# SLOPE STABILITY ANALYSIS OF EMBANKMENT DAM UNDER TOTAL AND EFFECTIVE STRESS

Tri Hartanto[1,*], Widjojo Adi Prakoso[2]

[1]Department of Civil Engineering, University of Indonesia, Depok, Indonesia

[*]*Corresponding author: adhe.trie@gmail.com*

**Abstract**

Very often many new construction and operating embankment dams need to be evaluated in terms of the slope stability. The necessity of considering body forces, pore-water pressures, and a variety of soil types in the analysis vitiates the application of methods that are well founded in the mechanics of continua and employ representative constitutive equations.

This study comparing stability analysis using total stress after the end of construction with effective stress couple of years later after the first impounding. Studies have indicated the advantages to be obtained employing an effective stress failure criterion (Bishop, 1952, Henkel and Skempton, 1955 and Bishop, 1960) for analysis and design of embankment dams. Pore-water pressure are determined from piezometer readings during the construction until the dam was operated.

This paper presents the results of stability analysis of embankments dam with both parameters and conditions, resulting that pore water pressures influence slope stability of the embankment.

Keywords: Slope stability, pore water pressure, total and effective stress, dam embankment

## 1. Introduction

Dams as large water retaining structures are one of the few civil engineering structures with very high risks and consequences. It is typically created by the emplacement and compaction of a various compositions of soil, sand, clay and/or rock, this type of dam is describing as earth fill dams. Soil compaction can lead to modification of soil structure and thus reduce permeability. Principal purposes of compaction in earthen dam are to increase stiffness to minimize settlements during and after construction, to increase strength to prevent sliding shear failure of dam and to make water tight to obtain required imperviousness of the core zone (Kanchana & Prasanna, 2015). Rather than concrete dam, the earth fill dam continues to be the most common type of dam, principally because its construction involves utilization of materials in their natural state with a minimum processing.

Geotechnical issues for earth fill dam could experience during construction, first filling or their operational stages are related to slope stability and seepage. Safety evaluation during these stages is becoming so important and must be taken carefully.

ICOLD carried out analyses of the data compiled to determine the most common cause of dam incidents. Others, including USCOLD (1975, 1988), USCOLD Committee on Dam Safety (1996), ANCOLD (1992), Charles and Boden (1985), Olwage and Oosthuizen (1984), and Gomez et al. (1979), have compiled data on incidents for various countries. Internal erosion-related dam collapse accounts for 46% of total dam collapse events worldwide. One of the causes of internal erosion is high pore water pressure, where it will cause water seepage to continue to enlarge, bringing soil particles and forming piping (Foster et al., 2000).

Table 1. Historical frequencies of failures and accidents in embankments of large dams constructed from 1800 to 1986, excluding dams constructed in Japan pre 1930, and in China (Foster et al, 2000)

| | No. of cases | | % failures (where known) | | Average frequency of failure ($\times 10^{-3}$) | |
|---|---|---|---|---|---|---|
| Mode of failure | All failures | Failures in operation | All failures | Failures in operation | All failures | Failures in operation |
| **Overtopping and appurtenant** | | | | | | |
| Overtopping | 46 | 40 | 35.9 | 34.2 | 4.1 | 3.6 |
| Spillway–gate | 16 | 15 | 12.5 | 12.8 | 1.4 | 1.3 |
| Subtotal | 62 | 55 | 48.4 | 47.0 | 5.5 | 4.9 |
| **Piping** | | | | | | |
| Through embankment | 39 | 38 | 30.5 | 32.5 | 3.5 | 3.4 |
| Through foundation | 19 | 18 | 14.8 | 15.4 | 1.7 | 1.6 |
| From embankment into foundation | 2 | 2 | 1.6 | 1.7 | 0.18 | 0.18 |
| Subtotal | 59 | 57 | 46.1 | 48.7 | 5.3 | 5.1 |
| **Slides** | | | | | | |
| Downstream | 6 | 4 | 4.7 | 3.4 | 0.54 | 0.36 |
| Upstream | 1 | 1 | 0.8 | 0.9 | 0.09 | 0.09 |
| Subtotal | 7 | 5 | 5.5 | 4.3 | 0.63 | 0.45 |
| Earthquake–liquefaction | 2 | 2 | 1.6 | 1.7 | 0.18 | 0.18 |
| Unknown mode | 8 | 7 | | | | |
| Total no. of failures | 136 | 124 | | | 12.2 (1.2%) | 11.1 (1.1%) |
| Total no. of failures where mode of failure known | 128 | 117 | | | | |
| No. of embankment dams | 11 192 | 11 192 | | | | |

