Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

SEISMIC STABILITY OF CONCRETE GRAVITY DAMS STRENGTHENED BY ROCKFILL

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ABSTRACT

Rockfill buttressing is often considered to strengthen existing gravity dams that have inadequate stability to resist the estimated hydrostatic and seismic loads. The rockfill stabilises the concrete dam by exerting "earth pressure" on the downstream face in opposition to the reservoir hydrostatic loads. Determination of magnitude of backfill pressure is a key point in seismic stability evaluation of composite dams. Simplified methods for seismic stability analysis of composite concrete-rockfill dams are discussed. Numerical analyses are performed using a nonlinear rockfill model and nonlinear dam-rockfill interface behavior to investigate the effects of backfill on dynamic response of composite dams. A typical 35 m concrete gravity dam, strengthened by rockfill buttressing is considered. The results of analyses confirm that backfill can improve the seismic stability of gravity dams by exerting pressure on the dam in opposition to hydrostatic loads. According to numerical analyses results, the backfill pressures vary during earthquake base excitations, and the inertia forces of the backfill are the main source for those variations. It is also shown that significant passive (or active) pressure cannot develop in composite dams with a finite backfill width. A simplified model is also proposed for dynamic analysis of composite dam by replacing the backfill with by a series of vertical cantilever shear beams connected to each other and to the dam by flexible links.

Introduction

Rockfill placed in embankment against existing concrete gravity dams has been adopted as a possible strengthening method to improve the stability of existing dams for hydrostatic or seismic loads. The rockfill stabilizes the dam by exerting earth pressure on the downstream face of dam. The horizontal earth pressure component is acting in the upstream direction, in opposition to the reservoir hydrostatic loads. The developed earth pressure is varying during the earthquake but its horizontal component generally remains in opposition to the hydrostatic force that helps the seismic stability of composite dam against downstream sliding. Figure 1 shows some examples of gravity dams strengthened by rockfill buttressing. Upper Glendevon gravity dam (Fig. 1a, Kerr 1995) in U.K. has been provided with a rockfill buttress on its downstream face to improve its seismic stability that was considered inadequate, especially in the presence of leaking vertical monolith joints and horizontal construction joints leading to significant uplift pressures. A 10mm thick bituminous slip membrane was installed at the concrete rockfill interface to reduce the interface friction angle and therefore maximize the horizontal thrust provided by the rockfill on the dam. The objective of the slip membrane was to reduce the tensile stresses at the heel of the dam. In seismic stability analysis of this composite dam, only static rockfill forces were considered and seismic variations of rockfill forces were neglected because of the different stiffness properties of the concrete and rockfill materials. The slope stability of embankment was verified separately for seismic stability.

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Rockfill placed on the downstream faces of Spullersee dams, two gravity dams of 38.4m and 27.6 m height in Austria (Bremen et al. 2004), was also used to replace the stability function of 40 year-old pre-stressed anchors (Fig. 1b). To assess the integrity of concrete dams during earthquake, a response spectra dynamic analysis was performed. The fill pressure was considered as added mass for the determination of the modal frequencies of the composite dam. The seismic stability analysis of the rockfill was carried out separately using a displacement approach by Newmark (1965).

Arya and Thakkar (1973,1977) investigated the feasibility of strengthening existing masonry dams for resisting earthquake forces by means of earthbacking on the downstream face of Bhatgar masonry dam in India. For dynamic analysis, they modeled the masonry dam portion and the earthbacking as two vertical cantilever beams connected by rigid links. Assuming that the contact between the two materials will be maintained during strong shaking, they found that at certain instant of time, the earth pressure may be lost near the top because of the presence of tensile stress at the dam-rockfill interface. The results of an experimental shake table study on a geometrically similar model showed that the effective viscous damping coefficient increase from ξ=3.5%, for the masonry dam only, to ξ=13.0% for the dam with backfill.

Chang and Oncul (2000) investigated the earthquake-induced separation along the fill-concrete interface of a hypothetical 30.48 m composite dam with earthfill at the downstream and upstream sides of the concrete dam. They used a non-linear dynamic analysis code NIKE3D, with Ramberg-Osgood non-linear material rockfill model and interface algorithms that allow separation and frictional sliding. The foundation was assumed as a fixed boundary condition and the concrete dam was assumed as elastic. According to the results, separation may occur along the concrete dam backfill upstream interface during earthquake.

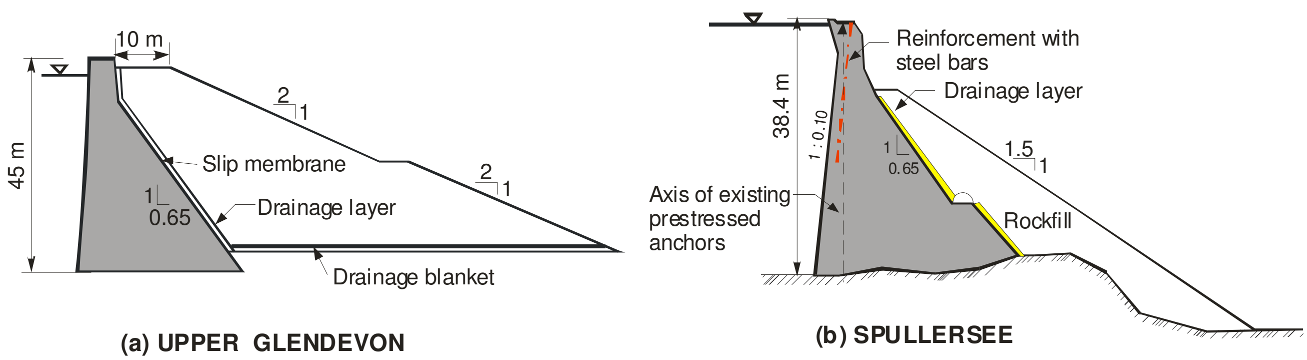

Figure 1. Example of composite gravity dams.

