Coupling Internal Erosion through Concrete Face Rockfill Dams with Damage in Face Slab under Impounding

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Abstract. This study describes a way to realistically model the internal erosion after damage in the concrete face of a concrete face rockfill dam (CFRD). The study uses a 2D plane strain model of Tianshengqiao dam’s geometry as an example, with and without consideration to erosion of the underlying cushion and transition layers. In order to accomplish this, Mazars’ concrete damage model, which is a continuum damage model, was coupled with permeability relationships on the face slab; and poroelastic model was used in conjunction with Sterpi’s erosion model for the underlying cushion and transition layers. Two scenarios were studied: without considering erosion and considering erosion. Results reveal that when the reservoir is linearly impounded damage in the face slab is concentrated to the lower quarter of the slab, and internal erosion will occur at the toe of the dam. After more fine particles are lost, the mechanical and hydraulic properties of the dam will change and the dam safety can be compromised.

1. Introduction
The scientific design and assessment of Concrete Faced Rockfill Dams (CFRDs) began with their introduction in the ASCE convention by Cooke and Sherard [1]. Later the same authors wrote two papers on their design and assessment which would become guidelines for future works[2, 3]. Today, CFRDs are one of the most widely in use in the recent past especially in China. This because these dams are amongst the most cost-effective options in dam building, are easy to construct and are relatively safer in case of an earthquake[4].

Multiple studies have been conducted to have been conducted to characterize the behaviour of CFRDs. Hunter and Fell conducted a study to predict the settlements, concrete face deformation and rockfill moduli based on data collected from a large number of existing dams[5]. Kim and Kim developed an Artificial Neural Network Model to predict crest settlements for the same [6]. Modares and Quiroz[7] developed a nonlinear scheme capable of accounting for the construction sequence, slab-rockfill interaction and reservoir impounding.

Arici[8, 9] was one of the first researchers to try and connect the damage of the concrete face under conditions of impounding, seismic excitation[10] and long-term settlements[8]. In these, studies relating impounding and cracking of face slab Arici estimated crack width along the face slab using the empirical Gergely-Lutz equation[11] and concluded observing tensile cracking at only the bottom one-third of the face slab. However, the change in material properties due to seepage that might be initiated due to the cracking of face slab is not considered. On the other hand, Chen et. al.[12] studied the deformation of CFRDs under non-steady seepage flow with stress-dependent change in
permeability. But, the change in the volume of water entering the dam and location of entry, which is dependent on the damage of concrete face slab and its location slab is not considered.

This study aims to understand the internal erosion mechanisms in CFRDs by coupling the deformation of rockfill and face slab and the damage in the face slab with the influx of water through damaged area and the subsequent internal erosion due to resulting infiltration of water. Understanding such phenomena become important because as Woo et. al.[13] reported up to 28.9% of concrete dams fail due to leakage and up to 7.9% fail due to erosion in the US alone. Zhang et. al.[14] also found that the rockfill in the Gouhou Dam, another CFRD, was internally unstable. This eventually led to erosion and further segregation of rockfill forming a perched water table and ultimately led to failure due to seepage.

The simulation uses a linear elastic model to deal with the mechanical aspects, although an elastic damage model is used to characterize concrete face damage, the rockfill is assumed to be elastic for simplicity. Darcy’s law is used to deal with fluid flow and to calculate pore pressures. For the cushion and transition zone, the gradation curves are assessed for internal erosion and poroelastic storage is assumed to account for the change in pore pressure due to change in porosity due to erosion. Internal erosion is quantified using Sterpi’s formula and a mass balance equation for fluid flow.

2. Theory

2.1. Concrete Damage Models

The concrete face is the primary water retaining element in CFRDs. Studies by Wang et. al. [15] were amongst the first to study the relationship between crack characteristics and concrete permeability suggesting an increase in flow rate as crack openings grew. Aldea et.al.[16] later suggested that regardless of the cause of cracking flow for the same crack level were similar and that Crack Mouth Opening Displacements (CMODs) or crack area had a direct relationship.