**Note:** Subtotals and totals do not necessarily sum to 100%, as some failures were classified as multiple modes of failure.

US Army Corps of Engineers (2003) mention that the pore water pressure must be monitoring to analyse the stability of the dam. These issues have been recognised (Rojas *et al.*, 2008; Pagano *et al.*, 2008; Pagano *et al.*, 2010*a*; Toll *et al.*, 2011) and the slopes of earth dams are no exception to this phenomenon. Changes in pore water pressure distribution within the dam body and foundation soils may also provide information with regard to safety related to other feared mechanisms, such as erosion of watertight zones. Reliable monitoring and an appropriate interpretation of pore water pressures are thus fundamental activities through which dam safety conditions are checked over time and possible issues concerning slope stability and erosion phenomena may be brought to light (Jappelli, 2003, Amorosi *et al.* 2008 on Fontanella *et al*., 2012).

The objective of this paper is to show that different mechanisms acting and behaviour within the dam body after construction and steady seepage, with or without earthquake which is recorded by piezometer instruments.

This study is using Geostudio 2012, which has own system and capability to analysing geotechnical issues, such as slope stability on embankment.

## 2. Material and Methodology

The case study conducted on Raknamo dam (Fig. 1), a zoned earth dam, 37 m high, 438 m length, with 1V : 3H upstream slope, 1V : 2,5H, located on Raknamo river, East Nusa Tenggara Province, about 3 hours heading east, by plane from Jakarta, the capital city of Indonesia. The main benefit is to provide irrigation on 841 ha dan raw water supply. The first filling started from January 9$^{th}$, 2018. This dam is constructed and owned by Ministry of Public Works and Housing, Republic of Indonesia.

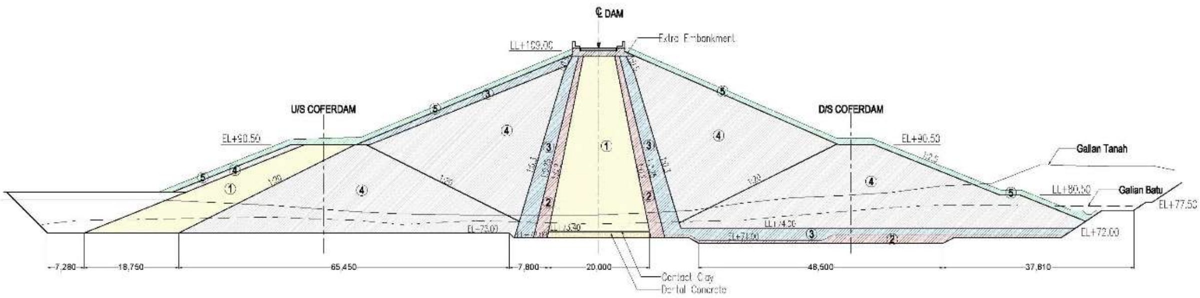


Fig. 1 Typical cross section of Raknamo dam

It consists of clay core as impervious material, is protected by 2 layer upstream and downstream layer, fines and coarser filter, and random material as a shoulder. Outer layer is covered by thins rip rap material. Treatment foundation has been carried out by curtain grouting below the clay core provides water-tightness inside the foundation.

The data used for empirical studies are divided into two categories, dam engineering and instrument data. Material properties for evaluation shown on Table 2.

