic formulation incorporating range values for the first significant modes

$$\beta=(2\xi_1\omega_1-2\xi_m\omega_m)/(\omega_1^2-\omega_m^2);\alpha=(2\xi_1\omega_1-\beta\omega_1^2)$$

the modal damping ratio ξ for the soil system would typically be much higher than other civil engineering structures, say 15 to 20%.

6. Free Vibration Analysis

The free vibration analysis of the dam is performed in ANSYS 10 using the subspace method. The subspace method uses the subspace iteration technique, which internally uses the generalized Jacobi iteration algorithm. It is highly accurate because it uses the full [K] and [M] matrices. For the same reason, however, the subspace method is slower than the reduced method. This method is typically used in cases where high accuracy is required or where selecting master DOF is not practical.

The fundamental natural periods of the model in this research have been calculated to be1.090227sec for model without considering the shell-core interface. Considering a gap-friction element between the shell and core the fundamental natural period increased to 1.096972 sec.

7. Initial Static Analysis

When the vertical displacement (settlement) profile is observed within the core of the dam for linear static analysis under the self-weight load at the end of construction, an increasing trend of displacement from base towards crest until some depth below the crest is observed where maximum settlement has occurred and again decreased for the portion above it. The profile with similar pattern but higher magnitude is obtained when slippage is considered.

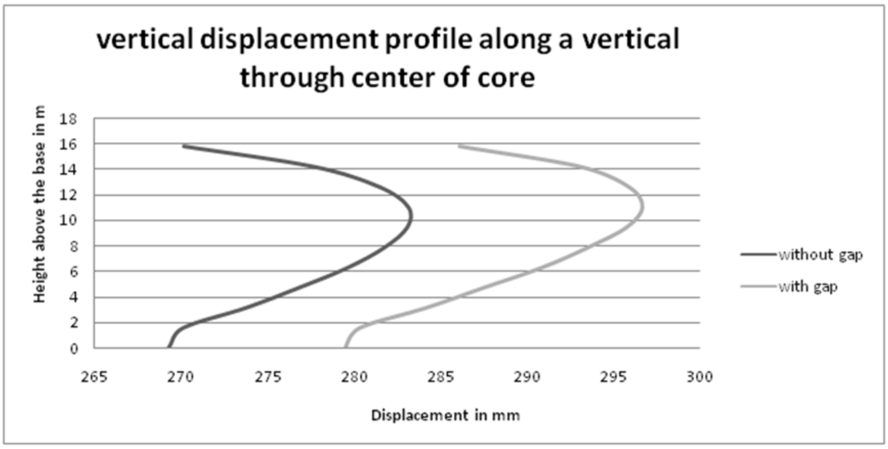

Figure 7.1: Vertical displacement profile along vertical

Figure 7.2: Vertical displacement profile along horizontal

8. Time History Analysis

The linear time history analysis is performed for both types of dam models excited by EL Centro earthquake. The step-by-step direct integration method based on Wilson-θ incremental algorithm, in built in SAP 2000 was used with θ=1.42 as mentioned in section. Rayleigh damping coefficients were computed based on the reference formulation. The damping range, as prevails for the geotechnical system, has been taken between 10 to 20% corresponding to first and the twelfth significant modes considered.

Table 8.1: Rayleigh damping coefficients

α	β
0.951925	0.005925

Table 8.2: Peak horizontal crest acceleration of the history

Model definition	Acceleration, in units of g
WOR	0.51
WR	0.50

Table 8.3: Permanent settlement result for Likhu hydropower dam model

Model definition	Permanent Crest settlement in, m
VC_WOR	0.55
IC_WOR	0.55

9. Conclusion

Under the assumed coefficient friction based on various literature surveys, it is found that the core settlement is considerably affected by the influence of core-shell interface with the increase in excess settlement of about 10-20 mm and also participates in the increase of natural time period. Parametric analysis of the dam also revealed that settlement within the core increases with the decrease in the coefficient of friction of the interface.

The result from time history analysis and permanent settlement from Newmark's chart showed insignificant difference for WOR and WR cases.

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