Failure of the Fujinuma Dams during the 2011 Tohoku Earthquake

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ABSTRACT: The M_w 9.0 Tohoku Earthquake resulted in failures of the two dams impounding Fujinuma Ike, in Fukushima Prefecture, Japan. After the event, several mechanisms were postulated regarding the failure modes of the dams. The authors made detailed observations at the site and performed numerical dynamic analyses whose principal aim was to determine the dams' potential failure mechanisms. Our numerical analyses of the Fujinuma main dam predicted over 5 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 main 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.

INTRODUCTION

The Magnitude 9.0 (M_w) Great East Japan Earthquake that struck Japan on March 11, 2011 is the fifth largest earthquake ever recorded. This earthquake, which is also well known by its Japanese name as Tohoku earthquake, resulted in more than 20,000 deaths and an estimated US\$210 billion damage (EERI, 2011). The earthquake triggered a destructive tsunami along the eastern coast of Japan, caused severe structural damage, and produced widespread ground failure (i.e., liquefaction, dynamically induced settlement, and landslides) up to 300 miles (480 km.) away from the epicenter (Pradel et al., 2012). Comparatively, there were relatively few seismically induced slope and dam failures, although the few slope failures we observed were very significant in nature (Pradel et al., 2011).

According to JCOLD (2011), dams generally behaved very well, suffering only minor to moderate damage during the Tohoku earthquake. The single exception was the 18¹/₂ m high Fujinuma earthdam (Figure 1) which was constructed between 1937 and 1949, near Sukagawa City in Fukushima Prefecture (N37.302°, E140.195°).



FIG. 1. Photograph of the breached Fujinuma main dam and reservoir.

Prior to the Tohoku earthquake, the Fujinuma dams retained a small reservoir known as Fujinuma Ike (Ike means pond or small lake in Japanese), which was used for irrigation and leisure. The maximum volume of the lake was approximately 1.5 million cubic meters; the crest length and width were 133 m and 6 m, respectively (JCOLD, 2011).

When the earthquake occurred, the reservoir level was almost full; according to a witness, water was seen overflowing the crest 20 to 25 minutes after the earthquake (Matsumoto et al., 2011). A photo in the EERI (2011) report shows almost the entire length of the dam being overtopped approximately 25 minutes after the earthquake. The uncontrolled discharge from the breach of the Fujinuma main dam was channeled through a narrow valley, destroying a small village at the mouth of the valley (Figure 2), and killing 8 people (Towhata et al., 2011).

According to EERI (2011) the dam was overtopped due to a drop in crest elevation. The EERI report postulated several 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, (c) downstream sliding through poorly compacted fill, or (d) an erosion mechanism with flow through cracks along the crest or internal seepage. Please note that EERI indicated that mechanism (d) was possible, suggesting that EERI considered mechanism (d) less likely than (a) through (c).

A small saddle dam at the site which retained the southeast end of Fujinuma Ike also failed as a result of the earthquake. This secondary or auxiliary dam was not breached and experienced a slope failure towards the reservoir.



FIG. 2. Photograph of the flood path downstream of Fujinuma main dam in the village of Naganuma (Fukushima Prefecture).

The saddle dam failure is depicted in Figure 3.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 aims is to document the failures and through numerical modeling analyze the potential modes of failure of the two dams that impounded Fujinuma Ike.

SITE OBSERVATIONS

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 Ike site, which included detailed observations, Gigapan imagery (Wartman, 2011) 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 to Japan was to document the geotechnical conditions of landslides, levee failures and the failures of dams during the Great East Japan Earthquake.

Fujinuma Main Dam

Although most of the upper portion and a significant section of the core of the main dam had been washed away by overtopping and breaching, our team was able to make key observations that revealed the nature of the dam construction (including foundation preparation), and its potential modes of failure:



FIG. 3. Two photographs of the failed saddle dam of Fujinuma Ike.

• Along the breached section, nearly vertical exposures up to 5m in height of cohesive fill materials were exposed (Figures 4a and 4b). Evidence of compaction layering was also observed (Figure 4a).

