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Numerical analysis of effect of mitigation measures on seismic performance of a liquefiable tailings dam foundation

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ABSTRACT

This paper addresses the earthquake response of a tailings dam with liquefiable sandy-gravel foundation. Geometry of the analyzed dam and the considered foundation conditions are typical for valley compacted earth and rockfill dams in the Andes of South America. Loose to medium-dense saturated sandy-gravel foundation conditions can compromise stability of the tailings dam, especially in active seismic zones where large magnitude earthquakes may induce liquefaction of the saturated alluvial foundation material eventually culminating in flow failure beneath the downstream section of the dam. Results of dynamic liquefaction analyses of the tailings dam – foundation system, considering alternatives for liquefaction mitigation (i.e., downstream buttress and/or in-situ foundation improvement) under two different seismic loading scenarios are presented and discussed. The effectiveness of the analyzed liquefaction mitigation measures is evaluated in terms of earthquake-induced deformations of the existing starter dam and of the proposed ultimate dam configuration. The dynamic liquefaction analyses were conducted using a non-linear two-dimensional finite-difference scheme along with a fully-coupled effective stress constitutive model. The selected geotechnical parameters and input earthquake time records as well as constitutive model calibration to laboratory test results are discussed in the paper. Results from the idealized case study analyses of both buttress and foundation improvement alternatives showed that the foundation improvement is the most effective to mitigate the liquefaction hazard for the ultimate dam configuration subject to earthquake loading.

Keywords: tailings dam, liquefaction, finite-difference scheme, earthquake-induced deformations, mitigation

1 INTRODUCTION

Some of the tailings dams located in the Andes of South America have been constructed over a foundation consisting of relatively loose to medium-dense, alluvial soil (silty gravel and/or silty sand with gravel). Such foundation materials are in most cases fully saturated as a result of abundant precipitation and/or operation conditions of the tailings dam. This condition could compromise the stability of the dam, especially in earthquake-prone regions where large magnitude earthquakes may induce liquefaction of the foundation soils and subsequently cause large displacements or flow failure beneath the dam.

This paper presents results from non-linear dynamic liquefaction analyses of a tailings dam with an underlying foundation comprising a layer of medium-dense saturated sandy-gravel alluvium. The two-dimensional non-linear dynamic analyses were performed using the Fast Lagrangian Analysis of Continua (FLAC) computer code FLAC Ver. 6.0 - (Itasca, 2008). The saturated alluvial foundation soil was modeled considering the elasto-plastic formulation implemented in UBCSAND constitutive model.

The idealized dynamic liquefaction analyses for dam stability addressed the following general cases: a) an existing starter dam constructed on medium-dense alluvial foundation soils with high potential for liquefaction, b) two alternatives of the starter dam remediation (i.e., buttress and buttress foundation improvement) to mitigate liquefaction related hazards such as large permanent dam displacement, and c) two alternatives (i.e., buttress and extensive foundation improvement) to mitigate the liquefaction related hazards for the expanded dam at the ultimate configuration.

The adopted geotechnical parameters, selection of earthquake time histories, calibration of the model, numerical analysis results and conclusions are discussed throughout the paper.

2 TAILINGS DAM CONFIGURATIONS

Six different dam configurations were considered for analyses. Figure 1 shows the starter dam-foundation system without any remedial measures (i.e., Case 1) along with the materials included in various components of the system. The starter dam, assumed to be in an operating stage, has a crest width of 8 m, upstream and downstream slopes of 1.5(H):1.0(V), and 2m of the freeboard is considered. A liner system consisting of geomembrane and geosynthetic clay liner (GCL) covers the upstream slope face and extends below the impoundment area. The tailings dam consists of three zones: a transition fill; a structural fill (gravelly soils); and a waste rock/rockfill. Foundation soils consist of a 10-meter-thick medium-dense liquefiable alluvial soil (silty sand with gravel), which is underlain by bedrock (see Figure 1). Groundwater table (GWT) is located at 2 m depth below the original ground surface. The embankment dam stores hydraulically-deposited tailings assumed to be in a slurry state.

