Back-analysis of a PVC geomembrane-lined pond failure

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ABSTRACT: This paper presents a case history of veneer slope failures in a non-potable water pond. The paper includes a description of slope construction and post-construction movements, field and laboratory investigations, and stability analyses used to evaluate the cause of the slides. The pond lining system was designed at a 3H : 1V slope and consists from bottom to top of native subgrade, a 0.75 mm thick poly (vinyl chloride) geomembrane, a 270 g/m^2 non-woven geotextile, and a 0.3 m thick layer of compacted soil cover over the geotextile. This paper discusses the back-analysis of the slope failures and presents recommendations for back-analyses of other slides involving geosynthetic-lined slopes.

KEYWORDS: Geosynthetics, Poly (vinyl chloride) geomembrane, Interface strengths, Direct shear test, Shear strength, Slope stability, Water pressures

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1. INTRODUCTION

This project involved an innovative water supply system for a new housing development near Greeley, Colorado. The system consists of a non-potable irrigation pond, a vertical turbine pump station with an automatic self-cleaning filtration system, and a distribution pipeline system. The non-potable irrigation pond is filled with water from an adjacent agricultural irrigation canal that distributes non-potable water to various non-potable storage ponds and downstream agricultural users. The non-potable water is used for irrigation throughout many large subdivisions, parks, golf courses, and commercial developments and thus saves the cost of water treatment for this application. Water flows by gravity from a 'turnout structure' in the canal to the storage pond and then is filtered and pumped to the distribution pipeline system. Figure 1 shows the pond and pumping station building before the slope failures. The distribution canal is located behind the fence at the top of the photograph in Figure 1(a) and the fence on the right side of the photograph in Figure 1(b). The irrigation distribution canal adjacent to this project has a concrete lining and an underdrain system. The canal is located just north of the detention pond. The flowline of the irrigation canal is approximately 2.4 m above the normal water surface elevation of the non-potable storage pond. The underdrain system collects the seepage water from the irrigation canal and discharges it through a 150 mm diameter poly (vinyl chloride) (PVC) pipe into the non-potable storage pond. The housing development and the distribution pipeline are located south and east of the non-potable storage pond.

The storage pond was excavated into native soil materials. The original design specified side slopes for the geosynthetic-lined portion of the pond of 3H : 1V. The pond was excavated by enlarging an existing shallow stormwater detention pond that was approximately 1 m deep. The total depth of excavation for the new pond was approximately 5 m below the existing grade. The pond is roughly 65 m across and 140 m long. The new storage pond did not have stormwater detention requirements and stormwater was diverted through a 0.9 m reinforced concrete pipe to another location in the project.

The pond liner system consists from bottom to top of native subgrade, a 0.75 mm thick PVC geomembrane, a

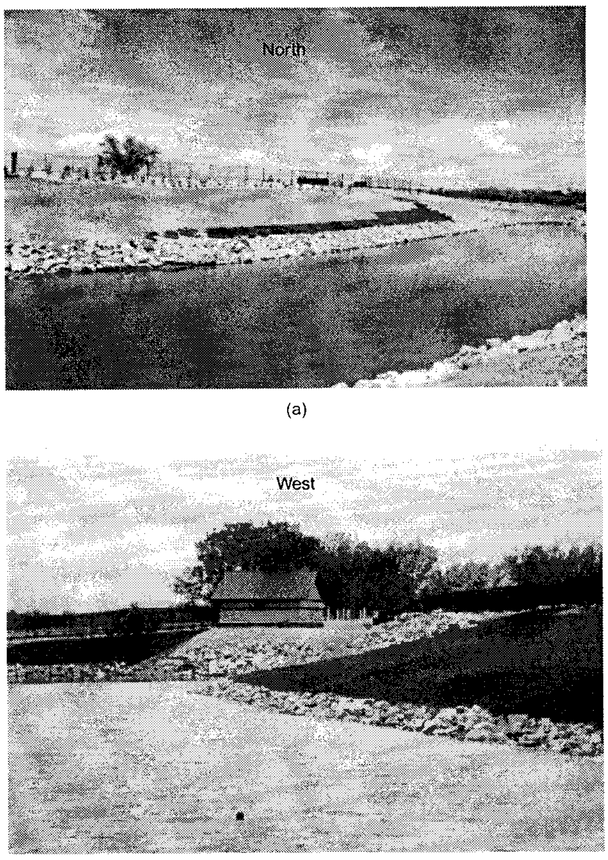


Figure 1. Water storage pond and pump station (a) during and (b) after construction