In this study, the theory continuum damage mechanics pioneered by Kachanov[17] is used to try and quantify the damage in the concrete face. Of the many damage models adapted to concrete that are in practice today the scalar damage model proposed by Mazars and Pijaudier-Cabot[18] is used in this study. The model parameters for the damage model were obtained from studies of Jason et. al.[19], who calibrated the damage model based on the experiments performed by Giakoumelis et. al.[20], Sinha et. al.[21] and Sfer et. al.[22].

According to Mazars et. al.[18] the process of damage calculation is given. If \( \sigma' \) is the effective stress and the isotropic damage variable is denoted by \( D \). The applied stress tensor \( \sigma \) is calculated as

\[
\sigma = (1 - D) \cdot \sigma'
\]

Where \( D = 0 \) for virgin material and 1 for completely damaged material.

Given that \( C \) is the elasticity tensor, the strain tensor \( \varepsilon \) is calculated as:

\[
\varepsilon = C^{-1} \sigma'
\]

The strain tensor is then used to calculate the equivalent strain \( (\varepsilon_{eq}) \) which quantifies the amount of extension the material will undergo during load application. Here, \( \varepsilon_i \) are the principal strains and \( \varepsilon_{eq} \) is calculated as:

\[
\varepsilon_{eq} = \sqrt{\sum_{i=1}^{3} (\varepsilon_i) ^2}
\]

And

\[
\langle \varepsilon_i \rangle = \frac{|\varepsilon_i| + \varepsilon_i}{2}
\]

Here \( \kappa \) is defined as the threshold of damage growth. The damage loading function is defined as:

\[
f(\varepsilon_{eq}, \kappa) = \varepsilon_{eq} - \kappa
\]
\[ \kappa_0 = \frac{\sigma_p}{E_{eq}} = \varepsilon_{D_h} \]  

And, \( \kappa_p \) represents the threshold from which damage begins.

Where, \( \sigma_p \) is the related peak stress and \( E_{eq} \) is the initial elastic modulus of concrete.

If \( f(\varepsilon_{eq}, \kappa) = 0 \) and \( \dot{f}(\varepsilon_{eq}, \kappa) = 0 \) then,

\[
\begin{cases}
    d = d(\varepsilon) \\
    \kappa = \varepsilon_{eq}
\end{cases}
\quad \text{with} \quad \dot{d} \geq 0, \quad \text{else} \quad \begin{cases}
    d = 0 \\
    \kappa = 0
\end{cases}
\]  

In other words, if \( \varepsilon_{eq} \geq \varepsilon_{D_h} \),

\[
d(\varepsilon) = \alpha_t(\varepsilon)D_t(\varepsilon_{eq}) + \alpha_c(\varepsilon)D_c(\varepsilon_{eq})
\]  

\[
D = \max_{t/2}(d,0)
\]  

Here \( d(\varepsilon) \) is the damage evolution law is a weighted sum of two pairs of scalars for tension (\( \alpha_t, D_t \)) and compression (\( \alpha_c, D_c \)) which are defined as follows:

\[
D_t = 1 - \frac{\varepsilon_{D_h}(1 - A_t)}{\varepsilon_{eq}} - \frac{A_t}{\exp[B_t(\varepsilon_{eq} - \varepsilon_{D_h})]}
\]  

\[
D_c = 1 - \frac{\varepsilon_{D_h}(1 - A_c)}{\varepsilon_{eq}} - \frac{A_c}{\exp[B_c(\varepsilon_{eq} - \varepsilon_{D_h})]}
\]  

\[
\alpha_t = \sum_{i=1}^{3} \left( \frac{c_i}{e_{eq}^{2}} \right)^{\beta}, \alpha_c = \sum_{i=1}^{3} \left( \frac{c_i}{e_{eq}^{2}} \right)^{\beta}
\]  

As suggested by Pijaudier-Cabot, Mazars and Jason et al.\[18, 19, 23\] \( C_{D_h} \), \( A_t \) and \( B_t \) are received from a tensile test, \( A_c \) and \( B_c \) are received from a cyclic compression test and \( \beta \) is fitted from the material’s response to shear (Here \( \beta \) is assumed to be 1 and other values were taken from studies by Jason et al.\[19\]).