The dam installed with some geotechnical instruments, such as piezometer to measure pore pressure, multilayer settlement, inclinometer, benchmark, strong motion accelerograph, v-notch for seepage measurement and AWLR. For this evaluation, we’ve only use piezometer reading from embankment started to initial impounding (February 2021) data. There are 2 cross section piezometer, MD 10 and MD 16. For this evaluation, we only use MD 16 because the section more depth rather than MD 10 section and representing behaviour of the dam and only piezometer which installed on the embankment, instruments on the foundation exclude for this study. The foundation model is isotropic with low permeability.

Table 2. Material properties for evaluation.

| No | Zone | γsat | γwet | CU | | UU | | Direct Shear | Permeability |
|---|---|---|---|---|---|---|---|---|---|
| | | | | c' | phi' | c | deg | deg | cm/sec |
| 1 | Core | 1.888 | 1.846 | 0.25 | 19.55 | 0.3 | 16 | - | 4.19E-8 |
| 2 | Fine Filter | 2.055 | 1.831 | - | - | - | - | 33 | 7.35E-3 |
| 3 | Coarse Filter | 2.065 | 1.867 | - | - | - | - | 35 | 7.34E-3 |
| 4 | Random | 1.947 | 1.860 | - | - | - | - | 37.23 | 7.49E-4 |
| 5 | Rip rap | | | | | | | 40 | 1.00E-2 |

## 3. Result and Discussion

Fig 2 shows layout plan of 2 section piezometer, MD 10 and MD 16. MD 16 more depth rather than MD 10 section, because MD 16 is located nearby the old river.

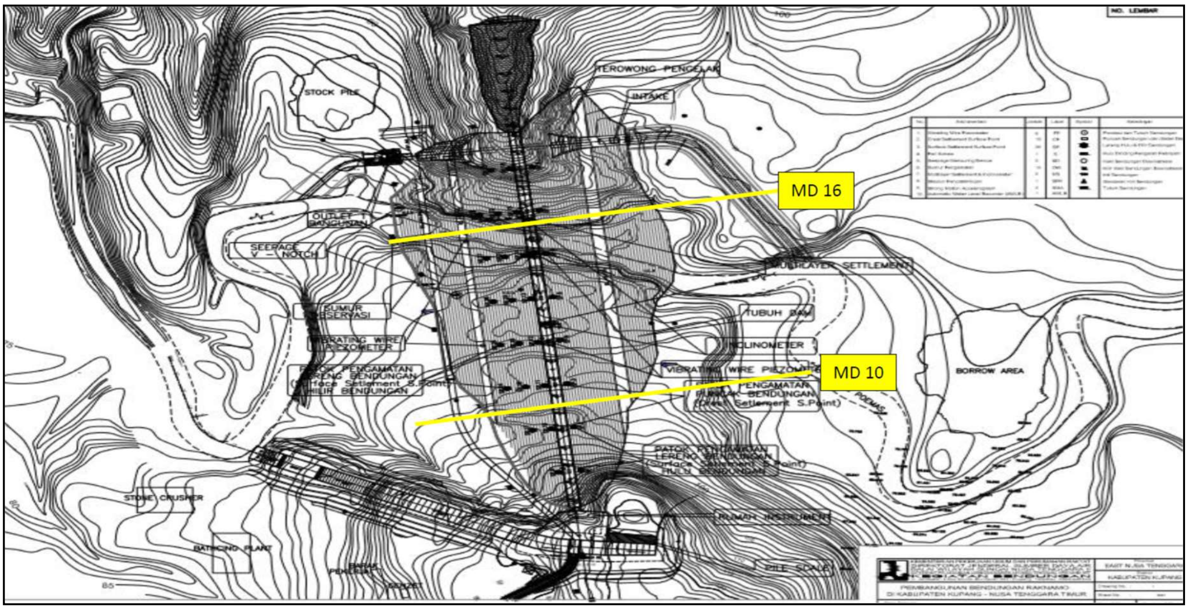


Fig 2. Layout of 2 section piezometer, MD 10 and MD16.

