Seismic Displacement Analysis in Kulekhani Rockfill Dam

B. R. Pandey [1] and G. Zenz [1]

[1] Graz University of Technology, Institute of Hydraulic Engineering and Water Management,
Stremayrgasse 10/2, A-8010 Graz, Austria

E-Mail: binaya.pandey@tugraz.at

Abstract

In seismically active zones like Nepal, dam design and analysis must prioritize the ability to withstand large deformations, especially under dynamic excitations caused by high seismic activity. The region's intense earthquake accelerations significantly amplify structural responses, revealing pronounced nonlinear behaviour. To capture and predict this nonlinear behaviour, an equivalent linear model is applied. The higher ground motion impact assessment through FEM on a dam is a prerequisite analysis before construction. The acceleration time history of horizontal ground motion, including a highest peak of 0.6g, felt in the Kulekhani Reservoir rock-fill dam, during the Earthquake 2015 in Nepal, was evaluated using Geo-studio. The equivalent linear model (ELM) together with the Newmark approach for a sliding mass were considered during the simulation, to evaluate the permanent displacement in the dam due to earthquake acceleration. The vertical displacement at the crest was evaluated and compared with the post-event field observation. The maximum vertical settlement was found to be comparable with the observed 10 cm of subsidence. The study shows that the crest is at higher risk of subsidence during the ground motion. The slope displacement over the dam's upstream and downstream sides was tested considering a critical slip circle through the Newmark approach. The higher displacement was observed almost 4.8 times on the dam's downstream side compared with the upstream side, indicating that the higher horizontal positive peak ground motion affects more in the downstream slope rather than the dam's upstream side. In this case, the reservoir water load was found to be stabilizing the upstream slope. However, the study concluded that the Newmark approach overestimates the displacement compared with the field observations. The only noticeable rip-rap boulders were found to be shifted in the field during post-event observation whereas 0.72m of potential sliding mass is obtained from the simulation.

Introduction

Earth-fill dams, constructed using compacted soil and rock, are essential for irrigation, water supply, and hydroelectric power generation. Ensuring their stability and integrity during seismic events is crucial due to the significant forces and displacements induced by earthquakes, which can lead to structural failure. Seismic analysis is thus a vital aspect of their design and operation. The Lower and Upper San Fernando Dams of the Van Norman Complex in the San Fernando Valley, California, experienced strong near-source ground motions during the 1994 Northridge and 1971 San Fernando earthquakes [1]. The Lower San Fernando Dam was reported as a near failure during the 1971 San Fernando earthquake. The seismic event caused the upstream slope of the dam to slide, significantly reducing the dam's freeboard and almost leading to a catastrophic release of water. Emergency measures, including reservoir drawdown and sandbagging, prevented a complete failure. This well-known incident highlights the critical importance of seismic analysis and resilient design for earth-fill dams.

Implementing the Finite Element Method (FEM) during the design phase of earth fill dams is crucial for ensuring their seismic resilience. FEM allows for precise modelling of the dam's complex geometry and material properties, enabling accurate simulation of its dynamic response to seismic events. This detailed analysis helps identify potential stress concentrations and deformation patterns, guiding engineers to reinforce vulnerable areas and optimize the overall design. By incorporating FEM, engineers can test various scenarios to achieve a robust and economical structure that meets safety regulations, ensuring the dam's stability and performance during earthquakes. Gordan, B. carried out a FEM analysis over a physical model test under seismic loading to capture the displacement and acceleration at the dam crest. The results have shown the reasonable maximum displacement and acceleration at the crest of the dam as expected [2]. Basudhar, P. K studied the seismic response of earth and rockfill dams using MSC_Nastran, focusing on the Tehri Dam in the Himalayas. A 2D FEM analysis with linear, elastic, non-homogeneous material properties and Bhuj Earthquake base acceleration data was conducted. The findings highlight that the maximum acceleration occurs at the dam crest, displacement is predominantly horizontal, and shear stresses vary significantly between the shell and core of the dam. Velocity-time histories show notable phase differences between the crest and other dam sections [3]. Durmaz, S. proposed a preliminary design procedure for CFRDs, focusing on earthquake-induced permanent displacement and acceleration response. Using numerical models with varied properties and real earthquake records, dynamic analyses were performed with finite element method-based software under two-dimensional plain-strain conditions. Acceleration responses were recorded and permanent displacements were calculated using Newmark’s sliding block approach. The procedure was validated through case histories, offering a method for engineers to estimate likely permanent displacements at the design stage [4]. Han, B. used a dynamic hydro-mechanically coupled FE method to simulate the seismic response of the Yele rockfill dam, validating the model with data from the Wenchuan earthquake. The analysis, which includes deformed shape, crest settlements, and acceleration patterns, confirms the dam’s safety during the event and explores the effects of material permeability and vertical ground motion on its response [5].

