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Seismic Slope Stability and Liquefaction Potential of Large Existing Local Dams

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Abstract **-** In order to increase the local material dam capacity to sustain from the big floods as the effect of climate change recently, the existing dams in Vietnam are frequently considered the two remedial design proposals either increasing the spillway discharge capacity or raising the dam height to increase the storage volume of the reservoir during the flood season. In addition, regarding the dam constructed on the earthquake prone area, the seismic slope stability and liquefaction potential caused by strong earthquake need to be carefully examined. Numerical study was implemented on the Phu Vinh large dam based on the site geotechnical data with many scenario loading cases. The analysis results showed that the static slope stability factors of safety could be satisfied by the existing standards; however, seismic factors of safety decreased significantly under unity with three cases of earthquake (T=475 years, 145 years and 10,000 years). In addition, liquefaction potential could be possible at the sandy layer under downstream rockfill. The safety measures need to be implemented in the design and construction of the existing and /or the new large dams.

Keywords**:** In-situ dam, earthquake, liquefaction, cyclic loading, seismic slope stability

1. Introduction

Local material dams are widely used in Vietnam for the construction of reservoirs for irrigation and hydropower. At present time, Vietnam has a total of about 6648 hydraulic reservoirs with a total storage capacity of about 13.5 billion m^3, including 702 large reservoirs and 5,946 small reservoirs [5]. In addition, there are more than 1,100 hydropower dams being either constructed or planned.

In order to increase the capacity of local material dams to sustain from the big floods as the effect of climate change recently, the existing dams in Vietnam are frequently considered the two main proposals, namely either increasing the spillway discharge capacity or raising the dam height to increase the reservoir volume to keep the flood water during the flood season.

Earthquakes are one of the major disasters for dam safety ([1],[8],[10]). The territory of Vietnam is located in a land with a complex geological - tectonic structure. Some strong earthquakes have occurred in the northwest region of the country such as the earthquake (M=7) in Lai Chau in 1914, Dien Bien in 1920 occurring in the Lai Chau-Dien Bien fault, the 1926 Son La earthquake on Son La fault, the 6-7 level Ta Khoa earthquake (M=5) and a number of level 7 earthquakes, which were aftershocks of the 1983 Tuan Giao earthquake [15]. Historically, reservoir dams have been observed damaged in strong earthquakes ([1],[10]). The damage could be due to the liquefaction of the embankment and/or foundation soil ([1][10]).

Note that some of the local material dams in the country are large dams that are filled with water-saturated gravel and sand and/or use coarse-grained backfill material that has a risk of liquefaction during strong earthquakes. Seismic design for dams has been included in design standards in several countries ([3],[14]). However, the seismic design standard for local material dams in Vietnam has not yet been released.

In the present study, numerical work was implemented on Phu Vinh dam with many scenarios loading cases to check the static and seismic slope stability and liquefaction potential in the design during the period of construction drawings.

2. Study Project

Phu Vinh reservoir project is located in the central province of Quang Binh, about 7 km west of Dong Hoi city centre (Fig.1). The headwork was built on the Phu Vinh river with the original purpose of creating a source of water for irrigation in the downstream area (including Dong Hoi city and related communes). The project provides an irrigated area is about

1,510 ha and creates water supply for domestic and industrial use 18,000 m^3/day. The reservoir has a capacity of about 22.36 million m^3 of water. Because the reservoir was built in 1992, so far, some components have been degraded and damaged noticeably, for example, the upstream slope protection have been eroded, the damaged water intake has caused seepage problem. Every year in the rainy season, technical problems can occur. The unsafety of the dam could affect human life at the downstream of the reservoir; thus, the repair and upgradation to ensure the safety of the dam is an urgent task.

