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Numerical investigation of the response of the Yele rockfill dam during the 2008 Wenchuan Earthquake

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Abstract. In this paper the seismic response of a well-documented Chinese rockfill dam, Yele dam, is simulated and investigated employing the dynamic hydro-mechanically (HM) coupled finite element (FE) method. The objective of the study is to firstly validate the numerical model for static and dynamic analyses of rockfill dams against the unique monitoring data on the Yele dam recorded before and during the Wenchuan earthquake. The initial stress state of the dynamic analysis is reproduced by simulating the geological history of the dam foundation, the dam construction and the reservoir impounding. Subsequently, the predicted seismic response of the Yele dam is analysed, in terms of the deformed shape, crest settlements and acceleration distribution pattern, in order to understand its seismic behaviour, assess its seismic safety and provide indication for the application of any potential reinforcement measures. The results show that the predicted seismic deformation of the Yele dam is in agreement with field observations that suggested that the dam operated safely during the Wenchuan earthquake. Finally, parametric studies are conducted to explore the impact of two factors on the seismic response of rockfill dams, i.e. the permeability of materials comprising the dam body and the vertical ground motion.

Key words: seismic response of rockfill dams, finite element analysis, hydro-mechanical coupling, Wenchuan earthquake, Yele dam

1 Introduction

In May 2008, the Wenchuan earthquake ($M_s=8.0$) occurred in west China, damaging approximately 391 dams in varying degree, including 4 large-scale earth and rockfill dams (dam heights exceed 100m) (Xu, 2009). Seismic damage such as crest settlements and concrete slab cracks were widely observed. The safety of these dams is critical for millions of people living in the downstream area. It is therefore important to thoroughly investigate well-documented case studies from the Wenchuan earthquake which will allow us to systematically assess the seismic resistance of earth and rockfill dams and to improve our numerical capabilities in predicting their seismic response and potential failure mechanism.

The seismic response of earth and rockfill dams has been extensively studied by researchers employing different methods of analysis, such as pseudo static limit equilibrium approaches, methods for the evaluation of the permanent displacements based on the sliding block concept of Newmark (1965) and numerical methods (e.g. Shear Beam, FE, Boundary Elements). Among these most widely used methods, the FE method has gained its popularity due to its ability of accurately simulating complex dam geometry, soil-structure interaction effects and realistic soil behaviour, through the implementation of soil constitutive models, boundary conditions and HM coupled consolidation formulation.

Indeed, over the years, the FE method has been widely used to investigate the seismic response of earth and rockfill dams, employing constitutive models of varying degree of sophistication. In these applications, the constitutive models can be usually categorised into: linear and equivalent-linear models (Clough and Chopra, 1966; Seed et al., 1969; Vrymoed, 1981), cyclic nonlinear formulations coupled with simple elasto-plastic models (Prevost et al., 1985; Pelecanos, 2013; Pelecanos et al., 2015), advanced elasto-plastic models (e.g. multi-surface kinematic hardening models) usually simulating clay core dams (Sica et al., 2008; Elia et al., 2011) and advanced elasto-plastic models (e.g. bounding surface plasticity models) aiming to simulate flow liquefaction failures (Elgamal et al., 2002; Aydingun and Adalier, 2003; Ming et al., 2011).

Another critical aspect of the numerical modelling of earth and rockfill dams is the rigorous simulation of the HM coupled response of the soil. This often requires the adoption of HM coupled formulation to accurately compute the development and

dissipation of pore water pressures, and account for their impact on the response of earth and rockfill dams (e.g. Lacy and Prevost, 1987; Elia et al., 2011; Pelecanos, 2013). The adoption of a HM coupled formulation is usually more critical for the accurate simulation of the dynamic behaviour of rockfill materials due to their large permeability. In rockfill dams, depending on the range of permeability and loading duration, consolidation can occur during the dynamic loading and therefore the response cannot be considered as drained or undrained. Notably in Lacy and Prevost (1987), the insufficiency of the single-phase, uncoupled formulation, in predicting soil material damping is highlighted and the adoption of the HM coupled formulation is suggested for a more accurate simulation of the dynamic behaviour of earth and rockfill dams. The HM formulation enables the simulation of an additional damping generated by the interaction between the soil skeleton and the pore water fluid.

Apart from the constitutive models and the HM coupled formulation, realistic predictions of the dynamic response of dams also require the appropriate treatment of other aspects of the computational model, such as time integration and boundary conditions. There is therefore a need for well-documented field case studies of dams to properly validate these important aspects of dynamic analysis. In this paper, the seismic response of a well-documented Chinese rockfill dam, the Yele dam, is investigated employing the dynamic HM formulation of the Imperial College Finite Element Program (ICFEP, Potts and Zdravković (1999)). A detailed description for the FE formulation of ICFEP can be found in Potts and Zdravković (1999) in static condition and in Kontoe (2006), Kontoe et al. (2008) and Han et al. (2015) in dynamic condition. Through the numerical investigation, different aspects of the numerical modelling for static and dynamic analyses of rockfill dams are validated against the available monitored data of the Yele dam before and during the Wenchuan earthquake. Subsequently, the predicted seismic response of the Yele dam is analysed, in terms of the deformed shape, crest settlements and acceleration distribution pattern, in order to understand its seismic behaviour, assess its seismic safety and provide indication for the application of any potential reinforcement measures. Finally, parametric studies are conducted to explore the impact of two factors on the seismic response of rockfill dams, i.e. the permeability of materials comprising the dam body and the vertical ground motion.

2 Yele dam

The Yele dam is an asphaltic concrete core rockfill dam located at the Nanya River in Sichuan Province, China. It is the highest asphaltic concrete core rockfill dam among all the dams of this type in China (124.5m high). The Yele dam serves solely as an electricity generating supply with a reservoir capacity of 298 million cubic metres. On the 12th of May 2008, the Wenchuan earthquake ($M_s=8.0$) occurred in west China damaging approximately 391 dams at a varying degree, including 4 large-scale ones (heights exceeding 100m) (Xu, 2009). Among all these dams, only the seismic monitoring equipment of the Yele dam (258km from the epicentre) was in good working order. Eight seismometers recorded the multi-directional ground motion at different locations on the dam, providing valuable monitoring data for a detailed investigation of its seismic response.