This paper studies the seismic impact on Kulekhani Reservoir Rock-Fill Dam, considering the devastating 7.8 magnitude Earthquake event of 2015 with FEM through the “Equivalent Linear Model (ELM)” using Geo-studio. The study particularly aims to evaluate and compare the field observed settlement data with the following objectives:

- Evaluating the maximum dam crest settlement or subsidence.
- Evaluating slope stability using the “Newmark approach”.

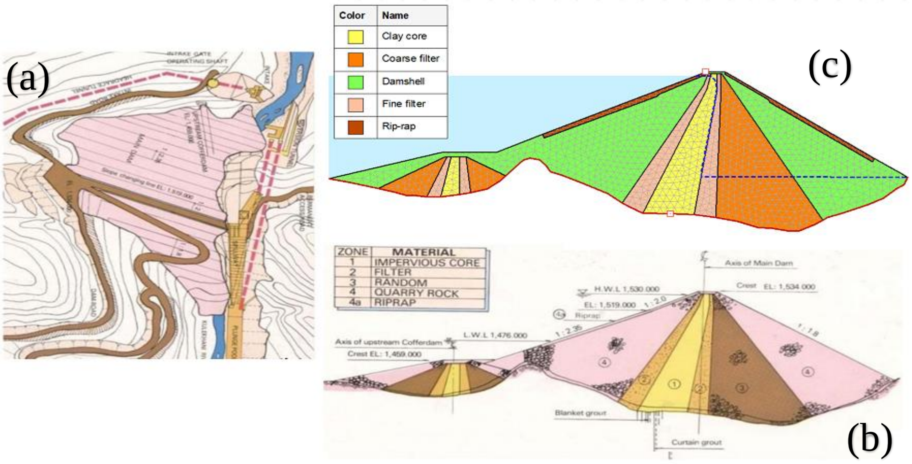

Figure 1:Kulekhani Reservoir Rock-fill Dam plane view (a), built designed cross-section (b), and FEM cross-section (c).

Methodology

The Kulekhani reservoir dam, situated in Dhorsing, Makwanpur, is Nepal's oldest rock-fill dam. The project is owned by the Nepal Electricity Authority (NEA). The dam was primarily constructed for irrigation and hydroelectric power generation, which can handle a Probable Maximum Flood (PMF) of up to 2540 m³/s, considering a watershed area of 126 km² [6,7]. The zonal rockfill dam features an inclined clay core, along with coarse and fine filter materials and boulder riprap for slope protection, as shown in Figure 1. It has an upstream slope of 1:2.35 and a downstream slope of 1:1.8. The clay core extends down to bedrock and is equipped with a grout curtain to prevent seepage [6,7]. The dam has a crest width of 10 meters, a bottom length of 97 meters at the central axis, and a top length of 397 meters, reaching a height of 114 meters.

Problem of Statement

According to the US Geological Survey (USGS National Earthquake Information Center, KTP Station), an earthquake hit the Kulekhani Dam area in the Himalayan region, with a maximum peak ground acceleration (MPGA) of 0.60g for a fraction of a second [8]. According to NEA, post-event field observations revealed that the dam had subsided by 10 cm and developed a crack along the middle of the dam crest [9]. Figure 2 (a) shows the repaired crest crack and (b) represents the recorded 10 seconds of horizontal ground motions with a maximum peak acceleration of 0.60g.

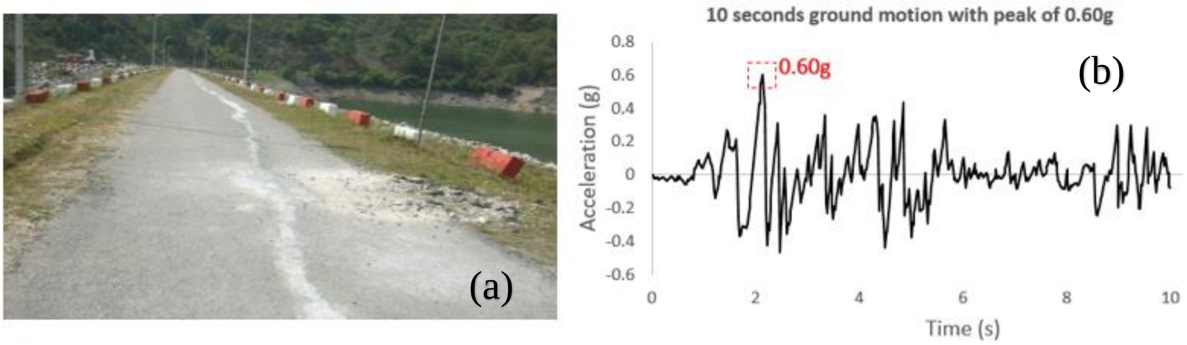

Figure 2: Repaired dam crest cracks (a) and recorded horizontal ground motion (b)

Governing Equations

The governing equation for the dynamic response of a system in finite element formulation can be expressed as follows [10]:

$$[M]\{\ddot{a}\} + [D]\{\dot{a}\} + [K]\{a\} = \{F\} \tag{1}$$

Where, $[M]$ is the mass matrix, $[D]$ is the damping matrix, $[K]$ is the stiffness matrix, $\{F\}$ is the vector loads, $\{\ddot{a}\}$ is the vector of nodal accelerations, $\{\dot{a}\}$ is the vector of the nodal velocities and $\{a\}$ is the vector of the nodal displacements.