The lack of knowledge in the evaluation of seismic response of composite dam is obvious from different approaches used for design by practicing engineers. In this paper, first simplified methods for seismic analyses of composite dams are discussed. Then the nonlinear numerical modeling of composite dams using the computer program FLAC is explained. The dynamic response of a composite dam during a sinusoidal base excitation and the resulting stabilized condition at the end of the excitation is investigated. Analyses are performed to study the effects of various modeling parameters such as: frequency and friction angles of the concrete-fill interface in the response of a composite dam, its seismic sliding stability, and intensities of developed forces along backfill and dam interface in complement to previous investigations (Léger and Javanmardi 2006a). The analyses results are used to verify the validity of assumptions made in simplified methods used in the past and to present an improved simplified dynamic analysis method.

Structural Analysis and Failure Mechanisms of Composite Dams

Figure 2 shows the failure mechanisms of composite dams subjected to earthquakes. Failure surfaces could develop in the dam, in the rockfill, or in the foundation as well as at the interface between these components. A failure surface initiated in the dam could extend in the rockfill (Fig. 2 (11)). The top portion of the rockfill could settled in case of earthquakes (Fig. 2 (14)), or concrete-rockfill separation could occur at the interface (Fig. 2 (12)). A portion of the rockfill could also be washed out in case of overtopping during severe leakage caused by the cracking or separation of the top section and adequate protection (e.g. armouring) must be provided if this failure mechanism is possible.

Simplified analysis methods replace the action of rockfill by force resultants acting on the dam's downstream face. The stability of the dam is then assessed by the gravity method to estimate the safety against sliding, overturning and overstressing along horizontal failure planes. The Mononobe-Okabe method is widely used for simplified seismic stability analysis of retaining walls (ASCE 1994). This method is an extension of the Coulomb sliding-wedge theory taking into account horizontal and vertical inertia forces developed in the soil during earthquakes (Fig. 3a). It is assumed that the wall is free to yield sufficiently to enable active or passive conditions to be mobilized. The resultant active and passive backfill forces $\mathbf{R_{AE}}$ and $\mathbf{R_{PE}}$ include both the static and dynamic earth pressure components for a backfill of height h is:

$$\mathbf{R_{AE} = 0.5\ K_{ah}\ \gamma\ (1 - K_V)\ h^2 \quad ; \quad R_{PE} = 0.5\ K_{ph}\ \gamma\ (1 - K_V)\ h^2} \tag{1}$$

The formulas for the dynamic active $\mathbf{K_{ah}}$ and passive $\mathbf{K_{ph}}$ earth pressure coefficients as well as the vertical acceleration coefficient, **Kv** are given in (ASCE 1994). Alternatively, in a "general wedge earthquake analysis" method, the inertia forces is simply estimated by the mass of the rockfill wedge multiplied by the horizontal acceleration and active or passive pressure can be estimated by Coulomb formulas (ASCE 1994). In case of composite dams, the hydrostatic and uplift pressures as well as inertia forces of reservoir and dam should also be considered in the analysis. Application of Mononobe-Okabe method for the case of horizontal acceleration and passive state is shown in Fig. 3b. The angle $\boldsymbol{\beta}$ for this case should be considered negative in Mononobe-Okabe $\mathbf{K_{ah}}$ and $\mathbf{K_{ph}}$ formulas.

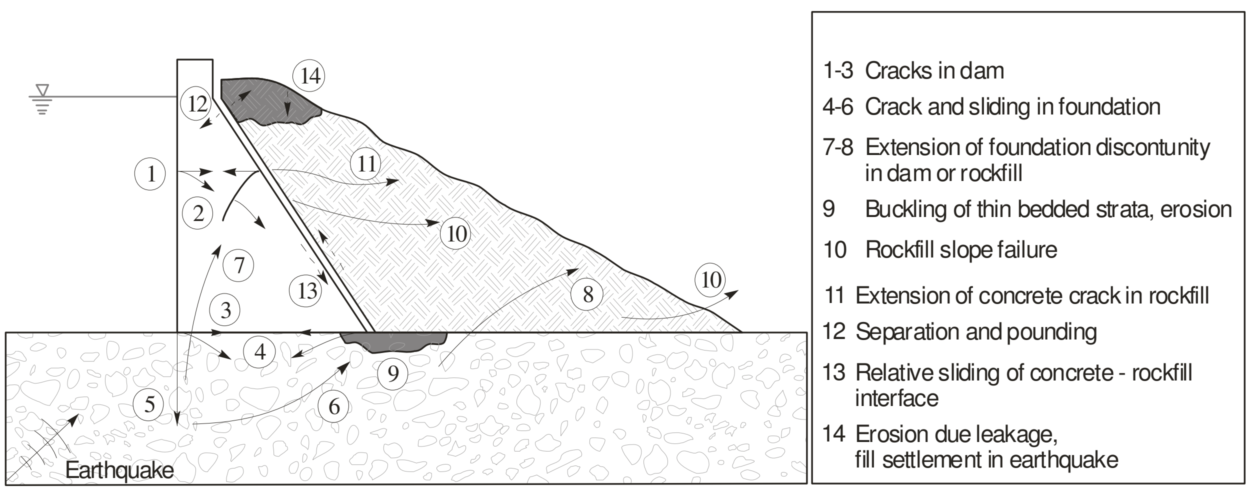

Figure 2. Failure mechanism of composite dams.