2.1 Dam geometry and construction sequence

The plan view, longitudinal section and maximum transverse section of the Yele dam are shown in Figure 1. The maximum height of the Yele dam is 124.5m and the designed operational reservoir level is 117m. The dam axial length is 411m and the crest width is 15m. The Yele dam body consists of several filling zones, i.e. an asphaltic concrete core, transition layer, filter layer, coffer dam, cap dam and rockfill zone, as shown in Figure 1c. The dam foundation consists of an alluvium layer of varying thickness, where the average depth for the whole foundation layer is approximately 53m.

Figure 1: Geometry of the Yele dam

According to He et al. (2006), the construction of the Yele dam started in October 2003 and reached a height of 75m in January of 2005, which corresponds to the designed minimum water level in dry season. Its construction was completed in November 2005, reaching the maximum dam height of 124.5m. The reservoir impounding started in January of 2005, while the dam was still under construction and had reached a height of 75m. In November of 2005, the reservoir reached a height of 107m and the power plant started to generate electricity. During the subsequent 2.5-years of operation, the reservoir experienced annual water level fluctuations due to seasonal rainfall variations. Before the occurrence of the Wenchuan earthquake, the reservoir level reduced to its minimum level due to the dry season at 75m. In order to numerically simulate the dam construction, reservoir impounding and operation processes, the construction and reservoir impounding sequence is simplified in the static analysis as shown in Figure 2.

Figure 2: Simplified construction and reservoir impounding sequence adopted in numerical analysis

2.2 Material properties

According to Xiong (2009), the foundation layer of the Yele dam mainly consists of gravelly and silty soils. The filter and transition layers of the dam are composed of fine sands to discharge seeping water and to accommodate any differential deformations between the core and the outer rockfill. The materials for the core and grouting curtain are asphalt concrete and concrete respectively. The rockfill zone, coffer dam and cap dam were constructed with rockfill materials excavated and processed from the site near the dam, where the particle size distribution for the rockfill materials is show in Figure 3. The soil properties of all dam materials are summarised in Table 1 (He et al., 2006), obtained from in-situ investigations and laboratory tests. In particular, the effective cohesion and angle of shearing resistance were obtained from laboratory direct shear tests. The shear wave velocity was measured by in-situ tests, but the test type was not reported. The permeability was determined by laboratory constant head permeability tests.

Figure 3: Particle size distribution curves of the Yele dam rockfill materials

Table 1: Soil properties of the Yele materials

Parameter Material	Cohesion c' (kPa)	Angle of Shearing ϕ' (degree)	Density ρ (g/cm ³)	S-wave Velocity v_s (m/s)	Permeability k (m/s)
Foundation layer	80	38	2.3	800	1.0E-8
Core			2.4		
Grouting curtain			2.4		
Transition layer	0	45	2.3	224	1.0E-5
Filter layer	0	45	2.3	224	1.0E-5
Rockfill	0	50	2.3	365	1.0E-3
Coffer dam	0	50	2.3	365	1.0E-3
Cap dam	0	50	2.3	365	1.0E-3

The dynamic soil properties for the Yele dam rockfill materials were obtained from Xiong (2009) through laboratory cyclic triaxial tests and are presented in Figure 4, in the form of shear modulus degradation and material damping curves. These are also compared with the corresponding standard curves proposed by Rollins et al. (1998) for gravels considering a wide confining pressure range (0-500 kPa). Clearly, the Yele rockfill materials exhibit a more rapid change of dynamic properties in the strain level, compared to the curves of Rollins et al. (1998). As there is no site-specific information available for the remaining materials and the rockfill material was excavated and processed near the dam site, it was assumed that the foundation materials are characterised by the same stiffness degradation and damping curves. Furthermore, since the volume of the transition and filter layers is relatively small, the employment of the same dynamic soil properties for these two materials is not expected to significantly affect the dynamic deformations of the whole dam.

Figure 4: Dynamic soil properties of the Yele rockfill materials

3 FE mesh and calibration of constitutive model

A two-dimensional plane strain model of the Yele dam was analysed with the FE code ICFEP (Potts and Zdravković 1999) under static (to simulate the construction sequence and operation prior to the earthquake) and dynamic conditions (dynamic coupled formulation of ICFEP in Kontoe (2006); Kontoe et al. (2008); Han et al. (2015)). A detailed sensitivity study was first conducted to establish an appropriate FE mesh for numerical analyses, in terms of element size and lateral extent. As a result, the element dimensions are 8m by 6m in the horizontal and vertical directions respectively and the lateral boundaries are placed at 300m away from the dam toe and the dam heel for the downstream and the upstream sides respectively. The resulting FE mesh of the Yele dam

consists of 2440 8-noded isoparametric quadrilateral solid elements and is shown in Figure 5.

Figure 5: FE mesh of the Yele dam and boundary conditions for the static analysis

For the static and dynamic analyses of the Yele dam, a cyclic nonlinear model, i.e. the Imperial College Generalised Small Strain Stiffness model (ICG3S model, Taborda and Zdravković (2012); Taborda et al. (2016)) coupled with a Mohr-Coulomb failure criterion is employed to simulate the elasto-plastic soil behaviour of the dam and foundation materials. The ICG3S model is formulated in three-dimensional stress-strain space, meaning it can simulate nonlinear soil behaviour associated with different deformation mechanism when subjected to multi-directional loading (i.e. shearing and volumetric response), in terms of shear and bulk/constrained modulus degradation and corresponding material damping. Furthermore, the model can realistically simulate the material damping at very small strain levels unlike most constitutive models of this class. A more detailed description of the employed constitutive model can be found in the Appendix and in Taborda et al. (2016). It should be noted that the asphalt concrete core and the grouting curtain are assumed to behave as linear elastic materials due to their large stiffness.

The calibration of the ICG3S model is conducted by matching the reproduced shear modulus degradation and damping curves with the reference ones, as shown in Figure 6a. However, due to the lack of laboratory data for the constrained modulus degradation and damping curves, identical model parameters are employed for the calibration related to the compressional deformation, as shown in Figure 6b and Table 2. The only exception is that a higher value was adopted for parameter s , which controls the constrained modulus degradation, than the corresponding parameter b which controls shear modulus

degradation (see Table 2). This is based on the investigation of Han (2014) and Han et al. (2016b), where by back-analysing downhole array seismic data it was found that the constrained modulus has a wider linear plateau but its degradation appears to be slightly steeper afterward, than that of the shear modulus.