QUAKE/W employs the Cholesky Factorization technique to solve finite element equations, a method closely related to Gaussian elimination. QUAKE/W uses Implicit methods to solve the global system of equations that represent the entire finite element model. Factorization is a key step for efficiently solving large matrix systems, in this case, Cholesky factorization is utilized. The details can be followed by the QUAKE/W theory manual [11].

Materials properties

The material properties of the dam shell, fine filter, coarse filter, rip-rap of unit weight (γ), permeability (k_s), the volume of water content (w.c), residual water content (res.w.c), compressibility coefficient (m_v), frictional angle ($\boldsymbol{\phi}$), cohesion (c), Modulus of elasticity (E) and Poisson's ratio (ν) were considered from Pandey. B.R [11] and AASHTO [12], as presented in the Table *1*.

Table 1: Material properties

Materials	k_s (m/s)	W.C	Res. W.C	m_v (kPa^{-1})	γ (kN/m^3)	$\boldsymbol{\phi}$	c (kPa)	E (MPa)	ν
Clay core	1×10^{-8}	0.50	0.050	3.3×10^{-5}	16	25	10	30	0.45
Dam Shell	1×10^{-4}	0.40	0.040	2×10^{-5}	20	30	8	50	0.3
Fine Filter	1×10^{-3}	0.30	0.030	4×10^{-5}	18	32	2	25	0.25
Coarse Filter	1×10^{-2}	0.25	0.025	1.6×10^{-5}	22	35	1	60	0.3
Rip-rap	1×10^{-1}	0.20	0.020	1×10^{-5}	27	37	0.5	95(GPa)	0.35

Dynamic materials properties

The dynamic shear modulus (G) and damping ratio (ξ) as a function of the cyclic shear strain are dominating factors in the seismic evaluation. This relation is generally obtained through the laboratory test of the dynamic triaxial test as shown in Figure *3* [13].

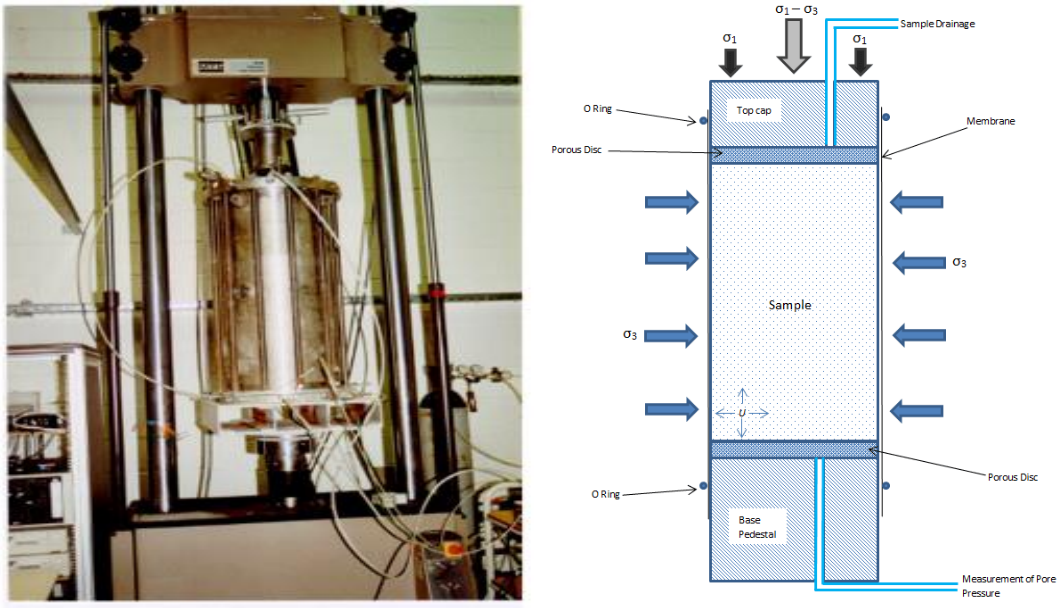

Figure 3: Dynamic triaxial test setup, Wieland, M [13]

G-reduction function (G/G_{max})

A soil subjected to dynamic stresses tends to 'soften' due to cyclic shear strain. The reduction of dynamic shear modulus (G/G_{max}) as a function of the cyclic shear strain (%) was obtained through the well-known validated empirical relation defined by Ishibashi and Zhang (1993) through Geo-studio [10]:

$$\frac{G}{G_{max}} = K\ (\gamma, \mathrm{PI})(\sigma'_m)^{m(\gamma,\mathrm{PI})-\mathrm{m}_0} \qquad (2)$$

The empirical relation is associated with the plasticity index (PI) and mean stress (σ'_m).