polypropylene 270 g/m^2 non-woven, needle-punched geotextile, and a 0.3 m thick layer of compacted soil cover over the geotextile. The geomembrane was faille-finished on one side and smooth on the other, although no guidance was given as to which side should be installed against the geotextile and which side against the subgrade. The faille-finished surface of the PVC geomembrane is embossed with a pattern like that of a file. The pond was lined with a 0.3 m thick ring of riprap spanning 2.6 m of elevation at the level of wave action to reduce erosion (see Figure 1(a)). The riprap specification called for a mean diameter of 0.3 m. The geosynthetics were anchored in a 0.3 m by 0.3 m anchor trench along the side slope. The 0.3 m of cover soil was to be placed over the geosynthetics but no guidance was provided on how to place the cover soil, for example, bottom to the top of the slope, and the excavation contractor had not encountered geosynthetics on any prior project. This led to the contractor not placing the cover soil from the bottom to the top of the slope.

Design and construction of the pond occurred in 2004 and 2005, which corresponded to a recent period of drought in Colorado. The pond construction, including soil cover placement, was completed on 2 February 2005 with riprap installation completed on 7 February 2005. Initial filling of the pond began on 11 March 2005. Pond filling continued until 14 March 2005, when the pond was approximately 70% full. By 6 May 2005, the pond water level was increased to 90% full and operation of the pond pump system began. By 17 May 2005, the pond was 100% full. The first observed slope failure occurred on 4 to 5 June 2005 after an intense 75 mm rainfall event.

Figures 2(a) and (b) provide some insight to the construction procedures employed by the excavation contractor. Figure 2(a) shows the earthwork contractor constructing an access road down a side slope to gain access to the bottom of the pond. Figure 2(b) shows the tracked-excavator on the south side of the pond, namely the side closest to the residences and furthest from the distribution canal, placing cover soil well ahead of the access road. This resulted in a substantial amount of cover soil being placed and spread in an unsupported manner. As discussed subsequently, it is hypothesised that this unsupported load induced shear stresses along the geotextile/PVC geomembrane interface, which may have resulted in mobilisation of a post-peak interface strength. It is frequently recommended that cover soil be placed from the bottom to the top of a slope so the soil buttresses itself and reduces the