It should be noted that the calibration of the cyclic nonlinear model aims to simulate accurately the damping at small deformation levels (i.e. approximately $<10^{-1}\%$ defined by Vucetic (1994)), due to the relatively weak bedrock motion for the Yele dam during the Wenchuan earthquake. This was achieved by employing the ICG3S model which can simulate accurately the material damping at small deformation levels. The material damping is underestimated by most other cyclic nonlinear models and to alleviate this, Rayleigh damping is usually employed which can lead to inaccurate predictions (Kontoe et al. 2011). On the other hand, at intermediate and large deformation levels (i.e. approximately $>10^{-1}\%$), material damping is usually overestimated by cyclic nonlinear models (Puzrin and Shiran, (2000); Phillips and Hashash (2009); Taborda, (2011)). This can be limited by prescribing a minimum value for the shear modulus (parameters c or t smaller than 1 in ICG3S model), which unavoidably leads to a gradual reduction in damping beyond the stiffness cut-off (see Figure 6). It should be noted that the minimum stiffness cut-off is widely employed in cyclic nonlinear models implemented also in commercial geotechnical FE programs, such as PLAXIS and recently Amorosi et al. (2016) also showed the damping reduction associated with the stiffness cut-off. The adopted calibration leads to a satisfactory damping prediction at small (approximately $<4 \times 10^{-2}\%$) and intermediate (approximately $4 \times 10^{-2}\% - 3 \times 10^{-1}\%$) strain levels, but to an under-prediction at large strain levels (approximately $>3 \times 10^{-1}\%$), as shown in Figure 6. However, this damping underestimation does not affect the overall prediction of the dynamic response of the Yele dam during the Wenchuan earthquake, as the induced strain levels are low due to the relatively weak bedrock motion.

Table 2: Parameters for the Yele dam materials by using the ICG3S model

Depth (GL. meters)	a	b	c	d ₁	d ₂	d ₃	d ₄
0.0 ~ -100.0	7.0E-04	15.0E+00	5.0E-02	100.0E+00	2.5E-01	500.0E+00	8.3E-01
Depth (GL. meters)	r	s	t	d ₅	d ₆	d ₇	d ₈
0.0 ~ -100.0	7.0E-04	20.0E+00	5.0E-02	100.0E+00	2.5E-01	500.0E+00	8.3E-01

Figure 6: Calibration of the ICG3S model for the Yele dam materials

4 Static analysis of the Yele dam

4.1 Numerical model

The static analysis aims to reproduce the static behaviour of the Yele dam before the Wenchuan earthquake by simulating the geological history of the dam foundation, the dam construction process, the reservoir impounding process and the subsequent operational period. The HM coupled consolidation FE formulation is employed for the foundation materials, whereas the behaviour of the dam materials is assumed to be drained due to their high permeability and the long duration of the construction period. The simulated construction and impounding sequence is summarised in

Table 3.

The displacement boundary conditions for the entire static analysis are shown in Figure 5, where both horizontal and vertical displacements are restricted at the bottom boundary and the horizontal displacements are restricted at the vertical boundaries of the mesh. Furthermore, appropriate hydraulic boundary conditions are employed for different stages of the analysis, as listed in Table 4. Three types of hydraulic boundary conditions are

mainly employed in the static analysis. In particular, the pore water pressure at some boundaries is prescribed as no change (i.e. $\Delta p_f=0$), such as at the far vertical boundary B-C. Water flow is restricted at some boundaries, such as at the impermeable bedrock surface A-B and at the boundary D-E during the dam construction and reservoir impounding/draw down to allow the pore water pressure accumulation. Pore water pressure at some boundaries is gradually increased or decreased to adapt to the new hydrostatic conditions during the reservoir impounding or draw down respectively, such as at boundary G-A and in the upstream dam materials. It should be noted that after the dam construction, a dual hydraulic boundary condition, the precipitation boundary condition in ICFEP (Potts and Zdravković, 1999), which allows an automatic dual prescription of either pore water pressure or water flow, is applied at the bottom of the filter layer (i.e. D-I), in order to simulate the dissipation mechanism of excess pore water pressure in the foundation generated during the dam construction and water impounding.

Table 3: Stages of the static analysis of the Yele dam

Stages	Duration	Dam height (m) /analysis steps	Reservoir level (m) / analysis steps	Construction sections	Notes
0. Initial stress state				Deactivation of sections 2-9	1. Hydrostatic pore water pressure 2. Vertical stresses; $K_0=0.5$
1. Simulation of geological history of dam foundation	100 years	0.0-0.0 / 1-60		Construction of section 2 & long term consolidation	
2. First stage of dam construction before impounding	431 days	0.0-75.0 / 61-160		Construction of sections 3-9 in layers	Boundary stresses are perpendicularly applied on upstream dam surface, to simulate the hydrostatic water pressure induced by reservoir impounding and draw down.
3. Second stage of dam construction /start of reservoir impounding to maximum level	321 days	75.0-124.5 / 161-240	0.0-107.0 / 161-240		
4. Operational stage /reservoir draw down to the level before earthquake	905 days	124.5-124.5 / 241-293	107.0-75.0 / 241-280		

Table 4: Hydraulic boundary conditions in the static analysis

Stages	Dam height (m) / analysis steps	Reservoir level (m) / analysis steps	A-B	B-D	D-I	I-E	E-F	F-G	E-H	G-A
1. Simulation of geological history of dam foundation	0.0-0.0 / 1-60		No water flow	$\Delta p_f=0$	$\Delta p_f=0$	$\Delta p_f=0$	N.A.	N.A.	No water flow	$\Delta p_f>0$
2. First stage of dam construction before impounding	0.0-75.0 / 61-160		No water flow	$\Delta p_f=0$	No water flow	No water flow	No water flow	$\Delta p_f=0$	N.A.	$\Delta p_f=0$
3. Second stage of dam construction /start of reservoir impounding to maximum level	75.0-124.5 / 161-240	0.0-107.0 / 161-240	No water flow	$\Delta p_f=0$	No water flow	No water flow	No water flow	No water flow	N.A.	$\Delta p_f>0$
4. Operational stage /reservoir draw down to the level before earthquake	124.5-124.5 / 241-293	107.0-75.0 / 241-280	No water flow	$\Delta p_f=0$	Precipitation	No water flow	No water flow	No water flow	N.A.	$\Delta p_f<0$ (241-280) $\Delta p_f=0$ (281-293)