The maximum dynamic shear modulus (G_{max}) is obtained through the following equations [10]:

- **For granular soil:**

$$G_{max} = 22K\sqrt{P_a \sigma'_m} \tag{3}$$

Where, Pa is the atmospheric pressure, K is the dimension less modulus value, σ'_m is the mean stress.

- **For cohesive soil:**

$$G_{max} = 625\left(\frac{1}{(0.3+0.7e^2)}\right)(OCR)^k\sqrt{P_a \sigma'_m} \tag{4}$$

Where, e is the void ratio, OCR is the over-consolidation ratio, k is the exponent related to PI.

Damping ratio function (ξ)

Following by the reduction of dynamic shear modulus (G/G_{max}), Ishibashi and Zhang (1993) also developed an expression that can be used to estimate the damping ratio (ξ) function (Kramer, 1996) which has been implemented in Geo-studio and has been used during the simulation [10]. The relation is expressed as:

$$\xi = 0.333\,\frac{1+\exp\left(-0.0145PI^{1.3}\right)}{2}\left[0.586\left(\frac{G}{G_{max}}\right)^2 - 1.547\frac{G}{G_{max}} + 1\right] \tag{5}$$

The plotted Figure 4 (a) shows the reduction of dynamic shear modulus (G/G_{max}) and (b) shows the damping ratio (ξ) as a function of the cyclic shear strain (%) which have been implemented during the simulation for different materials of the dam.

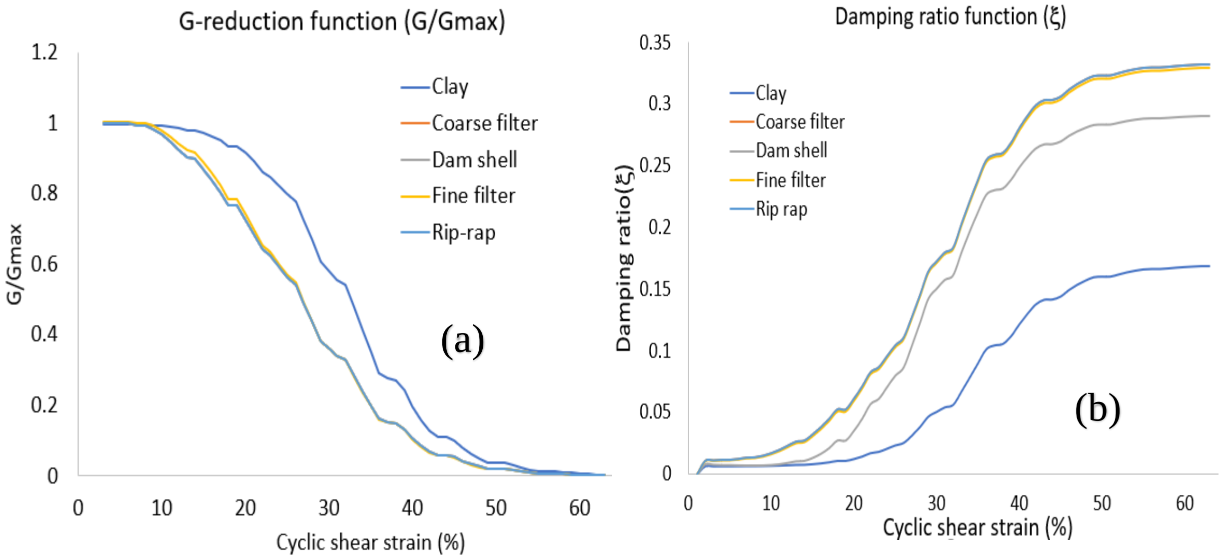

Figure 4: G-reduction function plot (a) and Damping ratio plot (b) for different used materials.

Equivalent Linear Model (ELM)

The equivalent linear model (ELM) is widely used in seismic evaluation for geotechnical engineering because of its balance between computational efficiency and reasonable accuracy. It simplifies the complex non-linear behaviour of soils during seismic events into an iterative process, making it accessible and practical for engineers. ELM considers several critical factors [10]:

1. Initial Soil Properties: The model investigation starts with the estimated linear properties of the soil, including shear modulus and damping ratio.
2. Strain Calculation: It calculates the strain levels in the soil layers from the initial analysis, focusing mainly on shear strain.
3. Modulus Reduction and Damping Curves: The model uses predefined modulus reduction and damping curves to update soil properties based on the calculated strains. These curves account for the non-linear degradation of soil stiffness and the increase in damping with strain.
4. The analysis iterates, updating the soil properties after each run based on the new strain calculations, until the changes between iterations converge within a set tolerance.
5. Seismic Loading: The model incorporates seismic loading conditions, such as acceleration time histories, which are applied at the boundaries or within the domain to simulate real-world seismic events.

