New multiscale numerical simulation approach to assess the seismic stability of gravity dams on rock foundations

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ABSTRACT

This work proposes a new Finite Element (FE) approach to assess the seismic stability of gravity dams while modeling the rock foundation roughness at various scales. First, an original technique is developed by implementing special Finite Elements to account for the effect of cohesion and irregular dam-rock interface geometries in practical sliding safety analyses. The proposed technique is then applied to investigate the seismic stability of an existing gravity dam with a stepped rock foundation. The results emphasize the significant effects of the mechanical and geometrical properties of dam-rock interfaces on the dynamic response of dams. However, difficulties are still associated with the selection of cohesion and friction coefficient values, mainly due to uncertainties related to the effects of roughness along dam-rock interfaces. Therefore, an original procedure is also developed in this paper to implement rough geometry of contact interfaces into non-linear FE models. This procedure is first experimentally validated using small scale shear-tested concrete-rock contact specimens drilled from existing dams. Then Bathymetric and LiDAR surveys of typical large rock foundation surfaces close to existing dam sites are processed to provide realistic rough dam-rock interface geometries. The generated profiles are implemented using the proposed procedure into non-linear FE models to conduct stability analyses of gravity dams. Overall, this paper presents a detailed investigation of the effects of rock foundation roughness on the shear strength of dam-rock interfaces and related dam seismic stability, an issue that has not attracted much attention in the literature.

Keywords: Dam-rock interface, Seismic stability, Finite Element Method, Cohesion, Roughness.

INTRODUCTION

Assessing the seismic stability of gravity dams, especially aging ones, has been a major research and engineering concern for the last decades. Dam-rock interfaces are generally considered as potential weakness planes where dam failure may be triggered during earthquakes, as shown in Fig. 1 (a) [1-8]. The outcome of a dam stability analysis might directly impact decisions relevant to its rehabilitation, generally associated with high costs and significant operational issues ([2] and [5]). Many guidelines recommend determining the Sliding Safety Factor SSF along the dam-rock interface using the Gravity Method (GM) coupled with the Mohr-Coulomb (M-C) criterion, defined by friction angle ϕ and cohesion c ([2] and [4-6]). According to this approach, the dam-rock interface must be simplified into weakness planes corresponding to possible failure modes, as shown in Fig. 1 (a). The M-C criterion has also been widely used in the literature to evaluate the seismic stability of concrete dams using the Finite Element (FE) Method [7-8]. However, lot of uncertainties are still associated with the selection of ϕ and c. Experimental investigations conducted on dam-rock interfaces show that the friction angle ϕ may vary from 28° to 68°, while the cohesion c can be comprised between 0 and 3 MPa ([1], [6] and [9-10]). Furthermore, due to high uncertainties associated with the presence of chemical links between rock and concrete, most guidelines assume an unbonded dam-rock interface with no cohesion c ([5-6] and [11]). As a result, modeling complexities and lack of experimental evidence motivate simplifying assumptions that are commonly adopted in practical seismic stability analyses of gravity dams. These primarily consist of: assuming fully unbonded dam-rock interfaces with null cohesion; neglecting the effects of large-scale irregular geometries of dam-rock interfaces; adopting the M-C criterion with no explicit consideration of roughness effects.

Previous experimental direct shear tests conducted on concrete-rock contact specimens, as shown in Fig. 1 (b) and (c), highlight that the shear strength τ_p of such interface is mainly controlled by: chemical cohesion due to the presence of a bond ([1], [6], [9-10] and [12]); small and large scale roughness [12-14]; applied normal stress σ_n for unbonded [13-14] and partially bonded [12] contacts; rock and concrete mechanical properties ([6] and [9]); and scale effects due to the size of the specimens [12]. Other research also highlighted the importance of large-scale irregular geometries of rock foundation surfaces on the stability of concrete dams [10-12]. Large-scale roughness can be observed from bathymetric and Light Detection and Ranging (LiDAR) surveys conducted by dam owners, although the implementation of these data into structural FE models is not yet a popular practice in dam engineering projects [15]. Moreover, extensive data on shear strength at dam joints is still needed. However, experimental investigations may require expensive and difficult in-situ testing handlings [10-12].

In rock mechanics, roughness is considered as a key factor in the evaluation of the shear strength τ_{p} of rock discontinuities [16-17]. Figure 1 (c) illustrates a typical non-linear failure criterion, such as the JRC-JCS model [16], representing the variation of measured peak shear strength as a function of applied normal stress σ_{n} during shear testing of an unbounded rock joint. This nonlinear failure curve can be linked to the classical M-C criterion by drawing straight lines tangent to the curve at various applied normal stresses, as illustrated in Fig. 1 (d). These lines provide an estimate of the apparent cohesion c' and friction angle ϕ' under a given normal stress σ_{n} [17]. Nevertheless, most of the available research addressing the shear behaviour of unbonded joints is related to mining and rock slope stability engineering applications, which involve a single material and, in some cases, high stress levels, as opposed to dam-rock interfaces ([11] and [13]). Moreover, only little published works have been dedicated to the numerical modeling of rough concrete-rock contact for dam applications [12]. Therefore, further research is still required to validate failure criteria or to recommend adequate values of apparent cohesion c' and friction angle ϕ' for the evaluation of the shear response of unbonded dam-rock interfaces.