Once the action of the rockfill on the dam, R, has been estimated, the sliding safety factor along a horizontal plane in the dam is defined as:

$$\mathbf{SSF = [(W_C - U + R_V)\ (\tan \Phi_c) + (CA)]\ /\ (P_{hs}+P_{hd} - R_H+P)} \tag{2}$$

Where $\mathbf{W_C}$ = Weight of concrete above the failure plane ; **U** = Uplift force acting on the plane
$\mathbf{\Phi_c}$ = Concrete-concrete or concrete-foundation friction angle; **C** = Cohesion
A = Area on which cohesion could be developed; $\mathbf{P_{hs}}$ = Hydrostatic force
$\mathbf{R_V}$ = Vertical force resultant provided by the rockfill; $\mathbf{P_{hd}}$ = Hydrodynamic force
$\mathbf{R_H}$ = Horizontal force resultant provided by the rockfill

The horizontal force resultant provided by the rockfill, $\mathbf{R_H}$, is considered as an actively applied force. It is thus included in the denominator of Eq. (2).

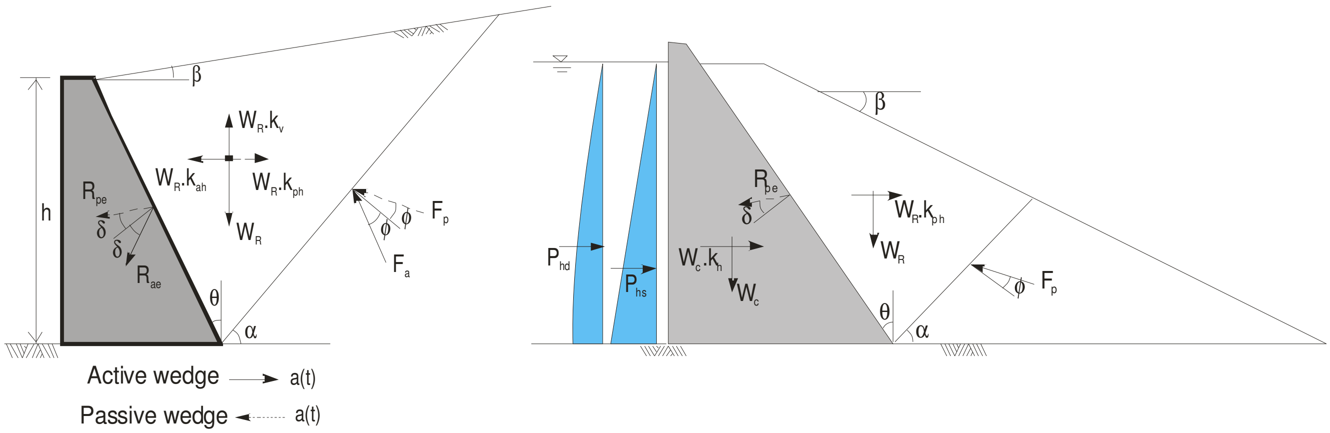

Figure 3. Mononobe-Okabe method.

System Analysed

To investigate the significance of backfill on the seismic response of composite dam, a typical 35 m gravity dam section subjected to sinusoidal base accelerations or earthquake records is first considered. The finite difference computer program FLAC was used for numerical analyses. FLAC allows modelling of large displacements, interface nonlinearities through gap-friction elements, and rockfill material nonlinearities. A plain strain model was used for the composite dam considered in this study.

Figure 4 shows the geometry of the dam and the grid geometry used in FLAC with a maximum grid size of 5m in the rockfill. The flexibility of the foundation was introduced by a "mass less" layer of rock. A nonlinear Mohr-Coulomb material model was used for the rockfill in all numerical simulations (elastic modulus = 100 MPa, mass density =1950kg/m^3) with a linear strength envelope defined using the internal rockfill friction angle. The interfaces where modelled using "no-tension" gap-friction elements. In this study the concrete section was considered elastic (elastic modulus = 24 000 MPa, mass density = 2400 kg/m^3) with nonlinearities (opening and /or sliding) confined to the interfaces I_1 to I_3 in Fig. 4. Normal and tangential stiffnesses of interface elements are assumed to be the same and equal to 100 GPa/m to model displacement continuity in joint closed condition. The friction angle is assumed equal to ϕ_c=45^o for the dam-foundation interface and δ=37^o for the dam-rockfill and rockfill-foundation interfaces. Modeling of additional horizontal masses (to be active in horizontal direction only) at the upstream face of the dam to represent Westergaard added mass is not explicitly possible in FLAC. A row of thin elements are added to the dam upstream face to generate these additional masses. The specific mass of these elements is computed such that the total mass of each element is equal to the Westergaard added mass at the elevation of element. The weight of these elements will affect the dam loading conditions and dam analysis results. A series of upward concentrated vertical loads, equal to the weight of these elements, are added at the location of added nodes to cancel the effect of additional weight (Fig. 4).