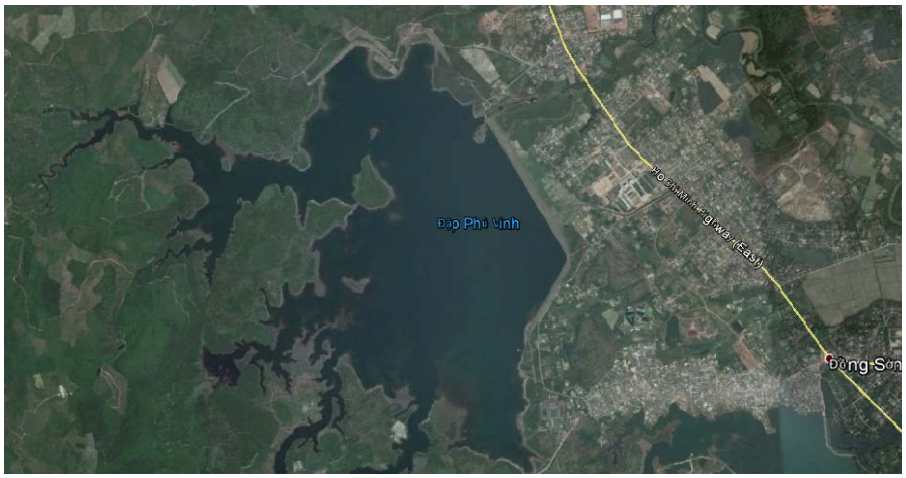

Fig. 1: Phu Vinh reservoir.