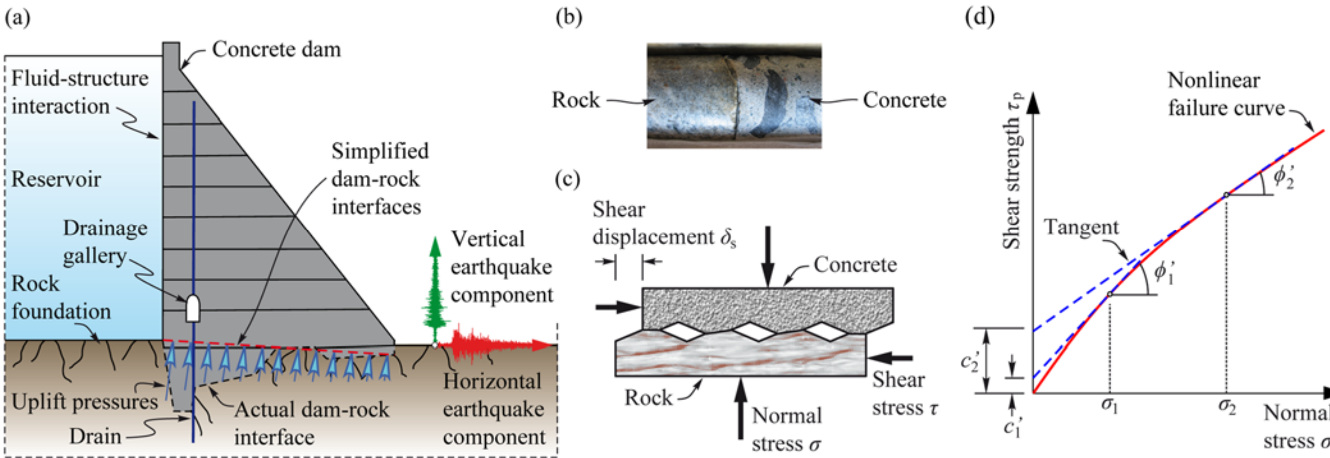

Figure 1. Unbonded rough concrete-rock interface: (a) located in a typical concrete gravity dam subjected to various loads, (b) drilled from an existing dam site, (c) shear tested, and (d) corresponding nonlinear shear strength.

In this paper, a new FE approach is proposed to assess the seismic stability of gravity dams while accounting for cohesion and roughness along dam-rock interfaces. First, an original technique is developed by implementing special non-linear Finite Elements to account for the effect of cohesion and irregular dam-rock interface geometries in practical sliding safety analyses. The proposed technique is then applied to investigate the seismic stability of an existing gravity dam with a stepped rock foundation. The results emphasize the significant effects of the mechanical and geometrical properties of the dam-rock interface on the dynamic response of dams. However, difficulties are still associated with the selection of cohesion and friction coefficient values, mainly due to uncertainties related to the effects of roughness along dam-rock interfaces. Therefore, a multi-scale FE procedure is developed to model the effect of rock foundation roughness on dam seismic stability. This procedure is first experimentally validated in the case of small-scale shear-tested concrete-rock contact specimens drilled from existing dams, then applied to model large-scale rough dam-rock interfaces. The corresponding irregular geometries are generated by processing Bathymetric and LiDAR surveys of typical rock foundation surfaces close to existing dam sites. Finally, values of apparent cohesion and friction angle are suggested to account for the effects of roughness in the M-C criterion. Overall, this paper presents original findings on the effects of cohesion and large-scale irregular geometries of dam-rock interfaces on dam seismic stability, an issue that has not attracted much attention in the literature.

PROPOSED TECHNIQUE FOR PRACTICAL DAM SEISMIC STABILITY ANALYSES

Treatment of nonlinearities along dam-rock interfaces

Two types of nonlinearities localized along a dam-rock interface must be modeled to investigate the seismic stability of a gravity dam: uplift and sliding. Denoting η the normal gap between the dam and the rock, and assuming that contacting normal stresses σ are positive in compression, the uplift conditions between the dam and the foundation can be expressed as

$$\eta\,\sigma = 0; \qquad \sigma \geq 0; \qquad \eta \geq 0; \tag{1}$$

Therefore, full contact can be expressed as $\eta = 0$ and $\sigma > 0$ while uplift at dam-rock interface occurs as soon as $\eta > 0$ and $\sigma = 0$. The sliding conditions along a dam-rock interface can be divided into two categories: non-cohesive and cohesive. The non-cohesive sliding condition is solely controlled by friction. Denoting τ the shear stress at contact, the ratio ζ_0 is introduced

$$\zeta_0 = \frac{\tau}{\sigma\,\tan(\phi)} \tag{2}$$

The non-cohesive condition along a dam-rock interface is governed by the M-C criterion: sliding occurs as soon as $|\zeta_0| \geq 1$. To account for the influence of cohesion c, the cohesive sliding conditions are defined by the parameters

$$\zeta_c = \frac{\tau}{\sigma\,\tan(\phi) + c} \tag{3}$$

and

$$\zeta = \begin{cases} \zeta_c \text{ as long as } |\zeta_c| < 1 \\ \quad \zeta_0 \text{ otherwise} \end{cases} \tag{4}$$

