Safety analyses of Srinagarind dam induced by earthquakes using dynamic response analysis method.

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ABSTRACT: Srinagarind dam is located in the seismic risk area. In the past, the stability of Srinagarind dam was designed with 0.10g seismic coefficient (k_h) by pseudostatic method. Nowadays, the dam has to be re-analyzed by dynamic response analysis method in order to understand more dynamic behavior. Due to inadequate of acceleration time history data, 213 records from 35 earthquakes events at rock site were selected for analyses. The largest peak ground acceleration (PGA) used in this analysis is 1.17g. The dynamic response behaviors of dam are different in each of dam material. The displacement and acceleration in the horizontal direction of dam significantly increases with dam height above elevation of +145 m.MSL and reach the maximum values at elevation of +180 m.MSL. The permanent slope displacement from seismic force is found to be depended on dam geometry, material properties, reservoir water level, and ground motion parameters. According to the results, the permanent slope displacement calculated by Swaisgood method (1998) is closed to Newmark's deformation method (Newmark, 1965). The seismic slope displacement in the upstream slope at normal high water level has a maximum value of 6.82 m (3.86 m settlement). The results show that Srinagarind dam can be withstanding the large earthquakes event with some damage but no immediate breaching will be triggered since the freeboard distance of the dam is 5 m. However, the dam crest and some instruments in the dam crest might be damaged significantly.

1 INTRODUCTION

As the request of the Electricity Generating Authority of Thailand (EGAT), Geotechnical Engineering Research and Development Center (GERD) of Kasetsart University has performed a seismic safety evaluation of Srinagarind dam. The purpose of this study is to assess the dam safety including the dynamic behavior of the dam.

The Srinagarind dam is located at the Quae Yai river, Kanchanaburi province, about 190 km northwest of Bangkok, Thailand. The dam is a center impervious core rockfill dam with a maximum height of 140 m and a crest length of 610 m. The construction began in 1973 and received the first filling on 1977. The dam slope stability was designed with 0.10g seismic coefficient (k_h) by pseudostatic method. The pseudostatic method does not cover the dynamic behavior of dam materials. As shown by seed (1979), dams designed by pseudostatic analyses later failed during earthquakes (Table 1). Therefore, the dynamic response analysis has to be used for Sringarind dam to re-evaluate its seismic safety.

The study begins by collecting the ground motion data and analyzes the ground motion components such as peak ground acceleration, predominant period and duration. Natural period of the dam body was determined by the dynamic response analysis using the horizontal sinusoidal input acceleration-time histories with PGA of 0.05g, 0.1g and 0.5g and period range from 0.1 to 1.2 second

Due to the lack of actual acceleration time history data recorded nearby Srinagarind dam, 213 rock site recorded data from 35 earthquakes events in the world were selected for analyses especially the well known events with magnitude between 5.4 Ml to 8.4 Mw. The largest PGA used in this analysis is 1.17g from the 1971 San Fernando earthquake.

Table 1. Results of pseudostatic analyses of earth dams that failed during earthquakes.

Dam	K_h	F.S.	Effect of Earthquake
Sheffield Dam	0.10	1.2	Complete failure.
Lower San Fernando Dam	0.15	1.3	Upstream slope failure.
Upper San Fernando Dam	0.15	~2-2.5	Downstream shell including crest slipped about 6 ft downstream.
Tailing Dam (Japan)	0.20	~1.3	Failure of dam with release of tailings.

Source: Seed (1979)

Those ground motions were used as an input for dynamic response analyses, seismic permanent deformation analyses and liquefaction potential analyses. To evaluate the validity of the analyses, the computed seismic deformation of Srinagarind dam using Newmark's deformation method (1965) (Numerical Analysis) were compared with the results from Swaisgood method (1998) (Statistical Method).

2 SEISMICITY AND DESIGN EARTHQUAKES

Seismicity map of Thailand had been done by the Thai Meteorological Department. The epicenter of earthquake events from 1983-2003 were plotted as shown in figure 1. The map indicated that most of events occurred along the contact between Indian-Australian Plate and Eurasian Plate. The earthquakes that have the epicenter within Thailand are mostly happen in the northern and western regions of the country.

The Srisawat fault zone is the closet potentially seismogenic fault to Srinagarind dam (Fig. 2). The Srisawat fault zone is an active fault with a northwest-strike, right-lateral, strike-slip fault zone located within a few kilometers of dam site. The largest earthquake event recorded in the study area is the reservoir induced earthquakes on April 22, 1983 with the magnitude of 5.9 Ms. The earthquake epicenter is about 58 km away from the dam. No damage was found. The accelerometer on the dam crest triggered with PGA of 0.051g.