The seismic response of a cracked dam is also investigated by introducing a predefined horizontal joint (I4, in Fig. 4) in elevation 25 m. The same mechanical properties as joint I1 is assumed for joint I4, except the friction angle that is assumed 32^o for Joint I4.

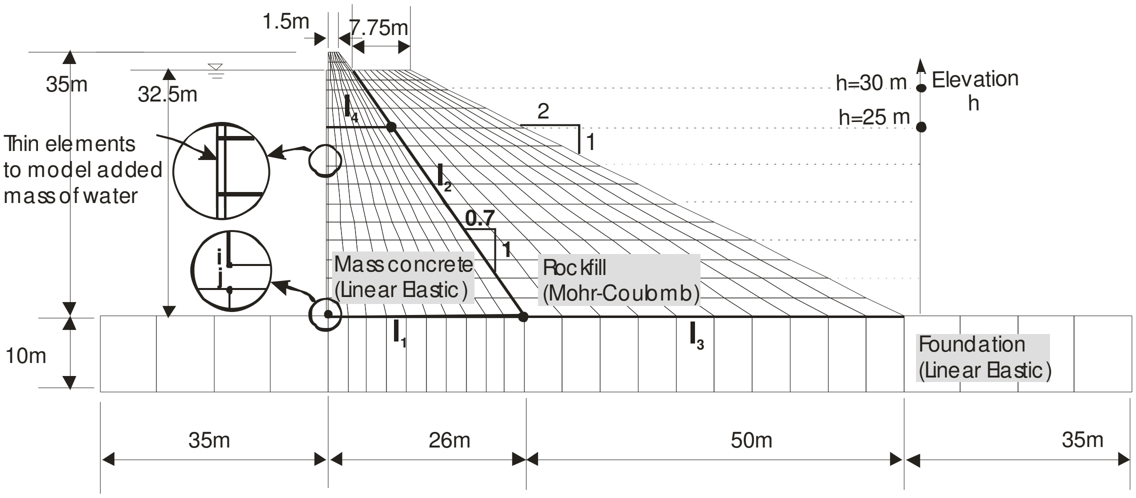

Figure 4. Geometry of system analysed and FLAC model.

Seismic Response of Composite Dam

The response of the composite dam to a 2 Hz sinusoidal base acceleration with peak ground acceleration (PGA) equal to 0.3g is first studied. The sinusoidal acceleration is applied for 5 s, but the analysis is continued for 2 s to study the static stability of the composite dam at the end of the base excitation. Results of these analyses show that the developed backfill thrust on the dam is not constant during earthquakes and changes with time but its horizontal component remains in opposition to the hydrostatic force that help the seismic stability of composite dam against downstream sliding. For example in case of 2 Hz sinusoidal acceleration with a PGA=0.3 g, no sliding occurs on composite dam-foundation interface (Fig. 5a). A similar dam without backfill, subjected to the same ground accelerations, slides more than 0.03 m in each period of excitation and the final sliding at the end of excitation is almost 0.24 m (Fig. 5a).

Investigation of rockfill deformation shows that lateral sliding of the rockfill along the dam downstream face is occurring during the base excitation. The magnitude of rockfill downward sliding is increasing with time and remains constant at the end of the excitation. Sliding of rockfill as well as nonlinear deformation in rockfill body causes permanent settlement of the backfill that increase its width at the middle height.

The case of dam with internal joint as shown in Fig. 4 is also investigated and compared to a dam without backfill. In case of dam without rockfill, joint downstream sliding occurs due to 2 Hz sinusoidal base excitation (Fig. 5 b). In case of dam with the backfill, the upper block of concrete slides more than 0.03 m, upstream, at the end of base excitation. While addition of backfill prevent dam base sliding in this case, it may causes upward sling along the existing joint in the dam body. The upstream base sliding may also occur in case of empty composite dam. The possibility for the upstream sliding should also be taking into account in seismic stability assessment of composite dams.

Active or Passive Pressure

Application of 2 Hz sinusoidal base acceleration with a doubled magnitude (PGA=0.6 g) compared to the previous case (PGA=0.3g) causes an accumulated total downward sliding equal to 76 mm at the end of the excitation. According to the passive pressure concept, the magnitude of backfill normal pressure at the end of the excitation should be larger than the initial static pressure due to permanent sliding of the dam that can mobilize some passive pressure state. The magnitude of backfill normal pressure at the end of the excitation is larger than the initial static pressure (compare "dynamic sliding=76 mm" in Fig. 6b, that shows the backfill normal stress distribution at the end of dynamic sliding, with the curve "no sliding", that shows the initial backfill normal stress distribution before base excitation).