Fig 3 shows installed vibrating wire piezometer at MD 16 cross section. There are 15 installed piezometers in this section, 4 piezometers on foundation, 8 piezometers on clay core and 3 piezometers installed on random zone. VP-1, VP-2, VP-3 and VP-4 are on the foundation, exclude for evaluation on this study.

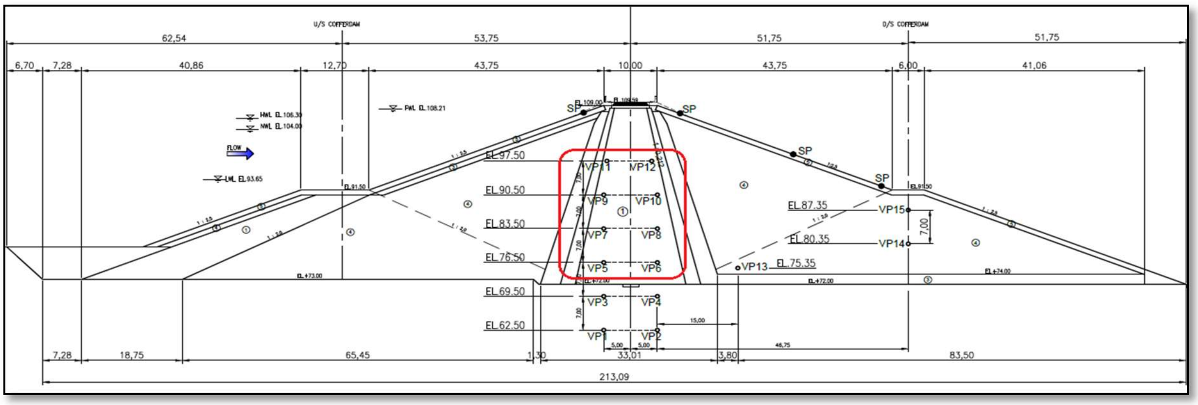

Fig 3. Installed vibrating wire piezometer at MD 16 cross section.

Fig 4, 5 and 6 shows plots time histories during construction, first impounding levels and piezometric heads during the 3 years after impounding.

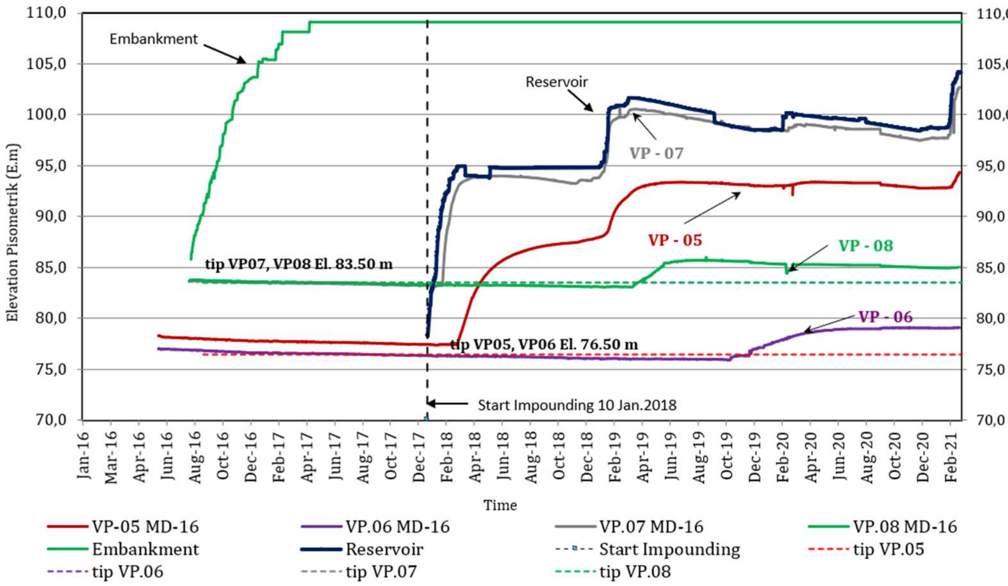


Fig 4. Piezometer reading of VP-05, VP-06, VP-07 and VP-08.

