

## FAILURE OF FUJINUMA DAM DURING THE 2011 TOHOKU EARTHQUAKE

Daniel Pradel<sup>1</sup>), Joseph Wartman<sup>2</sup>), and Binod Tiwari<sup>3</sup>)

1) Principal Engineer, Group Delta Consultants, Torrance, and Adjunct Associate Professor, University of California, Los Angeles, USA

2) Associate Professor, Dept. of Civil Engineering, University of Washington, Seattle, USA

3) Assistant Professor, Department of Civil and Environmental Engineering, California State University, Fullerton, USA

dpradel@ucla.edu, joseph.wartman@uw.edu, btiwari@fullerton.edu

**Abstract:** The Mw=9.0 Tohoku Earthquake resulted in failures of the two dams impounding Fujinuma Lake, in Fukushima Prefecture. After the event, several mechanisms were postulated regarding the failure modes of the dams. The authors have performed numerical dynamic analyses whose principal aim was to determine the dams' likely failure mechanism. Our numerical analyses of the Fujinuma Main Dam predicted several meters of lateral displacement of the downstream face due to seismic shaking, i.e., sliding, and a corresponding large drop in the crest elevation. This drop rendered the dam vulnerable to overtopping which ultimately breached the dam. Additional numerical stability analyses also found that the Fujinuma Saddle Dam had a static factor of safety below unity under rapid drawdown conditions, and was therefore vulnerable to a rapid release of the reservoir water when the main dam was breached.

### 1. INTRODUCTION

On March 11, 2011, the Magnitude Mw=9.0 Tohoku Earthquake triggered an extremely destructive tsunami that killed thousands of people, caused wide spread liquefaction, and resulted in tens of billions of dollars in damage. Comparatively, there were relatively few seismically induced slope and dam failures.

According to JCOLD (2011), dams generally behaved very well, suffering only minor to moderate damage during the Tohoku Earthquake. The single exception was the 18½ m high Fujinuma earthdam (Figure 1), which was constructed between 1937 and 1949, near Sukagawa City in Fukushima Prefecture (N37.302°, E140.195°). The excellent performance of Japanese dams during one of the largest earthquakes in recent history, attests to the excellence of Japanese dam construction and long term maintenance.

Prior to the Tohoku Earthquake, the Fujinuma dam retained a small reservoir known as Fujinuma Ike (maximum volume of approximately 1.5 million cubic meters), that was used for irrigation and leisure. According to EERI (2011), the main dam was reported to have begun breaching within 20 minutes of the earthquake and a photo, taken approximately 25 minutes after the earthquake, shows almost the entire length of the dam being overtopped. The uncontrolled discharge from the breach was channeled through a narrow valley, destroying a small village at the mouth of the valley, and killing 8 people (Towhata, 2011).

According to EERI (2011) the dam was overtopped due to a drop in crest elevation. The EERI report postulated three possible causes for the lowering of the crest: (a) an upstream slope failure, (b) a downstream slope failure resulting from sliding on thick organic paleo-soils, or (c) downstream sliding through poorly compacted fill. EERI also indicated that an erosion mechanism was possible, with flow through cracks along the crest or internal seepage.

A small Saddle Dam at the site which retained the southeast end of Fujinuma Lake also failed as a result of the earthquake. This secondary dam was not breached and experienced a slope failure towards the reservoir (Figure 2). According to EERI (2011) the saddle dam' failure could have been caused by strength loss during the earthquake, i.e., sliding, or by rapid drawdown from the rapid release of the reservoir water through the main dam breach.

The authors have performed a study whose principal aim was to present a likely cause for the failure of the two dams that impounded Fujinuma Lake.

### 2. SITE OBSERVATIONS

#### 2.1 Introduction

In April 2011 the authors visited Japan as members of the American Society of Civil Engineers (ASCE) Embankment, Dams and Slope Committee (EDS). During their reconnaissance mission (Wartman et al., 2011) the authors performed an investigation of the Fujinuma Lake

site, which included detailed observations and measurements of the failures of the main dam (which was breached) and of the saddle dam (which was not breached). The main purpose of their visit was to document the geotechnical conditions of the two failed dams and to determine their probable failure mechanisms.