During the ground motion, the shear stresses along the dam-rock interface increase gradually. This process continues until the shear fracture of cohesive bonds along the interface occurs when $|\zeta| \geq 1$, which triggers the sliding motion. Eq. (4) implies that if $|\zeta_c|$ becomes larger than unity at an instant t_1 corresponding to fracture of cohesive bonds, then $\zeta = \zeta_0$ during the rest of dam motion for $t > t_1$. Therefore, the degradation of the area over which cohesion is effective can be tracked during the seism. The presence of a bond may also involve the presence of substantial tensile strength f_{t}. Detailed methodology to model f_{t} is provided in [8]. Although some coupling between cohesion and tensile strength at dam-rock interfaces has been observed ([1] and [12]), the modeling approach developed here assumes that these two parameters are independent to give the analyst more freedom in specifying most appropriate values for the dam being studied.

Finite Element modeling of partially bonded interfaces

Special attention is paid in this work to the explicit modeling of irregular geometries along dam-rock interfaces using the FEM. Uplift and non-cohesive sliding conditions along dam-rock interfaces are implemented through basic frictional Finite contact Elements. The gravity dam is considered as the contactor block which can slide or rock over the target block, represented by the rock foundation. Contactor and target interfaces are created to simulate contact conditions. The contact algorithm used is capable of detecting contact points during motion, i.e. they are assumed not known a priori, and also of preventing penetration of segments after each contact. As shown in Fig. 2 (a), two series of N_I coincident nodes are created on each interface: (i) nodes n_i^{D} and n_i^{F}, $i = 1 \ldots N_I$, at contactor and target interfaces, respectively. To model cohesion, a new interface Truss Element for Cohesion modeling, denoted hereafter as TEC, is introduced. For this purpose, new nodes $\bar{n}_i^{\mathrm{F}}$, $i = 1 \ldots N_I$, are created, as TECs connect nodes n_i^{F} to nodes $\bar{n}_i^{\mathrm{F}}$, as shown in Fig. 2 (a). Nodes $\bar{n}_i^{\mathrm{F}}$ are positioned and constrained to have the same displacements parallel to the dam-rock interface as node n_i^{D} and perpendicularly to the dam-rock interface as node n_i^{F}, thus ensuring that TECs remain parallel to dam-rock interface during sliding. Truss elements are adopted as they constitute basic elements available on most FE software. TECs are characterized by their rigidity $k_i^{(\mathrm{TEC})}$ and are implemented using bilinear materials including a rupture option which triggers the disappearance of the elements once maximum the elastic strength is reached. $\Delta_i^{(\mathrm{TEC})}$ denotes the relative displacements of node n_i^{D} with respect to node n_i^{F} along the parallel direction of the dam-rock interface, as illustrated in Fig. 2 (b). As soon as $|\zeta_0| \geq 1$ is satisfied, non-cohesive sliding controlled by the basic contact elements occurs and each node n_i^{D} moves from node n_i^{F} by a distance of $\Delta_i^{(\mathrm{TEC})}$ as shown in Fig. 2 (c). Each TEC sustains a shear force $F_i^{(\mathrm{TEC})}$, given by

$$F_i^{(\mathrm{TEC})} = k_i^{(\mathrm{TEC})} \Delta_i^{(\mathrm{TEC})} = [\tau - \sigma\,\tan(\phi)]\,S_i \tag{5}$$

where S_i denotes for the tributary area associated with each node i. Using Eq. (5), the following equation can be obtained

$$k_i^{(\mathrm{TEC})} \Delta_{\max}^{(\mathrm{TEC})} / S_i = c \tag{6}$$

where $\Delta_{\max}^{(\mathrm{TEC})}$ denotes the maximum displacement that can be sustained by the TECs. $\Delta_{\max}^{(\mathrm{TEC})}$ can also be interpreted as the displacement required to reach the peak shear strength of the dam-rock interface. A verification example of the proposed numerical procedure is provided in [8].