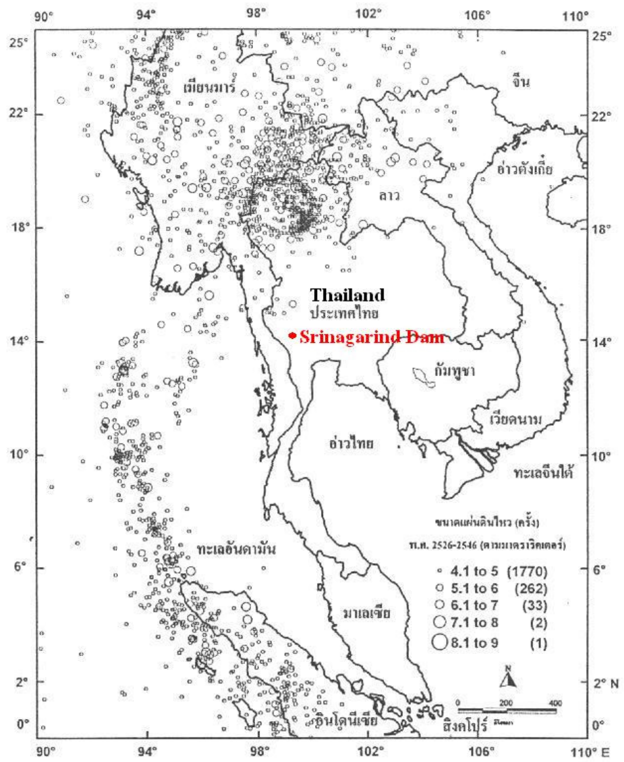

Figure 1. Seismicity map from 1983-2003.
Source: Prajub and Wetbuntueng (2006)

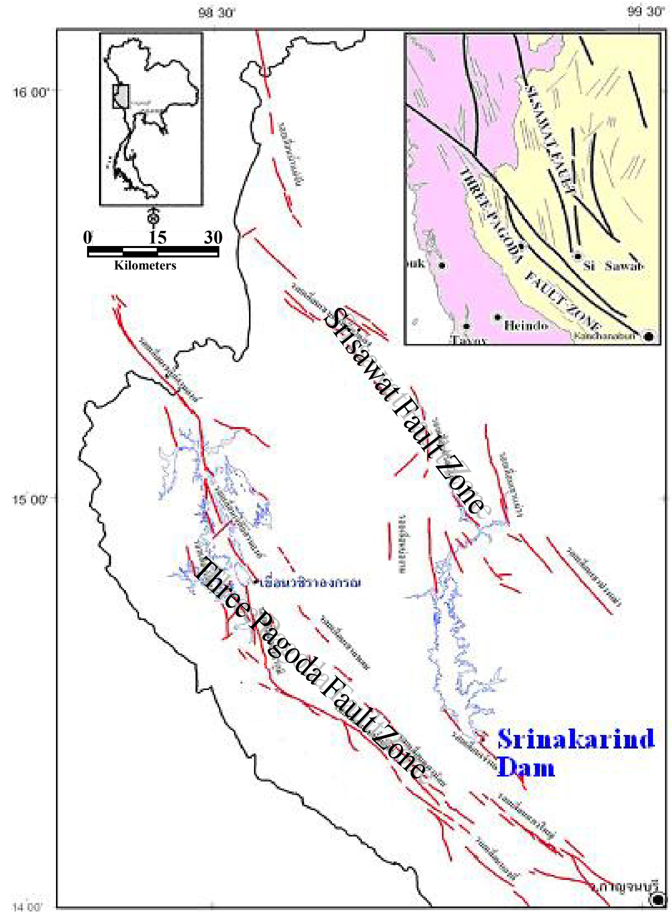

Figure 2. Active fault zone in study area.
Source: Charusiri et. al. (2004)

3 DAM CONSTRUCTION AND MATERIALS

The dam has a height of 140 m from the foundation rock and a crest length of 610 m. The downstream slope angle is 1:1.8 and the upstream slope angles are 1:2.2 and 1:2. The reservoir capacity is 17,745 million m^3 at the maximum retention level of 180 m.MSL. The dam consists of 5 zones (Figure 3), the impervious core made of clayey sand (SC), the filter material is obtained from a river alluvium and the transition zone is obtained from the foundation excavation and a query of quartzite. The rockfill material is a hard durable limestone with the maximum size of 0.70 and 1.50 meters. The static properties of the dam materials are shown in Table 2.

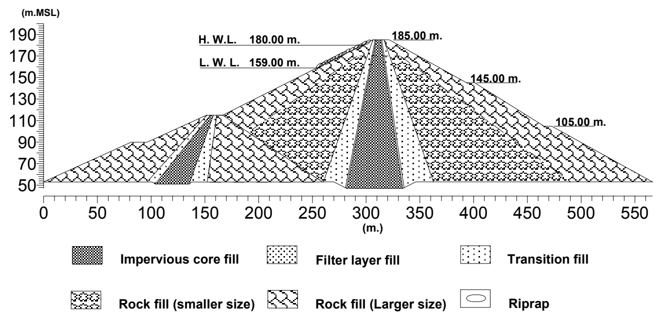

Figure 3. Cross section of Srinagarind dam.

Table 2. The static properties of the dam materials.

Zone	Description	Unit Weight t/m^3 Dry	Wet	Sat.	Cohesion (t/m^2)	Coefficient of Internal Friction
1	Impervious	1.80	2.03	2.13	4	0.30
2	Filter	2.00	2.04	2.25	-	0.70
3	Transition	2.00	2.04	2.25	-	0.70
4	Rockfill (Smaller)	1.80	1.82	2.13	-	0.65
5	Rockfill (Larger)	1.75	1.77	2.09	-	0.80

Source: Champa and Mahatraradol (1982)

4 ANALYSIS PROCEDURE

The composition of all the performed analyses is shown in Figure 4. The analyses started with pseudostatic analysis and then dynamic response analysis with the case of vary input ground motions, various storage water levels from 130 to 180 m.MSL and rapid drawdown condition. After that the seismic deformation analysis was performed using the Newmark's deformation method and Swaisgood's method to compare with the availability of dam freeboard. Finally, liquefaction analysis was carried out. If the dam material is liquefied, stability analysis will be done by using the residual strength of liquefied material.