And,

\[
\varepsilon'_t = (1 - D)C^{-1}\sigma'_t, \varepsilon'_c = (1 - D)C^{-1}\sigma'_c
\]  

Where, \( \sigma_t \) and \( \sigma_c \) are stress tensors which contain only the positive and negative principal stresses from the original principal stress tensor \( \sigma \), so, \( \sigma = \sigma_t + \sigma_c \).

In this study, instead of relating CMODs with flow rates, the damage is correlated with change in permeability of the material. Bary et al.\[24\] was amongst the first to theoretically relate concrete’s permeability to damage and thereby study the mechanical effects on concrete due to pore pressure using the effective stress principle. In this study, however, uses the results from Pijaudier-Cabot et al.\[25\], shown in Figure 1, relating permeability with damage variable obtained by combining diffused damage with localized damage through a matching law. The model parameters were calibrated from Pijaudier-Cabot et al. as well, who calibrated them using experiments conducted by Gerard et al.\[26\].
2.2. **Internal erosion**

Internal erosion has caused damage to CFRDs before as in the case of Gouhou as shown in [14, 27]. The effect of internal erosion on the change in porosity in zones IIB (cushion layer) and IIIA (transition layer) is studied here.

Of the many internal stability criteria in existence (Kenney and Lau[28], Lafleur[29], Mao[30], Li[31], Wan and Fell[32], Moffat[33]), Kenney and Lau’s criterion is chosen in this study due to its ease in use and higher accuracy as observed by Bi[34]. In accordance with this criterion, F and H are defined as the mass percentages of soil with particles smaller than random diameter D and mass percentages of soil with particle sizes between D and 4D respectively. The soil is considered internally stable if the H/F ratio remains greater than 1 when the cumulative mass percentage of D is swept from 0 to 20 percent.

The entire soil sample is assumed to have been made up of non-erodible particles making up the soil skeleton, fine particles that make up the erodible mass and voids occupied by air or water.

The erosion relationship used in this study is the one proposed by Sterpi[35] which states:

\[ \mu_e = \mu_0 \left[ 1 - \exp \left( - \left( \frac{t}{t_0} \right)^b \frac{i^c}{a} \right) \right] \]  
(14)

Where \( \mu_e \) is the weight of eroded fine particles, \( \mu_0 \) is the weight of initial erodible fine particles, \( t \) is the time elapsed in hours, \( i \) is the hydraulic gradient and \( t_0 = 1 \) h. \( a, b \) and \( c \) are experimental constants calculated by Sterpi[35] as well.

As erosion progresses the porosity of the material changes as more of the fine particles are moved. For unit volume, if \( \rho_p \) is the particle density (taken as 2500 kg/m³ for limestone) this is expressed as,

\[ \frac{\partial \phi}{\partial t} = \frac{1}{\rho_p} \left( \frac{\partial \mu_c}{\partial t} \right) \]  
(15)

As porosity changes the permeability of rockfill material changes as well. This is characterized by the modified Kozeny Carman equation which is as follows:

\[ k = k_0 \left( \frac{\phi}{\phi_0} \right)^3 \left( 1 - \frac{\phi_0}{1 - \phi} \right) \]  
(16)

Here \( k \) and \( \phi \) are the changed permeability and porosity whereas \( k_0 \) and \( \phi_0 \) are the initial permeability and porosity respectively.
2.3. Poroelasticity

In this study, the flow through the concrete face, buffer layers, and rockfill layers are assumed to be laminar and viscous forces are assumed to be predominant and therefore the flow is assumed to follow Darcy’s law. This is described as,

\[ v_f = -\frac{k}{\mu} \nabla \psi; \]

with, \( \psi = z + \frac{p_i}{\gamma} \) \hfill (17)

Here, \( v_f \) is the velocity of the fluid, \( k \) is the permeability of the medium, \( \mu \) is the fluid viscosity, \( \psi \) is the total pressure head, \( z \) is the potential head, \( p_i \) is the pore water pressure and \( \gamma \) is the specific weight of the fluid.