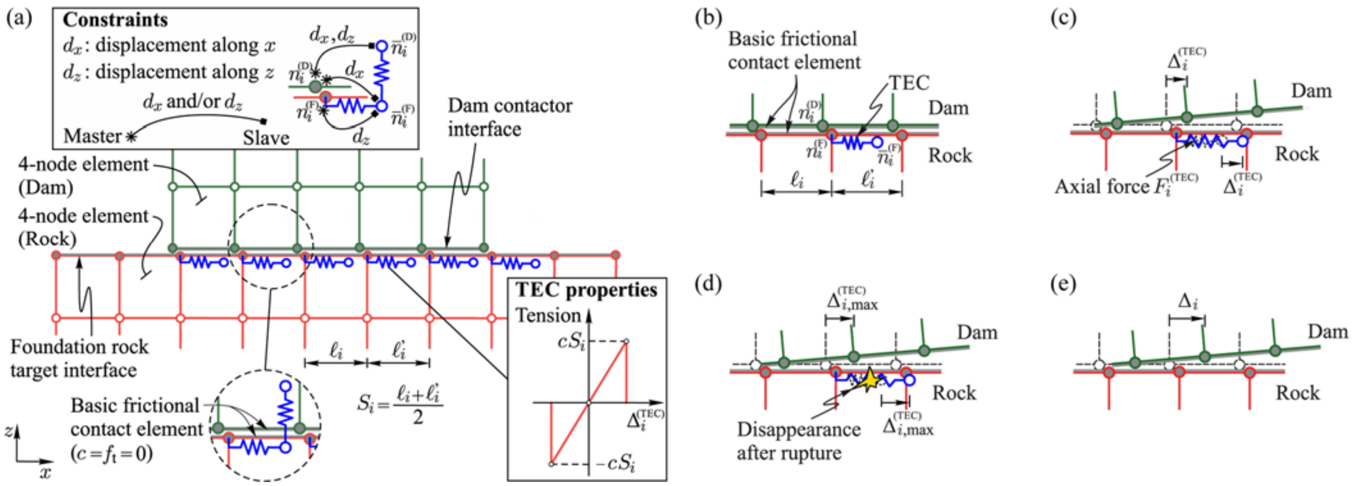

Figure 2. Proposed TEC to model cohesion along dam-rock interface: (a) Properties; (b) Bonded conditions; (c) The basic contact elements strength is exceeded; (d) The TEC disappears as its strength is exceeded; (e) Post-peak sliding motion.

APPLICATION TO AN EXISTING GRAVITY DAM

Dam studied, simplified geometry variants and applied loads

In this section, the methodology presented above is applied to investigate the effects of the mechanical and geometrical properties of the dam-rock interface on the seismic response of an actual concrete gravity dam with a stepped rock foundation shown in Fig. 3 (a) located in Québec. Two simplified dam sections are also considered, as illustrated in Figs. 3 (b) and (c). For brevity of notation, the dam with the actual stepped rock foundation is designated by $D^{(S)}$, and the dams with simplified dam-rock interfaces modeled as an inclined and a horizontal planes are designated by $D^{(I)}$ and $D^{(H)}$, respectively. Simplified dam-rock interfaces are required when assessing dam seismic stability using the GM. The principal sliding directions for dams $D^{(S)}$, $D^{(I)}$ and $D^{(H)}$ are horizontal, inclined, and horizontal, respectively. A modulus of elasticity $E_{c1} = 21.375$ GPa is considered for all the dam structure, except the area around the drainage gallery where a modulus of elasticity $E_{c2} = 22.775$ GPa is used to account for the presence of steel reinforcement. A Poisson's ratio $\nu_c = 0.164$ and a density $\rho_c = 2295$ kg/m^3 are also adopted for the dam concrete. These concrete mechanical properties were obtained experimentally for the actual dam studied. A mass density $\rho_w = 1000$ kg/m^3 is considered for water. The horizontal and vertical components of the Imperial Valley ground motion (1940) at station El Centro are considered in this work. A Rayleigh damping equivalent to a viscous damping $\xi = 5\%$ is adopted for the concrete dam. Uplift pressures along the dam-rock interfaces are determined according to USACE [2]. A drain is located at 6 m from the upstream dam face with efficiency of 66.7 %.

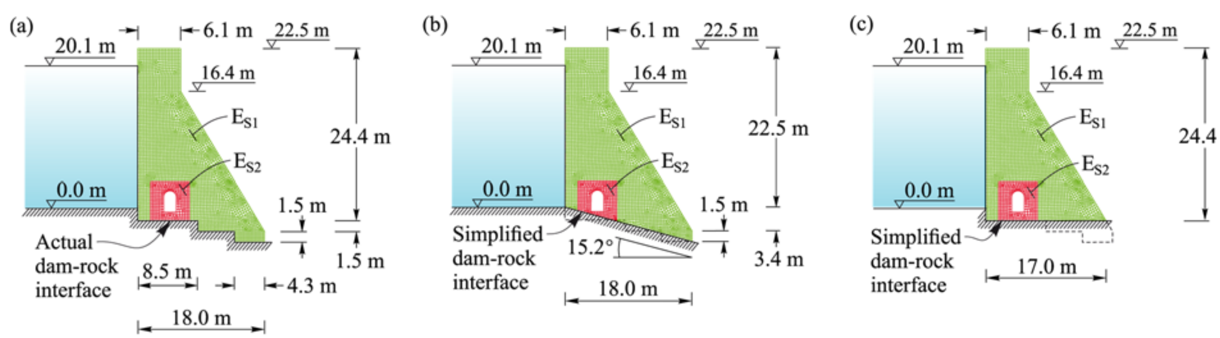

Figure 3. Studied gravity dams: (a) dam $D^{(S)}$ with stepped dam-rock interface; (b) dam $D^{(I)}$ with simplified inclined dam-rock interface; and (c) $D^{(H)}$ with simplified horizontal dam-rock interface.