Note: compression positive for pore water pressure

4.2 Results

During and after the dam construction, a number of displacement monitoring bolts were placed at different locations of the Yele dam, in order to monitor the real time deformation and to ensure the safe operation of the dam (Wang et al., 2008). These observations include the monitored horizontal displacements at different elevations of the concrete core (shown as a solid line in Figure 7a) and the recorded settlements at five locations on the maximum transverse dam section (denoted as the upper data in Figure 7b). It should be noted that the bolts in the concrete core were placed during the construction of the core section and started to operate immediately after the installation and the data in Figure 7a were obtained at 22 months after the dam construction (i.e. at step 280). The bolts at the downstream berms and at the crest were installed at 3 and 6 months after the dam construction respectively and the data in Figure 7b were recorded at 22 months after the dam construction (i.e. settlements due to the post-construction consolidation at sub-step 245-280 and 250-280 respectively).

The static predictions for the Yele dam are compared with the monitored data to

validate the employed numerical model. Firstly, the predicted horizontal displacement variation along the asphalt concrete core is compared with the monitored data in Figure 7a. Results show the horizontal deformation of the concrete core at two time points, i.e. after the reservoir impounding and after the reservoir draw down (step 240 and 280 respectively). Due to impounding, the concrete core experiences large-magnitude horizontal deformations. The maximum displacements are located at the bottom 1/3 part of the concrete core, reaching about 400mm. However, after lowering the reservoir level in the operational stage, the horizontal deformation of the concrete core reduces by about 25% and the maximum displacements decrease to 300 mm, still at the bottom 1/3 part of the core. The decrease of the horizontal displacements is attributed to the reduced water pressure on the upstream dam surface. In this case, the elastic deformation of the dam materials induced by the water impounding could be recovered due to unloading. Furthermore, a good agreement is observed between the predicted results and the monitored data at step 280, where the magnitude and variation of the horizontal displacements along the concrete core are reasonably simulated by the static analysis.

The settlements predicted at different locations on the Yele dam are compared with the monitored data in Figure 7b, i.e. the incremental settlements on the two downstream berms between the time step 245 and 280 and the incremental settlements on the crest between the time step 250 and 280. The numerical results agree well with the monitored data at different locations. This agreement confirms the validity of the numerical model for the static analysis which ensures that the correct initial stress state is employed in the subsequent dynamic analysis.

(a): Horizontal displacement

(b): Settlements at different locations on the dam

distribution along the core

Figure 7: Static numerical results of the Yele dam

The seepage pattern of the Yele dam is presented in Figure 8, in terms of the compressive pore water pressure contour plots at different stages of the analysis. In particular, Figure 8a shows the pore water pressure contours after the long-term geological history simulation of the dam foundation, where hydrostatic conditions are achieved in the foundation before the dam construction. Figure 8b shows the pore water pressure contours at the end of the first construction stage and before the reservoir impounding (the end of stage 2 in

Table 3). During the construction process, the pore water pressure in the foundation under the dam body increases due to the applied weight of the construction materials and the assumed no-flow hydraulic boundary condition at the interface between the dam body and the foundation soil. However, the pore water pressure variation at the far boundary is not significantly affected and it remains at hydrostatic level. Figure 8c shows the pore water pressure contours just before the Wenchuan earthquake (the end of stage 4 in

Table 3). It can be seen that, following a period of 222 days of consolidation, the excess pore water pressure has been gradually dissipated in the upstream foundation through seeping to the downstream side, approaching the hydrostatic conditions.

Figure 8: Contour plots of pore water pressure of the Yele dam at different stages
(Compression positive: kPa)

5 Dynamic analysis of the Yele dam

The seismic response of the Yele dam subjected to the Wenchuan earthquake is simulated in time-domain FE analysis using the u-p HM coupled consolidation formulation in ICFEP. The Generalised- α time integration method (CH method), proposed by Chung & Hulbert (1993) and extended to the HM coupled formulation in ICFEP by Kontoe et al. (2008), is employed in the dynamic analysis. The initial stress state, before the Wenchuan earthquake, is reproduced by the static FE analysis simulating the dam construction. The boundary conditions for the dynamic analysis are shown in Figure 9, where the cone boundary condition (Kellezi, 1998 & 2000; Kontoe et al., 2009)

is employed at the lateral boundaries and the recorded 2-D bedrock motion is uniformly prescribed at the bottom boundary as the input motion in both the horizontal and vertical directions. For the hydraulic boundary conditions, water flow is restricted at the bottom boundary and the pore water pressure at other external boundaries is prescribed as no change (i.e. $\Delta p_f=0$). The precipitation boundary condition is employed under the filter layer in order to simulate the dissipation mechanism of excess pore water pressure generated in the dam foundation. The time step and duration for the dynamic analysis are 0.02 s and 162 s respectively. The parameters employed for the time integration are listed in Table 5, which satisfy the stability conditions of the CH method under coupled formulation studied in Han (2014) and Han et al. (2015).

Figure 9: FE mesh and boundary conditions for the dynamic analysis of the Yele dam

Table 5: The integration parameters for the CH method

Parameter	δ	α	α_m	α_f	β	ρ_∞
CH method	0.9286	0.5102	-0.1429	0.2857	0.8	0.6

5.1 Acceleration time histories and response spectra

Eight seismometers were installed on the Yele dam to capture the dynamic response in the East-West (EW), North-South (NS) and Up-Down (UD) directions. The EW, NS and UD directions denote the transverse, longitudinal and vertical directions respectively. Usable monitored data were obtained from the seismometers 4 and 7 at the dam crest and base on the downstream slope at the maximum transverse section respectively and from the seismometer installed in the grouting gallery at the left bank (see Figure 10). It should be noted that the grouting gallery was constructed just above the bedrock at the left bank

and therefore the data monitored by this seismometer can be considered as the bedrock motion.