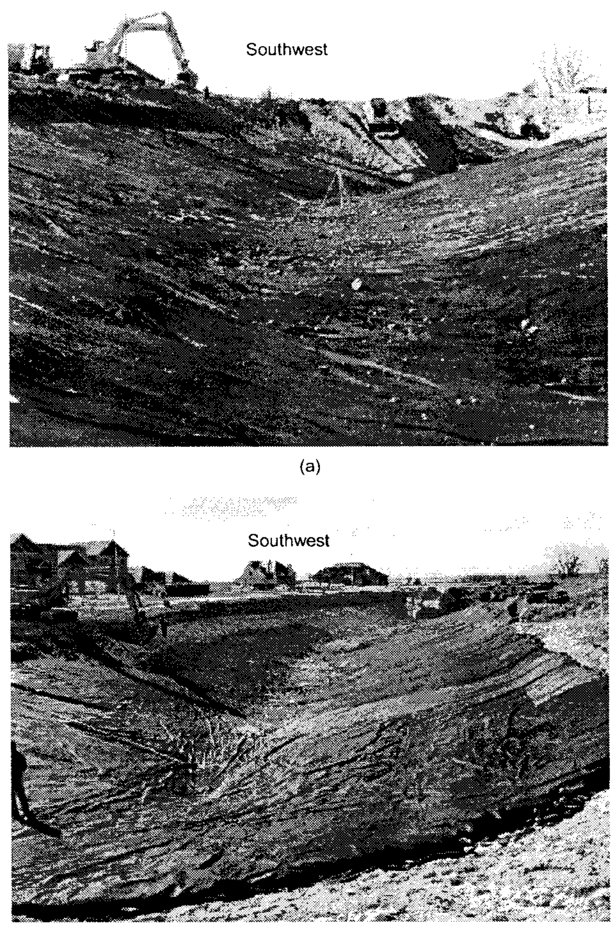


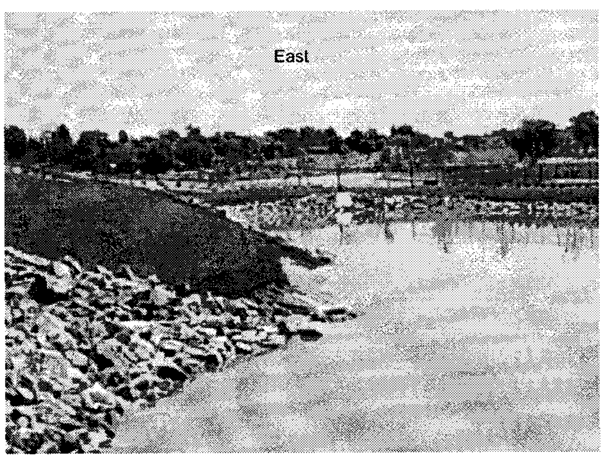
Figure 2. (a) Placement of unsupported cover material while access ramp to pond bottom is being constructed, and (b) unsupported cover material placed well in advance of buttressing soil

shear stress and shear displacement applied to underlying geosynthetic interfaces (Koerner and Soong 2005).

2. DESCRIPTION OF SLIDES AND FAILURE MECHANISM

The initial slope failure occurred on 4 or 5 June 2005, which was about 4 months after construction was complete and 3 months after pond filling. Figure 3(a) shows the initial slope failure on 4 June 2005, which is the area of the photograph where the riprap is missing. The following important observations can be made from this photograph.

1. No pond drawdown or other detrimental operation event occurred prior to the first observed slope failure on 4 or 5 June 2005.
2. The newly placed sod, riprap, soil cover, and cushion geotextile slid below the water line, leaving the PVC geomembrane exposed on the slope.
3. This failure appeared to be isolated because at the time of the photo the surrounding slopes had not failed and signs of potential failure such as tension cracks were not observed.
4. This failure continued to progress and expand over the next few days. Expansion of the slide was augmented by the drawing down of the pond level after the initial failure was observed. Figure 3(b) shows the initial slide 2 days later on 6 June 2005.



(a)

(b)

Figure 3. (a) Initial slope failure with pond level at or near capacity on 4 June 2005, and (b) slope failure on 6 June 2005

Prior to the slope failure, the new sod had been heavily irrigated. In addition, from 3 to 5 June 2005, the site received approximately 75 mm of rainfall. The City Engineer reported that during and after the failure, the 150 mm diameter pipe from the concrete lined irrigation canal underdrain system was flowing completely full and discharging into the storage pond. During normal operation of the irrigation canal, the underdrain pipe flowed approximately one-third to one-half full into the storage pond.

Figure 4 shows the initiation of a smaller failure on the south side of the pond. This photo was taken on 6 June 2005 and thus this slide movement occurred after the initial slide movement on the north slope shown in Figure 3. The slide in Figure 4 also involved the new sod, riprap, soil cover, and geotextile sliding off the PVC geomembrane.

Figure 5(a) shows the failure on 8 June 2005 with the pond level drawn down about 75%. As the slide progressed, small holes started to appear in the PVC geomembrane because the geomembrane was in extreme tension. As the slide movement continued, the small holes enlarged on the left side of the slide mass shown in Figure 5(a). In Figure 5(a), however, the PVC geomembrane remained in place on the slope and thus the critical interface for sliding was the geotextile/PVC geomembrane interface.