Seismic stability analyses

Pseudo-dynamic and non-linear dynamic analyses are conducted to assess the seismic stability of the studied gravity dams. Pseudo-dynamic analyses are carried out using the software CADAM [4] which combines the GM with the simplified method of Fenves and Chopra [18] to determine the *SSF* along dam-rock interfaces. As previously mentioned, the GM can only be used to assess the seismic stability of monoliths $D^{(I)}$ and $D^{(H)}$ with simplified dam-rock interfaces. This approach considers dynamic amplification due to the dam structural flexibility, water compressibility and fluid-structure interaction using the fundamental mode and the design spectrum of the dam-reservoir system assessed. The horizontal spectral acceleration corresponding to the fundamental period of the studied dam-reservoir system, $T_{\text{s}} = 0.081$ s, is $a_{\text{s}} = 0.65$ g. However, the dam is submitted to sustained acceleration equal to $2/3\ a_{\text{s}}$ as longer seismic loads are required to trigger sliding along dam-rock interfaces [3]. The vertical seismic load is multiplied by 0.3 to account for the probable non-simultaneous occurrence of horizontal and vertical peak accelerations. The SSF obtained with dams $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ with pseudo-dynamic analyses are presented in Table 1 with various friction angles ϕ and cohesions c corresponding to available values in the literature ([1], [6] and [9-10]). f_{t} is assumed negligible in this work to isolate the effects of cohesion. It can be seen that the SSF obtained for dams $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ can substantially differ. Moreover, the values of ϕ and c are as critical for dam stability assessment as dam-rock geometry.

Table 1. SSF obtained with dams $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ through pseudo-dynamic analyses.

Dam Monolith	$\phi = 35°$ $c = 0.01$ MPa	$\phi = 35°$ $c = 0.5$ MPa	$\phi = 35°$ $c = 1$ MPa	$\phi = 55°$ $c = 0.01$ MPa	$\phi = 55°$ $c = 0.5$ MPa	$\phi = 55°$ $c = 1$ MPa
$\text{D}^{(\text{I})}$	0.605	1.845	3.110	1.208	2.448	3.712
$\text{D}^{(\text{H})}$	0.944	2.470	4.028	1.893	3.419	4.977

Non-linear dynamic analyses are conducted next using the FE code ADINA [19]. The dam and rock foundation are modeled using 4-nodes plane strain Finite Elements. Basic frictional contact elements programmed in ADINA [19] are used to model non-cohesive sliding and uplift conditions along the dam-rock interface. The cohesion is modeled using the TECs previously developed with the conservative value of $\Delta_{\text{max}}^{(\text{TEC})} = 0.2$ mm [12]. Prior to earthquake loads, the dams are subjected to static gravity loads, uplift and hydrostatic pressures applied gradually through a ramp. Since non-linear analyses may induce large displacements in the reservoir, the Westergaard added-masses formulation is preferred to model the hydrodynamic effects. As recommended by the USACE [2], uplift-pressures are assumed constant during earthquake shaking, independently of possible cracking. The foundation is assumed massless and infinitely rigid in the non-linear dynamic models.

Denoting L_{p} the length along the dam-rock interface between the heel of the dam and the point of interest, Figures 8 (a), (b) and (c) illustrate the stress distributions σ_t tangential to the principal sliding direction determined along the dam-rock interfaces of $\text{D}^{(\text{S})}$, $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$, respectively, subjected to Imperial Valley (1940) ground motion with $\phi = 55°$ and $c = 0.01$ MPa. These figures highlight the high sensitivity of stress distributions to the geometry of dam-rock interfaces. Indeed, the distributions of σ_t along the simplified dam-rock interfaces of dams $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ are clearly different from the one along the stepped dam-rock interface of dam $\text{D}^{(\text{S})}$. Table 2 present the horizontal residual displacement d_{r} obtained at the heel of dams $\text{D}^{(\text{S})}$, $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ with values of ϕ and c considered identical as in Table 1. It is shown that d_{r} for dam $\text{D}^{(\text{I})}$ is substantially larger than for dams $\text{D}^{(\text{S})}$ and $\text{D}^{(\text{H})}$ with $c = 0.01$ MPa. Although the values of d_{r} show some differences between dams $\text{D}^{(\text{S})}$ and $\text{D}^{(\text{H})}$, the results suggest that the horizontal dam-rock interface is best suited as a simplification to the initial stepped dam-rock foundation than the inclined interface. For $c \geq 0.5$ MPa, no significant residual displacement occurs, thus demonstrating the importance of cohesion for the seismic stability analyses conducted along dam-rock interfaces.