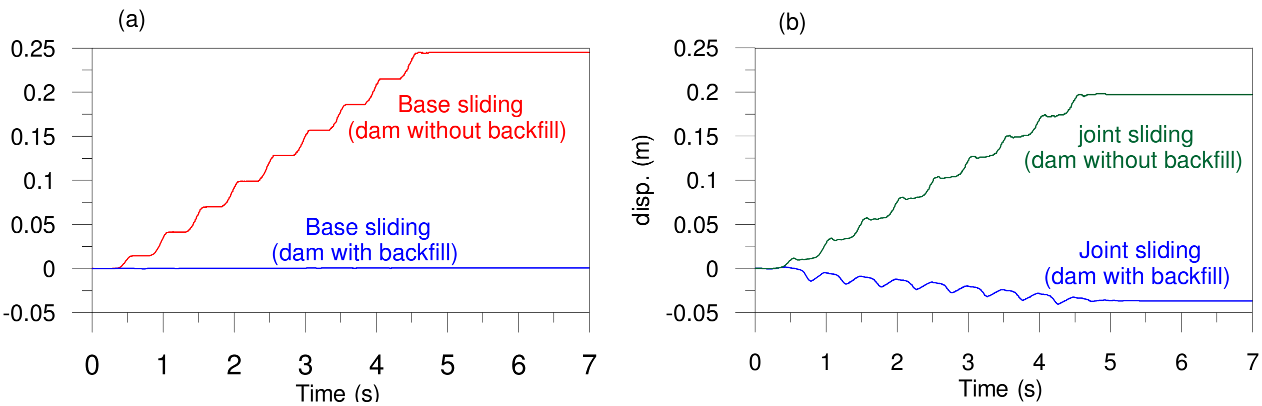

Figure 5. Seismic sliding of dam, a) dam without joint, b) dam with joint

It seems that larger pressure is associated to nonlinear deformation of the backfill and not to a significant passive state of stress. A static analysis was performed to compare the magnitude of backfill pressure after monotonic static sliding with the backfill pressure at the end of dynamic oscillatory sliding. The objective was to study the effect of cyclic sliding motions versus monotonic sliding motion. To induce the downward dam monotonic static sliding, the reservoir level was increased from its normal level, h=32.5 m, in 2.5 m steps and static analyses were performed to determine the dam sliding and the resultant backfill normal force in each step (without any concern for overtopping). Figure 6a shows the variations of the dam downward sliding and backfill normal resultant force versus reservoir elevations. The state of stress in the backfill can be categorized as at rest condition as long as reservoir level is below h=47.5 m and the magnitude of the total lateral force remains constant with no sliding.

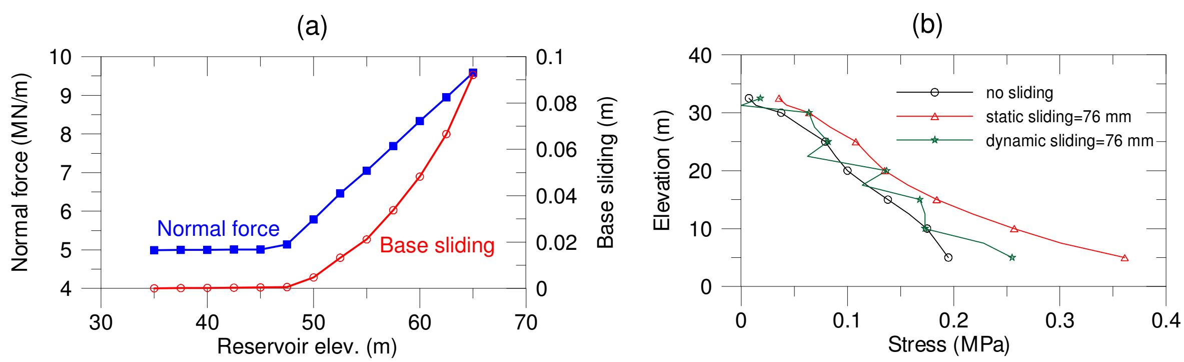

Figure 6. a) developing of passive force during dam static sliding, b) backfill normal stress distribution.

Increasing the reservoir level from h=47.5 m, in each 2.5 m loading step, leads to dam sliding due to increase in applied driving force. The increased rockfill force due to dam sliding brings it again to a stable equilibrium condition. Increase in backfill force confirms some mobilization of passive force in rockfill due to dam downstream sliding. According to Fig. 6a, a total sliding of 76 mm occurs when reservoir elevation is h=63 m. The normal backfill stress distributions along the dam-backfill interface when the water level is h=63 m (76 mm static sliding) is also shown in Fig. 6b. The differences between normal stress distributions after 76 mm static and dynamic sliding show the passive pressure due to 76 mm downward static sliding of the dam. Although the dam sliding displacements are equal to 76 mm in the static (hydrostatic overload) and dynamic cases, the normal pressure is smaller in the dynamic case. It can thus be concluded that the passive state can not be significantly developed during vibration of the backfill. The backfill transient deformation and settlement relieve the potentially mobilized passive pressure during dynamic excitation of composite dams. Application of the Mononobe-Okabe method for composite dam is not recommended because a passive or active force does not develop significantly. This is an important finding of this study.

It should be mentioned that this behavior is representative of composite dams where the dimension of the backfill is limited. In the case of a retaining wall this phenomenon may not occur because of the confined fill displacement due to the infinite dimension of the backfill.