Figure 5(b) shows the slide mass after the pond had been drained. This continued drawdown and slide progression resulted in the geomembrane being torn across the initial slide. Without having the photograph of the initial slide, someone investigating the post-failure condition

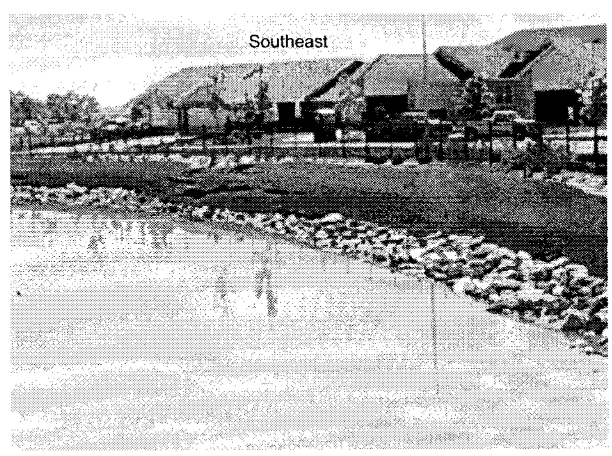
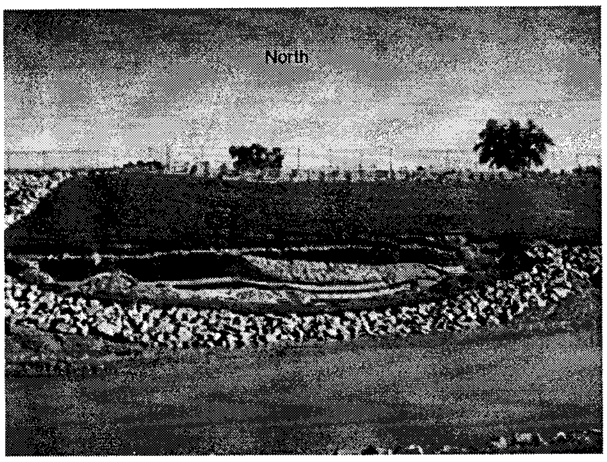
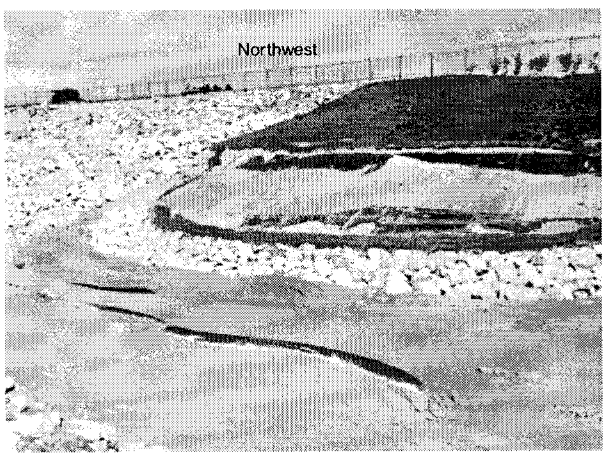


Figure 4. Slope failure on south side of pond near houses



(a)



(b)

Figure 5. (a) Progression of slope failure as pond level is drawn down, which results in geomembrane tearing, and (b) tearing of geomembrane across slide after pond level is drawn down

might think sliding occurred on the PVC geomembrane/soil interface as shown, which was not the case.

Figure 5(b) also shows another initially confusing aspect of the failure, which was the development of bulges or bubbles in the PVC geomembrane from excess hydrostatic pressure underneath the geomembrane at the base of the pond. Without having the initial photograph, it might be postulated that sliding was caused by hydraulic pressure under the geomembrane and failure of the geomembrane subsequently occurred in tension as shown in Figure 5(b). This hypothesis could be supported by the fact that groundwater was encountered on the north slope during construction and the observation that the 150 mm underdrain pipe system was flowing completely full at the time of failure, possibly indicating that groundwater was accumulating from the agricultural irrigation at a rate exceeding the capacity of the underdrain system. Figure 6 presents a plan view of the pond that shows the location of the slope failure and the bulges or bubbles under the geomembrane found after draw down of the pond water.