Newmark approach for sliding

The Newmark integration approach for sliding mass is used to evaluate the permanent deformation or displacement during seismic loading. The average acceleration (a_{ave}) is envaulted using the following relation for a sliding mass [10]:

$$a_{ave} = \frac{\sum S_m - \sum S_{m(static)}}{\sum W} \tag{6}$$

Where, S_m is the current and $S_{m(static)}$is the initial (static) mobilized shear force, and W is the slice weight.

The yield of average acceleration (a_y) is estimated based on the assumption that the slice tends to move when the factor of safety (FOS) is less than 1. Time integration of the area above yield acceleration leads to the time-velocity distribution with the following relation [10]:

$$V = \int_t^{t+dt} \left(a_{ave} - a_y\right) dt \tag{7}$$

Further time integration leads to a permanent displacement as follows [10]:

$$d = \int_t^{t+dt} V \, dt \tag{8}$$

The limitation of the Newmark sliding approach:

- Inapplicability to Situations with Excess Pore-Water Pressure Generation.
- Not Suitable for Soils with Structural Collapse or Liquefaction.
- Limited Applicability for Soils with Significant Shear Strength Degradation.

Results and Discussions

The vertical settlement at the crest of the dam was evaluated considering a 10-second time series of horizontal ground motion with a maximum peak acceleration of 0.6g. The time series of displacement in Figure 5 shows the maximum displacement of 9.1 cm at the dam crest, which was found to be close to the post-event field observed displacement of 10cm. The results show that 10 cm of subsidence is capable of forming a crack in the dam section, increasing a risk to the dam for further damage. Due to higher crest subsidence risk during seismic events, the Nepal Electricity Authority (NEA) was restricted from filling up the reservoir to avoid additional structural damage until 1525 m.a.s.l. Higher subsidence will reduce freeboard and ultimately increase the risk of over-topping. Furthermore, a higher water level in the reservoir increases the pressure on the dam which further enhances to widens the crack. However, the quick response of sealing the crack with clay or bentonite slurry to fill the crack, followed by grout injection for additional stabilization, has reduced the risk, and no further damage has been observed lately. The study has shown that the crest is at risk of subsidence during the ground motion to cause damage.

