Geotextiles and Geomembranes 46 (2018) 52–65

Contents lists available at ScienceDirect

Geotextiles and Geomembranes

journal homepage: www.elsevier.com/locate/geotexmem

Failure analysis of a geomembrane lined reservoir embankment

Riya Bhowmik*, J.T. Shahu, Manoj Datta

Department of Civil Engineering, IIT Delhi, Hauz Khas, New Delhi 110 016, India

ARTICLE INFO

Keywords:
Geosynthetics
Embankment
Failure
Geomembrane
Liners
Slope stability
Piping analysis

ABSTRACT

This paper presents case study and failure analysis of an embankment enclosing a raw water reservoir at a coal-based thermal power plant. The embankments and the base of the reservoir were all lined with geomembrane. Major breaches occurred in the embankment separating two compartments of the reservoir (i.e., the partition embankment) approximately one year after the filling of one of the reservoirs. Seepage and slope stability analyses were carried out to detect the causes of failure. The post–failure field observations and results of stability analyses indicated that the use of a single layer geomembrane as the sole component of barrier layer was inadequate. Pipe drains provided at the base of the reservoir to intercept rising groundwater level acted as a flow pathway for water seeping from tears and punctures in geomembrane liner at the base of the reservoir. The design of internal drainage system for both the partition embankment and peripheral embankment (i.e., the embankments other than the partition embankment surrounding the reservoir) was insufficient. The remedial measures which could be adopted for geosynthetic lined reservoir and embankment were evaluated and presented in the paper. The study highlights the need to provide a secondary liner in form of clay or geosynthetic clay liner whenever a geomembrane is used as a barrier layer. In cases where use of single layer of geomembrane is unavoidable, seepage and safety analysis should be carried out with the assumption that it may leak. This is important when an adequate quality control in laying the geomembrane is lacking or the embankment facilities would continue to be operated at full head even after the design life of the geomembrane is exceeded.

1. Introduction

According to ICOLD (2011), small dams are defined as “dams with height in the range of 5–15 m, where the product of square of the height (H, in m) and square root of the storage volume (V, in million m^3) is less than 200 ($H^2 \times \sqrt{V} < 200$)”. Geomembranes are used as liners for small dams at sites where impervious material is not easily available. The present paper describes a case study where geomembrane liner was used in small dam and failure occurred as the geomembrane liner was not properly integrated with the design of the dam.

In Indo–Gangetic plain which encompasses most of northern and eastern India, the soil deposits consist of uniformly graded fine sand or silty sand few hundred meters below ground surface. Impervious clay is, thus, not easily available. Geomembranes or geosynthetic clay liners usually provide the most cost effective solution as a barrier layer (ICOLD, 2010). However, for small dams and embankments, internal drainage system is not given due importance (refer Sowers and Sally, 1962) and this general negligence is also extended to geomembrane lined small embankments and reservoirs. This may lead to serious flaws in design, giving rise to failure of the embankment. Pisaniello et al. (2015) highlights the lack of adequate safety assurance and management practices for small dams in developing countries through a case study of 22 small dams in Vietnam. According to ICOLD (2011), although the overall failure in small dams is estimated at 2% only, total number of victims has been ten times higher than for failures of very high dams. The risk is higher for small dams as a consequence of poor care usually taken during the design, construction and maintenance of such dams.

Case studies of failures of geomembrane–lined dams and failures of dams due to internal erosion have been reported and reviewed in the literature. Girard et al. (1990) presented an example of partial slip of PVC geomembrane on the upstream slope of Aubrac dam, which illustrated the difficulties in laying an upstream geomembrane protective facing. Messerklinger (2014) presented a failure case history of internal erosion through geomembrane–lined embankment. The root-cause analysis showed that the failure of the embankment was caused when water from the reservoir entered through an open joint in the adjacent concrete structure, and the seepage water eroded embankment material from below the geomembrane. The geomembrane ruptured due to continuing seepage and erosion of embankment material, and this resulted in the full reservoir head being applied to the embankment fill material. This increased the seepage through the embankment fill

* Corresponding author.
E-mail addresses: riyabhowmik89@gmail.com (R. Bhowmik), shahu@civil.iitd.ac.in (J.T. Shahu), mdatta@civil.iitd.ac.in (M. Datta).

http://dx.doi.org/10.1016/j.geotexmem.2017.10.005
Received 17 May 2016; Received in revised form 7 September 2017; Accepted 18 October 2017

which ultimately lead its failure. Foster et al. (2000) conducted statistical analyses of structural failures of large dams, and found that 40% of the failures were due to piping through body of the dams. For homogeneous earth–fill dams, internal erosion constituted 50% of the failures. Fell et al. (2003) showed that the time for potential development of piping is short in earth-fill dams having poor internal erosion and seepage control measures. They also described the process of failure due to internal erosion as a result of following sequential events: (i) initiation of erosion, (ii) continuation of erosion, (iii) progression to form pipes and (iv) formation of breach. Richards and Reddy (2007) compiled the data from case histories of piping failure of dams, with the objective of categorizing different modes of piping. They concluded from the comprehensive review that very few studies were conducted on piping through cohesionless soil, and thus very few advances have been made in this area.