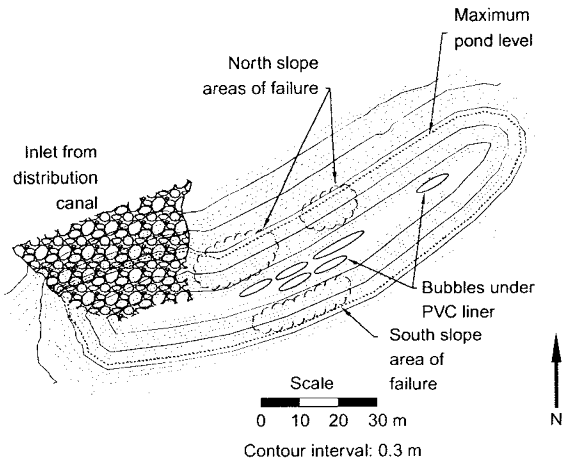


Figure 6. Plan view showing location of failures and bubbles under the geomembrane

3. SLIDE INVESTIGATION

3.1. Subsurface Investigation

No site-specific borings were performed for the irrigation pond design, which was separate from the residential development. A geotechnical investigation for the adjacent residential development was, however, conducted prior to pond construction. The investigation consisted of shallow borings (three of which were in the vicinity of the pond) and limited laboratory testing. The borings near the pond indicated that the pond is probably underlain by clayey sand of alluvial origin and a claystone formation with sandstone interbedded. The thickness of the clayey sand is 3 to 6 m at the test boring locations. No groundwater was noted in these borings because they were drilled in the year 2000 during the initial stages of a drought. Other borings drilled for the residential development show the presence of groundwater at depths of 2.4 to 4.3 m.

After the slope failures occurred, four piezometers were installed to monitor groundwater levels. Three of the piezometers were installed north of the irrigation distribution canal; that is, on the other side of the canal from the pond, and one was installed south of the pond. These peizometers indicated groundwater at a depth of 4.6 to 7.0 m below the ground surface on the north of the canal and at 5.8 m below the ground surface south of the pond.

3.2. Site observations

The following site observations made by the first and third authors during site visits supplement the observations presented previously in the photographs.:

1. In the slope areas that failed, the geotextile had pulled out of the anchor trench and not torn. The exception was an area where the geotextile tore at a seam.
2. The PVC geomembrane remained anchored in the anchor trench.
3. The slope of the soil below the level of the riprap was surveyed to be 2.5H : 1V in the vicinity of the

initial slide. This was steeper than the design slope inclination of 3.0H : 1V. This survey of 2.5H : 1V probably underestimated the inclination of the excavated native subgrade because the cover soil below the riprap had already moved to the bottom of the pond creating a flatter slope. It is estimated that the inclination of the excavated native subgrade was between 2.5H : 1V and 2.0H : 1V.

4. The slope of the soil cover on the south slope below the level of the riprap was surveyed to be 3.0H : 1V. Thus, the south slope appeared to be flatter than the north slope, which correlated with slope failures initially occurring on the north slope and more and larger failures occurring on the north slope than the south slope.
5. The thickness of the riprap was at least 0.5 m, which is larger than the 0.3 m indicated in the design documents. This resulted in shear stresses that were greater than the design values being applied to the PVC geomembrane interface.
6. A filter geotextile was not placed below the riprap to reduce the potential of erosion of the cover soil.
7. The non-woven cushion geotextile was overlain by 0.3 m of cover soil as required by the design.
8. Several bulges were observed in the geomembrane at or near the bottom of the pond.
9. One of the liner bulges was pierced with a knife by another investigator in August 2005 and clear water was observed beneath the liner.
10. On a portion of the north slope exposed by a slope failure, seepage was observed by another investigator to be flowing out of the native subgrade in August 2005.

3.3. Possible failure mechanisms

Following the review of the available data, an on-site investigation by the first and third authors, and a meeting with the engineer of record, the following failure mechanisms were considered in the causation analysis.

1. Sliding along the PVC geomembrane/subgrade interface with or without underlying hydraulic pressures.
2. Bulging of the geomembrane by hydraulic pressure causing the geomembrane to stretch and eventually tear.
3. Sliding along the geotextile/PVC geomembrane interface.

The investigation and analyses presented subsequently attempt to identify which of these failure mechanisms caused the initial slide observed on 6 June 2005.