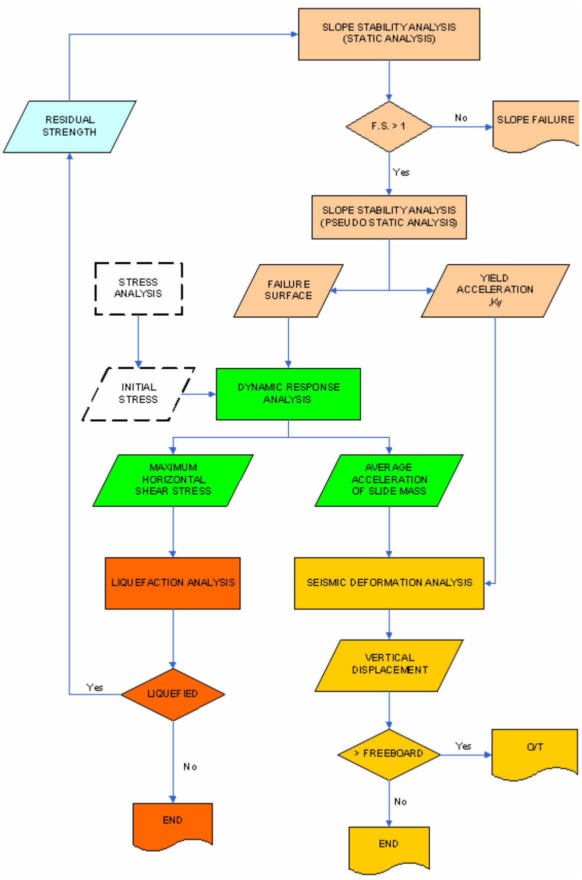

Figure 4. Analysis procedure chart.

5 SLOPE STABILITY ANALYSIS

The slope stability analysis of dam using the Simplified Bishop method shows that the down stream slope of the dam has lower safety factor (Table 3). The yield acceleration (k_y) for the interested sliding surface at the upstream slope of the dam provided lower values than that of the downstream slope because of the buoyancy effect.

Table 3. Results of slope stability analysis.

Slip Surface	F.S.	K_y
Down Stream Slope	1.456	0.174
Up Stream Slope	1.628	0.130
Rapid Drawdown (From +180 to +155 m.MSL)	1.514	0.166

6 INITIAL STRESS ANALYSIS

The static stress at present condition is quite important for dynamic analysis since the maximum shear modulus of the dam material depends on this value. The dam was modeled by finite element method (FEM) in 2D using the maximum cross section (Fig. 5). The simulation of loading from staged construction to present in the total period of 30 years was done. The dam was assumed to be attached to the rigid rock foundation since the elasticity of the rock foundation is much higher than the dam materials. The mean effective stress of dam in case of normal high reservoir water level is shown in Figure 6. The verification of the result was done by comparing the dam displacement and pore water pressure with the instrumentation data. Adjustment of input data was done in order to obtain the close to real condition.

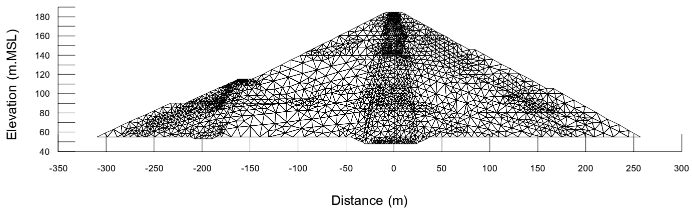

Figure 5. The model of maximum cross section of dam.

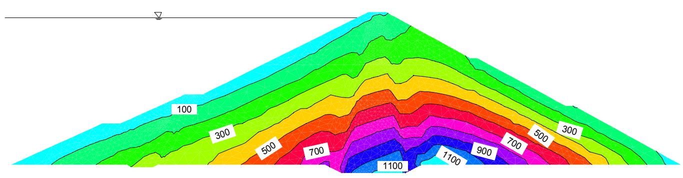

Figure 6. Mean effective stress of dam in full reservoir condition (unit: KPa).

7 DYNAMIC ANALYSIS

7.1 *Dynamic properties*

The maximum shear modulus (G_{max}) of dam materials was obtained from the field test using spectrum analysis of surface wave (SASW) method. Empirical equations (Kokusho and Esashi, 1982., Hardin and Black, 1968) were also used for comparison. According to the results, the G_{max} from both methods is difference especially with higher effective stress (Fig. 7). The G_{max} from SASW was use for dynamic analysis because the limitations in actual condition of empirical equation such as the maximum size of sample and the confining pressure.