• Along the right abutment large blocks had been displaced and uplifted along the downstream face of the dam (Figures 4b and 5). The position of the blocks indicated several meters of translational movement, i.e., a slope instability.



FIG. 4. Two photographs of the Fujinuma main dam and right abutment.

• The breach exposed a dark layer of residual soil which blanketed the base of the dam and large areas of the left abutment (Figures 1, 4b and 6). This residual soil layer, or paleo-soil, was rich in organics (e.g., roots and even tree trunks) and had been locally reworked into compacted fill along the base of the dam. The location of the paleo-soil layers was geometrically consistent (Figure 4) with the sliding failure mechanism suggested by Figure 5 and previously described above.

• A thin layer of sand blanketed the displaced blocks of cohesive soils and grass which covered the downstream slope prior to the failure (Figure 5). Similarly,

sand was observed covering vast areas of the reservoir (Figure 1), portions of the breach zone (Figure 4), and the left abutment (Figure 1). The sand appeared to have been deposited by water during the breach (i.e., post-earthquake) and we did not observe evidence of soil liquefaction (such as sand boils) along the intact portions of the dam and abutments.



FIG. 5. Uplifted and displaced blocks along the right abutment of the main dam.



FIG. 6. Organic rich layer of paleo-soil.

Please note that our observation regarding the cohesive nature of the dam's compacted fill is consistent with:

• Matsumoto et al. (2011): "The structure is a homogeneous type and most materials seem to be cohesive soil."

• Harder et al. (2011): "Embankment fill materials appeared to be generally cohesive and placed in lift thicknesses ranging between 20 and 45 centimeters."

Based on our field observations, our team concluded that the evidence clearly indicated that a landslide with a basal plane located through the highly organic residual soil layer likely occurred during the Tohoku earthquake. Please note that Harder et al. (2011) proposed a similar potential mode of failure independently.

Based on the large lateral displacements and uplifting of the downstream blocks, our team reached the conclusion, that the seismic displacements must have been sufficient to significantly lower the crest of the dam. This would have allowed, initially, a slow release of water over the crest of the dam, followed by overtopping and further erosion of the top of the dam, which would have culminated in a full blown breach, and the overtopping of the almost full crest of the dam, i.e., consistent with the photograph taken 25 minutes after the Earthquake and published in EERI (2011).

Fujinuma Saddle Dam

Although the failure of the saddle dam is reminiscent of images from the 1971 Lower San Fernando dam liquefaction failure, Figure 3 clearly shows a failure through cohesive materials. These materials appeared similar to those of the main dam which was built during the same period. Furthermore, careful examination by our team 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 water after the main dam failed.

Please note that the shores of the reservoir have numerous localized slope failures which are also consistent with rapid drawdown. LiDAR images of these failures are shown in Kayen et al. (2011)

POST-FAILURE INVESTIGATIONS

Two teams have investigated the properties of materials of the Fujinuma main dam after the dam failures. Shortly after the Tohoku earthquake a team from the University of Arkansas and USGS performed noninvasive surface wave testing at 56 sites affected by the earthquake, using the Spectral Analysis of Surface Waves (SASW) and the Multi-channel Analysis of Surfaces Waves (MASW) methods (Cox et al., 2011). Fujinuma dam was among the sites investigated and their measurements across the top of the remaining section of the main dam are reported in Table 1. The shear wave velocities measured indicate that the top 11 m were extremely soft. Based on our field investigation we believe that layers 1 through 3 in Table 1 consist of compacted fill from the base of the dam and underlying foundation materials, which include both natural and reworked residual soils (paleo-soils) similar to those in Figure 6.

Between August 2011 and January 2012 a government panel of Japanese experts was created to "review the cause of Fujinuma-Ike failure". The panel published a summary report on January 25, 2012 for release to the "public" which was translated into English by JCOLD (2012). Reportedly, the panel performed detailed mapping of the failed area and conducted a subsurface investigation consisting of borings and laboratory tests. The published report only contains their detailed mapping and conclusions. To date, the panel's boring logs and laboratory test results remain unavailable to the public.