The development of poroelasticity began with Biot [36]. Poroelasticity relates the strain of the porous material, which in this case is considered to be cushion and transition material, to alterations in states of stresses and pore pressures in the material. Using the pressure head formulation,

\[ \rho_f \phi_f \frac{\partial \psi}{\partial t} + \nabla \cdot \left( \rho_f v_f \right) = -\rho_f \alpha_b \frac{d e_{vol}}{dt} \]

Where, \( \phi_f \) is the porosity, \( \rho_f \) is the density of fluid, \( v_f \) is the velocity of fluid described by Darcy’s law, \( \alpha_b \) is the Biot’s coefficient (assumed to be 1), \( e_{vol} \) is the volumetric strain, \( \psi \) is the pressure head and \( \chi_f \) is the compressibility of water (taken as \( 4 \times 10^{-10} \text{ Pa}^{-1} \)).

![Figure 2. Tianshengqiao model geometry](image)

3. Numerical Simulation

Tianshengqiao dam was taken as an example over which the numerical simulations were run. A representative section of the Tianshengqiao dam was taken and treated as a 2D plane strain model in this analysis. This assumption has been assumed to be valid because the crest length to dam height ratio in this dam exceeds 6. Major regions and dimensions for the model are depicted in Figure 2. Two model scenarios were run assuming the reservoir head is linearly increased from 0m to the normal water level head of 163.5m in 360 days. Scenario one considers the damage in concrete slab and seepage of water with an increment of face slab permeability while internal erosion is not considered. Scenario two considers the changes in material properties in filter along with concrete slab damage with the internal erosion of underlying cushion and transition layers.

The concrete face slab was modeled using Mazars’ elastic damage model. The thickness of the slab ranged from 0.9m at the bottom to 0.3m at the top. The physical parameters for the concrete are
described in Table 1. Since the slab is reinforced, for the concrete strain level to reach \( \varepsilon_{Dc} \) the strain level in the steel must reach \( \varepsilon_{De} \) first, therefore \( \varepsilon_{De} \) is set at \( \varepsilon_{Dc} \times (E_s/E_c) \).

### Table 1. Physical parameters for concrete

| Properties                              | Values                    |
|-----------------------------------------|---------------------------|
| Initial elastic modulus for concrete \((E_c)\) | 3.2x10^{10} Pa           |
| Poisson’s ratio                         | 0.2                       |
| Initial permeability                    | 1x10^{-17} m^2            |
| Density                                 | 2400 kg/m^3               |
| Porosity                                | 0.1                       |
| \(A_c\)                                 | 1.5                       |
| \(B_c\)                                 | 1391                      |
| \(A_t\)                                 | 0.88                      |
| \(B_t\)                                 | 8000                      |
| \(\varepsilon_{De}\)                   | 0.6                       |
| \(\lambda\)                             | 1.89                      |
| Elastic modulus for steel \((E_s)\)      | 200GPa                    |
| Area of steel/ Area of concrete         | 2.5%                      |

The rockfill materials are assumed to behave elastically in this study. The exceptions are the cushion and the transition materials where changes in porosity are considered while calculating the volumetric strain due to the low permeability values often associated with the materials used in these regions. The remainder of the model was meshed with a coarser triangular mesh. The infinite element domains on either side of the bedrock layers were meshed with a rectangular element. In total, the model consisted of 55876 domain elements, 5097 boundary elements and solves for 241358 degrees of freedom. The initial physical parameters of the different regions are listed in Table 2.