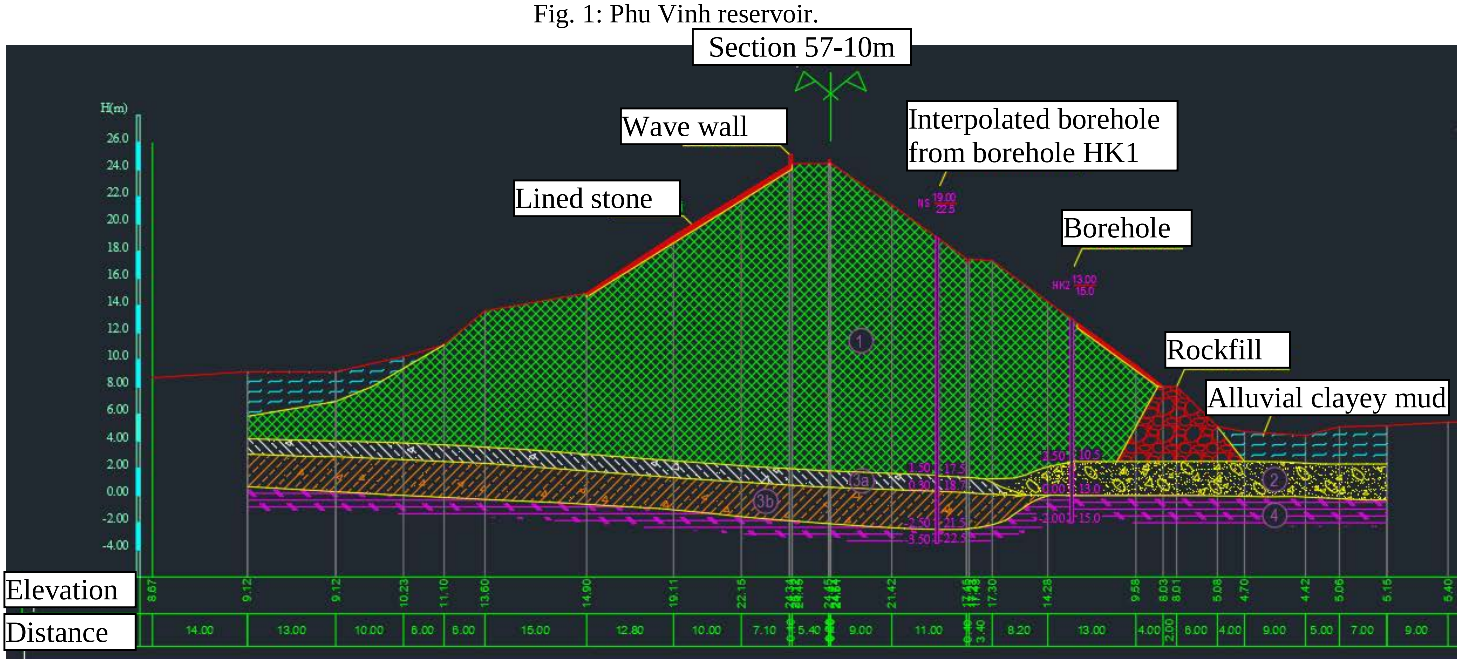

Fig. 2: Phu Vinh dam geotechnical existing cross section No. 57.

Table 1. Description of soil layers

Layer	Description
1a	A mixture of dark brown, yellow-gray clay gravel, compact structure, hard state. It is distributed on the surface down to the survey depth, being the filling soil on the existing dam crest taken in the neighbouring areas. The average thickness is about 0.4 meters.
2a	Distribution on the surface down to the survey depth, in the area of the dam crest line, the surface layer is covered by layer (1a). The main composition in the layer is clay and gravel with yellow-gray, light yellow color, from semi-hard to hard state. Max layer elevation +24.05, min layer depth +1.5. The thickness of the layer varies according to the dam surface. At the top of the dam, the average thickness is 22.5m.
2	Distributed just below the layer (2a), the distribution only occurs in the area downstream of the dam, which is the remnant of the old slit bed. Soil mixed with coarse-grained sand, gray-black, gray-white, yellow-grey, medium-tight texture, water saturation (alluvial origin, slit-bed accumulation). SPT (N_{30}) =16-34. At the the location of borehole HK1(BS), the layer surface at an elevation of +2.60, and at borehole HK2(BS), the layer surface at an elevation of +3.07. The thickness of the layer varies from 2.0 to 3.3m.
3a	Distributed just below layer (2a), the distribution area is mainly in the area of the dam body towards the upstream, formed by the complete weathering process from the siltstone bedrock. The main composition is clay mixed with gray-green, dark gray gravel, hard plastic state to semi-hard (origin of remnants from siltstone). At the position of interpolated borehole NS(BS), the layer surface at +1.50, at borehole HK1(BS) and HK2(BS) layer did not appear. The average thickness is about 1.2 m.
3b	Distributed just below the layer (3a), the distribution area is mainly in the area of the dam body towards the upstream, formed by the complete weathering process from the siltstone bedrock. The main composition in the layer is clay mixed with yellow-gray, gray-white gravel, semi-hard ÷ hard state (origin of remnants from siltstone). At the interpolated borehole NS(BS) the layer surface at an elevation of +0.30, at borehole HK1(BS) and HK2(BS), the layer did not appear. The average thickness of the layer is about 2.8m.
IB	Distributed just below layers (2) and (3b), is the bedrock layer with uneven weathering, layer surface elevation changes complicatedly, the distribution area is spread over most of the construction area. The main composition in the layer is yellow-gray, white-gray siltstone, strong weathering degree ÷ medium, cracked rock, low hardness. At the interpolated borehole NS(BS) encountered the layer surface at an elevation of -2.50, and at borehole HK1(BS), the layer surface at +0.60. The thickness of the layer has not been determined.