Figure 1 Photograph of the breached Fujinuma Main Dam (the arrows point to a layer of dark and organic residual soil)



Figure 2 Photograph of the failed Fujinuma Saddle Dam



Figure 3 Photograph of a near vertical section of the Fujinuma Main Dam breach (looking South).

## 2.2 Fujinuma Main Dam

Although a significant portion of the main dam had been washed away by overtopping, the EDS team was able to make key observations that revealed the nature of the dam construction and its likely mode of failure. For example, along the breached section, nearly vertical exposures up to 5m in height of cohesive fill materials and compaction layering were exposed (Figure 3). Furthermore, evidence of a translational sliding mechanism was observed along the right abutment which included a slightly elevated bulge and a large displaced block from the downstream face of the dam. Lastly, the breach exposed foundation materials at several locations along the base of the dam. Of particular interest to our team was a dark residual soil layer which blanketed the base of the dam and was also exposed downstream along the left abutment (Figure 1); this residual soil layer (paleo-soil) was rich in organics and had been locally reworked into compacted fill along the foundations (or base) of the dam.

Since no evidence of soil liquefaction was observed along the intact portions of the dam and abutments, our team concluded that the probable failure mechanism was a seismically induced landslide with a basal plane located through the highly organic residual soil layer. Please note that the GEER team proposed a similar mode of failure independently (Harder et al., 2011).

Based on their field observations, the EDS team reached the conclusion, that sliding of the downstream face probably occurred during the Tohoku Earthquake. The seismic displacements must have been sufficient to lower the crest of the dam and allowed, initially, a slow release of water over the crest of the dam. Overtopping caused erosion of the top of the dam, which culminated in a full blown breach, and the overtopping which photographed 25 minutes after the Earthquake and reported in EERI (2011).

## 2.3 Fujinuma Saddle Dam

Although the failure of the Saddle Dam (Figure 2) is reminiscent of the 1971 Lower San Fernando Dam liquefaction failure, careful examination of the downstream face of the dam and of the abutments did not reveal evidence of sand boils, lateral spread, or liquefaction related effects.

A LIDAR investigation by Kayen et al. (2011) found little deformation on the slope immediately adjacent to the dam. Hence, our team concluded that the failure was probably due to rapid drawdown, i.e., caused by the sudden release of the reservoir after the main dam failed. Please note that the GEER team (Harder et al., 2011) reached a similar conclusion independently.

### 3. ENGINEERING ANALYSES

#### 3.1 Introduction

To evaluate the likelihood of the postulated mechanisms, our team conducted independent engineering analyses of the two dam failures. Please note that we did not have the benefit of subsurface investigation or testing and thus the properties used in our study are solely based on the detailed observations we performed at the site, and on our experience.

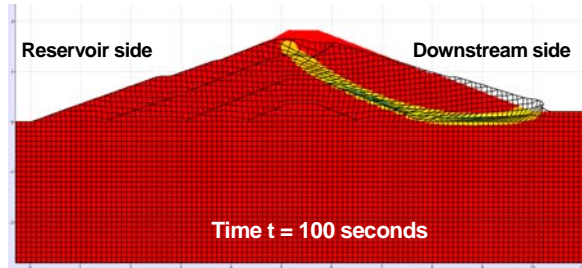


Figure 4 Mesh used in our dynamic analyses of the Fujinuma Main Dam and predicted mode deformation (displacements are exaggerated). Shear strains contours are also shown.

#### 3.2 Main dam

To evaluate the likelihood of the postulated mechanism, we performed a relatively simple dynamic numerical analysis using the computer program FLAC version 7.0 (Itasca, 2011). Figure 4 depicts the mesh used in our analysis, as well as the predicted mode of deformation. The ground motions used in our analyses were obtained from several stations of the KNET network where bedrock was shallow, namely stations FSK015 (PGA=0.28g) TCG001 (PGA= 0.4g) and FSK025 (PGA=0.15g). Due to manuscript length limitations only the numerical results using the FSK015 motions are presented herein (the velocities from FSK015 depicted in Figure 5 were the input at the base of our FLAC model). To allow damping to vary with time during the earthquake, we adopted, for simplicity, hysteretic damping (Figure 6) using as backbone shear moduli based on the “upper range” curves by Seed & Idriss (1970). The numerical approximation shown in Figure 7, was considered reasonable for the on-site cohesive materials.