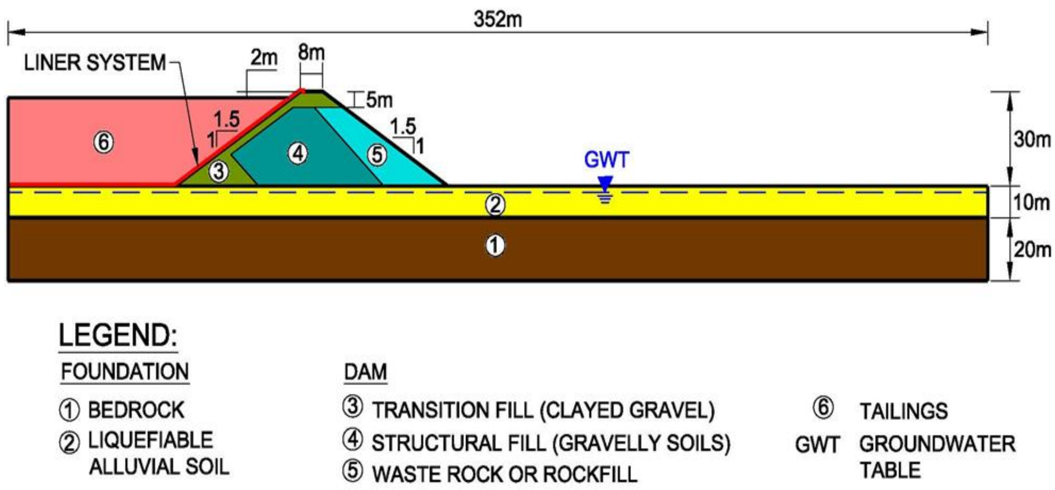


Figure 1. Geometry and materials of the analyzed starter dam (Case 1).

The configurations for the other cases considered are shown in Figure 2. Case 2 considers the case of the starter dam with a 60-m wide buttress placed downstream of the dam (Figure 2a). A 25-m wide buttress and local foundation improvement beneath the buttress are considered in the Case 3 configuration (Figure 2b). The toe buttress for Cases 2 and 3 could be considered a portion of the dam raise for the next construction stage. In Case 3, the alluvial layer underneath the buttress will be excavated at a 1.0(H):1.0(V) slope with a base width of 10m, and will be replaced with a non-liquefiable drained compacted structural fill, as shown in Figure 2b) .

Cases 4 through 6 consider the ultimate dam configuration without and with potential remedial measures for the next construction stage (expanded TSF). Figure 2c shows the Case 4 configuration representative for the ultimate dam with no remediation measures. Case 5 includes a toe buttress with 40m crest width and 20m height with no foundation improvement (Figure 2d). Lastly, Case 6 includes an extensive improvement of the alluvial foundation soil layer below the downstream portion of the dam expansion, where the alluvial foundation material is replaced with a compacted drained structural fill to reduce the consequences of liquefaction (Figure 2e).