### Table 2. Physical parameters for dam material [39]

| Region   | Modulus of elasticity (Pa) | Poisson’s Ratio | Density \((kg/m^3)\) | Porosity | Permeability \((m^2)\) |
|----------|-----------------------------|-----------------|----------------------|----------|-----------------------|
| IA       | 7x10^7                      | 0.2             | 1600                 | -        | -                     |
| IB       | 6x10^7                      | 0.2             | 1500                 | -        | -                     |
| IIB      | 8.19x10^7                   | 0.2             | 2200                 | 0.19     | 5.11x10^{-12}         |
| IIIA     | 6.33x10^7                   | 0.32            | 2150                 | 0.21     | 5.11x10^{-10}         |
| IIIB     | 6.10x10^7                   | 0.33            | 2150                 | 0.22     | 6.13x10^{-8}          |
| IIIC     | 6.33x10^7                   | 0.34            | 2120                 | 0.22     | 1.02x10^{-10}         |
| IID      | 6.33x10^7                   | 0.35            | 2050                 | 0.24     | 6.13x10^{-7}          |
| Bedrock and infinite element domains   | 2.5x10^8         | 0.2             | 2500                 | -        | -                     |

The time steps for various impounding rates were defined so that they had the same number of intervals. For example, if the dam was assumed to fill in 90 days the step size would be 0.9 days so that there are 100 steps, whereas if the dam was assumed to fill in 1100 days the step size would be 11 days so that there were 100 steps as well. This has been done so that the results at the same hydraulic head but at different times can be effectively compared.

Fixed constraints are applied to segments 1-2, 2-3 and 2-4, as shown in Figure 2. And the horizontal movement is constrained on segments 4-5, 5-6, 6-7, 1-8, 8-9 and 9-10. Water pressure is applied to segments 11-12, 12-13 and 13-14. Segments 15-16, 16-17, 17-18 and 18-19 are under atmospheric pressure.

The dam elements were first initialized with stresses from the self-weight of the dam so that conditions of compaction were considered and settlements before impounding were controlled.
initialization, the stiffness matrix in use was not subjected to the damage model as it is assumed here that all damage in the concrete face occurred here as a direct result of water loads alone and because in practice the concrete slabs are usually only cast after the rockfill is compacted to minimize damage on the face slab.

For scenario one the porosities and permeability of cushion and transition layers were as defined in Table 2. However, for scenario two both the porosities and permeability change by relations defined in section 2.4. The initial values are however taken to be as described in Table 2. The bedrock layers and associated infinite element domains were not subjected to Darcy flow or resulting pore pressure as they are assumed to be impervious. Transport of particles and subsequent change in porosity is only considered in regions IIB and IIIA. In these regions, the total erodible mass is the mass of particles smaller than the minimum equivalent diameter of pore channels \(d_0\), calculated by the expression that Indraratna et al.[37] came up with:

\[
d_0 = 2.67 \frac{\phi}{1-\phi} \frac{D_h}{\alpha}
\]  

(19)

Where, \(\alpha\) is the shape coefficient and is equal to 6 for spherical particles and \(D_h\) is the equivalent diameter of gradation in mm and is calculated by the relationship given by Kovacs[38]:

\[
D_h = \frac{1}{\sum \Delta S_i / D_i}
\]  

(20)

If the particle size gradation curve is divided into \(i\) equal parts along the particle size diameter axis. The average diameter of each \(i^{th}\) segment is \(D_i\) and the weight of particles in this interval is \(\Delta S_i\).

The gradations for zone IIB was chosen so that it fell between the limits of gradation suggested by the International Committee on Large Dams (ICOLD)[39]. No such limits have been specified for IIIA. However, deviations in the gradations curves for both zones IIB and IIIA had to be made in order to match the maximum particle sizes and permeability reported by Feng et al.[40]. Specifically, the \(D_5\), \(D_{10}\) and \(D_{15}\) were set so that the hydraulic conductivities specified were close to those predicted by Sherard[41] (equation 21) and Indraratna[42] (equation 22) the relations:

\[
K = 0.35(D_{15})^2
\]  

(21)

\[
K = 1.02(D_5D_{10})^{0.93}
\]  

(22)

Where, \(D_5\), \(D_{10}\) and \(D_{15}\) are the particle size diameters (in mm) of the medium such that 5%, 10%, and 15% of particles by weight are finer respectively. \(K\) is the hydraulic conductivity of medium in cm/s. The maximum size of particles included in the gradation was based on the details of the maximum particle sizes which were 8cm and 30cm for IIB and IIIA respectively[43]. The assumed gradations for zones IIB and IIIA can be seen along with the limits for zone IIB specified by ICOLD in Figure 3.