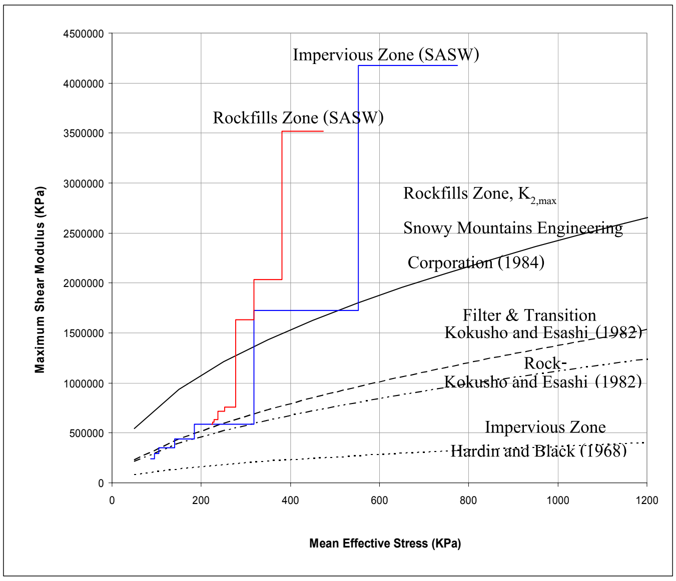

Figure 7. The relation between maximum shear modulus and mean effective stress obtained from SASW and empirical equations.

The strain dependency of shear modulus and damping ratios used in this analysis for rockfill and transition zone are based on the curve suggested by Gazetas (1992), filter materials by Seed et.al. (1985) and impervious core by Vucetic et al. (1991) (Fig. 8).

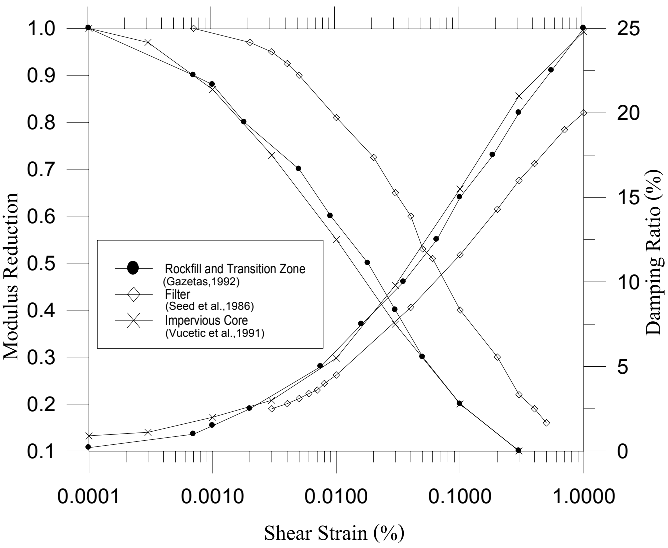

Figure 8. The modulus reduction and damping ratio curves use in the analysis.

7.2 *Input seismic motion*

Due to inadequate of ground motion data nearby the dam, 213 rock site recorded data from 35 earthquake events in the world were selected for analyses especially the well known events with magnitude between 5.4 Ml to 8.4 Mw 400 kilometers from epicenter. The largest peak ground acceleration (PGA) used in this analysis is 1.17g. The correlation between the PGA and epicenter distance of 6.0 Mw to 6.9 Mw is shown in Figure 9. The PGA decreases with the increasing of epicenter distance. The PGA is less than 0.1g when epicenter distance is far beyond 90 km. The predominant periods of earthquakes are mostly between 0.1 to 0.4 second (Fig. 10). Bracketed duration also decreases with epicenter distance, however the data is quite scattered (Fig. 11).

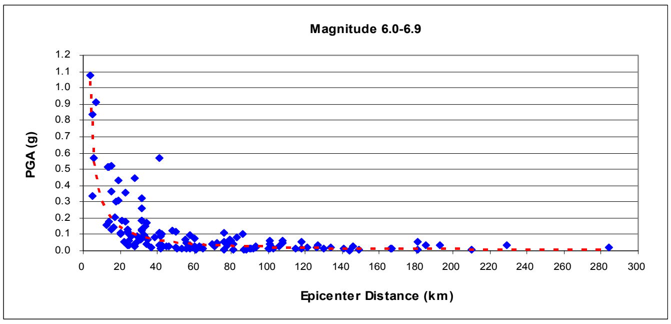

Figure 9. Relationship between PGA and epicenter distance of earthquake magnitude 6.0 to 6.9.

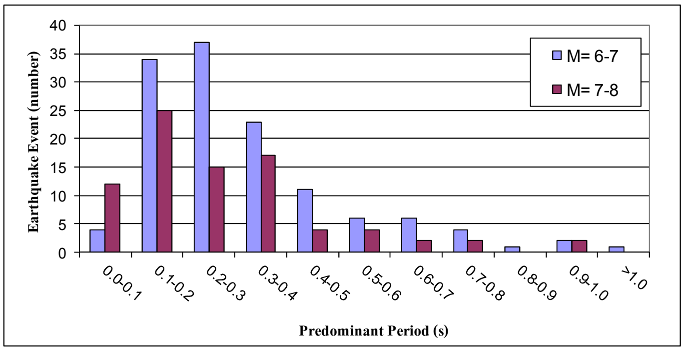

Figure 10. Predominant periods distribution.