It appears that the hydraulic pressures developed under the geomembrane after the initial slide and the subsequent drawdown of the pond. Thus, sliding along the geomembrane/subgrade interface might have occurred without hydraulic pressures. However, failure along the geomembrane/subgrade interface without hydraulic pressures is not a plausible failure mechanism because the initial sliding occurred on the geotextile/PVC geomembrane, as shown in Figure 3(a) and (b). Thus, the geomembrane/subgrade interface was not tested during the laboratory shear test programme.

A complicating aspect of this failure was the post-failure presence of hydraulic pressures below, and bulging of, the geomembrane. The bulging and stretching of the geomembrane induced by the underlying hydraulic pressures (and associated loss of interface shear resistance) caused the geomembrane to eventually tear and/or move along the geomembrane/subgrade interface. This mechanism might provide a plausible explanation for the initial failure if not for the fact that the initial movement was observed to have occurred along the geotextile/PVC geomembrane. The hydraulic pressures also did not influence the geotextile/PVC geomembrane interface because the interface was isolated from the hydraulic pressure below the geomembrane by the presence of the geomembrane. Thus, the hydraulic pressure did not cause or trigger the slide along the geotextile/PVC geomembrane interface. However, if the initial movement had occurred on the geomembrane/subgrade interface, hydraulic pressure could/would have been a contributing or primary factor to the slide.

Based on Figure 3(a) and (b), the most plausible failure mechanism is sliding along the geotextile/PVC geomembrane interface and this was the focus of the back-analysis described subsequently.

4. GEOSYNTHETIC INTERFACE SAMPLING AND TESTING

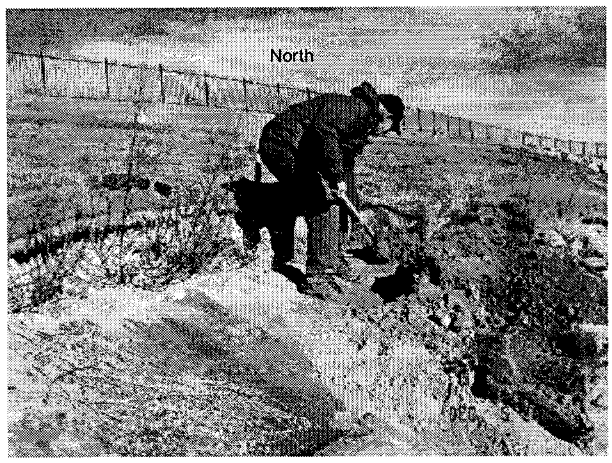
During the design process, no geosynthetic–geosynthetic or geosynthetic–soil interface tests were performed for the pond liner design. Interface tests were not performed during design or before construction because the design had been successfully used previously. Veneer instability has, however, been observed with other PVC geomembrane installations (Amaya *et al.* 2006). PVC geomembranes are less variable than textured polyethylene geomembranes because PVC goemembranes rely on their softness and pliability for interface shear resistance and not the aggressiveness of the texturing (Hillman and Stark 2001). Thus, interface testing may not appear warranted for PVC geomembranes. The softness and pliability of PVC geomembranes is fairly uniform because of the requirements of the PVC Geomembrane Institute material specification PGI 1104 (PVC Geomembrane Institute 2004). There can, however, be a difference in the shear resistance of the faille finish in comparison with the smooth finish. The smooth finish usually yields a higher interface shear resistance than the faille finish (Hillman and Stark 2001). Thus, the design should specify which finish is facing which interface so the critical interface utilises the smooth side of the PVC geomembrane. If both sides of the PVC geomembrane utilise a smooth finish or a faille finish, the placement arrangement is irrelevant. Thus, site-specific interface testing is important even for PVC geomembranes because the finish of the geomembrane can vary, albeit to a lesser extent than polyethylene geomembrane texturing.

The difference in PVC geomembrane finish is also important in the investigation of slope failures. For example, another investigator conducted direct shear interface tests according to ASTM D 5321 on the PVC geomembrane/geotextile interface using new samples of the geomembrane and geotextile that were thought to be representative of the installed geosynthetics. Thus, the geosynthetics actually installed were not tested and, more importantly, it was not reported whether or not the smooth or faille finish was in contact with the geotextile. In the field, the faille side was in contact with the overlying geotextile and the smooth side was in contact with the subgrade. To increase interface shear resistance, the smooth side of the geomembrane could have been specified to be in contact with the geotextile although the interface probably still would not have been stable because the slope inclination exceeded 2.5H : 1V.