The recorded dynamic data are shown in Figure 11 in terms of acceleration time histories and the interpreted dynamic data are shown in Figures 12 and 13 in terms of acceleration response spectra (5% material damping was employed for all response spectra in this paper) and response amplification spectra respectively. The response amplification spectra are calculated by normalising the spectral accelerations at the crest by the corresponding values at the base or at the bedrock, i.e. crest/base and crest/bedrock amplification. Furthermore, the main features of the seismic data are also summarised in Table 6, in terms of the PA (Peak Acceleration) (see Figure 11), the PA amplification factors and the predominant frequencies of the seismic motions (see Figure 12) at the three locations (i.e. crest, base and bedrock) and the dam fundamental frequencies obtained from the response amplification spectra (see Figure 13). It is interesting to note that the strongest component of the ground motion recorded at bedrock is the vertical one. Furthermore, considering points of increasing elevation, the PA increases from the bedrock to the dam crest, reflecting the amplification effects at different dam locations. The amplification effects are found to be more significant in the horizontal direction than in the vertical direction. The predominant frequencies of the ground motions recorded at the bedrock and the dam crest are larger in the UD direction than in the EW direction, while at the dam base the predominant frequency is slightly smaller in the UD direction. The vertical ground motion is richer in higher frequencies as it is usually the case, although the predominant frequencies in both directions are relatively low and are all below 5Hz. The actual dam response, expressed in terms of spectral ratios (i.e. crest/bedrock and crest/base), indicates that the fundamental frequency of the dam is also larger in the vertical direction which is associated with the larger stiffness in compression mode.

Furthermore, based on the recorded data, the fundamental frequency of the dam vibration is 1.64Hz ($f_{\text{crest/base}}$) in the horizontal direction. The nonlinear fundamental periods of embankment dams can be estimated using an equation proposed by Papadimitriou et al. (2014) which was derived semi-empirically based on a large set of parametric analyses. As a result, the nonlinear fundamental frequency of the Yele dam for the Wenchuan excitation based on the Papadimitriou et al. (2014) methodology is calculated as 1.09Hz (Han, 2014). This is 33% smaller than the monitored one (1.64Hz).

However, it is noted that the back-calculated fundamental frequency is based on the spectral amplification ratio of the recorded motions between the dam crest and the downstream berm, which only accounts for 78% of the entire dam height and is therefore theoretically larger than the actual fundamental frequency of the whole dam body.

Figure 10: The layout of seismometers on the Yele dam

(a): Horizontal-EW direction

(b): Vertical-UD direction

Figure 11: Seismic monitored data (acceleration time histories)

(a): Horizontal-EW direction

(b): Vertical-UD direction

Figure 12: Interpreted seismic monitored data (acceleration response spectra)

(a): Horizontal-EW direction

(b): Vertical-UD direction

Figure 13: Interpreted seismic monitored data (acceleration response amplification spectra)

Table 6: Summary of the seismic monitored data

Monitoring positions	EW direction			UD direction		
	Bedrock	Dam base	Dam crest	Bedrock	Dam base	Dam crest
PA value (m/s^2)	0.071	0.346	0.435	0.119	0.198	0.318
PA amplification factor	1.0	4.9	6.1	1.0	1.7	2.7
Predominant frequency (Hz)	3.41	3.48	2.55	5.00	3.13	3.34
Fundamental frequency (Hz)	$f_{\text{crest/bedrock}} = 2.48$ $f_{\text{crest/base}} = 1.64$			$f_{\text{crest/bedrock}} = 3.45$ $f_{\text{crest/base}} = 3.31$		

The numerically predicted seismic response of the Yele dam is compared with the monitored data in Figures 14 and 15, in terms of acceleration response spectra and acceleration time histories at two monitoring points on the dam crest and base (points 4 and 7 respectively in Figure 9), showing overall good agreement. In particular, the predicted horizontal and vertical response spectra at the dam crest and dam base match well with the monitored response in Figure 14, both in terms of frequency content and amplitude. Furthermore in Figure 15, a good agreement is observed for the corresponding comparison of the two-directional acceleration time histories at the dam crest and base. A small overestimation of the vertical response at the dam base is observed in the frequency range of 4-10 Hz, which is probably due to the low damping assumed for compressional deformation in the small strain range.

Figure 14: Dynamic response of the Yele dam (acceleration response spectra)

(a): Horizontal-EW direction

(b): Vertical-UD direction

Figure 15: Dynamic response of the Yele dam (acceleration time histories)

5.2 Seismic deformation of the Yele dam

In this section and the subsequent two sections, the predicted seismic response of the Yele dam is analysed, in terms of the deformed shape, crest settlements and acceleration distribution pattern, in order to understand its seismic behaviour, assess its seismic safety and provide indication for the application of any potential reinforcement measures. In particular, Figure 16 plots the vectors of accumulated displacement at the end of the Wenchuan earthquake which include the deformations induced by both the static and dynamic loadings. It can be seen that after the Wenchuan earthquake, the deformation of the Yele dam is mainly downward and towards the downstream side, indicating that the overall response is dominated by the effects of the reservoir impounding. The maximum deformation is 0.564 m and occurs at the middle height of the upstream rockfill zone, close to the core. The position and orientation of the maximum displacement are indicated by a grey vector. The predicted maximum displacement of the Yele dam satisfies the Chinese design standards for earth and rockfill dams. In particular, according to the Chinese Specifications for Design of Hydraulic Structures (2000), the maximum permanent displacement on the dam should be less than 1.0% of the dam height, i.e. 1.245 m for the Yele dam. Furthermore, based on the numerical results, no failure is observed on the dam. Therefore the overall deformations of the Yele dam are not significant and are well within the limit for its safe operation during the Wenchuan earthquake. This is also in agreement with the post-earthquake field observations of Cao et al. (2010) and Wu et al. (2009), which concluded that the Wenchuan earthquake did not cause severe damage on the Yele dam. Only some local damage was observed, which however did not influence the satisfactory seismic performance of the dam or the normal operation of the power plant.