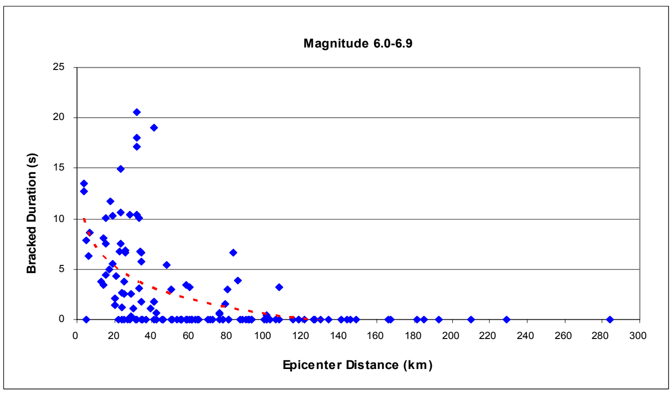

Figure 11. Relationship between bracketed duration and Epicenter distance of earthquake magnitude 6.0 to 6.9.

7.3 *Natural period of the dam body*

Natural period of the dam body was performed by the dynamic response analysis using the horizontal sinusoidal input acceleration-time histories with PGA of 0.05g, 0.1g and 0.5g and motion period of 0.1 to 1.2 second. The dynamic response shows that the natural periods of the dam are between 0.62 - 0.90 second as shown in the figure 12 for the 0.1g input motion. The natural period of the dam body seems to be longer than the predominant period of the input seismic motion. However, it can be seen that even though the dam is quite high (140 m) but its natural period can be compared as the natural period of 6 to 9 story building.

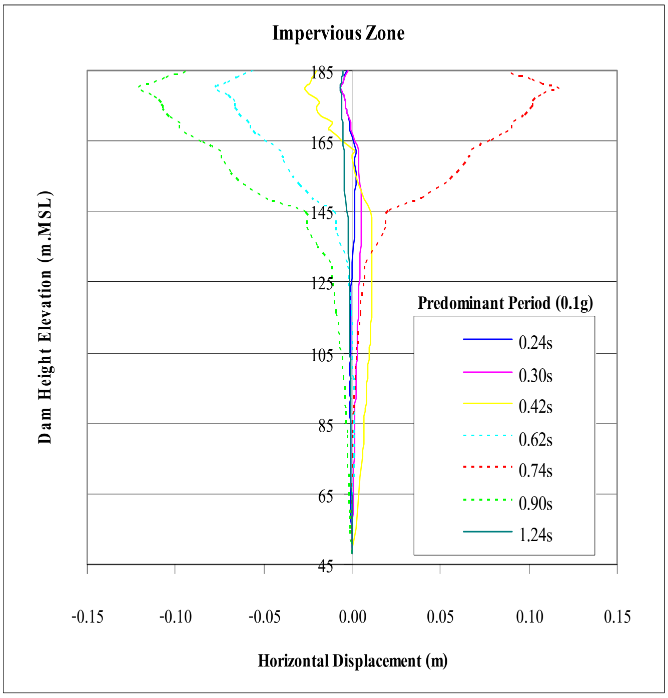

Figure 12. Determination of natural period of the dam by dynamic response analysis.

7.4 *Dynamic Response Analysis*

Plane strain finite element dynamic response analysis was carried out using the computer program QUAKE/W. The equivalent linear method was used to account for the nonlinear behavior of the dam material. The dynamic response behaviors of dam are different in each of dam zone as illustrative in Figure 13 which shown an example of relative horizontal displacement of dam induced by 1971 San Fernando earthquake at the dam base during the period of peak ground acceleration. The major movement is at the upper third the part of the dam particularly in the upstream slope. The displacement and acceleration in the horizontal direction of dam significantly increases with dam height above elevation of +145 m.MSL and reach the maximum values at elevation of +180 m.MSL (Fig. 14).

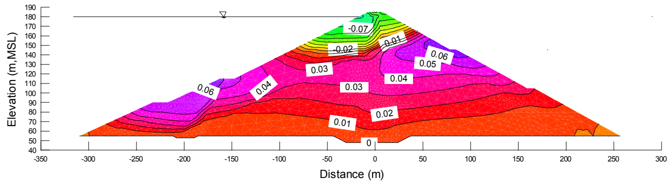

Figure 13. Relative horizontal displacement of dam in case of San Fernando 1971 earthquake induced at the dam base. (Unit:m)

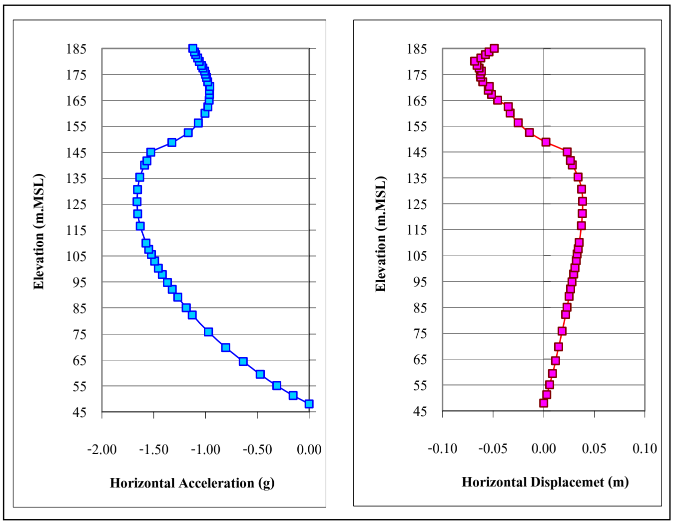

Figure 14. Relative horizontal acceleration and displacement in the impervious core zone induced by San Fernando1971 earthquake.