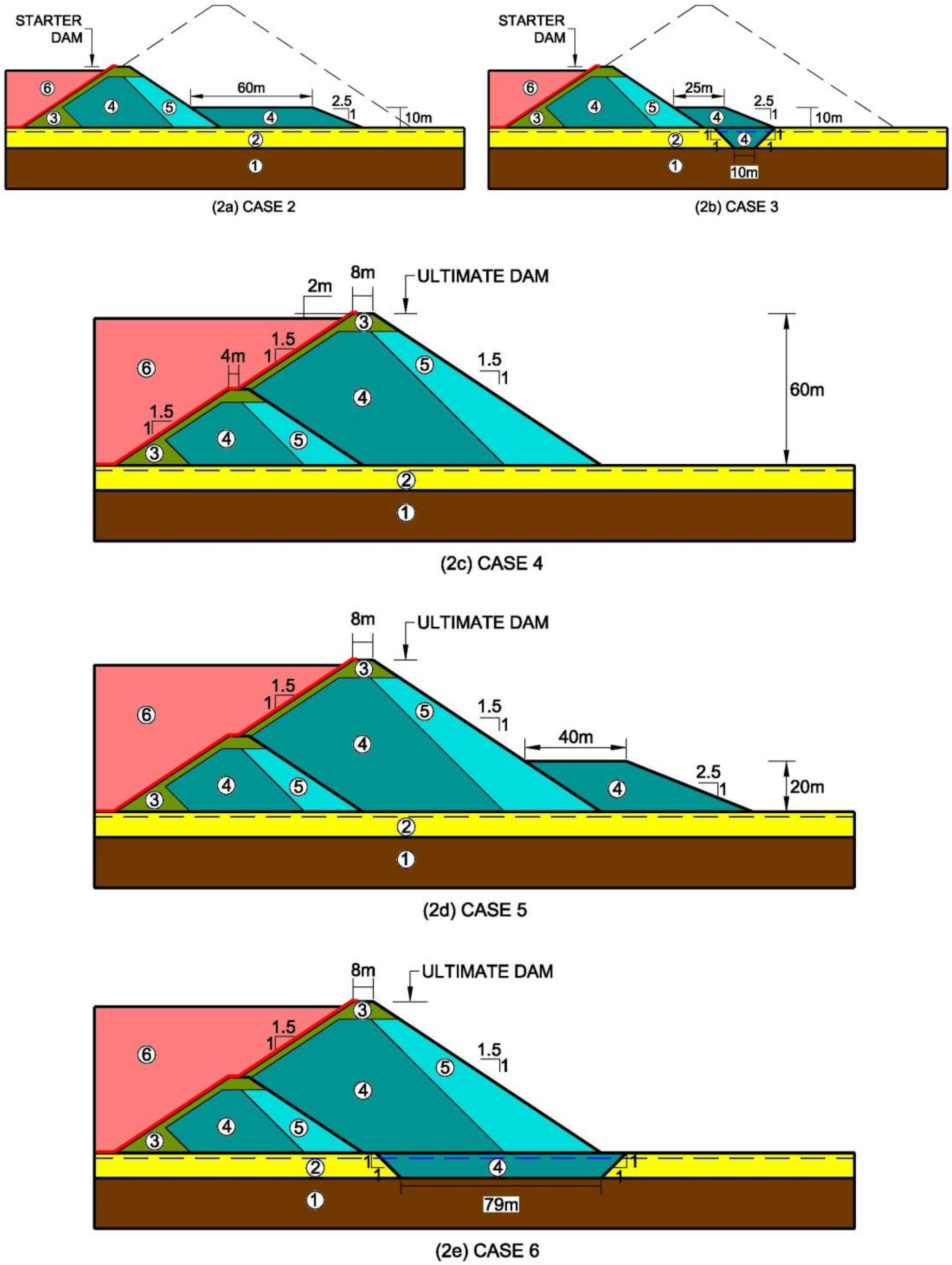


Figure 2. Dam geometry for Case 2, Case 3, Case 4, Case 5, and Case 6.

3 MATERIAL PROPERTIES

The geotechnical properties of the dam and foundation materials considered herein are provided in Table 1. The elasto-plastic Mohr-Coulomb model was used to characterize the stress-strain behavior of the bedrock and engineered materials of the dam in the numerical analysis. Additionally, the liquefiable alluvial soil below the groundwater table was modeled using the UBCSAND advanced constitutive model for evaluating liquefaction described later.

Saturated tailings with a total density of 1.73 t/m^3 were assumed to be fully liquefied and were modeled as a pressure applied to the upstream face of the dam and foundation ground surface in the upstream impoundment area.

Table 1. Geotechnical properties of embankment and foundation materials

Material Type	Density	Elastic Properties		Effective Shear Strength		
	t/m^3	Bulk Modulus (kPa)	Shear Modulus (kPa)	Cohesion (kPa)	Friction Angle (°)	Friction Angle at constant volume(°)
Bedrock	2.65	1.51x10^6	1.13x10^6	4000	36	-
Liquefiable Alluvium below GWL	1.93	1.74x10^5	8.08x10^4	0	35	33
Alluvium above GWL	1.85	1.74x10^5	8.08x10^4	0	35	-
Transition Fill	1.89	6.44x10^5	3.36x10^5	5	38	-
Structural Fill / Buttress	2.24	1.38x10^5	7.78x10^5	0	45 to 36*	-
Waste Rock or Rockfill	2.35	1.38x10^5	7.78x10^5	0	52 to 46*	-
Saturated Tailings	1.73	-	-	-	-	-

Note: * Friction angle varies with depths as presented by Leps (1970).

4 NUMERICAL MODELING APPROACH

The finite-difference dynamic analyses were conducted using a two-dimensional non-linear elasto-plastic model with fully coupled liquefaction triggering in Fast Lagrangian Analysis of Continua (FLAC) code (Itasca 2008). The FLAC code solves the equations of motion in explicit form in the time domain using very small time steps that allows a non-linear inelastic stress strain soil behavior.