The seismically induced deformation in terms of the vectors of accumulated displacement on the Yele dam only due to the Wenchuan earthquake are shown in Figure 17, focusing only on the deformation induced by the dynamic loading. The final seismic deformation in the downstream part is downward and towards the upstream side, whereas the deformation in the upstream part is slightly downward and also towards the upstream side. The maximum seismic deformation occurred on the upstream slope surface, approximately at the middle height and close to the berm. Its magnitude is 0.074 m and the position and orientation are indicated by a grey vector. Overall, the predicted maximum residual seismic deformation satisfies the safety requirements for the Yele dam

and no failure is observed within the dam body based on the numerical simulations.

Figure 16: Vectors of accumulated displacement on the Yele dam after the Wenchuan earthquake
(displacement scaling factor: 1:50)

Figure 17: Vectors of accumulated displacement on the Yele dam only due to the Wenchuan earthquake
(displacement scaling factor: 1:50)

The shear stress contours of the asphalt concrete core are shown in Figure 18 at two stages, i.e. just before and at the 46th second of (corresponding to the peak acceleration of the horizontal input motion) the Wenchuan earthquake. These two plots show the shear stress distribution in the core induced by the static and dynamic loads respectively. The behaviour of the asphalt concrete is simulated by a linear elastic model in the analysis due to the high strength of the material. Based on the results, it can be seen that both the statically and dynamically induced shear stresses are well below the tensile strength of the material (approximately 2.5 GPa according to Huang (1993)). This indicates that cracking or leakage are not likely to occur through the core, in agreement with the observations of the safe operation of the Yele dam before and during the Wenchuan

earthquake. It is noted that the dimensions of the core are skewed for ease of presentation, i.e. X-scale=1:50 and Y-scale=1:1000.

Figure 18: Statically and dynamically induced shear stresses in the asphalt concrete core (kPa)

5.3 Crest settlements

Figure 19 shows the accumulated vertical displacement time histories due to the Wenchuan earthquake at the two monitoring points on the dam crest (point a and d in Figure 20). It can be seen that the vertical displacements at point a (downstream side) are mainly downward and that the final settlement is 3.43 mm. The vertical displacements at point d (upstream side) fluctuate in the upward and downward directions reaching a negligible permanent settlement of 0.52 mm. This is consistent with the overall seismically induced deformation pattern observed in Figure 17, where the downstream part shows a more significant downward deformation trend than the upstream part. Furthermore, the predicted maximum settlement on the dam crest is smaller than the freeboard allowance, i.e. 2.9 m, calculated by subtracting the post-construction consolidation settlement predicted by the static FE analysis (0.1 m) from the design freeboard (3 m), indicating the safe-operation status of the Yele dam during the Wenchuan earthquake.

Figure 19: Accumulated crest vertical displacement time histories at two locations on the dam crest only due to the Wenchuan earthquake

Figure 20: Monitoring points on the Yele dam

5.4 Acceleration distributions within the dam body

The acceleration time histories at six locations on the upstream and downstream slope surfaces are obtained from the dynamic FE analysis (points a-f in Figure 20). The peak accelerations at the six points are plotted against their elevation in Figure 21, showing that from the dam base to the dam crest, the peak acceleration first decreases and then increases. This acceleration distribution pattern is in agreement with the observed seismic response of the Yele dam subjected to a previous low-intensity earthquake in 2007 (Xiong et al. 2008). Furthermore, the peak horizontal acceleration is larger than the peak vertical acceleration on the dam. Considering that the bedrock motion is actually stronger in the vertical direction (see Figure 11), it becomes obvious that the dams amplifies significantly more the horizontal ground motion. The directions of the peak accelerations at the six locations are also marked in Figure 21. The peak accelerations on the downstream slope are mainly downward and towards the upstream side, whereas those on the upstream slope are mostly upward and towards the upstream side. Again, this is consistent with the overall seismically induced deformation pattern shown previously in Figure 17. Finally, at the same elevation, the peak accelerations on the upstream slope are larger than those on the downstream slope, indicating more significant seismic deformation in the upstream

part of the dam. These numerical observations can provide indication for the application of any potential reinforcement measures on rockfill dams.

Figure 21: Peak acceleration variation on the Yele dam

6 Parametric studies of the Yele dam

In this part, the effects of two critical factors on the seismic response of the Yele dam are investigated through parametric studies, i.e. the permeability of the materials comprising the dam body and the vertical ground motion, denoted as parametric study 1 and 2 respectively. The previous dynamic analysis of the Yele dam is employed as the reference case for the presented parametric investigations.

6.1 Effect of permeability of the dam materials

In this section, the effect of the permeability of the materials comprising the dam body on the dynamic response is investigated. The previously employed values of soil permeability are listed in Table 1, while in this parametric study, the permeability values for the dam materials (i.e. the rockfill, coffer dam, cap, transition layer and filter layer) are prescribed to be $1.0\text{E-}8\text{m/s}$ assuming a homogeneous earthfill dam to simulate essentially an undrained response. All other aspects of the numerical model are the same as those for the reference case.

The seismic response at the dam crest and base employing the new lower values of permeability is compared to the reference results in Figures 22 and 23, in terms of

acceleration response spectra and acceleration time histories. The dynamic response in both directions is affected by the soil permeability. Larger response is predicted when employing low permeability ($1.0\text{E-}8$ m/s), both in terms of response spectra amplitudes and transient acceleration values. These differences are more pronounced in the vertical response and they can be attributed to the influence of the fluid-induced viscous damping discussed in Han (2014) and Han et al. (2016a). In particular, for low permeability soils subjected to vertical motions, lower or no viscous damping is introduced as there is very little or no interaction between the solid and pore fluid phases, leading to larger vertical dynamic response. However, slightly larger horizontal response is also predicted by the analysis employing the lower permeability values, which reflects the coupling effects between the responses in the two directions.

The seismic deformation of the Yele dam is further investigated by comparing the vertical displacement time histories at two crest points (points a and d in Figure 20) in Figure 24. It can be observed that, compared to the reference results, the low permeability analysis predicts larger settlements at the downstream crest and more pronounced upward movement at the upstream crest.