The maximum acceleration of impervious core induced by various earthquakes is shown in Figure 15. The San Fernando Earthquake (1971) gave the maximum horizontal acceleration and displacement at the elevation of +180 m.MSL (Fig. 16).

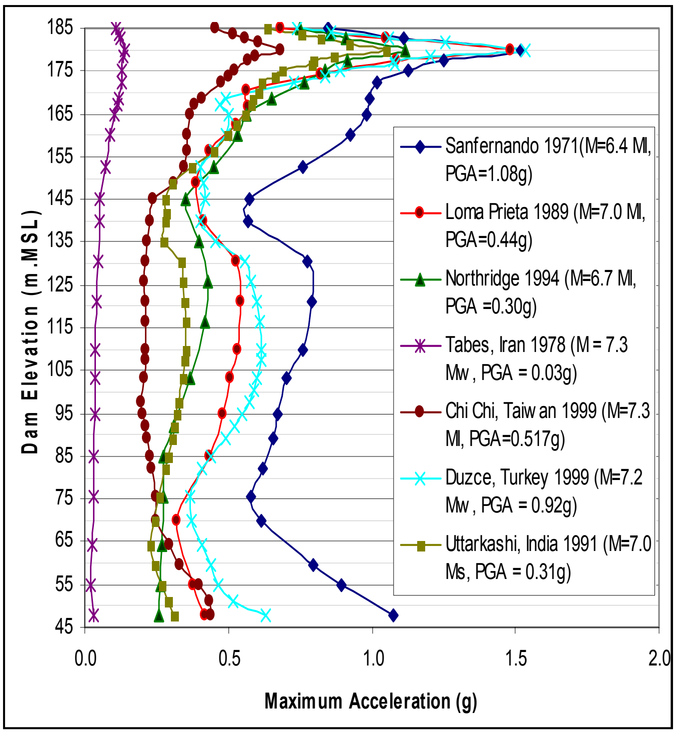

Figure 15. Maximum acceleration of dam in impervious core zone induced by varies earthquakes.

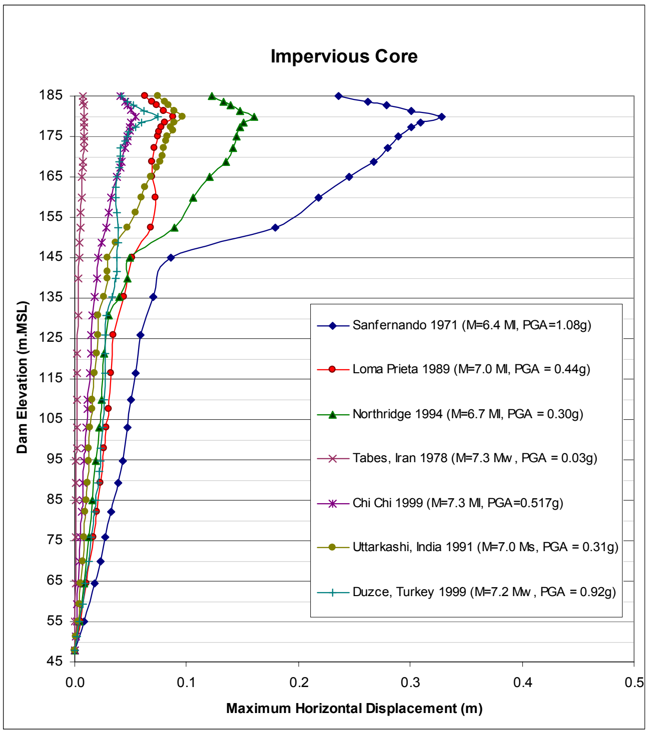

Figure 16. Maximum displacement of dam in impervious core zone induced by various earthquakes.

As for the effect of reservoir water level, it was found that the horizontal displacement increases when the reservoir water level increases as shown in Figure 17. However, the affect is not significant to the safety of the dam.

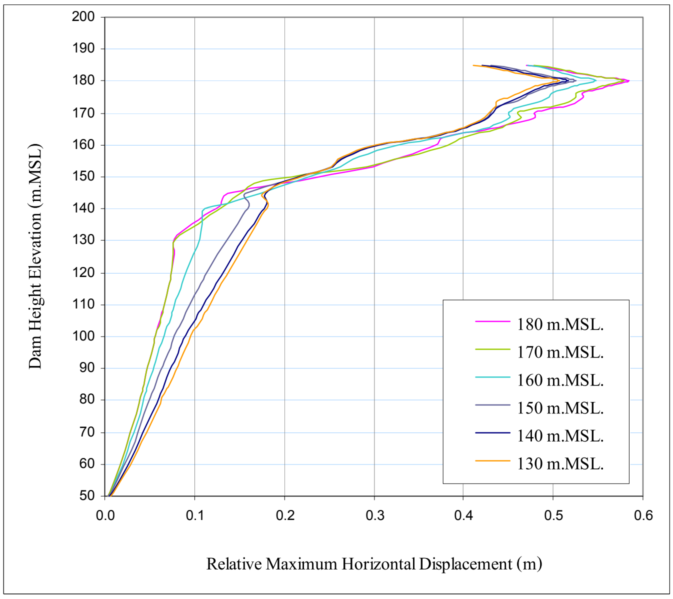

Figure 17. Relative maximum horizontal displacements with reservoir water levels.