The saturated liquefiable alluvial soils were modeled using UBCSAND, a user-defined model incorporated into FLAC. The UBCSAND model was developed by Professor Peter M. Byrne and his colleagues at University of British Columbia (Byrne et al, 2003 and Byrne, 2009). The model simulates the stress-strain behavior of soil under static or cyclic loading for drained, un-drained, or partially drained conditions by using an elasto-plastic formulation at all stages of loading rather than just at the failure state. In this way plastic strains, both shear and volumetric, are predicted at all stages of loading. The plastic parameters in the model are selected to give agreement with results from simple shear element tests, which can be considered most closely replicate conditions in the field during earthquake loading.

Conventional state-of-practice procedures for evaluating liquefaction are implemented in separate analyses for evaluating liquefaction triggering and flow slide as well as for predicting permanent deformations, but are not capable of predicting the generation of excess pore-water pressure, dynamic stress-strain response, and displacement patterns simultaneously. The UBCSAND is a fully-coupled flow effective stress plasticity model enabling the dynamic response analyses in terms of pore pressures, accelerations and displacements induced by a specific input seismic motion. In this manner, the liquefaction triggering, deformation and potential for flow slide are evaluated in a single integrated analysis.

4.1 Model Calibration

In order to implement the UBCSAND model incorporated into the FLAC, this model was initially calibrated by selecting the $(N_1)_{60}$ value that matches the results of cyclic triaxial testing. In the UBCSAND calibration the cyclic triaxial test results were reduced by applying a factor of 2/3 to represent comparable in-situ stress conditions (Idriss and Boulanger, 2008). The model calibration was accomplished by using a single element simulation in FLAC to model the laboratory tests. The single element was assigned elastic and plastic parameters based on a $(N_1)_{60}$ value as described by Byrne et al. (2004). By comparing the predicted number of cycles to liquefaction to the measured number in the laboratory tests, the $(N_1)_{60}$ was adjusted and the simulations were then iterated until the predicted and measured number of cycles were almost the same. Results of the cyclic triaxial tests and UBCSAND calibrations are shown in Figure 3. For $(N_1)_{60}$ =12, as shown in Figure 3, the calibration curve generated by the UBCSAND model fits the cyclic triaxial test data quite well. In order to better represent in-situ stress conditions, as shown by the dotted line of Figure 3, the model was also calibrated by correcting the triaxial cyclic stress ratio (CSR) values to in-situ CSR ones (Kramer, 1996).

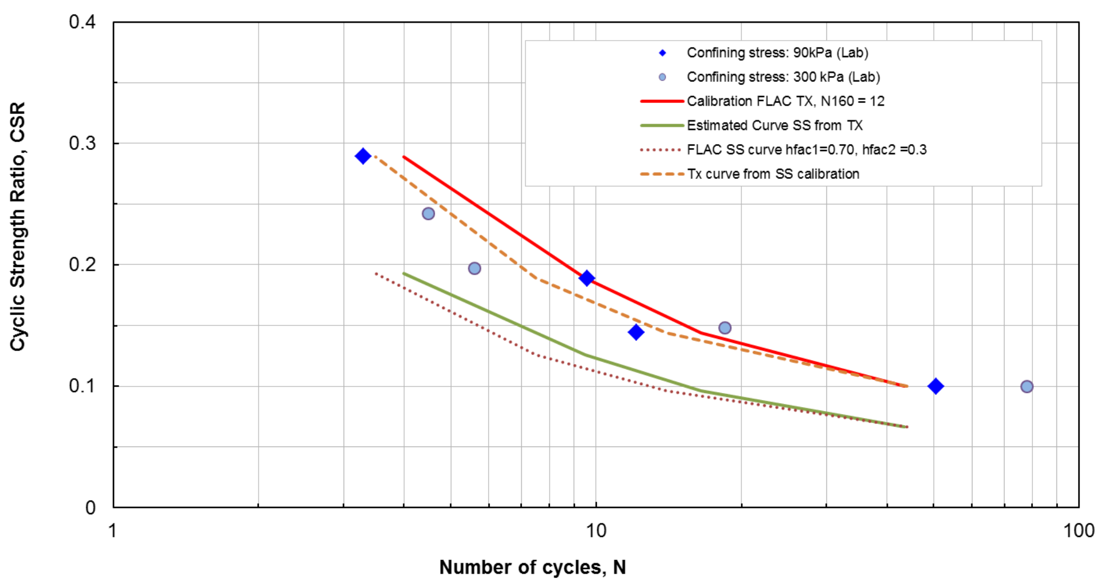


Figure 3. Predicted and measured liquefaction response for the considered alluvial soil