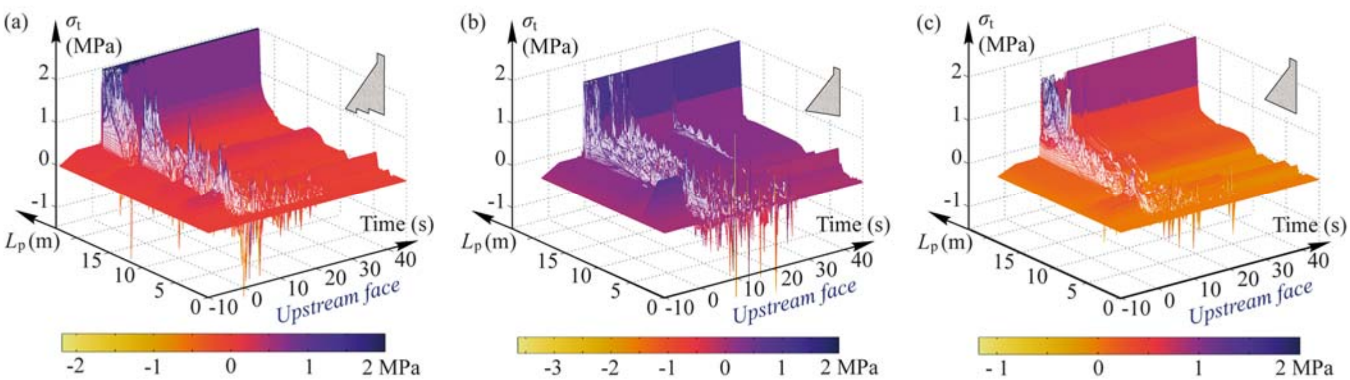

Figure 4. Tangential stress to the principal sliding direction σ_t obtained through non-linear FE dynamic analyses along the interfaces of: (a) dam $D^{(S)}$; (b) dam $D^{(I)}$; and (c) $D^{(H)}$.

Table 2. Residual displacement d_r (mm) obtained with dams $\text{D}^{(\text{S})}$, $\text{D}^{(\text{I})}$ and $\text{D}^{(\text{H})}$ through non-linear FE dynamic analyses.

Dam Monolith	$\phi = 35°$ $c = 0.01$ MPa	$\phi = 35°$ $c = 0.5$ MPa	$\phi = 35°$ $c = 1$ MPa	$\phi = 55°$ $c = 0.01$ MPa	$\phi = 55°$ $c = 0.5$ MPa	$\phi = 55°$ $c = 1$ MPa
$\text{D}^{(\text{S})}$	84	0.1	0.1	8.8	0.5	0.1
$\text{D}^{(\text{I})}$	–*	0.1	0.1	25.4	0.1	0.1
$\text{D}^{(\text{H})}$	143.8	0.1	0.1	3.9	0.1	0.1

–: Dam unstable under static gravity loads.*

MULTISCALE APPROACH TO MODEL ROUGH UNBONDED DAM-ROCK INTERFACES

Difficulties are still associated with the selection of cohesion and friction coefficient values, mainly due to uncertainties related to the effects of roughness along dam-rock interfaces. Therefore, an original procedure is introduced in this section to model the shear response of rough concrete-rock contacts using non-linear FE analyses. These interfaces are assumed unbonded with no cohesion and are implemented in the FE code ADINA [19] through basic frictional contact elements whose corresponding uplift and non-cohesive sliding conditions are already described in Eqs. (1) and (2). The assumption of an unbonded contact is motivated by the need to isolate the effects of chemical cohesion from those of roughness. The irregular geometry of a concrete-rock contact can be distinguished into primary and secondary roughness, corresponding to larger and much smaller asperities, respectively. Primary roughness is simulated through direct modeling of the irregular geometry of concrete-rock contacts, while the basic friction angle ϕ_b implemented into the basic contact elements is used to account for the effects of secondary roughness, as shown in Fig. 5 (a). Material elasto-plasticity is also introduced along the contact to account for the crushing of asperities due to the shearing process [15]. Therefore, a compressive strength $f_c = 30$ MPa is considered for rock and concrete. The following material properties are also considered for the mass rock in the multiscale approach: a Poisson's ratio $\nu_r = 0.3$, a mass density $\rho_r = 2400$ kg/m^3 and a modulus of elasticity $E_r = 40$ GPa corresponding to in-situ mechanical properties of dam foundations. In order to get realistic contact geometries, six small-scale concrete-rock contact specimens drilled from existing gravity dams are scanned, and four large-scale Bathymetric and LiDAR surveys are carried out on typical large rock foundation surfaces close to existing dam sites. Such profiles are cleaned and filtered as described in [14-15] as they initially consist of clouds of irregularly-spaced points which comprise noise and irrelevant data.