Layer No.	Thickness (m)	P-wave velocity (m/s)	Shear wave velocity (m/s)	Poisson's ratio	Unit weight (kN/m ³)
1	2.01	150	80	0.3	16
2	1.52	1520	80	0.5	16
3	7.62	1520	180	0.49	16
4	12.31	1520	370	0.47	19

 Table 1. Surface Wave Measurements by Cox et al. (2011)

The panel reported that evidence was found for seven types of slides at the location of the main dam. Some are postulated as the overtopping obliterated some of them. Slides labeled No.1 and 2 are directed towards the lake, and slide No.4 is in a downstream direction. It is interesting to note that slide No.4 is shown on both the left and rights abutments. The right abutment portion of slide No.4 coincides with our observations (e.g., Figure 5).

The panel concluded that "the primary cause of the failure of Fujinuma Dam is the nature of its upper embankment and middle embankment and the triggering cause is the strong earthquake motion and its long duration." More specifically the panel indicated:

- That the dam experienced large acceleration for 100 seconds with a peak acceleration at the dam crest estimated at 0.45g;
- The compaction of the embankment was very low (a standard proctor of 84% was reported to the first author in a personal communication), especially in the upper embankment which consisted of sand rich materials;
- The slide that damaged the saddle dam occurred in the same manner as the failure of the main dam.

The main difference between our team's observations and the panel's summary report is that our team did not observe sand rich materials in the upper portions of either the main dam or saddle dam (e.g., Figures 3 and 4a). Please note that sandy materials in the upper portion of the dam are also not reported in Harder et al. (2011), Matsumoto et al. (2011) or JCOLD (2011).

Another difference is that our team concluded that the saddle dam experienced a rapid drawdown type of failure, whereas the panel suggested a dynamic failure similar to that of the main dam. A rapid drawdown failure is one of the mechanisms postulated by EERI (2011) and Harder et al. (2011).

GEOMECHANICAL MODELING

Introduction

To evaluate potential mechanisms of both Fujinuma dam failures our team conducted numerical analyses to investigate their performance using the computer program FLAC version 7.0 from Itasca (2011). We prepared pre-failure cross-sections, using topographic data from LiDAR (Kayen et al., 2011) and field surveys (JCOLD, 2012). We also used the shear wave velocities from Cox et al. (2011) which are reported in Table 1. Please note that we did not conduct shear strength testing and thus the strength properties used in this study are based on conventional static back-calculations using the detailed observations we performed at the site.

Shear Strengths

As seen in Figure 4a, the failure resulted in near vertical sections that exposed compacted fill materials (measured angles ranged from 60° to vertical). Based on the height of the vertical sections we estimated that dam fills had a 20 kPa minimum static undrained shear strength using Taylor charts (Taylor, 1937).

When this minimum static strength was used to analyze the stability of the saddle dam and evaluate the likelihood of the rapid drawdown failure mechanism postulated by our team, we obtained a Factor of Safety of 0.97 by the strength reduction method. The mesh used and the predicted mode of failure are shown in Figures 7 and 8. The predicted failure of the saddle dam is consistent with our field observations and independently supports our strength back-analyses of the near vertical face of the main dam using Taylor charts.

For sliding and permanent displacement to occur a hypothesis of the shear strength of all the fill materials was required in our numerical models. As previously described 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 an 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, $S_u/\sigma_z = 0.25$. This ratio is in the middle range of values in Ladd (1991) and is consistent with numerous soft soil sites (e.g., New Orleans). Please note that no differentiation was made in terms of strength between the natural residual soils (paleo-soils) and the dam's compacted fill, as some of the residual soil was observed to have been reworked as compacted fill.

Ishihara (1996) has shown that the dynamic shear strength of cohesive materials, τ_{cyc} , is generally 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 when subjected to a large number of cycles. The shear strengths used in our numerical analyses are consistent with Ishihara's findings (Figure 9).



FIG. 7. Mesh used for the stability analyses of the Fujinuma saddle dam.



FIG. 8. Mode of failure predicted by the Strength Reduction Method during rapid drawdown (Factor of safety = 0.97); also shown are shear strain contours and displacement vectors.