4.2 Earthquake Input Motions

For the dynamic analysis, two different seismic records were considered: a) to represent operational design earthquake producing a peak ground acceleration (PGA) of 0.27g, a 475 year return period earthquake; and b) to represent closure design earthquake producing a PGA of 0.34 g, a 2500 year return period earthquake. The seismic records were corrected by baseline and high frequencies were eliminated to assure better transmissibility of seismic waves in the numerical models.

The operational earthquake with a return period of 475 years was considered in assessments of seismic performance for Cases 1 to 3 (i.e., starter dam cases). For Cases 4 through 6 (i.e., the cases with the ultimate dam configuration) including the closure conditions, the 2500 year return period earthquake was taken into account.

5 DISCUSSION OF NUMERICAL ANALYSIS RESULTS

Table 2 summarizes estimated earthquake-induced deformations for each analyzed case. It should be noted that these deformations do not include post-earthquake deformations. The computed seismic response is representative for the permanent deformations of the dam and foundation at the end of the earthquake shaking.

For all cases considered in this paper, the levels of estimated vertical displacement at the dam crest are less than the design freeboard of 2.0 m, thus indicating no potential overflow of tailings at the end of the earthquake, as seen in Table 2.

Most of the cases analyzed showed large horizontal displacements ranging from approximately 2 m to 3 m throughout the dam and foundation, which would be able to compromise integrity of the liner system, exceeding tolerable limits of tensile strength and strain of the geomembrane liner.

Table 2. Summary of earthquake-induced deformation at the end of earthquake shaking

Case	Buttress	Foundation improvement	Earthquake	Maximum Displacement in model (m)		Displacement at dam crest (m)			
						Upstream edge		Downstream edge	
			PGA (g)	Horiz.	Ver.	Horiz.	Ver.	Horiz.	Ver.
1	no	no	0.27	2.58	-0.81	2.06	-0.10	2.29	-0.38
2	yes	no	0.27	2.66	-0.66	2.01	-0.36	1.99	-0.50
3	yes	yes (local)	0.27	1.79	-0.29	1.78	-0.08	1.78	-0.17
4	no	no	0.34	2.97	-1.11	2.22	-0.43	2.28	-0.53
5	yes	no	0.34	2.82	-0.88	2.48	-0.66	2.48	-0.76
6	no	yes (extensive)	0.34	1.42	-0.35	1.41	-0.02	1.42	-0.07

Note: Positive horizontal displacement indicates the dam moves toward downstream.
Negative vertical displacement indicates the dam moves downward.

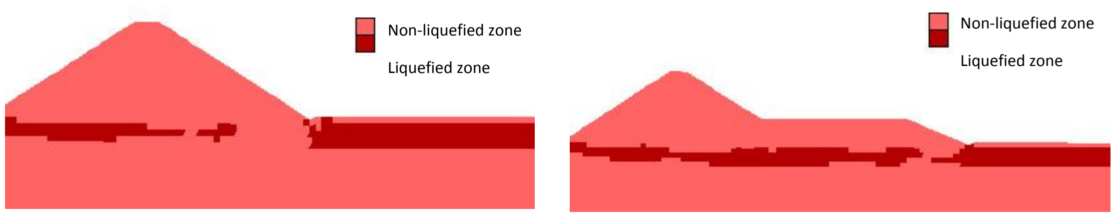
5.1 Case 1

The horizontal displacements at the dam crest elevation varied between approximately 2.06 m and 2.29 m, and the vertical displacements between approximately 0.10 m and 0.38 m, as shown in Figures 4a and 4b. The maximum displacements occurred near the downstream toe of the starter dam and were estimated to be approximately 2.58 m horizontally and 0.81 m vertically. Based on the estimated horizontal displacements, it is anticipated that the seismic deformations would be able to compromise the liner integrity. Additionally, the level of deformation along the lower downstream dam slope might result in a relatively large post-earthquake flow slide. An uplift of the ground surface adjacent to the downstream toe was observed. As expected, liquefied zones within the alluvial foundation soil could be identified, as shown in Figure 5a. However, it is noted that the alluvial foundation soil below the lower portion of the dam downstream slope did not experience liquefaction because the lateral migration of the foundation soils towards the downstream free field produced a dilative response of the underlying alluvial foundation soil as indicated by the uplift of the ground surface adjacent to the downstream toe.