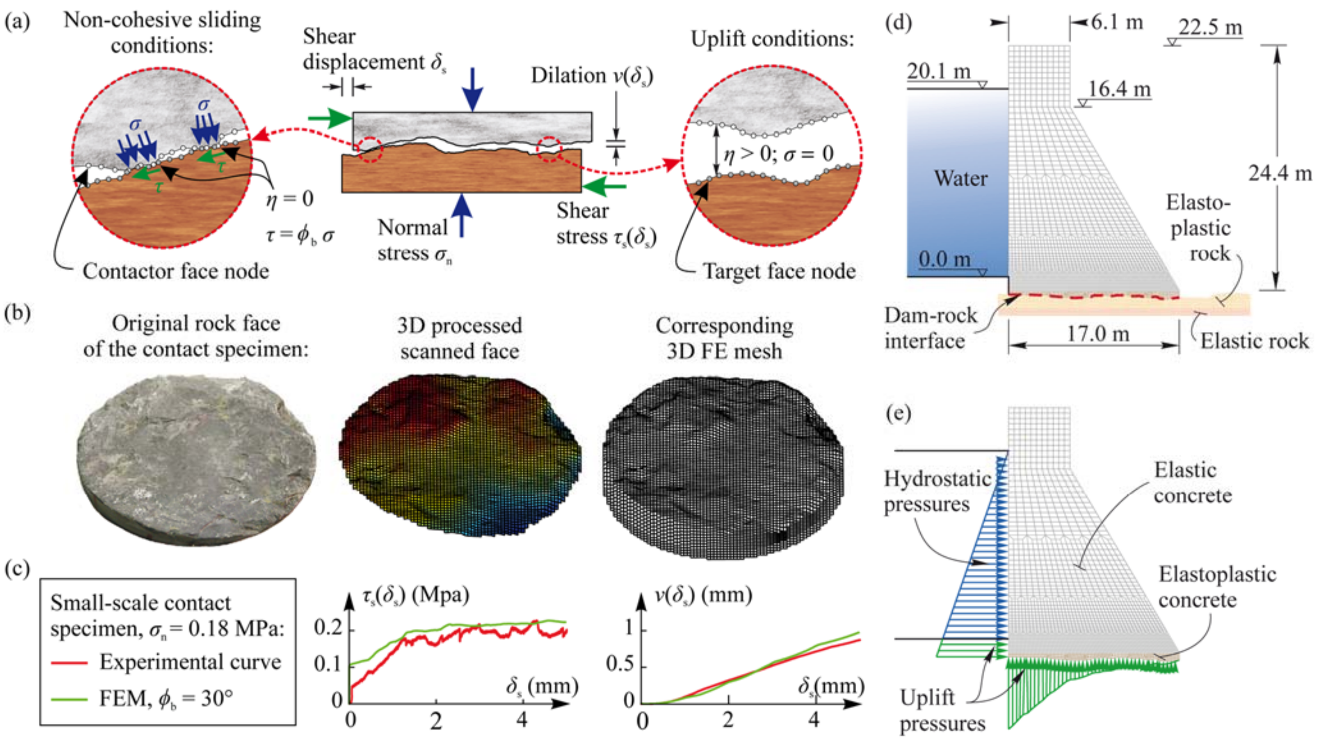

Figure 5. Multiscale non-linear FE modeling of rough concrete-rock contact: (a) Contact conditions along a sheared interface; (b) Implementation of scanned rock surface in 3D FE mesh; (c) Experimental validation with small-scale contact specimen; (d) Implementation of Bathymetric and LiDAR surveys in 2D FE model of dams with (e) corresponding loads.

As shown in Figs. 5 (a) and (b), 3D non-linear FE models of shear-tested concrete-rock contacts are constructed using the six scanned small-scale contact specimens whose diameters are comprised between 80 and 145 mm. The inferior block of the modeled interface remains fixed. The superior block is subjected to a constant normal stress σ_n and a shear displacement δ_s causing a shear stress $\tau_s(\delta_s)$ and a vertical displacement $v(\delta_s)$, as shown in Fig. 6 (a). Detailed analysis of the experimental shear test results can be found in [14]. The experimental data is then compared to the numerical shear response of the studied contact specimens, as shown in Fig. 6 (c). The FE analyses provide reasonable predictions of $\tau_s(\delta_s)$ and $v(\delta_s)$ compared to the trends followed by the experimental results, especially with $\phi_b = 30°$, thus validating at small scale the proposed numerical procedure. The friction angle ϕ of the concrete-rock contact specimens are also numerically and experimentally found to vary from 40° to 60° depending their roughness properties. However, such values should not be directly considered in dam seismic stability due to the associated significant variability and probable scale effect.

Figure 6 presents the four 2D dam-rock interface profiles P1, P2, P3 and P4 obtained based on 3D rock surfaces resulting from bathymetric and LiDAR surveys. A plane horizontal rock surface profile, denoted as P0, is also considered for comparison purposes. Such profiles are used to construct the five 2D non-linear FE models of gravity dams, as illustrated in Fig. 5 (d), with the same geometry than the studied dam introduced in Fig. 3. For each dam-rock model, static sliding stability analyses are conducted under the effects of the static gravity loads, uplift and hydrostatic pressures, as shown in Fig. 5 (e), to determine the failure friction angle ϕ_f for which sliding occurs. A threshold displacement of 1 mm at the dam toe is considered as the maximum acceptable displacement and therefore a trigger for failure.