Figure 22: Dynamic response of the Yele dam from the parametric study 1 (acceleration response spectra)

Figure 23: Dynamic response of the Yele dam from the parametric study 1 (acceleration time histories)

Figure 24: Accumulated crest vertical displacement time histories at two locations on the Yele dam only due to the Wenchuan earthquake from the parametric study 1

6.2 Effect of vertical ground motion

In this section, the effects of the vertical ground motion on the dynamic response of the Yele dam are investigated. For this parametric study, only the recorded horizontal bedrock input motion is imposed on the Yele dam and the vertical displacements at the bottom boundary are restricted. All other aspects of the numerical model are the same as those employed for the reference case.

The resulting response at the dam crest and base considering only the horizontal ground motion is shown in Figures 25 and 26, in terms of horizontal acceleration response spectra and acceleration time histories respectively. Compared to the reference results, the horizontal dynamic response is not found to be significantly affected by considering only the horizontal ground input motion. Only in the frequency range $>6.0\text{Hz}$, spectral accelerations are slightly smaller, also reflected by the smaller transient acceleration values.

However, Figure 27 shows the comparison of accumulated crest vertical displacement time histories at points a and d (see in Figure 20) due to the earthquake, from analyses employing the 2-D and 1-D input motions. Clearly, when ignoring the vertical ground motion, the vertical displacement amplitudes as well as the stabilised values of settlement

are smaller. The predicted residual settlements by employing the 1-D input motion are approximately half of that subjected to the 2-D input motion indicating that more plasticity is introduced in the reference case. Overall, this comparison highlights the importance of considering 2-D ground motion for a more robust simulation and evaluation of the seismic deformation of rockfill dams.

Figure 25: Dynamic response of the Yele dam from the parametric study 2 (acceleration response spectra)

Figure 26: Dynamic response of the Yele dam from the parametric study 2 (acceleration time histories)

Figure 27: Accumulated crest vertical displacement time histories at two locations on the Yele dam only due to the Wenchuan earthquake from the parametric study 2

7 Conclusions

In this paper, the seismic response of a well-documented Chinese rockfill dam, the Yele dam, was investigated by employing dynamic HM coupled FE analysis. Through the conducted numerical investigation, different aspects of the numerical modelling for static and dynamic analyses of rockfill dams were validated against the available monitoring data for the Yele dam during the 2008 Wenchuan earthquake. The seismic safety of the Yele dam was also assessed by analysing its dynamic behaviour predicted by FE analyses.

First, a detailed static analysis was conducted which simulated the response of the Yele dam under construction, impounding and operation loading conditions. Numerical predictions were compared against the static monitored data, showing satisfactory agreement in terms of the horizontal displacement variation of the core section and the settlements at different locations on the dam.

By employing the simulated static state as the initial stress profile for the dynamic analysis, the seismic response of the Yele dam was then analysed. A good agreement was observed between the numerical results and the dynamic monitored response at two points

at the dam crest and base, in terms of acceleration response spectra and time histories. Further observations regarding the computed seismic response of the Yele dam are summarised below:

- The dam deformation in terms of vectors of accumulated displacement showed that the overall dam deformations were well within the limit for its safe operation during the Wenchuan earthquake. The vectors of accumulated displacement of the Yele dam only due to the seismic event showed that the final seismic deformation in the downstream part was downward and towards the upstream side, whereas the seismic deformation in the upstream part was slightly downward and also towards the upstream side.
- The predicted accumulated crest settlement time histories due to the Wenchuan earthquake are in agreement with post-earthquake field observations which suggest the safe-operation of the Yele dam during the seismic event. The numerical predictions indicate more significant downward crest settlement at the downstream side than that at the upstream side.
- The variation of the peak acceleration within the dam body showed that, from the dam base to the dam crest, the peak acceleration decreases and then increases. This trend is in agreement with the recorded seismic response at the Yele dam during a previous lower intensity earthquake on the 23th of October 2007. Furthermore, at the same elevation, the peak accelerations on the upstream slope are larger than those on the downstream slope, indicating more significant seismic response in the upstream part of the dam.

Furthermore, the effects of two critical factors on the seismic response of the Yele dam were investigated through parametric studies, i.e. the permeability of materials comprising the dam body and the vertical ground motion.

The first parametric study indicated that the vertical acceleration response of the Yele dam was strongly affected by the permeability of the materials comprising the dam body. In particular, by employing lower permeability for the dam materials (i.e. $1.0\text{E-}8$ m/s assuming a homogeneous earth dam), larger vertical dynamic response was predicted due to the absence of viscous damping effects. This in turn resulted in slightly larger horizontal dynamic response due to the coupling of the response in the two directions. Furthermore, the consideration of low permeability values resulted in more significant downward settlements and upward movement at the downstream and upstream crest

locations respectively, compared to the reference analysis which employed considerably higher permeability values for the dam materials ($1.0\text{E-}3$ m/s).

The second parametric study investigated the influence of the vertical ground motion on the dynamic response of the Yele dam. The results indicated that by ignoring the vertical ground motion, the crest settlements can be significantly underestimated, which highlighted the importance of employing 2-D input motion for a more robust simulation and evaluation of the seismic deformation of rockfill dams.

It should be noted that the two parametric studies were performed for a relatively weak bedrock motion. The effect of the soil permeability on the seismic response of the Yele dam can be attributed to the interaction effect between the solid skeleton and the pore water. This hydraulic viscous effect is found to be independent of the intensity of the ground shaking according to Han et al. (2016a). However, the influence of the vertical motion on the seismic response of the Yele dam is likely to be affected by the intensity of the bedrock motion. A further study would be needed to investigate this problem in greater detail considering ground motions of higher intensity.