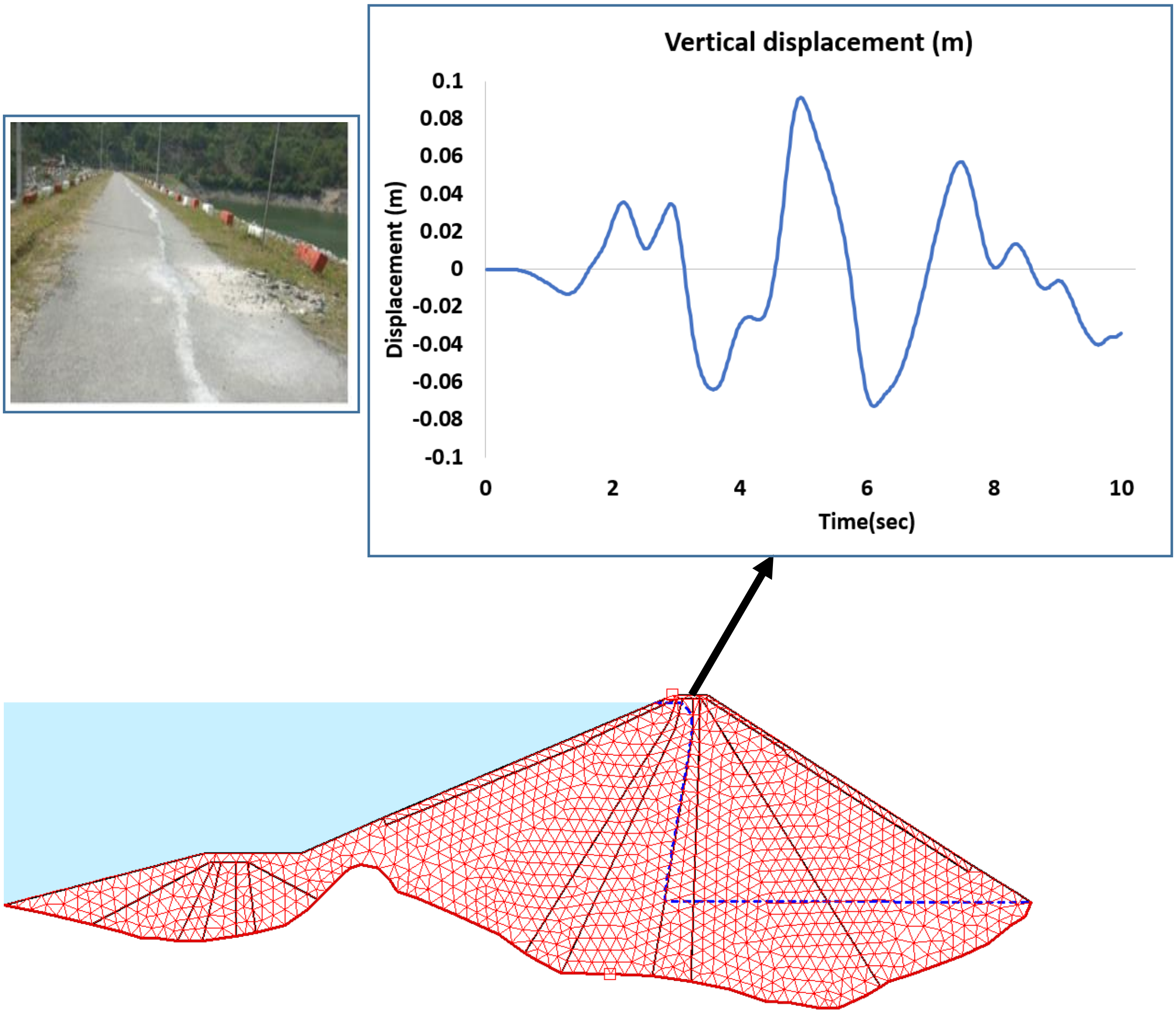

Figure 5: Vertical displacement at crest due to ground motion.

Further study of expected risk associated with permanent slope displacement was carried out with the Newmark approach over the upstream and downstream slopes of the dam. The critical slip circle was considered during the study. Figure 6 shows the implementation of time double-integration of average acceleration to adopt permanent displacement. The yield average acceleration of 0.05g and 0.12g were adopted concerning FOS less than 1 for the expectation of sliding for downstream and upstream slopes. The permanent displacement of 0.72m was obtained on the dam's downstream side as shown in Figure 6, which can be visualized in the displacement plot. Moreover, the upstream slope displacement of 0.15m was evaluated and expected to be less displaced than the downstream slope due to the stabilizing water load of the reservoir, as represented in Figure 7. The result has shown that the positive higher peak increases the risk of sliding on the downstream side rather than the upstream side. As per post-event field observation, only the noticeable movement of rip-rap was observed at the downstream side of the dam, though the sliding displacement was found to be 0.72m during the simulations.