Another factor influencing PVC geomembrane interface shear resistance is the surface texture of the non-woven geotextile. Some geotextiles are heat burnished, which can produce a rougher surface than other types of processing. In this particular case, the rougher side of the geotextile was in contact with the faille side of the PVC geomembrane.

On 5 December 2005, the first author travelled to the site with the third author to inspect the slope failures and obtain samples of the geomembrane and geotextile for interface testing. Samples of the geomembrane and geotextiles 1 m wide and 2.5 m long were hand excavated near the upper right portion of the slope failure, just downslope of the anchor trench as shown in Figures 7(a) and (b). The excavation of the geosynthetics revealed that the faille side of the PVC geomembrane was in contact with the rougher side of the geotextile.

Two direct shear interface tests were conducted according to ASTM D 5321 on the PVC geomembrane/geotextile interface at the University of Illinois. The resulting shear stress-horizontal displacement relationships are shown in Figure 8. These tests utilised normal stresses of 12.2 and 24.1 kPa to simulate the loading imposed by the cover soil and overlying riprap. Assuming an adhesion of zero because of the untextured finish of the PVC geomembrane, Figure 8(b) shows the linear failure envelope drawn through the peak shear resistance measured during these shear tests. The peak failure envelope corresponds to an interface friction angle (δ_P) of 28°. The large displacement failure envelope at 100 mm of shear displacement corresponded to an interface friction angle (δ_{100mm}) of 25°. The small post-peak interface strength loss of the PVC geomembrane was attributed to the large and intimate contact area between the geomembrane, the geomembrane not being textured and thus not damaging the geotextile (Stark *et al.* 1996), and the high flexibility/pliability of the PVC geomembrane (Hillman and Stark 2001).



(a)

(b)

Figure 7. Excavation and sampling of installed geosynthetics for laboratory testing on northern slope

5. BACK-ANALYSIS OF SLIDE AND CAUSATION

5.1. General

Back-analyses of slope stability case histories are usually performed using a two-dimensional (2-D) slope stability method, which does not account for three-dimensional (3-D) end or shear forces. These end effects increase stability, and thus 2-D back-analyses can yield unconservative estimates of the field shear strength because the end effects are incorporated in the back-calculated shear strength (Stark and Eid 1998). As the width of the initial slide mass in this case history ranged from 20 to 25 m and the depth of the slide surface was only 0.3 m, the 3-D effect for the initial slide was determined to be small. The depth of the slide surface, 0.3 m, corresponded to the nonwoven geotextile/PVC geomembrane interface. As a result, a 2-D analysis was used to back-calculate the slope inclination of the excavated native subgrade.

Back-analysis of the first slope failure was performed using the microcomputer program XSTABL Version 5 (Sharma 1996) and Janbu's stability method (Janbu 1973)as coded in XSTABL. Spencer's stability method (Spencer 1967) was not used for the back-analysis even though it satisfied all conditions of equilibrium and provided a better estimate of the factor of safety (Duncan and Wright 1980), because the abrupt entrance and exit of the failure surface from the geotextile/geomembrane interface resulted in numerical difficulties (Spencer 1968). In