Hence, a couple of remediation measures were taken into consideration: Case 2 including the 60-m wide buttress and Case 3 including the 25-m wide buttress and foundation improvement, as shown in Figures 2a and 2b.

(a) (b)

Figure 4. Case 1 (a) Horizontal displacement, and (b) vertical displacement contours



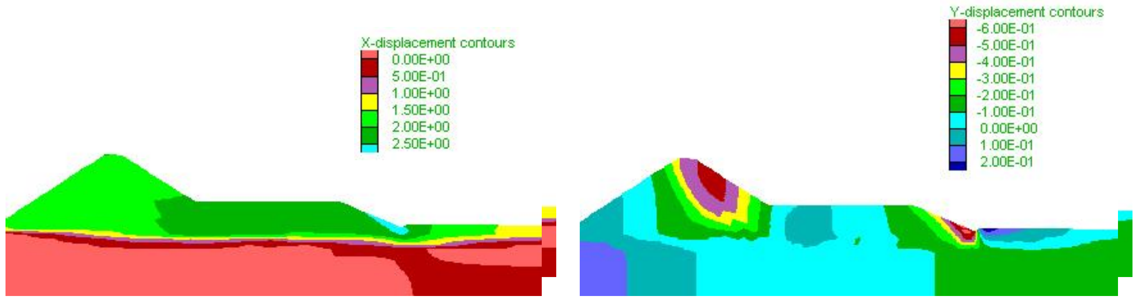
(a) (b)

Figure 5. (a) Liquefied zone for Case 1, (b) liquefied zone for Case 2

5.2 Case 2

The horizontal displacements at the dam crest elevation were estimated to be approximately 2.0 m, and the vertical displacements varied from approximately 0.36 m to 0.50 m, as shown in Figures 6a and 6b. The maximum displacements occurred at the downstream toe of the buttress, and the maximum horizontal and vertical displacements were estimated to be approximately 2.66 m and 0.66 m, respectively. A large horizontal displacement was predicted within the dam body and buttress because the alluvial foundation beneath these structures liquefied, as shown in

Figure 5b. Similar to Case 1, an uplift of the ground surface adjacent to the downstream toe of the buttress could be observed. A comparison of results from Case 1 and Case 2 reveals that the placement of the toe buttress increased significantly the vertical displacements at the crest zone. This is because the addition of the buttress to the dam toe did not allow for a dilative behavior of the alluvial foundation soil below the lower downstream slope (see Figure 5b). The horizontal displacement at the upstream crest was only slightly reduced. Thus, the results indicate that the buttress remediation approach would bring only little improvement to the seismic performance of the starter dam.

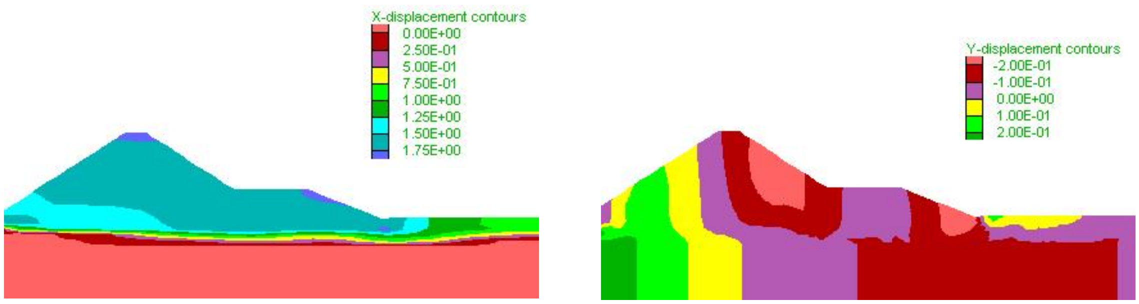


(a) (b)

Figure 6. Case 2 (a) Horizontal displacement, and (b) vertical displacement contours