Table 2. Soil material properties

Soil layer	K	γ	φ	C	E	ϑ	G
	m/s	kN/m^3	Degree	kPa	kPa	-	kPa
2a	3.96E-07	20.65	14.0	26.3	14793	0.35	5478.9
3a	8.25E-07	20.36	14.8	24.3	12557	0.35	4650.7
3b	2.18E-07	20.51	15.4	35.1	20036	0.35	7420.7
IB	3.96E-07	20.00	30.0	15.0	100000	0.20	41666.7
2 (sand)	5.00E-04	18.00	35.0	0.0	30000	0.20	12500.0
Downstream rockfill	1.00E-02	22.00	35.0	0.0	50000	0.20	20833.3
Filtration	1.00E-02	20.00	27.0	0.0	30000	0.20	12500.0
Upstream rockfill	1.00E-02	22.00	35.0	0.0	50000	0.20	20833.3
Filling soil	3.96E-07	18.60	15.0	21.0	14793	0.35	5478.9

Figure 2 shows the geotechnical existing main dam cross section No. 57. Based on the geological record [17], within the scope of the survey to a maximum exploration depth of 22.5m, the stratigraphy of the main dam route at the site and the distribution of soils in order from top to bottom, consist of the following main soil layers (Table 1).

In order to increase the Phu Vinh local dam capacity to sustain from the big floods in the coming time, two remedial design proposals were taken into consideration, namely either increasing the spillway discharge capacity or raising the dam height. Considering the complexity when widening the spillway at the left dam abutment as a result of difficult geological condition, the proposal of raising the dam height was selected. According to the design proposal, the top elevation is increased from +24.2 to +25.0; the top width widened from 5m to 6m; the top of wave wall raised from +25.2 to +25.8. Thus, the maximum height of dam increases from 20.0 to 20.8m. The slope of the upstream face is remained at 3.5, while the existing slope of downstream face (3.25) has been modified varyingly from 3.0 to 3.25. Checking slope stability and liquefaction for the new remedial design cross section were implemented under different loading scenarios as described below.

3. Modelling of seismic slope stability and liquefaction

3.1 Loading scenarios

There are 11 loading scenarios implemented as summarised in Table 3 below including different combination of water elevations at upstream and associated downstream of the dam by the relevant standard [13].

Table 3. Loading scenarios

Case	Description	UWL (m)	DWL (m)
Case 1	Basic case: Reservoir water at normal water level, steady state seepage	22.0	6.5
Case 2	Basic case: Reservoir water at design flood level (p=1%), steady state seepage	22.98	8.9
Case 3	Special case: Reservoir water at check flood level (p=0.01%)	24.8	8.9
Case 4	Special case, Reservoir water at normal water level, the filtration clogged	22.0	6.5
Case 5	Special case: reservoir water at check flood (p=0.01%), the filtration clogged	24.8	8.9
Case 6	Drawdown: Reservoir water from design flood level to normal water level	22.98→22.0	8.9
Case 7	Drawdown: Reservoir water from check flood level to normal water level	24.8→22.0	8.9
Case 8	Drawdown: Reservoir water from normal water level to spillway crest water level	22.0→17.0	6.5
Case 9.1	Special case. Reservoir at normal water level, design earthquake (T=475 years)	22.0	6.5
Case 9.2	Special case. Reservoir at normal water level, OBE earthquake (T=145 years)	22.0	6.5
Case 9.3	Special case. Reservoir at normal water level, SEE earthquake (T=10,000 years)	22.0	6.5

Note: UWL= upstream water level; DWL=downstream water level.

3.2 Seismic data

We consider the earthquake by Vietnamese standard TCVN 9386:2012[12] with a return period of 475 years, peak ground acceleration PGA=0.095g at Dong Hoi city, Quang Binh province (Case 9.1). Since Phu Vinh is a large dam, we consider the multi-level seismic design parameters, namely the Operating Basis Earthquake (OBE), return period T=145 years, PGA=0.064g (Case 9.2) and the Safety Evaluation Earthquake (SEE), T=10,000 years, PGA=0.26g (Case 9.3) as recommended by ICOD [7]. The OBE means "no damage" earthquake and for this seismic acceleration all required factors of safety should be >1.1. The SEE is a "no failure" earthquake for which a factor of safety on sliding could be <1, as long as the deformations are acceptable and there is no uncontrolled release of water. As the TCVN 9386:2012 only provides seismic accelerations for 1 in 475 year earthquake, the seismic accelerations for return periods different for 1 in 475 year are calculated using the Eurocode 8 (EN 1998-2)[2]. Due to the limitation in the observation stations at site, we imported the earthquake acceleration time history sample of QUAKE/W program [4] in all seismic computation (Fig. 3) for the three above earthquakes. For coarse grain materials like layer 2, the dynamic soil properties such as excess porewater pressure function, liquefaction curve, Ka, Ks functions were obtained by employing the QUAKE/W library functions for the loose sand (Fig. 4).