Fig 4 show time histories VP-05, VP-06, VP-07 and VP-08. Embankment started approximately on August 2016, same time with installed VP-5 and VP-6. Installed el. on 76.5 m, lowest part on clay core embankment. The embankment raising up until reaches el. 83.5 m, VP-07 and VP-08 were installed. Until el. 83.5m, VP-05 and VP-06 still no response regarding to raising the embankment. The overburden can't immediately be recorded by the instrument, probably because there is time-lag and the accuracy of the instrument itself, or some mistake when the instrumentation was installed. VP-05 and Vp-06 has response when first impounding started on January 10$^{th}$, 2018. During first filling, VP-05 responding very well, correlated with raising the reservoir level. VP-06 responding very well too, because of the location of VP-06 little bit downstream from VP-05 and close from downstream filter, the response quite different from VP-05. VP-06 started response at October 2019. VP-07 has higher reading from VP-05, reflect anomaly reading, probably caused by capacity of piezometer, the layer the piezometer was installed has moisture contents or the tip containing bubble air. Piezometer VP-5 reading should be > VP-07 reading,

After the reservoir at the steady level, approximately at el. 102 m, elevation level of VP-05, VP-06, VP-07 and VP-06 trend to decrease following time, the dissipation process happens and consolidation stages started.

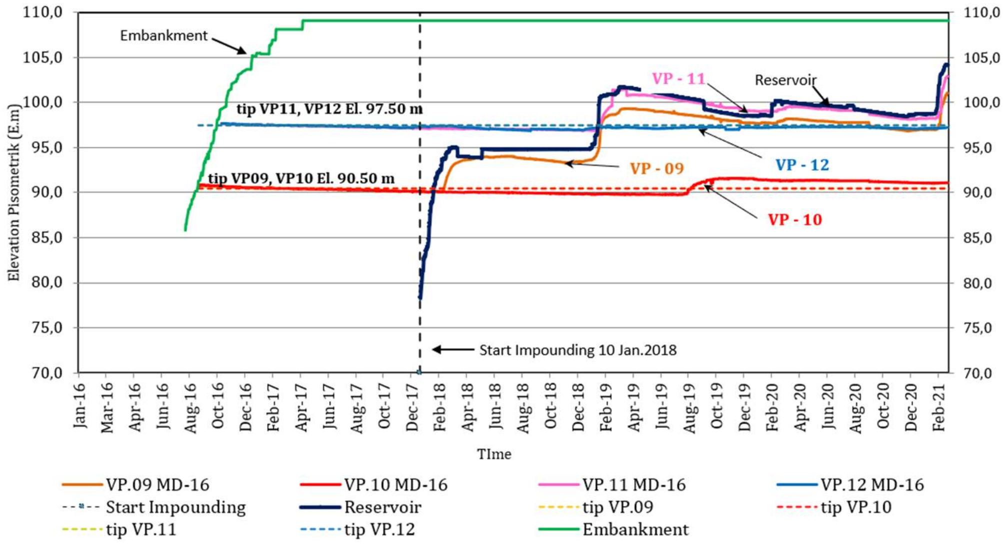


Fig 5. Piezometer reading of VP-09, VP-10, VP-11 and VP-12.

Fig 5 shows during the construction stages on VP-09, VP-10, VP-11 and VP-12 didn't response related to raising the embankment. VP-09 Soon after impounding was started, it's started to respond correspondent to reservoir level. These response same with other instruments below, reflected that the instruments doing good job to measure the pore pressure. VP-09 and VP-10 installed at el. 90.5 m, VP-11 and VP-12 installed at el. 97.5 m. VP-9 and VP11 has quick response regarding reservoir level, and VP-10 and VP 12 has slow response because its located downstream from VP-9 and VP-11.

For the same reasons, after consolidation, changes in piezometric heads are consistent with the change in the impounding level. As expected, the time lag is higher downstream and sensitivity to impounding changes is enhanced upstream.