8 Appendix

Imperial College Generalised Small Strain Stiffness Model (ICG3S model)

The ICG3S model was proposed by Taborda and Zdravković (2012) and Taborda et al. (2016) to simulate complex dynamic soil behaviour under cyclic loading. The model was developed based on the hyperbolic model by Kondner and Zelasko (1963) and the modified hyperbolic model by Matasovic & Vucetic (1993), but involves complex rules to account for some complicated aspects of soil behaviour, such as the independent simulation of shear and volumetric deformation mechanism, spatial variation of soil stiffness and adequate simulation of material damping at very small strain levels. The backbone curve for the ICG3S model is expressed by the integration of Equation (1), where G_{tan} and G_{max} are the tangent and maximum shear moduli and a , b and c are model parameters. It should be noted that in order to account for soil nonlinear behaviour under general loading conditions, the 3-D stress and strain invariants, i.e. J and E_d shown in Equations (2) and (3) (Potts and Zdravković, 1999), are employed to derive the formulation of the backbone curve. Furthermore, the modified 3-D strain invariant E_d^* is employed in Equation (1), which assumes both positive and negative strain values, as

explained by Taborda (2011).

$$\frac{G_{tan}}{G_{max}} = c + \frac{(1-c)}{1 + \left(\frac{|E_d^*|}{a} \right)^b} \quad (1)$$

$$J = \frac{1}{\sqrt{6}} \sqrt{(\sigma'_1 - \sigma'_2)^2 + (\sigma'_2 - \sigma'_3)^2 + (\sigma'_3 - \sigma'_1)^2} = \sqrt{\frac{1}{6} \left((\sigma'_x - \sigma'_y)^2 + (\sigma'_y - \sigma'_z)^2 + (\sigma'_x - \sigma'_z)^2 \right) + \tau_{xy}^2 + \tau_{yz}^2 + \tau_{xz}^2} \quad (2)$$

$$E_d = \frac{2}{\sqrt{6}} \sqrt{(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2} = \sqrt{\frac{4}{6} \left((\varepsilon_x - \varepsilon_y)^2 + (\varepsilon_y - \varepsilon_z)^2 + (\varepsilon_x - \varepsilon_z)^2 \right) + \gamma_{xy}^2 + \gamma_{yz}^2 + \gamma_{xz}^2} \quad (3)$$

where J and E_d are the 3-D deviatoric stress and strain invariants respectively, which can be only expressed by positive values, σ'_1, σ'_2 and σ'_3 are the principal effective stresses, and $\varepsilon_1, \varepsilon_2$ and ε_3 are the principal strains.

Most cyclic nonlinear models simulate hysteretic behaviour considering only the shear stiffness degradation, while bulk and constrained moduli are totally dependent on the shear modulus, assuming a constant Poisson's ratio, in terms of modulus degradation, material damping and reversal behaviour. However, the ICG3S model was developed to be capable of independently reproducing the shear and volumetric deformation mechanism. Therefore, a second backbone curve is specified for the volumetric behaviour, by integrating Equation (4), where ε_{vol}^* is the volumetric strain, K_{tan} and K_{max} are the tangent and maximum bulk moduli, and r, s and t are another three model parameters, corresponding to parameters a, b and c for the backbone curve of the shear deformation. Furthermore, the reversal behaviour for shear and volumetric deformations are also independently simulated by numerically implementing different reversal control procedures. It should be noted that the material Poisson's ratio simulated by the ICG3S formulation is not constant and depends on the respective nonlinear states of the shear and bulk moduli.

$$\frac{K_{tan}}{K_{max}} = t + \frac{(1-t)}{1 + \left(\frac{|\varepsilon_{vol}^*|}{r} \right)^r} \quad (4)$$

After employing the Masing rules, the expressions for the ICG3S model are shown in Equation (5), where two scaling factors, n_1 and n_2 , are employed for the shear and

volumetric stress-strain hysteretic loops respectively. These two scaling factors are independently controlled by the model parameters d_1 - d_4 and d_5 - d_8 . As mentioned in Taborda and Zdravković (2012), the soil material damping at very small strain levels is generally underestimated by the existing cyclic nonlinear models, which could lead to a non-conservative assessment for dynamic analysis of geotechnical structures and limit the applicability of cyclic nonlinear models. Therefore, the varying scaling factors n_1 and n_2 are employed within the ICG3S model to enable more accurate simulation of the material damping in the very small strain range.

$$\begin{cases} \frac{G_{tan}}{G_{max}} = c + \frac{(1-c)}{1 + \left(\frac{|E_d^* - E_{d,r}^*|}{n_1 a} \right)^b} \\ \frac{K_{tan}}{K_{max}} = t + \frac{(1-t)}{1 + \left(\frac{|\epsilon_{vol}^* - \epsilon_{vol,r}^*|}{n_2 r} \right)^s} \end{cases} \quad (5)$$

where

$$\begin{cases} n_1 = (d_1 + 2)^{|E_d^* - E_{d,r}^*|^{d_2}} \left(\frac{(d_3 + 1)|E_d^* - E_{d,r}^*|}{1 + (d_3 + 1)|E_d^* - E_{d,r}^*|} \right)^{d_4} \\ n_2 = (d_5 + 2)^{|\epsilon_{v,o}^* - \epsilon_{v,a,r}^*|^{d_6}} \left(\frac{(d_7 + 1)|\epsilon_{v,o}^* - \epsilon_{v,a,r}^*|}{1 + (d_7 + 1)|\epsilon_{v,o}^* - \epsilon_{v,a,r}^*|} \right)^{d_8} \end{cases}$$

Mohr-Coulomb yield function

The previously described cyclic nonlinear models can only simulate the pre-yield elastic soil behaviour. In this paper, cyclic nonlinear models are coupled with a yield surface defined by a Mohr-Coulomb failure criterion in FE analyses. Plastic deformations can be generated only when the stress state reaches the yield surface. The expression for the Mohr-Coulomb yield function is shown in Equation (6).

$$F = J - \left(\frac{c'}{\tan \phi'} + p' \right) g(\theta) = 0 \quad (6)$$

where

$$p' = \frac{1}{3}(\sigma'_1 + \sigma'_2 + \sigma'_3)$$

$$\theta = \frac{1}{\sqrt{3}} \left(2 \frac{\sigma'_2 - \sigma'_3}{\sigma'_1 - \sigma'_3} - 1 \right)$$

$$g(\theta) = \frac{\sin \phi'}{\cos \theta + \frac{\sin \theta \sin \phi'}{\sqrt{3}}}$$

where J is the deviatoric stress invariant, p' is the mean effective stress, c' is the soil material cohesion, ϕ' is the angle of shearing resistance, θ is the Lode's angle and $g(\theta)$ defines the shape of the yield surface on the deviatoric plane.

9 References

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