8 SEISMIC DEFORMATION ANALYSIS

The seismic deformation analysis was carried out both upstream and downstream slope in various conditions such as variation of reservoir water levels and rapid drawdown. 75 ground motion records were used. The analysis begins with all possible slip planes. By considering the effect of sliding mass to the breaching potential of dam, 4 and 5 slip planes in the upstream and downstream slope were analyzed for all input ground motion records (Figs 18-19). The permanent slope displacement is found to be depended on dam geometry, material properties, reservoir water level, and ground motion parameters. The maximum seismic slope displacement of 6.82 m (3.86 m settlement) is found at the upstream slope with normal high water level (Table 4-5). In the case of reservoir water level effect, it was found that, the slope displacement decreases when the reservoir water level decreases (Fig. 20). According to the results, it's also found that the permanent slope displacement calculated by Swaisgood's method (1998) is closed to Newmark's deformation method (1965) especially when PGA is below 0.6g (Fig. 21).

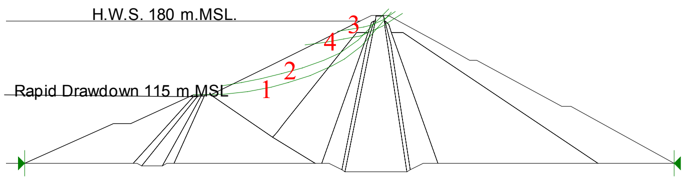

Figure 18. Considered slip planes for upstream slope deformation analysis.

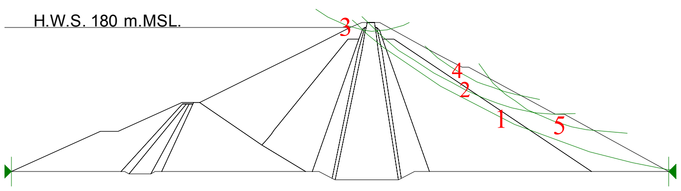

Figure 19. Considered slip planes for downstream slope deformation analysis.

Table 4. Results of upstream slope displacement by Newmark's deformation analysis.

Earthquake	Slope Displacement (m) Slip No.			
	1	2	3	4
Duzce, Turkey 1999 M=7.2 Mw, PGA=0.92g	0.004	0.006	0.856	0.468
Uttarkashi, INDIA 1988 M=7.0 Ms, PGA=0.295g	0.019	0.047	0.688	0.553
Loma Prieta, USA 1989 M=7.0 Ml, PGA=0.275g	0.007	0.023	0.677	0.542
Northridge, USA 1994 M=6.7 Ml, PGA=0.568g	0.119	0.171	2.173	1.410
San Fernando,USA 1971 M=6.4 Ml, PGA = 1.171g	0.527	0.639	**6.822**	4.750
Chi Chi, Taiwan 1999 M=7.3 Ml, PGA = 0.517g	0.001	0.006	0.296	0.270
Tabes, Iran 1978 M= 7.3 Mw, PGA = 0.381g	0.024	0.039	0.750	0.650

Table 5. Results of downstream slope displacement by Newmark's deformation analysis.

Earthquakes	Slope Displacement (m) Slip No. 1	2	3	4	5
Duzce, M=7.2 Mw, PGA=0.92g	0.026	0.106	0.260	0.234	0.201
Uttarkashi, M=7.0 Ms, PGA=0.295g	0.000	0.000	0.246	0.000	0.000
Loma Prieta, M=7.0 Ml, PGA=0.275g	0.000	0.000	0.260	0.000	0.000
Northridge, M=6.7 Ml, PGA=0.568g	0.017	0.031	1.229	0.042	0.029
San Fernando, M=6.4 Ml, PGA = 1.171g	0.170	0.204	**5.747**	0.248	0.170
Chi Chi, M=7.3 Ml, PGA = 0.517g	0.000	0.000	0.076	0000	0.001
Tabes, M= 7.3 Mw, PGA = 0.381g	0.000	0.008	0.202	0.000	0.000

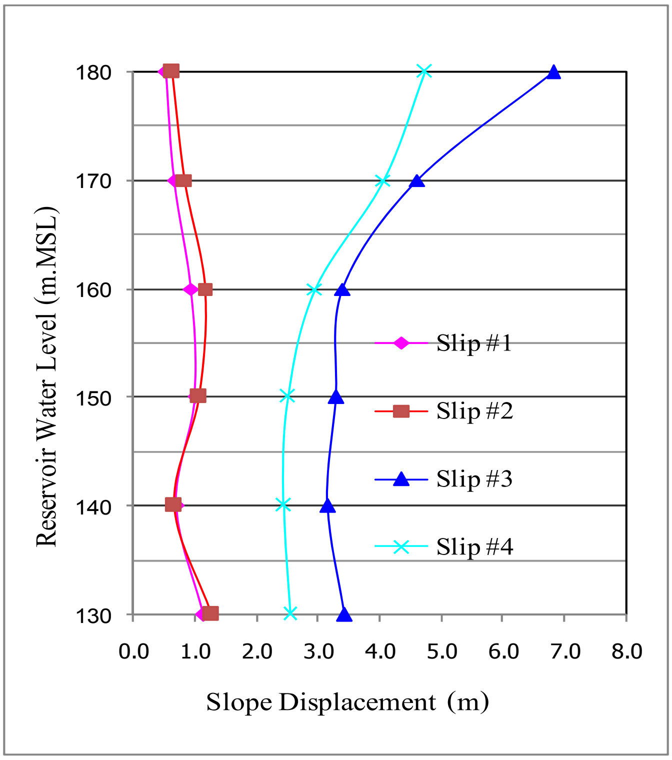

Figure 20. The correlation of upstream slope displacement and reservoir water levels.