5.3 Case 3

The horizontal displacements at the dam crest elevation were estimated to be approximately 1.78 m, and the vertical displacements varied from approximately 0.08 m to 0.17 m, as shown in Figures 7a and 7b. The maximum displacements occurred at the downstream toe of the buttress and were estimated to be approximately 1.79 m horizontally and 0.29 m vertically. Comparison of results from Case 3 and Case 1 shows that the local alluvial foundation improvement combined with placement of the toe buttress reduced the vertical displacements at the upstream crest by approximately 20 % and at the downstream crest by approximately 55 %. Additionally, the horizontal displacements at the upstream and downstream crests were reduced by approximately 14 % and 22 %, respectively. In general, it is anticipated that this combined remediation would be able to improve the seismic performance of the starter dam.



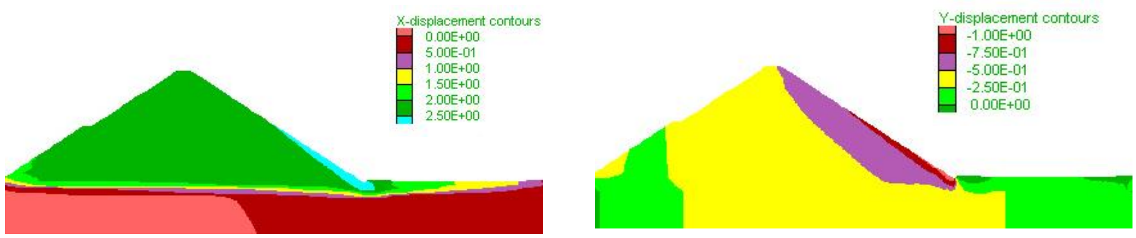
(a) (b)

Figure 7. Case 3 (a) Horizontal displacement, and (b) vertical displacement contours

5.4 Case 4

The ultimate dam configuration corresponding to Case 4 was subject to the 2500-year earthquake representative for closure conditions. Averaged horizontal displacement at the dam crest elevation was approximately 2.25 m, and averaged vertical displacement was approximately 0.48 m, as shown in Figures 8a and 8b. The maximum displacements occurred near the downstream toe of the ultimate dam and were estimated to be approximately 2.97 m horizontally and 1.11 m vertically. Likewise, it is anticipated that the seismic deformations predicted for the ultimate dam with no remediation, would be able to adversely impact the liner integrity. Additionally, the levels of deformation in the lower downstream dam slope might result in a relatively large post-earthquake flow slide. Uplift of the ground surface adjacent to the downstream toe was observed. Similarly to the starter dam with no remediation, the alluvial foundation soil below the lower portion of the downstream slope showed the tendency to dilate.

Hence, the following remediation alternatives were considered for the ultimate dam configuration: Case 5 which involves the 40-m wide buttress; and Case 6 which includes extensive foundation improvement, as shown in Figures 2d and 2e, respectively.



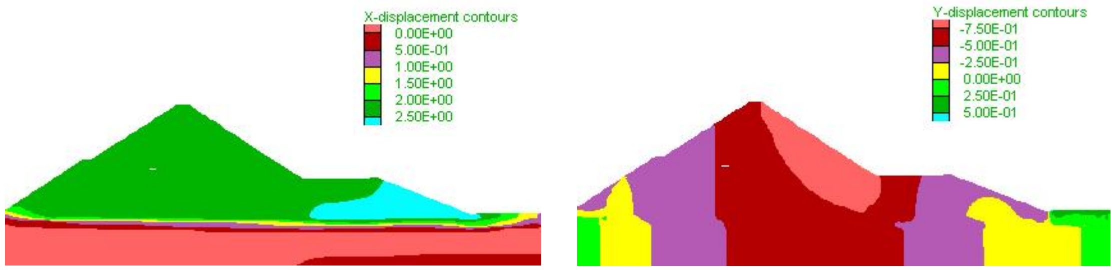
(a) (b)

Figure 8. Case 4 (a) Horizontal displacement, and (b) vertical displacement contours

5.5 Case 5

Averaged horizontal and vertical displacements at the dam crest elevation were estimated to be approximately 2.48 m and 0.71 m, respectively, as shown in Figures 9a and 9b. A maximum vertical displacement of approximately 0.88 m occurred at mid-height of the rockfill downstream shell due to the confinement of the toe buttress, while a maximum horizontal displacement of approximately 2.82 m occurred at the downstream toe of the buttress.