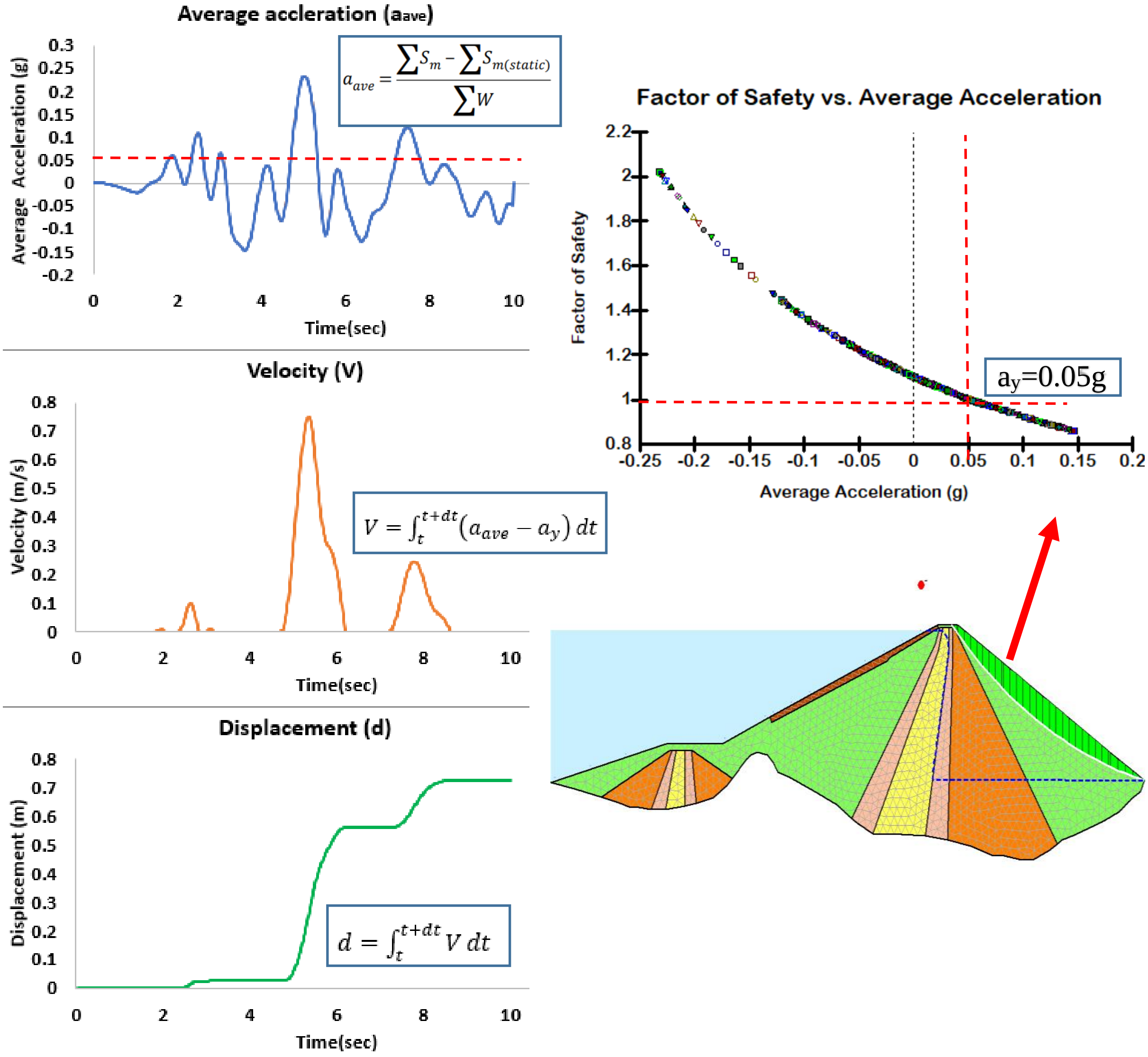

Figure 6: Evaluation of displacement at downstream slope

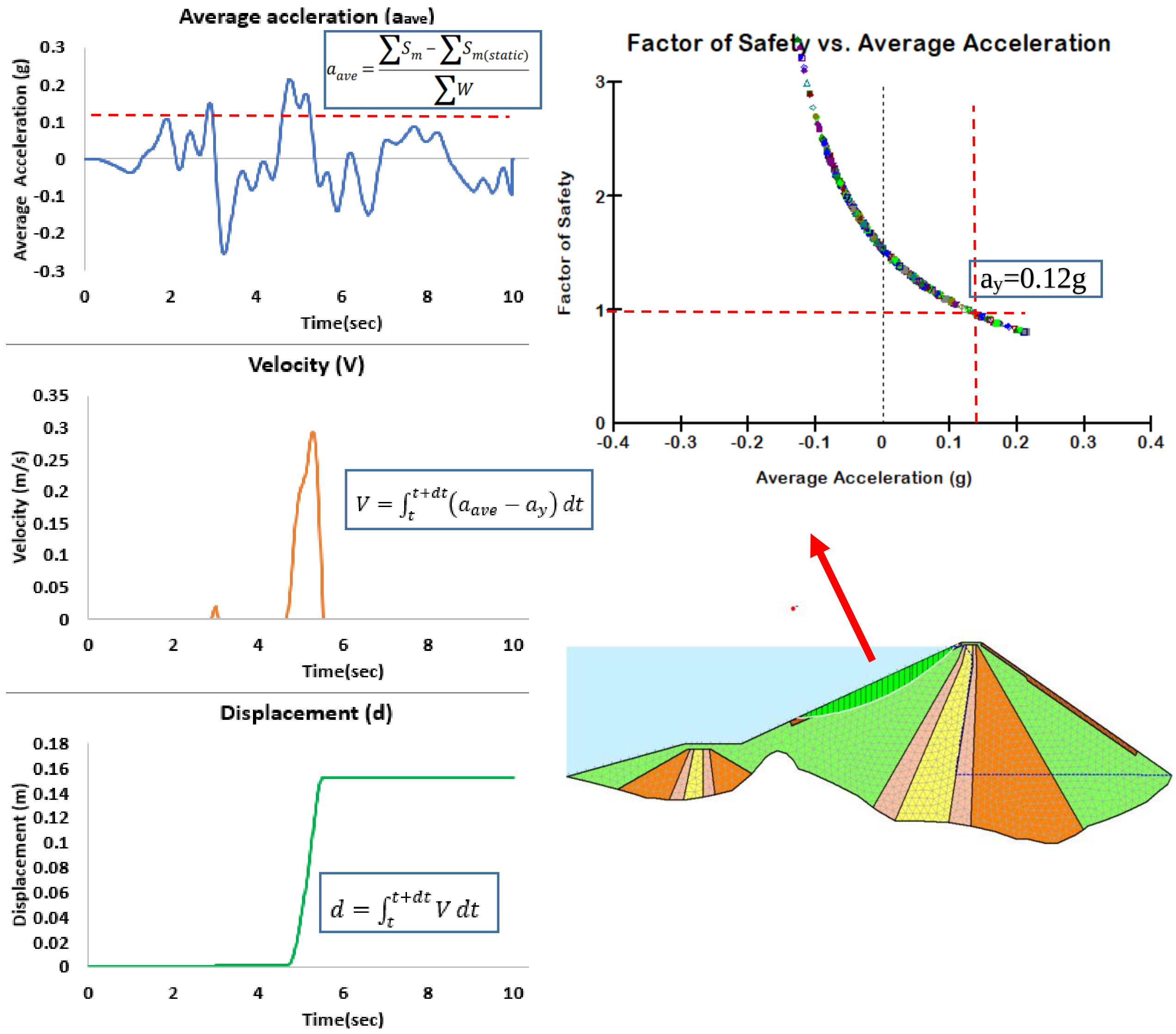

Figure 7: Evaluation of displacement at upstream slope

Conclusions

The seismic evaluation study on rock fill dam draws the following specific conclusions:

- The application of an equivalent linear model (ELM) was found to be feasible, as compared to the result of the vertical settlement of the field post-event observations. The reasonable vertical displacement of almost 10cm was re-evaluated by the means of FEM as compared to the field observation. The study emphasizes that the higher potential risk of crack formation is associated with vertical settlement in the crest during seismic events.
- A higher risk of dam displacement is observed on the downstream slope compared with the upstream slope during the higher horizontal peak acceleration of 0.6g. The displacement of the dam at the downstream slope was found to be 4.8 times higher than the upstream side of the dam.
- Although the higher risk of sliding mass of 0.72m was found at the downstream slope in the simulation results, no noticeable large movement was observed in the field than the movement of boulder rip-rap. Thus, the study of Newmark analysis overestimated the displacement results and is not able to fully capture the overall behaviour due to the complexity of geotechnical parameters and in situ placement of rip-rap.

List of References

[1] Bardet, J. P., & Davis, C. A. (1996). Performance of San Fernando dams during 1994 Northridge earthquake. Journal of Geotechnical Engineering, 122(7), 554-564.

[2] Gordan, B., Raja, M. A., Armaghani, D. J., & Adnan, A. (2022). Review on dynamic behaviour of earth dam and embankment during an earthquake. Geotechnical and Geological Engineering, 40(1), 3-33.

[3] Basudhar, P. K., Rao, N. S. V., Bhookya, M., & Dey, A. (2010). 2D FEM Analysis of Earth and Rockfill Dams under Seismic Condition.