Simplified Dynamic Analysis Method

In the proposed simplified seismic model, the rockfill is replaced by a series of cantilever shear beams, fixed at the base and connected to each other and to the dam by flexible links as shown in Fig. 6. Using several shear beams and distributing the rockfill mass between them allows for dynamic lateral deformation of rockfill relative to the dam. The number of shear beams is optional, but a minimum of 3 is recommended to simulate the backfill-dam relative motions during dynamic excitations as indicated from complimentary parametric analyses. To be consistent with the dam mesh, 13 nodes are assumed along the height of each beam at the elevation of each node of dam in Fig. 7. The lateral shear stiffness of the part of rockfill replaced by the beams can be used to determine the lateral stiffness of each beam element as:

$$\mathbf{K_C} = \mathbf{GA_S/H_C}\begin{bmatrix} \mathbf{1} & -\mathbf{1} \\ -\mathbf{1} & \mathbf{1} \end{bmatrix} \tag{3}$$

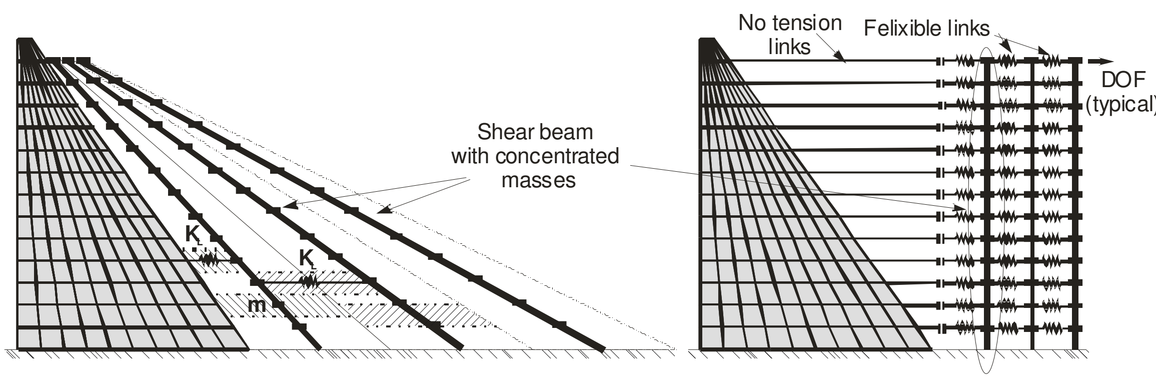

Figure 6. Simplified dynamic method.

Where **G** is the shear modulus of rockfill, $\mathbf{A_S}$ is shear area of the part of rockfill replaced by the beam, and $\mathbf{H_C}$ is vertical distance between nodes along the beam. The stiffness of each link element between two nodes is computed from axial stiffness of existing rockfill block between two nodes by:

$$\mathbf{K_L} = \mathbf{EA_C/L}\begin{bmatrix} \mathbf{1} & -\mathbf{1} \\ -\mathbf{1} & \mathbf{1} \end{bmatrix} \tag{4}$$

Where **E** is modulus of elasticity of the rockfill, $\mathbf{A_C}$ is the cross sectional area of the rockfill block between nodes, and **L** is horizontal distance between the nodes. The total mass of the rockfill is also concentrated at the location of each node according to its tributary area. The possibility for backfill-dam separation is considered by using no tension link elements to connect the first shear beam to the dam. Initial axial compressive forces in the no tension link elements computed from the tributary area of the element and existing static backfill pressure must be considered. Most commercially available structural analysis software with the capability of nonlinear link elements can be used to analyze the model. The SAP2000 program is used herein with damping equal to ξ=0.13 to compute the response of the simplified model subjected to the sinusoidal base accelerations with 3 different frequencies (2 Hz, 6 Hz, and 10 Hz). The natural frequency of the system was determined equal to 2.90 Hz, that is close to the frequency of backfill determined by the FLAC analysis (f=3.0 Hz). The computed backfill normal forces from SAP2000 (simplified) and FLAC nonlinear analyses are shown in Figs 8a,b,c.