For sliding and permanent displacement to occur a hypothesis of the shear strength of the materials was required in our model. Based on the height of the near vertical sections (measured angles ranged from 60° to vertical) we estimated that fills had a minimum static undrained shear strength of 20 kPa. This strength was adopted along the upstream face of the reservoir and doubled along the unsaturated downstream face to account

for the increase due to soil suction. Within the saturated core of the dam the increase of strength with depth was based on an undrained cohesion to total stress ratio,  $S_u/\sigma = 0.25$ . Please note that no differentiation was made in terms of strength between the natural residual clay (paleo-soil) and the dam’s compacted fill.

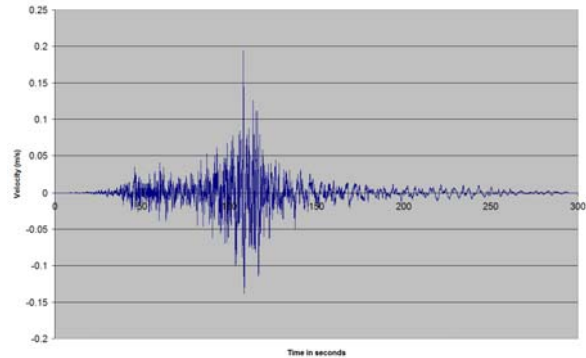


Figure 5 Velocity input at the base of the numerical model for the FSK015 recording (PGA=0.28g)

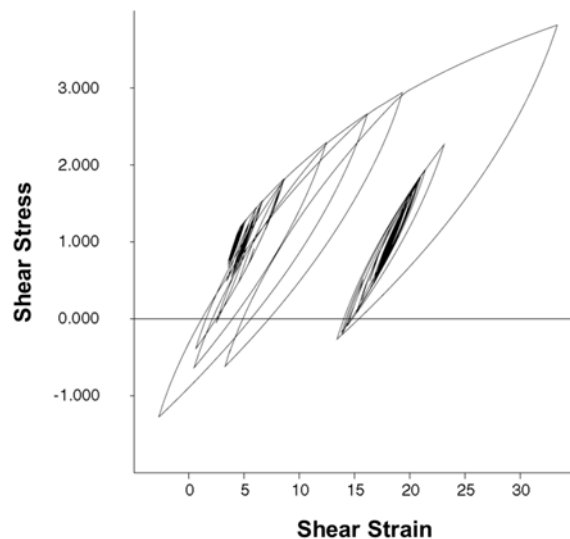


Figure 6 Hysteretic damping

Ishihara (1996) has shown that the dynamic strength of cohesive materials is larger than the static strength. At low confining stresses Ishihara’s tests on Nanamawari Izu clay showed an increase of up to 100% when subjected to a small number of loading cycles and almost no increase with large number of cycles. The range of adopted strength profiles used in our study are shown in Figure 8.

Our numerical analyses generally predicted meters of lateral displacement in the downstream direction due to seismic shaking (Figure 4), with slippage occurs taking place along the base of the dam, i.e., where the dark residual soil and fill were present (Figure 1). More importantly the model

showed a corresponding large drop in the crest elevation. The predicted drop in crest elevation would have rendered the dam very vulnerable to overtopping. This mode of failure is consistent with the conclusions of our teams' observations, Harder et al. (2011), and is one of the postulated mechanisms in EERI (2011). Lastly, the proposed failure mode is also consistent with the photograph taken by M. Yoshizawa approximately 25 minutes after the earthquake (contained in the EERI (2011) report), which does not show evidence of a slide towards the reservoir.