At the end of the analysed period when the impounding level follows oscillating up to a nearly constant maximum value, stable piezometric heads seem to indicate the reaching of nearly steady state conditions, to be understood with respect to an average value of the impounding level.

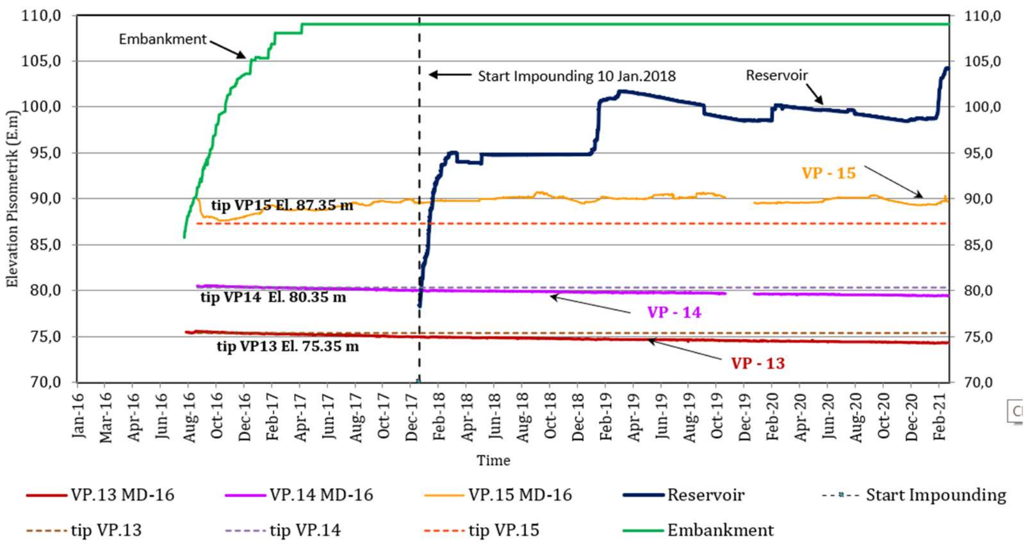


Fig 6. Piezometer reading of VP-13, VP-14, VP-15 and VP-16.

Because VP-13, VP-14, VP-15 and VP-16 installed higher elevation than other piezometer and close to crest of the dam, 10 m water head, until last monitoring (Feb 2021) still no significance response (Fig 6). This section should be placed lower than el. 80 m to read the phreatic line on random zone.

Table 3 and 4 shown result of stability analysis with OBE and MDE. Safety factors after construction has higher result than SF minimum rather than steady seepage with or without earthquake (OBE and MDE). This result representing that pore water pressure has influence on stability of embankment, with higher pore water pressure will be decreasing stability, opposite low water level will be increasing stability of embankment.

Table 3. Stability analysis with OBE

| No. | Condition | SF min | acceleration (g) | y/H | Safety Factor | |
|---|---|---|---|---|---|---|
| | | | | | US | DS |
| 1 | After construction, without earthquake, pore pressure 50% from soil weight. | 1.3 | - | - | 2.174 | 1.944 |
| 2 | After construction, with earthquake (50% OBE), pore pressure 50% from soil weight. | 1.2 | 0.100 | 0.25 | 2.077 | 1.899 |
| | | | | 0.50 | 1.738 | 1.589 |
| | | | | 0.75 | 1.977 | 1.623 |
| | | | | 1.00 | 1.730 | 1.570 |
| 3 | Steady flow, NHWL El. 104,00 m, without earthquake | 1.5 | - | - | 2.433 | 2.125 |
| 4 | Aliran langgeng (*steady flow*) normal height water level (NHWL) El. 104,00 m, with OBE | 1.2 | 0.200 | 0.25 | 1.516 | 1.498 |
| | | | | 0.50 | 1.275 | 1.377 |
| | | | | 0.75 | 1.344 | 1.431 |
| | | | | 1.00 | 1.237 | 1.416 |