![Figure 3. Particle size gradations](image-url)
4. Results

4.1. Internal stability of zones IIB and IIIA

The gradation curves for zones IIB and IIIA were analyzed using the Kenney and Lau criterion for internal stability. Figure 4 shows that all the particle gradations as well as the limits of particle gradations for IIB are lower than the suggested boundary and are therefore internally unstable. This means that the particles in this region are erodible. However, when the gradations for IIB and IIIA are tested using the $D_{15}/D_{85}$ criterion, defined by Terzaghi [44], the resulting values were well under 4 for both particle size distributions.

![Figure 4. Grain size distribution shape curve and the boundary between unstable and stable grading using Kenney and Lau criterion[28]](https://example.com/figure4.png)

4.2. Damage patterns for concrete face slab

Results from the simulations show that damage started from the lower part of the face slab, no matter internal erosion is considered or not. The damage when the reservoir level reaches the normal water level was found to be constrained to the lower quarter length of the face slab at all seven reservoir filling rate scenarios. Figure 5 shows the Damage variable ($D$) at the center line of the face slab, at the end of reservoir filling, plotted against the length of the face slab when the reservoir is filled at 360 days (considering erosion and ignoring erosion). It can be noted that most of the damage is concentrated at the bottom part of the face slab. This quite simply because this region receives the largest hydraulic pressures, deforms the most and is the first to exceed the $E_D$ threshold.

![Figure 5. Damage variable vs. Length of face slab at the end of reservoir filling when the reservoir is filled in 360 days and erosion is not considered](https://example.com/figure5.png)
Since, the plots for crack length with and without considerations for erosion overlap, it can be noted that the change in volumetric strain due to erosion is not significant enough to cause any appreciable change in concrete face damage. It must, however, be noted that this model does not consider the change in the elastic moduli of the rockfill due to the loss of eroded mass. When Figure 5 is compared to crack width (which is also a marker of damage) to distance along face plots by Arici [8] for impounding, we see that the trends for damage predicted from the simulations are consistent.

Figure 6. Damage variable with corresponding fluid pressure contours at various stages of loading when the reservoir is filled in 360 days and erosion is considered

Figure 6a) to Figure 6d) show the progression of damage from its initiation when the reservoir is filled from 0 to 163.5 m head in 360 days when the transport of particles is considered, is presented as an example here. Figure 6a shows that damage initiation began at 302.4 days (hydraulic head of 137.34m) in the region between 2.5 to 9 meters above the dam toe. Figure 6b shows an increment in damage variable (D) to a critical value is observed in the initiation region until 313.2 days (hydraulic head of 142.245m) while there is a steady increase in equivalent strain in the rest of the face slab. Figure 6b) and Figure 6c) show a sudden increase in the damaged area which extended up to of 29.5m above the dam toe between 313.2 days (hydraulic head of 142.245m) and 316.8 days (hydraulic head of 143.88m). This spike in damage is attributed to the brittle failure in concrete. Figure 6c and Figure 6d show that between 316.8 days (hydraulic head of 143.88m) and end of impoundment at 360 days (hydraulic head of 163.5 m) the damage increased only slightly at the edge of damaged area. This is thought to be because the stress could not be effectively transmitted after damage extended across the slab and the effective Young’s modulus declined to a large extent. Therefore, instead, the...
applied stress increased the equivalent strains towards the boundary of the damaged area. This is evident from the growth of equivalent strain values as shown in Figure 7a, b and c.

![Figure 7](image-url)

**Figure 7.** a) Enlarged view of the upper limit of damage (labelled A in Figure 6)
   b) Equivalent strains after brittle damage at A
   c) Equivalent strains at the end of impounding at boundaries if damaged areas at A

### 4.3. Effects of Internal Erosion

Figure 8a) and Figure 8b) show the porosities of zones IIB and IIIA with the corresponding damage of face slab at 302.4 and 360 days respectively. Figure 8b) shows that the porosity grew, with increment in time and hydraulic head, this is attributed to the flow of particles due to the incoming water. Porosity grew to a larger value in IIB because analysis of grading curves revealed that the diameter of erodible particles ($d_0$) was 0.0346 mm for IIB which accounted for almost 10% of the total particle mass.