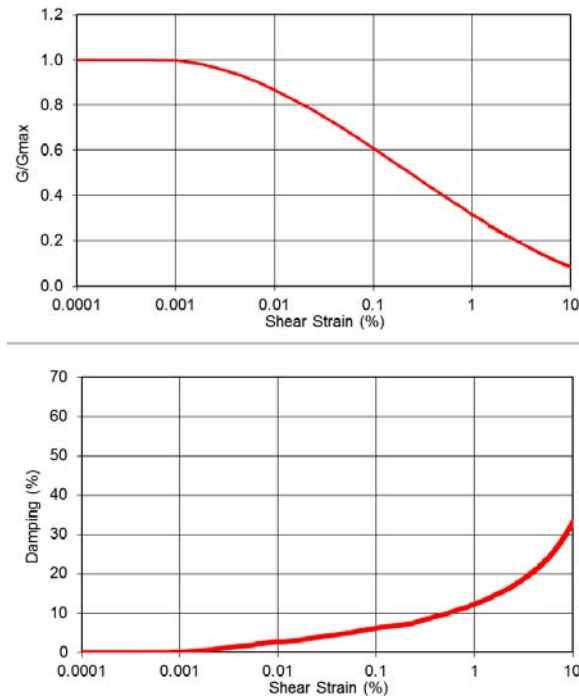


Figure 7 Backbone modulus reduction and damping ratios used for hysteretic damping

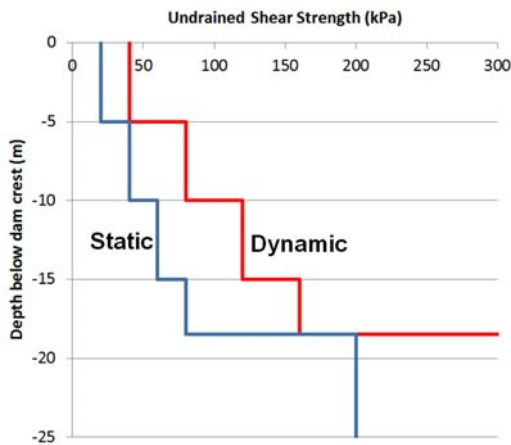


Figure 8 Range of adopted static and dynamic shear strengths

### 3.3 Saddle dam

To evaluate the likelihood of the rapid drawdown failure mechanism postulated by our team, we conducted a simple analysis of the stability of the saddle dam by the strength reduction method. The mesh used and predicted mode of failure are shown in Figures 9 and 10. In our analysis we assumed that both dams were built at the same time with similar compacted fill materials, hence, we used the static shear strength shown in Figure 8.

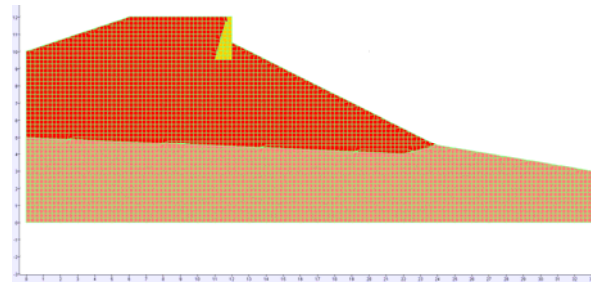


Figure 9 Mesh used for the rapid drawdown analyses of the Fujinuma Saddle Dam.

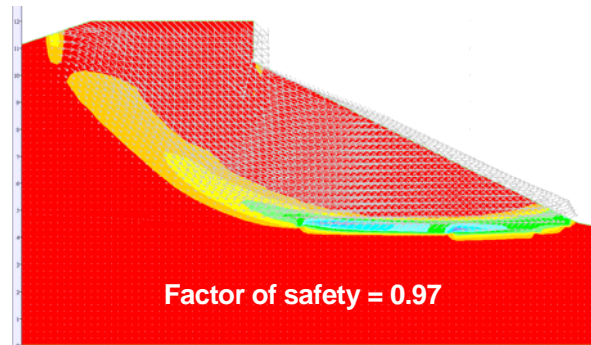


Figure 10 Mode of failure predicted by the Strength Reduction Method during rapid drawdown. Also shown are shear strain contours and displacement vectors)

Our numerical stability analyses found a static Factor of Safety of 0.97, which indicates that the saddle dam was vulnerable to rapid drawdown mode of failure. In our opinion, the rapid release of the reservoir water through the main dam breach was the likely cause of failure.