As the estimated displacements between Case 4 and Case 5 were compared, these were observed to have a similar tendency to the displacement patterns for Cases 1 and 2. The placement of the toe buttress increased both vertical and horizontal displacements at the crest elevation. The estimated vertical and horizontal displacements at the crest elevation were increased by approximately 48 % and 10 %, respectively. Therefore, the results indicate that buttress remediation does not improve the seismic performance of the ultimate dam configuration.



(a) (b)

Figure 9. Case 5 (a) Horizontal displacement, and (b) vertical displacement contours

5.6 Case 6

The horizontal displacements at the dam crest elevation were estimated to be approximately 1.42 m, and the vertical displacements less than 0.1 m, as shown in Figures 10a and 10b. A maximum vertical displacement of approximately 0.35 m occurred at the lower portion of the dam downstream slope, while a maximum horizontal displacement of approximately 1.42 m occurred at the downstream crest of the dam. As the estimated displacements for Case 6 were compared to those for Case 4, the foundation improvement involving replacement of the liquefiable alluvial layer below the downstream portion of the expanded dam resulted in a significant reduction of the overall displacements. The estimated vertical and horizontal displacements at the crest elevation were reduced by approximately 90 % and 37 %, respectively. The results indicate that this extensive foundation improvement of all mitigation measures considered in this study would be the most effective to improve the seismic performance of the ultimate dam. It is also anticipated that the post-earthquake performance of the dam will be improved because the liquefaction potential within the foundation soil was eliminated by the replacement of the liquefiable foundation soil with the compacted drained structural fill (non-liquefiable material). Therefore, no damage to the liner system is expected.

(a) (b)

Figure 10. Case 6, (a) Horizontal displacement, and (b) vertical displacement contours

6 CONCLUSIONS

A fully coupled dynamic liquefaction analysis was performed to assess the levels of earthquake-induced deformation in the dam and soil foundation. The dam foundation consisting of medium-dense saturated alluvial soils was observed to experience liquefaction under design earthquake loadings. Remediation measures considered for improving the seismic performance of the dam included downstream buttress and in-situ foundation improvement. The buttress placed at the downstream toe of a 30-m high starter dam was observed to make little improvement to the dam seismic performance, while the starter dam having a smaller buttress combined with local foundation improvement performed effectively under the operational design earthquake (475 year return period earthquake) considered in this study.

The dynamic response of a 60-meter high ultimate dam considered for the next construction stage was also investigated. Results from the numerical analyses indicate that the ultimate dam (expanded TSF) might have an adverse seismic performance under the closure design earthquake (2500 year return period earthquake) and the remediation measures might be still required to improve it. The results demonstrated that the buttress remediation could not contribute to an improvement in seismic performance of the ultimate dam. The extensive foundation improvement of the alluvial layer below the downstream portion of the expanded dam has the potential for a significant reduction of the overall displacements. It is anticipated that the ultimate dam will perform well during the closure design earthquake and even under post-earthquake loading conditions because the liquefaction potential within the foundation soil was eliminated by replacing the liquefiable foundation soil with the compacted drained structural fill (non-liquefiable material).

Most of the cases analyzed showed large horizontal displacements ranging from approximately 2 m to 3 m in the dam and foundation, which would be able to compromise integrity of the liner system, exceeding tolerable limits of tensile strength and strain of the geomembrane liner. Extensive foundation improvement on the ultimate dam configuration showed a significant reduction of the earthquake-induced deformations to an allowable level that can minimize the risk of geomembrane damage.

Buttress remediation did not function effectively for this particular case study involving a saturated medium-dense alluvial foundation soil susceptible to the earthquake liquefaction. However, the buttress could still function as an effective remedial measure to improve seismic performance of the embankment dam underlain by non-liquefiable foundation soils.

For all cases considered in this paper and an assumed design freeboard of 2.0 m, the levels of estimated vertical displacement at the upstream crest of the dam indicate no potential overflow of tailings. However, it is important to note that some cases might still experience tailings overflow under post-earthquake loading conditions.

The earthquake-induced deformations described in this paper do not include post-earthquake deformations. Total permanent displacements should be evaluated by considering liquefied undrained shear strength of the alluvial foundation soil in order to verify the potential for flow failure phenomena.

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