9 LIQUEFACTION ANALYSIS

Even thought the dam materials was well compacted and the dam body sit on the rock foundation, however with large earthquake, liquefaction could be triggered near the dam crest where the effective confining pressure is low.

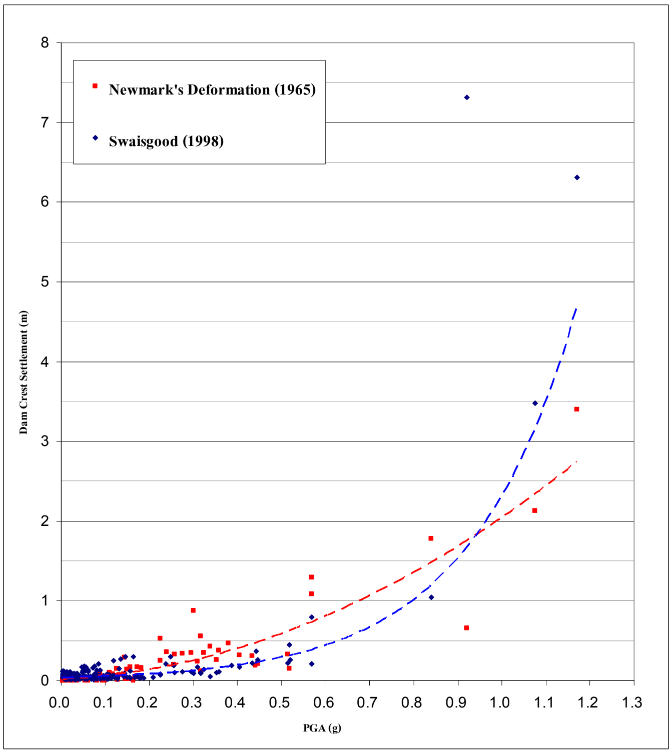

Figure 21. The correlation of dam crest settlement by Newmark's deformation and Swaisgood method.

The analyses were performed by obtaining the maximum shear stresses data from dynamic response analyses. The analyses were done based on the cyclic stress approach (Seed et al., 1985). Transition and filter zones of upstream slope and downstream filter material were analyzed by using the 1971 San Fernando earthquake records with PGA of 1.076g. The results show that the liquefaction will occur in the upper part of the filter and transition zone of upstream slope (Fig. 22). The result of the post liquefied stability analysis based on Simplified Bishop's circular slip method found that factor of safety is 1.542.

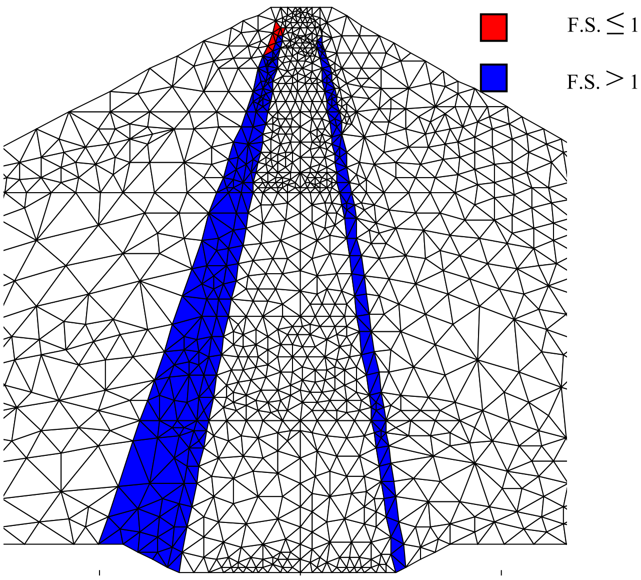

Figure 22. The liquefied zone.

10 COMPARISON BETWEEN DAM CREST SETTLEMENT AND DESIGN PGA

According to seismic deformation analysis using Newmark's deformation method (1985) it was found that the magnitude of crest settlement analyzed with various condition are less than the available dam freeboard (5 meters) as shown in table 6. The magnitude of PGA that can induced the crest settlement equal to available dam freeboard have to be at least 2.20g (using ground motion by scaling up the 1971 San Fernando Earthquake records).

Table 6. Maximum crest settlement for all designs PGA.

PGA at Dam Base (g)	Crest Settlement (m)	Remarks
0.10	0.10	Design PGA used for Srinagarind dam construction.
0.15	0.29	PGA at Srinagarind dam by earthquake risk map. (Wanitchaikul et al., 1996)
0.60	1.29	Design PGA in highly seismic region (Wieland, 2003)
1.17	3.40	Maximum PGA for this analysis.
2.20	5.00	PGA that make the crest settlement equal to freeboard.

11 CONCLUSION

After comprehensive analyses, it was found that Srinagarind dam is safe to the seismic force, considering that it locates in low to moderate seismic area. With large earthquake, the dam shall experience some damages near the dam crest. The post earthquake dam safety plan needs to be designed. Instrumentations capable of indicating the leakage shall be installed in the dam body far from the crest area, in order to avoid the damage from the permanent movements.

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