A rapid drawdown failure is one of the mechanisms postulated by EERI (2011). It is consistent with both our team's original conclusions and those of the GEER team (Harder et al. 2011). Please note that the absence of permanent displacements and distress in the downstream face of the saddle dam strongly supports a rapid drawdown failure, versus a mode of failure related to strength loss caused by earthquake shaking.

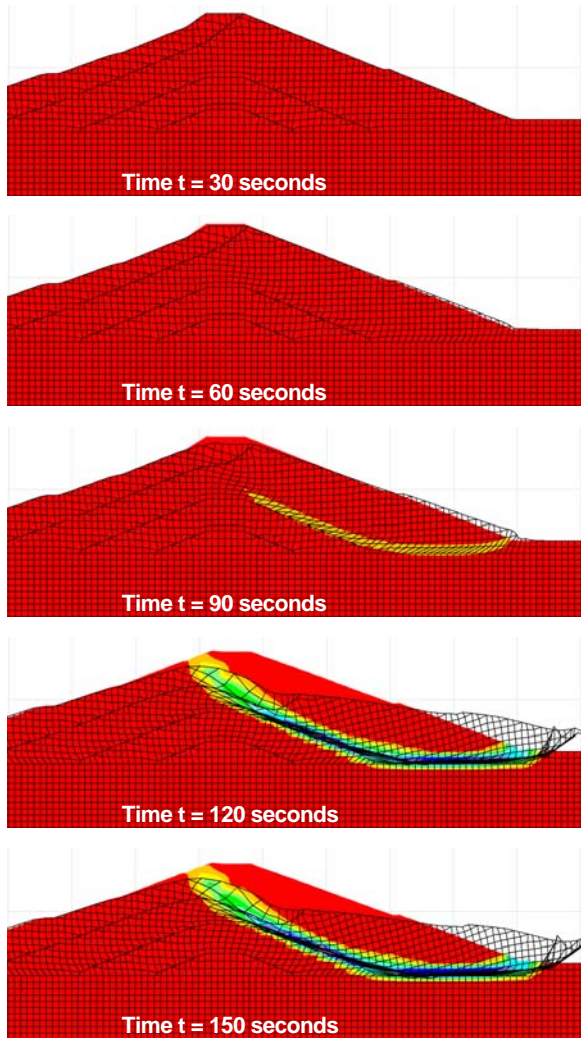


Figure 11 Predicted deformations and shear strains contours of Fujinuma Main Dam using static undrained shear strengths (displacements are exaggerated five times; maximum displacement at  $t = 150$  seconds is approximately 2 m).

Our study of the saddle dam stability also suggests that the undrained strengths in Figure 8 which were used for the two dams are reasonable, as they explain both the Saddle Dam failure and the maximum near vertical heights of fill on the main dam.

#### 4. CONCLUSIONS

Several failure modes have been postulated about the mode of failure of the Fujinuma dams (e.g., EERI, 2011). Based on our observations at the site and experience, simplified dynamic analyses were conducted to determine the dams' likely failure mechanisms.

Our numerical model of the Fujinuma Main Dam

predicted large lateral displacements (meters) of the downstream face due to seismic shaking, i.e., sliding, and a corresponding major drop in the crest elevation (Figure 11). This drop rendered the dam vulnerable to overtopping and ultimately to failure.

Our numerical stability analyses also found that the Fujinuma Saddle Dam had a static factor of safety below unity under rapid drawdown conditions, and was therefore vulnerable to a rapid release of the reservoir water when the main dam was breached.

#### Acknowledgements:

The authors acknowledge support of the American Society of Civil Engineers (ASCE) for funding our reconnaissance mission in April 2011, of the Japan Landslide Society for its assistance during our mission, and of the Center for Urban Earthquake Engineering (CUEE) in Tokyo Institute of Technology for their support of the forthcoming international conference.

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