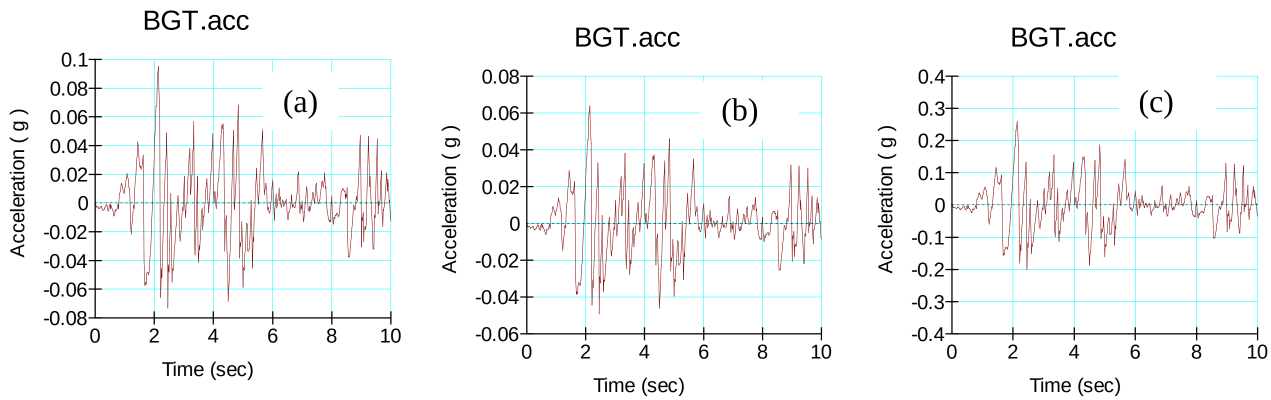

Fig. 3: Employed acceleration time histories a) T=475 years, b) T=145 years, c) T=10,000 years.

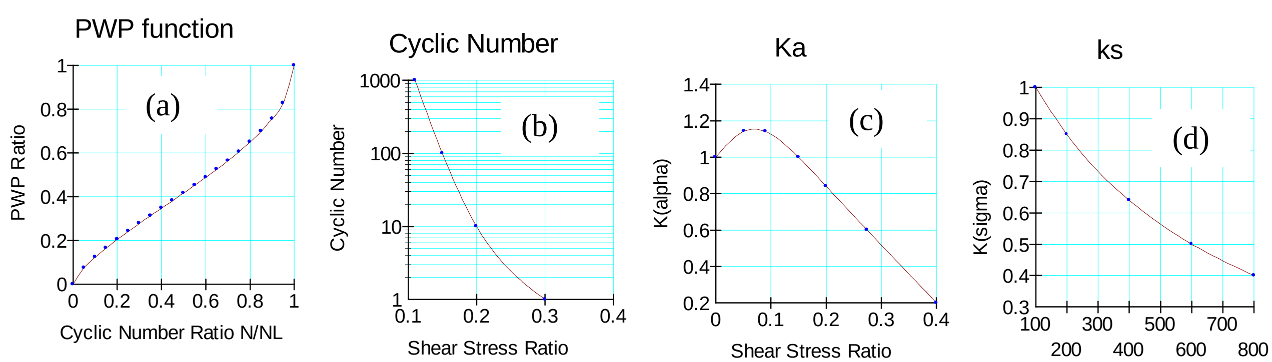

Fig. 4: Employed dynamic functions for sand layer 2.