The design of an embankment becomes complicated and chances of failure increase if the embankment serves as the partition embankment between twin reservoirs; the reasons being, (a) toe drain and rock toe cannot be provided on the downstream side because the other compartment of the reservoir would be present on the downstream side which requires liner on the downstream slope, and (b) it is also problematic to provide internal drainage system because the collection and eviction of the collected water in the sump of the internal drain become difficult and internal drains cannot discharge on the downstream side as that will provide a direct connection between the other compartment of the reservoir and the inside of the embankment.

There is a growing trend of the use of geomembrane as liner in small reservoirs (Xue-shan et al., 2015). Also, there is general negligence towards adequate provision of internal drainage system in small dams and embankments. This paper presents a case study of failure of an 8 m high embankment enclosing a raw water reservoir at a thermal power plant in India. The failed embankment was designed as the partition embankment of twin reservoirs. The embankments as well as the reservoir were all lined with geomembrane only. Two major breaches occurred in the partition embankment approximately one year after the filling of one of the reservoirs. To detect the causes of failure, seepage and slope stability analyses were conducted. The lessons learnt from this study are discussed and the design modifications for a geosynthetic lined reservoir and embankment are presented.

2. Project description

2.1. Twin reservoirs

A coal-based thermal power plant was set up in the state of Punjab in India and the first unit was commissioned in November 2013. The thermal power plant has 500,000 m^2 for raw water reservoir with storage capacity of 4,154,000 m^3 in two adjacent reservoirs. The twin reservoirs, RWR-I and RWR-II, are separated by a partition embankment. Fig. 1 shows the plan view of the twin reservoirs. At the time of failure, RWR–I had storage capacity of 2,077,800 m^3 and was used as raw water reservoir, whereas RWR–II was partly constructed; while the body of the peripheral embankments around RWR-II was constructed, upstream and downstream slope protection measures, liner and drainage system of the embankments, and liner at the base of the reservoir were not constructed.

2.2. Site investigation

Based on borehole data, the soil profile at the site was as follows: top 2–4 m thick layer consisted of loose to medium dense (bulk unit weight, $\gamma = 18\ kN/m^3$), greyish brown, silty sand (SM) with an average SPT-N value of 10. This stratum was followed by medium dense to very dense (bulk unit weight, $\gamma = 20\ kN/m^3$), brownish grey, silty sand (SM) up to the maximum depth of boring of 50 m. The average N value for the lower strata varied from 10 at a depth of 2 m to 70 at a depth of 20 m. Water level was encountered at an average depth of 7 m. Fig. 2 shows the logs of boreholes of different depths (20, 40 and 50 m) located in and around the reservoir as shown in Fig. 1. Drained triaxial tests on saturated specimens reconstituted at field density gave cohesion c' equal to 0 kPa and angle of shearing resistance ϕ' varying from 28° to 38° depending upon the depth of sampling from 0 m to 18 m. The values of angle of shearing resistance deduced from SPT-N values (IS 6403:1981) are shown in Table 1.

2.3. Embankments

RWR-I is enclosed by embankments on all four sides, namely, a partition embankment between RWR–I and RWR–II, and peripheral embankments on the other three sides (Fig. 2). The partition embankment is 8 m high on both upstream side and downstream side. The peripheral embankments are 8 m high on the upstream side, but the height varies from 5 m to 8 m on the downstream side due to varying ground level. Figs. 3 and 4 show the cross-section of the partition embankment and peripheral embankment, respectively. Both the embankments have an upstream slope of 2H:1V and a downstream slope of 2.5H:1V. Both upstream and downstream slopes are provided with 1 m wide berm at 5 m below the crest of the embankment. The embankments were constructed with the soil excavated from the site, which was compacted to the optimum moisture content of 12%, and at 98% of Proctor density.

2.4. Liner

The upstream slope of embankments and the reservoir base were constructed with a barrier layer (liner) to prevent infiltration of water from the reservoir into the embankment and surrounding soil. The liner on the upstream slope of embankment consisted of 1 mm thick smooth HDPE geomembrane overlain with consecutive layers of 12 mm thick cement mortar and 50 mm thick precast cement tiles (Fig. 5a). The liner in reservoir bed consisted of 1 mm thick smooth HDPE geomembrane overlain with 300 mm thick soil cover (Fig. 5b).

2.5. Downstream slope protection

The downstream slope of embankments were protected by two layers of 100 mm thick graded and compacted filter material overlain by a single layer of 300 mm thick stone-pitching (Figs. 3 and 4).

2.6. Internal drainage

The drainage system for the peripheral embankments comprised of a toe drain (Fig. 4). The internal drainage system was not provided (such as vertical drain, inclined drain, horizontal blanket drain, rock toe, etc.) for a number of reasons: First, it was assumed that the liners at the base of the reservoir and the upstream slope of the embankments would not leak, and thus seepage would not occur. Second, the embankments being in the category of 'small dams'; internal drainage system is considered not vital and thus, neglected in small dams. Third, the cost of bringing material for filter over a distance of 200 km for construction of internal drainage system was very high. Owing to the high cost, only nominal thicknesses (10 cm) of transition filter layers were provided on the downstream slope face, which were even less than the minimum specified by Indian Standard Specifications. Fourth, as the embankments were constructed with semi–pervious soil (silty sand), internal drainage was probably not considered necessary by the designers.

Due to presence of RWR-II, the partition embankment did not have a toe drain on the other side. The drainage was provided by a central drain parallel to the longitudinal axis of the embankment below the body of the embankment (Fig. 3). The central drain was trapezoidal in shape (0.3 m deep and 0.3 m wide at bottom), filled with gravel–sized