For clays, Boulanger and Idriss (2007) showed that the cyclic and monotonic undrained shear strengths are closely related and proposed a methodology to determine the cyclic softening that can occur in fine-grained soils subjected to a large number of cycles. Based on Boulanger and Idriss's methodology a cyclic strength well below the static undrained shear strength would be expected for Fujinuma dam due to the long duration and large number of cycles of the Tohoku earthquake. Hence, suggesting larger displacements than those predicted with our lowest shear strength hypothesis in Figure 9.

Ground Motions and Numerical Analyses

The ground motions used in our analyses were obtained from several nearby stations of the KNET network where bedrock was shallow, namely stations FSK015 (PGA=0.28g), TCG001 (PGA= 0.4g) and FSK025 (PGA=0.15g). To minimize wave reflections at model boundaries a quiet (viscous) boundary was specified along the base of the model, and free-field boundaries were specified along the edges. The

velocity input from FSK015, as shown in Figure 10, served as an input in some of our dynamic runs at the base of our FLAC model. To allow damping to vary with time during the earthquake, we adopted, for simplicity, hysteretic damping using as backbone shear moduli based on the "upper range" curves by Seed & Idriss (1970), and a minor amount of Rayleigh damping (0.1%) was also included. The adopted hysteretic damping used results in the damping curve shown in Figure 11, except when the shear strength of the materials is reached and plastic flow occurs.

The mesh used in our analyses is shown in Figure 12 with different colors indicating areas of varying shear strengths (Figure 9). As shown in Table 2, large displacements, i.e., failure was predicted for all the analyses using $\tau_{cyc} = S_u$. The predicted mode of failure for all three ground motions was very similar as shown in Figures 13 to 15.



FIG. 9. Adopted dynamic undrained shear strengths.



FIG. 10. Adopted Velocity input at the base of the numerical model for the FSK025 recording (PGA=0.15g).



FIG. 11. Modulus reduction and damping ratios from hysteric damping.



FIG. 12. Mesh used for dynamic analyses (grid base size = 1m x 1m).

Table 2.	Maximum	Displacements	Predicted by	v Dvnamic	Analyses
		Displacements	Liculture a	<i>,</i>	111111111111111111111111111111111111111

Ground Motion Record	PGA	$\tau_{cyc} = S_u$	$\tau_{cyc} = 1.5 \ _x S_u$	$\tau_{cyc} = 2_x S_u$
FSK015	0.28g	>10m	0.43m	0.09m
FSK025	0.15g	5.5m	0.02m	0.02m
TCG001	0.4g	>10m	0.02m	0.02m







FIG. 14. Predicted mode of failure for FSK025 and $\tau_{cyc} = S_u$.



FIG. 15. Predicted mode of failure for TCG001 and τ_{cyc} = S_u

CONCLUSIONS

Several failure modes have been postulated about the mode of failure of the two Fujinuma dams (e.g., JCOLD, 2012; EERI, 2011 and Harder et al., 2011). Observations at the site suggested that the downstream face of the Fujinuma main dam experienced a large slide during the Tohoku earthquake. Simplified dynamic analyses using shear wave velocities from surface wave testing and back-calculated shear strengths ($\tau_{cyc} = S_u$) show large lateral displacements (5+ meters) of the downstream face for three nearby recordings, and more importantly a corresponding major drop in the crest elevation (as seen in Figures 13 to 15). Such a drop would have rendered the main dam, which was almost full, very vulnerable to overtopping and ultimately to failure. Hence our simplified dynamic analyses show that a downstream failure is a likely failure mechanism of the Fujinuma main dam.

Since failure of the main dam is consistently predicted at $\tau_{cyc} = S_u$ (as shown in Table 2) it is unclear if cyclic softening of the type suggested by Boulanger and Idriss (2007) occurred or was a factor in the failure, since the Boulanger and Idriss methodology would have resulted in even larger displacements than those predicted in Table 2 with the lowest shear strength hypothesis.

Our numerical stability analyses also found that using back-calculated undrained shear strengths from the main dam, the similarly constructed Fujinuma saddle dam had a static factor of safety below unity under rapid drawdown conditions. Hence, it was vulnerable to a rapid release of the reservoir water when the main dam was breached, and thus, in our opinion, rapid drawdown was a likely failure mechanism.

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