3.3 Finite Element Modelling

The numerical analyses were carried out by the finite element method for a plane strain problem. The typical design section based on the existing cross section No. 57 was employed for the simulation (Fig. 2).

Note that the existence of saturated sand layer 2 in the dam foundation under the rockfill prism can trigger a liquefaction if the strong earthquake happens.

The finite element software GeoStudio [4] was employed. The linear elastic model was applied to all soil layers for simplicity. Table 2 shows the model parameters of the soil layers. Note that some of the soil parameters such as Poisson's ratio ϑ, damping ratio h (=0.1) were assumed with referring literatures; and shear modulus was deduced from Young's modulus and Poisson's ratio.

The initial porewater pressures in the dam body and foundation were simulated under the steady state seepage condition with the reservoir's water level by SEEP/W module (Fig. 5). After that, module QUAKE/W was employed to calculate the initial stress condition. Next, the dynamic analysis was activated to simulate the earthquake loading by applying the input acceleration time history record on the problem boundary (Fig. 3). The results from QUAKE/W analysis were then employed for dam slope stability analysis by finite element method, using module SLOPE/W. The time duration was assumed to be 10s, which was divided into 500 time steps.

For the static case (Case 1 to 8), the slope stability of dam was implemented by limit equilibrium method employing Morgenstern-Price method. In which, the grid 16x16 of centres of 16 potential slip surfaces for each centre, leading to 4096 potential slip surfaces were examined (Fig. 6).

Table 4 shows that the static factors of safety (FOS) of all the upstream and downstream dam slopes satisfied the allowable FOS by the existing standards ([9],[13]). However, in the event of an earthquake, the factors of safety, which were analysed with the three earthquakes, reached the smallest values at t=5.4s (Fig. 7). Even the OBE, the minimum FOS fell below unity (0.920) at t=5.4s (Fig. 7b). For SEE, with the return period of 10,000 years, the minimum FOS decreased to the smallest value of 0.454 at t=5.4s (Fig. 7c). Thus, it is necessary to pay much attention to safety measures for the dam when the strong earthquakes occur.

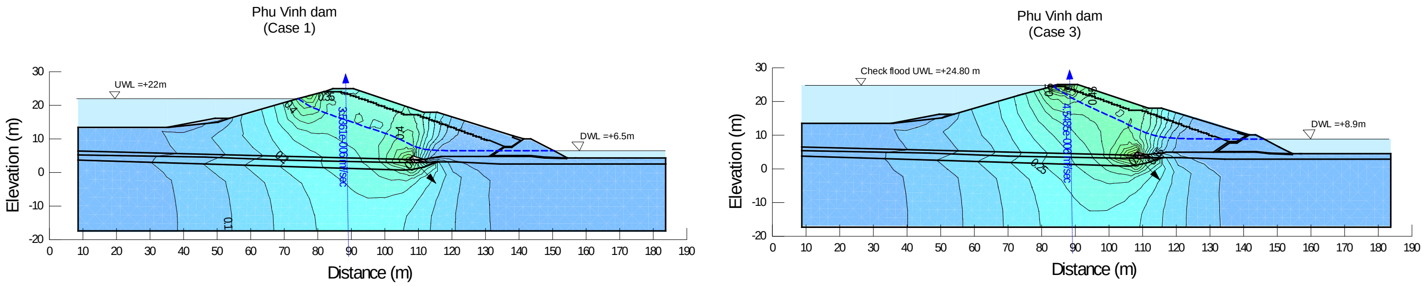

Fig. 5: Seepage calculation results with different cases (Case 1 and Case 3).

The seismic simulation results show that, there existed liquefied zones in the sand layer 2 under the drainage rockfill at downstream for all analyses (Cases 9.1, 9.2 and 9.3) (Fig. 6).

Note also from the above simulation results that the size of liquefied zone simulated with longer return period was generally bigger than that simulated with smaller return period.