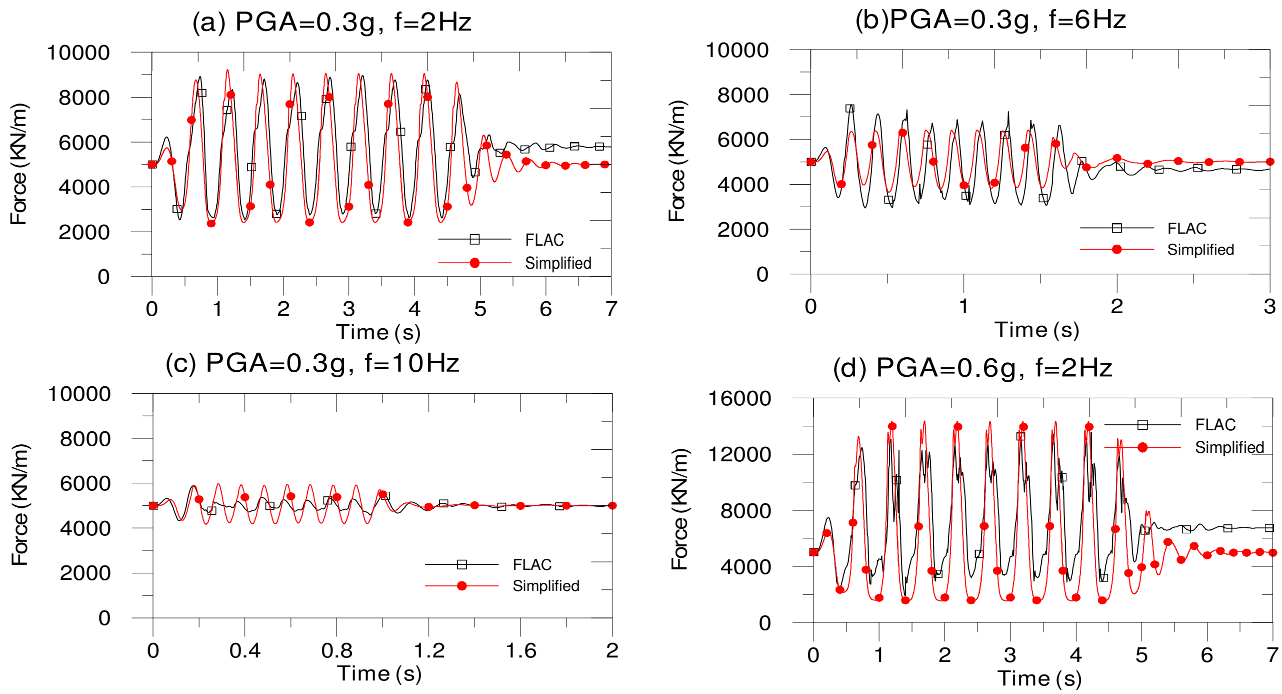

Figure 7. Comparison of FLAC and simplified method results.

The magnitude of existing static backfill pressures to be used as input in SAP2000 model, representative of the prevailing pre-seismic conditions, can be computed by representative simplified methods. FLAC static analysis results were used here for consistency in comparison of results. The backfill normal force variations from the simplified method are close to the FLAC results according to Figs 8a,b,c. The dominant nonlinear backfill deformations from FLAC analyses are the main sources for final differences between the results of FLAC and SAP2000 at the end of excitations. A case of base acceleration with 2 Hz and PGA=0.6g is also analyzed and the results are shown with FLAC results in Fig. 8d. The computed minimum dynamic force in each cycle and the final force at the end of excitation by the proposed method are smaller than FLAC results. This shows that the proposed simplified method becomes less accurate by increasing the degree of nonlinearity in the composite dam response. By increasing the base excitation intensity and modifying its characteristics (PGA, duration, number of cycle, dominant period), the dam base sliding and or rockfill nonlinear deformations increase. Using the simplified method is not accurate for these cases although it produces results on the conservative side (a small rockfill thrust value correspond to a smaller downstream sliding safety factor for the dam). According to the results the simplified method can be used confidently as long as the dam base sliding does not occur. The occurrence of dam base sliding can be verified by computing the dam sliding safety factor.

Parametric Analyses

The composite dam system is analyzed with two different dam-backfill friction angles $\delta=5^o$, $\delta=22^o$, in addition to the initially assumed $\delta=37^o$ to investigate the effect of a slip membrane at the dam-backfill interface. This construction feature was used at Upper Glendevon dam (Fig. 1a) as explained before. The results of analyses for two sinusoidal base accelerations with equal magnitude (PGA=0.3g) and different frequencies (2 Hz and 10 Hz) are summarized in Table 1. The dam-backfill friction angle was the only parameter considered in the parametric analyses. The backfill resultant force, its angle relative to normal to dam-rockfill interface, and sliding safety factors are shown in this Table. Comparison of static backfill resultant forces shows that it increases slightly with friction angle but its inclination (angle with normal to dam downstream face) decreases significantly by decreasing the friction angle. The horizontal component of resultant force will be larger in case of smaller friction angle that leads to greater static SSF as shown in Table 1.

Table 1. Parametric analyses; frequency of ground motions (2Hz vs 10Hz), dam-rockfill friction angle δ.

	Resultant force (MN/m)			**Resultant force angle**			**Sliding Safety Factor**		
PGA=0.3g, 2Hz	$R_{ini.}$	R_{min}	R_{max}	$\varphi_{ini.}$	φ_{min}	φ_{max}	$SSF_{ini.}$	$SSF_{d/s}$	$SSF_{u/s}$
$\delta=5^o$	5.1	2.6	8.4	5	-5	5	8.81	1.05	-1.65
$\delta=22^o$	5.4	2.8	8.4	22	-7	22	5.45	1.05	-1.65
$\delta=37^o$	5.9	2.6	8.9	33	-6	37	4.24	1.10	-1.65
PGA=0.3g, 10Hz									
$\delta=5^o$	5.1	4.6	5.3	5	0	4	8.81	3.00	-
$\delta=22^o$	5.4	4.7	5.5	22	15	20	5.45	2.75	-
$\delta=37^o$	5.9	5.6	6.0	33	29	34	4.24	2.40	-

In case of 2 Hz excitation, the minimum and maximum developed dynamic backfill force shows that range of resultant force variations are almost the same for different dam-backfill friction angle. Meanwhile, the maximum angles of resultant forces are different for different friction angles but it does not affect the minimum and maximum SSF significantly. It is concluded that decreasing the dam-backfill friction angle does not change the seismic sliding safety factor of dam during an earthquake. Investigation of rockfill deformation at the end of excitation shows that rockfill permanent deformations are smaller in case of smaller dam-backfill friction angle. Therefore using slip membrane can improve static sliding stability of composite dams but it does not change the seismic sliding stability of composite dam. However it improves rockfill response to seismic excitation by reducing its seismic permanent settlement. Comparing the results of 10 Hz with the results of 2 Hz reveals that the dynamic variations of backfill trust and SSF are considerably smaller in case of 10 Hz sinusoidal excitation. Therefore frequency of excitation can affect the seismic response of composite dams.