[4] Durmaz, S., & Ülgen, D. (2021). Prediction of earthquake-induced permanent deformations for concrete-faced rockfill dams. Natural Hazards, 105(1), 587-610.

[5] Han, B., Zdravkovic, L., Kontoe, S., & Taborda, D. M. (2016). Numerical investigation of the response of the Yele rockfill dam during the 2008 Wenchuan earthquake. Soil Dynamics and Earthquake Engineering, 88, 124-142.

[6] Shrestha, H.S. Sedimentation and Sediment Handling in Himalayan Reservoirs. Ph.D. Thesis, Norges Teknisk-Naturvitenskapelige Universitet, Trondheim, Norway, 2012. Available online: https://ntnuopen.ntnu.no/ntnu-xmlui/handle/11250/242352 (accessed on 25 July 2024)

[7] UN. Kulekhani Dam Break Study. 2012. Available online: https://digitallibrary.un.org/record/211850?ln=en (accessed on 25 July 2024)

[8] US Geological Survey. (2015). An earthquake with peak ground acceleration near Kulekhani Dam. Retrieved from [https://www.usgs.gov]

[9] NEA (2015, August 10). NEA not to let Kulekhani reservoir fill up completely. The Kathmandu Post. Retrieved from [https://kathmandupost.com/money/2015/08/10/nea-not-to-let-kulekhani-reservoir-fill-up-completely]

[10] Geostudio. (2024). QUAKE/W theory manual. Retrieved from [https://www.seequent.com/products-solutions/geostudio-2d/]

[11] Pandey BR, Knoblauch H, Zenz G. Slope Stability Evaluation Due to Reservoir Draw-Down Using LEM and Stress-Based FEM along with Mohr–Coulomb Criteria. Water. 2023; 15(22):4022. https://doi.org/10.3390/w15224022

[12] AASTHO. LRFD Bridge Design Specifications (SI); American Association of State Highway and Transportation Officials: Washington, DC, USA, 2007; p. 1037.

[13] Wieland, M., & Malla, S. (2002, September). Seismic safety evaluation of a 117 m high embankment dam resting on a thick soil layer. In 12th European Conference on Earthquake Engineering (Vol. 3, pp. 836-843).