Table 4. Stability analysis with MDE

| No. | Condition | SF min | Acceleration (g) | y/H | SF | |
|---|---|---|---|---|---|---|
| | | | | | US | DS |
| 1 | After construction, with earthquake (50% MDE), pore water pressure 50% soil weight | 1.00 | 0.300 | 0.25 | 1.293 | 1.130 |
| | | | | 0.50 | 1.211 | 1.049 |
| | | | | 0.75 | 1.394 | 1.089 |
| | | | | 1.00 | 1.287 | 1.098 |
| 2 | Steady flow, NHWL El. 104,00 m, with earthquake MDE | 1.00 | 0.600 | 0.25 | 0.686 | 0.632 |
| | | | | 0.50 | 0.612 | 0.664 |
| | | | | 0.75 | 0.636 | 0.708 |
| | | | | 1.00 | 0.550 | 0.725 |

## 4. Conclusions

Safety factors after construction has higher result than SF minimum rather than steady seepage. After construction condition, embankment has total stress, overburden pressure on embankment isn't reduced by pore water pressure ($u$), and factor of safety average between 1.7 – 2.0. Pore water pressure will be built up slowly regarding raising the reservoir, the embankment has effective stress ($\sigma_v - u$), factor of safety average between 1.2 – 1.5. With this result, pore water pressure has influence on stability of embankment, with higher pore water pressure will be decreasing the stability.

However, there is an anomaly in reading VP-07, who has higher reading from VP-05, probably caused by capacity of piezometer, the layer the piezometer was installed has moisture contents or the tip containing bubble air. Piezometer VP-5 reading should be > VP-07. This condition must be check very carefully and comprehensive later on.

Based on the calculation results of slope stability analysis, it can be concluded that the Raknamo Dam is safe against landslide hazards on the upstream and downstream slopes at normal water levels, flood water levels and in OBE conditions.

## 5. Recommendation

The calibration constants for piezometer should be checked from installation details (if available) and necessary correction should be done accordingly. Continuous monitoring should be done regularly, visual observation and instrument analysis to checking dam safety of Raknamo dam. On MDE if SF < 1, still we need to calculate permanent deformation which is proposed by Makdisi & Seed, 1977.

## 6. References


Caturbina Guna Persada (2017). *Laporan Akhir Pelaksanaan Konstruksi Bendungan Raknamo*. Balai Wilayah Sungai NT II. Ministry of Public Works and Housing. Not published.

Fontanella E., Pagano, L., Desideri, A., (2012). *Actual and Nominal Pore Water Pressure Distribution in Earth Dams*. EJGE Vol. 17

Foster, M., Fell, R., Spannagle, M., 2000. *The statistic of embankment dam failures and accident*. Journal Canada Geotech.

Geoslope (2012). *Seepage Modeling With Seep/W*. Geoslope International Ltd. Canada

Indonesian National Standard. (2016). *Standar Nasional Indonesia No. 8064:2016. Metode Analisis Stabilitas Lereng Statik Bendungan Tipe Urugan*

Olwage, H.J.W., and Oosthuizen, C. 1984. *Lessons learnt from South African dam incidents. In Safety of dams*. Edited by J.L. Serafim. A.A. Balkema, Rotterdam, The Netherlands, pp. 33–39.

USCOLD Committee on Dam Safety. 1996. *Anthology of dam modification case histories. United States Committee on Large Dams (USCOLD)*, Committee on Dam Safety, Denver, Colo.

US Army Corps of Engineers. (2004). *EM 1110-2-2300. General Design and Construction Considerations for Earth and Rock-Fill Dams*, Washington. 2-3-6 p.

US Army Corps of Engineers., (1993). EM 1110-2-1901. *Seepage Analysis and Control For Dams*, Washington. 4-21-24; 6-1-5; 8-1-10 p.

US Army Corps of Engineers., (1995). EM 1110-2-1908. *Instrumentation of Embankment Dam and Levees*. Washington.

Rancang Mandiri (2020). *Laporan Pendampingan Izin Operasi Bendungan Raknamo*, Balai Wilayah Sungai NT II. Ministry of Public Works and Housing. Not published.