Simplified Pseudo-Dynamic Method

We have also investigated a simplified pseudo-dynamic method for practical applications. The normal inertia force to the downstream face is estimated by the mass of a rockfill wedge multiplied by the related normal spectral acceleration. The procedure to compute estimates of the maximum and minimum backfill force during the earthquake is described below:

i. Compute static at rest normal backfill force, $\mathbf{R^{STA}}$ by a simplified method (Léger and Javanmardi 2006b);
ii. Define the shape of the effective backfill wedge using a sloped line passing by the toe and making an angle equal to the internal friction angle of the rockfill with the horizontal;
iii. The normal component of the backfill force is computed by

$$\mathbf{R}^{\mathbf{TOT}} = \mathbf{R}^{\mathbf{STA}} \pm \mathbf{M}_{\mathbf{w}} \left[\mathbf{PS}_{\mathbf{A}}{}^{\mathbf{N}}(\omega_{\mathbf{b}})\right]\lambda\,; \qquad \text{with } \omega_{\mathbf{b}} = \frac{\pi}{2\mathbf{H}}\sqrt{\frac{\mathbf{G}}{\rho}}\sqrt{1+\frac{2}{1-\mu}\frac{\mathbf{H}^2}{\mathbf{L}^2}} \tag{5}$$

where $\mathbf{M_w}$ is the mass of the backfill wedge, $\mathbf{PS_A{}^N(\omega_b)}$ is the pseudo spectral acceleration of the ground motions evaluated at the natural circular frequency of the backfill (ω_b), and $\boldsymbol{\lambda}$ is a coefficient that account for the replacing of oscillatory variations of forces with constant static values. The recommended values for λ are 0.67 and 0.5 for the Western North America earthquakes and Eastern North America earthquakes, respectively. To evaluate $\boldsymbol{\omega_b}$ in Eq.5 (Wu and Finn 1999) **H** is defined as the maximum height of the backfill, **G** its shear modulus, $\boldsymbol{\rho}$ its mass density, $\boldsymbol{\mu}$ its Poisson's ratio and **L** is the effective length of the rockfill to have a rectangular area equal to that of the wedge. This approximation gives $\boldsymbol{\omega_b}$ = 14.5 rd/sec (2.3 Hz) as compared to the value of 18.8 rd/sec (3 Hz) obtained from detailed numerical analysis (FLAC). Considering harmonic ground accelerations at 2Hz we obtained conservative pseudo-dynamic estimate for $\mathbf{R^{TOT}}$ min. = 1.6 MN/m as compared to 2.6 MN/m for numerical analysis. However, the results are not on the conservative side for 6Hz ground acceleration with $\mathbf{R^{TOT}}$ min = 4.5 Mn/m as compared to 3.1 MN/m for numerical analysis. More work need to be done to obtain a pseudo-dynamic method that yields conservative estimates over a wide frequency range of ground excitations and rockfill natural frequencies.

Conclusions

Rockfill buttressing is often considered to strengthen existing gravity dams that have inadequate stability to resist the estimated hydrostatic and earthquake loads. Numerical analyses of a 35 m composite dam were performed using a nonlinear rockfill model and nonlinear dam-rockfill-foundation interface behavior to investigate the composite dam response during earthquakes. Seismic analyses show that: (i) Rockfill buttressing can prevent or reduce the dam downstream sliding displacements during earthquakes; (ii) Backfill plastic deformations occur during the earthquake and remain as permanent deformations at the end of the earthquake excitation; (iii) Significant active or passive state can not be mobilized in the backfill during vibration of the composite dam because of the limited extent of the backfill behind the concrete dam; the backfill deformation and settlement relieves the potentially mobilized active or passive pressure during dynamic excitation of composite dam foundation systems, (iv) Because significant active or passive pressures do not develop in composite dam system during earthquake excitation, application of Mononobe-Okabe method for simplified pseudo-static analysis of composite dams is not recommended.

A simplified dynamic method is proposed to consider an effective mass of backfill as concentrated masses that are connected to the dam by no tension link and shear beam elements. The computed dynamic backfill forces from proposed simplified method as compared to numerical analyses using a continuum nonlinear rockfill constitutive model are generally in good agreement as long as there is no dam base sliding corresponding to severe nonlinear response in the backfill. The objective of the rockfill is actually to prevent dam downstream sliding. The seismic stability of backfill should be investigated in a complementary study using appropriate methods for embankment dam analysis. The above conclusions are drawn under the assumptions of (i) a 2D plain strain analysis that cannot fully take account of the 3D effects expected in the behaviour of rockfill in narrow-shaped canyons, (ii) a horizontal ground motions, and (iii) a rigid foundation. The conclusions may not be generalized to all composite dams.

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