Note that the existing Vietnamese standard [12] only provides seismic accelerations for 1 in 475 year earthquake. Given the event of a 1 in 475 year earthquake, effect of raising reservoir water level on liquefaction potential was implemented by taking into account three reservoir water levels, namely, the normal water level, the design flood level (probability 1%) and the check flood level (probability 0.01%). Our previous study shows that the liquefied zone size increase gradually with the water level [6]. Note that the probability that the checked flood water level happens concurrently the strong earthquake is not mentioned in the existing seismic standard [9],[13].

Table 4. Result of slope stability calculation

Case	Dam slope	Minimum FOS	Allowable FOS	Remark
1	Downstream	1.757	1.35	Satisfied
2	Downstream	1.719	1.35	Satisfied
3	Downstream	1.661	1.15	Satisfied
4	Downstream	1.431	1.15	Satisfied
5	Downstream	1.289	1.15	Satisfied
6	Upstream	2.828	1.35	Satisfied
7	Upstream	2.735	1.15	Satisfied
8	Upstream	2.035	1.15	Satisfied
9.1	Downstream	0.688	1.15	Not satisfied
9.2	Downstream	0.920	1.15	Not satisfied
9.3	Downstream	0.454	1.15	Not satisfied

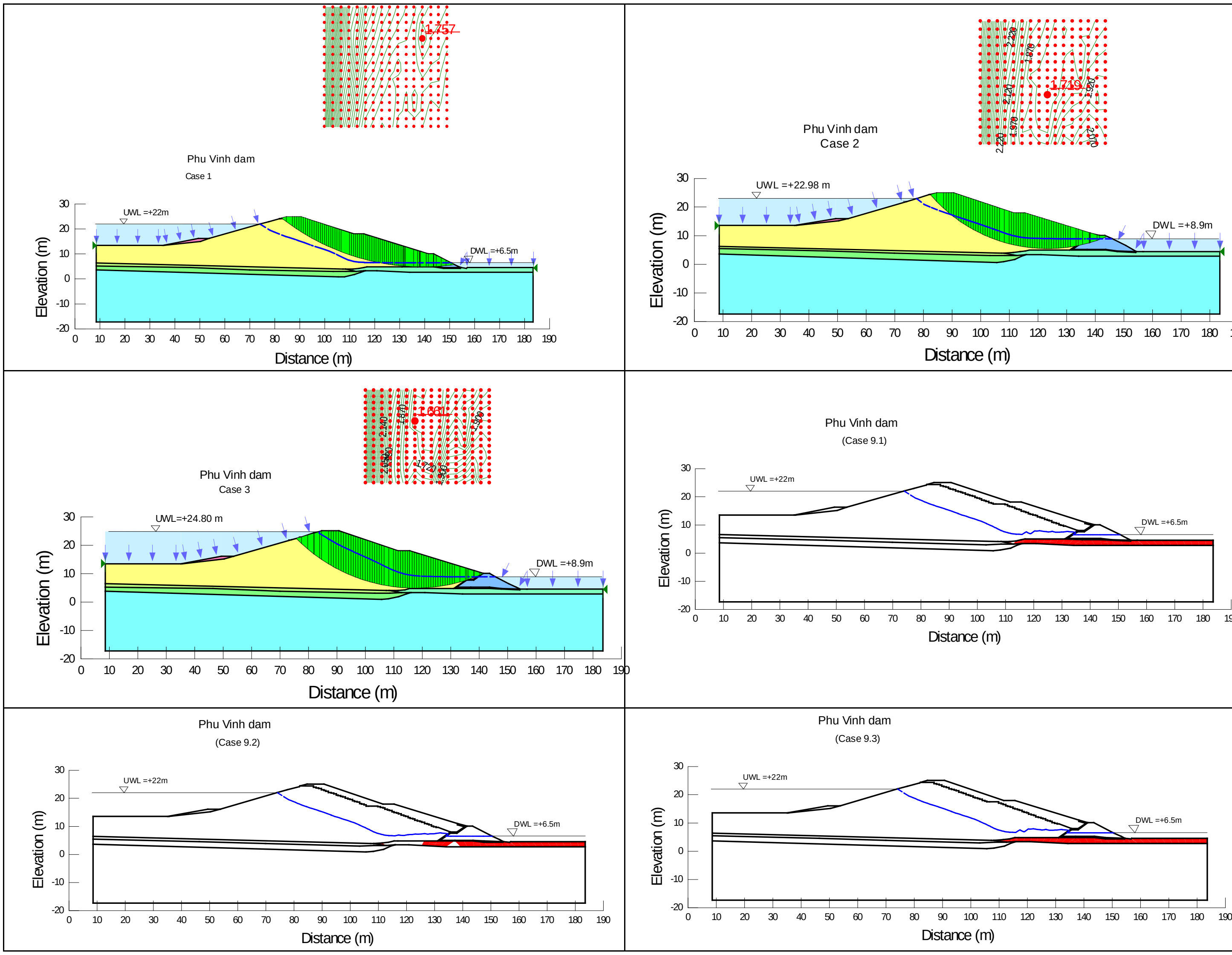

Fig. 6: Static and seismic slope stability and liquefaction calculation results.

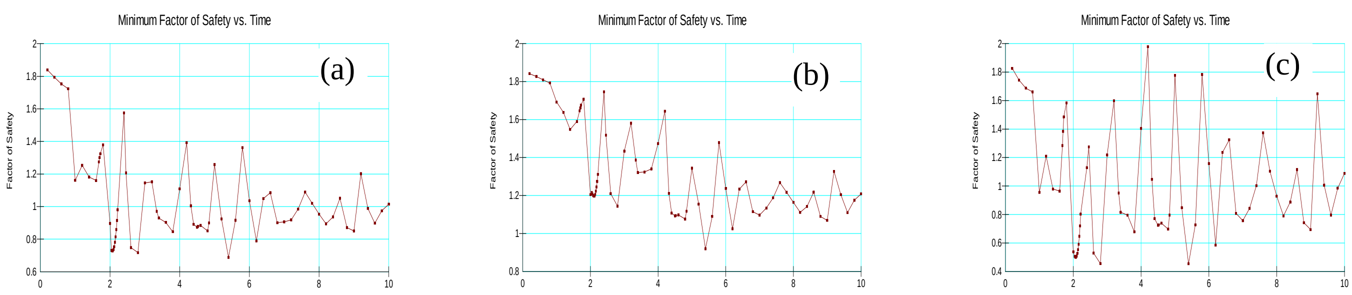

Fig. 7: Minimum factors of safety with time with three cases: a) Case 9.1, b) Case 9.2 and c) Case 9.3.

It is possible to prevent the liquefaction potential in the sand foundation layer No.2 by applying some appropriate methods such as the densification or deep mixing method.

It is interestingly to note that when employing the SPT number N_{30} at site (N_{30} varied from 16 to 34) [17], the FOS values calculated by the simplified procedure ([11],[16]) were greater than unity with depth. More studies on laboratory tests using cyclic triaxial tests are required to check this difference between simulation and field testing results.

4. Conclusion

The liquefaction analysis results of Phu Vinh dam show that the foundation sand layer No. 2 under the downstream drainage rockfill could be liquefied when subjected to the design earthquakes (T= 475 years), OBE and SEE earthquakes. The simulation results shows that the FOS of the dam slope during earthquakes reached values below unity at t=5.4s.

The liquefaction potential of the dam depends on some main factors such as seismic, geotechnical data, and reservoir water level. The liquefied zone becomes bigger subjected to stronger earthquake, and higher the reservoir water level.

It is necessary to take measures to prevent dam instability in the event of strong earthquakes, especially during the flood season by appropriate methods such as densification or deep mixing.

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