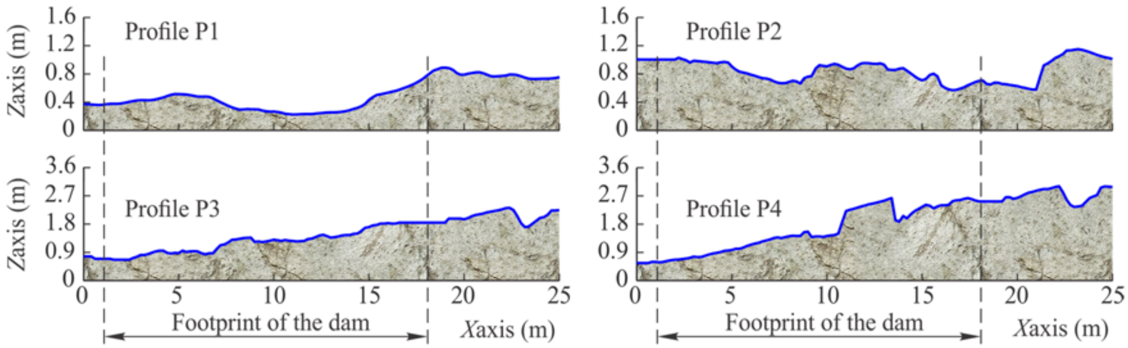

Figure 6. 2D realistic dam-rock interface profiles with various large-scale roughness.

It can be seen in Table 3 that a substantial higher ϕ_f is found for the plane horizontal Profile P0 than for the other rough profiles. Therefore, large scale rock surface roughness, even relatively small, can significantly increase the shear strength of dam-rock interfaces. The shear strength τ_p of an unbonded rough joint can be predicted by the JRC-JCS model [14] as

$$\tau_\mathrm{p} = \sigma_\mathrm{n} \tan[\phi_\mathrm{r} + JRC \log(JCS/\sigma_\mathrm{n})] \tag{7}$$

where JRC, JCS and ϕ_r denote for the Joint Roughness Coefficient, Joint Compressive Strength and the residual friction angle, respectively. It is interesting to note that the FE models provide the average normal and shear stresses $\bar{\sigma}$ and $\bar{\tau}$ along dam-rock interfaces while the JCS can be taken as f_c. When dam failure occurs, it is notable that $\phi_\mathrm{f} = \phi_\mathrm{r}$, $\bar{\sigma} = \sigma_\mathrm{n}$ and $\bar{\tau} = \tau_\mathrm{p}$. Therefore, the JRC for each large-scale profile is estimated in Table 3 by combining these equalities with Eq. (7). In the case of dam-rock interfaces, the primary and secondary roughness can be linked to the irregular geometry of large-scale profiles, and the friction angle ϕ of small-scale concrete-rock contacts, respectively. Thus, a value of $\phi_\mathrm{r} = 40°$ experimentally found conservative is selected, and obtained values of JRC can be used in Eq.(7) to predict τ_p as a function of σ_n for each dam-rock interface. The obtained five non-linear failure envelops can be linked to the classical M-C criterion through linear regressions. These regressions are carried out under the range of normal stresses found along dam-rock interfaces, and provide the values of apparent cohesion c' and friction angle ϕ' shown in Table 3. Finally, the seismic stability of the original existing dam, presented in Fig. 3, is investigated through pseudo-dynamic analyses conducted with the GM and the simplified method of Fenves and Chopra [18]. Although the GM requires to simplify the dam-rock interface into a unique weakness plane, effects of large-scale roughness are accounted with the proposed approach using the determined apparent cohesion c' and friction angle ϕ'. The simplified geometry variant $\mathrm{D}^{(\mathrm{H})}$ is used since it is best suited as a simplification to $\mathrm{D}^{(\mathrm{S})}$ with initial stepped dam-rock foundation than $\mathrm{D}^{(\mathrm{I})}$. The obtained SSF are presented in Table 3. The results clearly emphasize the major role of the dam-rock interface geometric irregularities on the dam seismic stability. Ongoing research is focusing on the verification of such results through dynamic non-linear analyses using TECs to model apparent cohesion.

Table 3. Effect of the large scale roughness on the dam-rock shear strength and the seismic stability of the studied dam.

	Profile P0	Profile P1	Profile P2	Profile P3	Profile P4
ϕ_f	23.8°	18.0°	10.8°	6.1°	–*
JRC	0	2.96	6.39	9.20	≥ 12.63
c' (MPa)	0	0.012	0.034	0.067	≥ 0.148
ϕ'	40°	44.8°	50.2°	54.4°	≥ 59.0°
SSF (dam $\mathrm{D}^{(\mathrm{H})}$)	0.947	1.332	1.670	2.029	≥ 2.631

–: No failure occurs when $\phi = 0$.*

CONCLUSION

This work proposed a new multiscale Finite Element (FE) approach to assess the seismic stability of gravity dams while modeling the rock foundation roughness at various scales. First, an original technique was developed by implementing special Finite Elements to account for the effect of cohesion and irregular dam-rock interface geometries in practical sliding safety analyses. For illustration purposes, the proposed methodology was applied to an existing concrete dam subjected to strong ground motions. It is shown that simplified dam-rock geometries, commonly adopted for practical modeling purposes, should be used with caution as they may lead to inaccurate results. Moreover, the values of cohesion and friction angle are found to be as critical for dam stability assessment as dam-rock geometry. Then, an experimentally validated multi-scale numerical procedure was developed to model the effect of rock surface roughness on dam seismic stability and suggest values of apparent cohesion and friction angle. Rough dam-rock interface geometries were generated using Bathymetric and LiDAR surveys of typical rock foundation surfaces and implemented into non-linear FE models of gravity dams. It is found that large scale roughness can significantly increase the shear strength of dam-rock interfaces.

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