![Figure 8](image-url)

**Figure 8.** Face slab damage with porosities of underlying layers IIB and IIIA at various stages when the reservoir is fully impounded at 360 days and erosion is considered

In addition, since zone IIB had a larger pressure gradient, as can be seen from the pressure contours from Figure 8a) and Figure 8b), loss of erodible mass was also comparatively faster than for IIIA. Almost all the particles that could be eroded, were eroded from the area immediately under the damaged zone in IIB, even though the permeability was relatively lower than that of zone IIIA. On the other hand, zone IIIA, in diameter for erodible particle ($d_0$) for IIIA was 0.0921mm that accounted for around 5% of total mass, had the highest porosities around the dam toe.

The arrows denote the velocity of the incoming fluid, which in this simulation is assumed to be the same as the velocity of moving particles, reveal that the velocity of flow too was higher near the dam toe, making it more vulnerable. Critical hydraulic gradients (as defined by Mantei et. al.[45]) of 0.23
were also only reached at areas from where particles had been eroded. Hydraulic gradients at the
downstream slops were well below critical so dam breach did not occur in this simulation.

Figure 9 shows the specific discharge passing through IIIA when erosion is not considered and
when it is. The graph clearly shows that while the spike in water flow began at the same time, the
specific discharge of water flowing through IIIA is higher, in this simulation, by just over 50% when
erosion is considered. The increase in specific discharge with the consideration of erosion is as
expected and is attributed to the change in permeability with the change in porosity which itself is a
function of the mass of eroded particles.

Figure 9. Specific discharge through IIIA vs. time

5. Conclusion
Internal erosion is one of the main factors inducing dam failure. If fractures appear on the face slab of
a CFRD, the water pressure will be applied to the rockfill directly and fine particles will be washed out
by the high water pressure gradient. The effect of concrete face damage on internal erosion is analyzed
in this study. Mazar’s damage model for the concrete face is employed to simulate the damage of
cement face. Sterpi’s erosion law is used to calculate the mass of eroded particles. The poroelasticity
theory is used to couple the internal erosion in rockfill, damage on face slab and displacement field of
the dam together. The permeability for cracked concrete and soil are calculated by the Pijaudier-
Cabot’s model and the Kozeny-Carman equation, respectively. The following conclusions can be
drawn:
1. Most codes for CFRDs don’t consider the internal stability of soils used in the dam. The
recommended gradations in codes can be internally unstable when estimated by commonly-used
criteria. This means that once the concrete face cracks, the fine particles will be washed out by the
seepage flow. The gradation of soil should be modified before the dam construction to avoid
internal erosion, especially for the transition and cushion layers.

2. Damage is concentrated at the bottom one-fourth of the face slab, where the water pressure is high.
Therefore, after damage, the water will flow in the dam with a high hydraulic gradient, which will
cause internal erosion in the soil. In the case of Tianshengqiao dam, the amounts of eroded
particles are greater than 10%. The mechanical and hydraulic properties of the dam after internal
erosion will dramatically change, which may eventually leave the dam susceptible to failure.

3. After internal erosion, the porosity and permeability of the soil will increase, which will allow a
higher velocity and pressure gradient. And more fine particles will be lost. Therefore, the internal
erosion is a gradually deteriorative process. At the same time, the modulus of the soil can decrease
due to fine particles loss. Then, more deformation can occur near the internal erosion zone, and the
damage of the concrete face will get worse. This means that the damage and internal erosion
mechanism form a process form a worsening cycle.

In this study, the force in the concrete face is only influenced by the small change of water pressure
during internal erosion. Therefore, the damaged zone when erosion is considered is almost the same as
the condition as when it is not. But it can be predicted that if the modulus reduction caused by fine particles loss is considered, the damage zone and the settlement of the dam will get larger. The theory for coupling internal erosion and mechanical performance of soil will be studied in the future work of the authors.

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