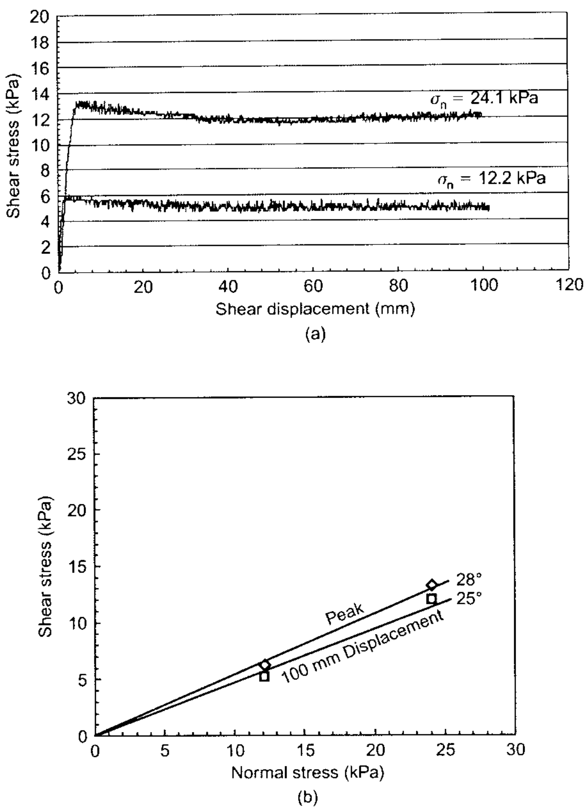


Figure 8. Non-woven geotextile/PVC geomembrane interface (a) shear test results and (b) peak and 100 mm displacement failure envelopes

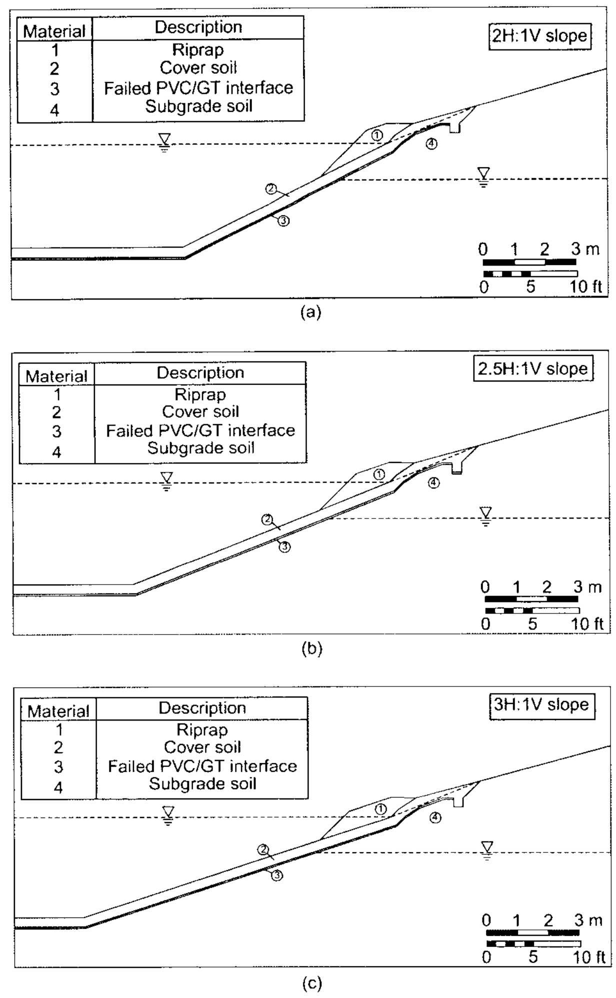


Figure 9. Cross-sections through slide on northern slope shown in Figure 6(a) with intermediate water level

the back-analysis, it was assumed that an FS less than or equal to unity indicated sliding.

As a result of uncertainties in the slope angle of the native subgrade, the slope of the native subgrade was varied from 3H : 1V to 2H : 1V to assess the effect of slope angle on the computed FS. The cross-sections used to depict the range of slope angles possible in the field, namely, 2H : 1V, 2.5H : 1V, and 3H : 1V, are shown in Figure 9. Observations of the north and south slope after the pond level was drawn down indicate that the failure mode was translational with the majority of the slip surface occurring along the geotextile/PVC geomembrane interface. The toe of the slide appeared to develop just below the riprap, and the top of the slide surface corresponded to the failure surface exiting the cover soil at the edge of the anchor trench.

Table 1 presents the input parameters used for the cover soil, the geotextile/PVC geomembrane interface, and the native subgrade for the stability analyses. The cover soil exhibits a total unit weight of 18.8 kN/m^3. The riprap was modelled using 14.4 kN/m^3. As noted previously, the peak effective stress shear strength parameters of the drainage sand were estimated to be a cohesion of zero and a friction angle of 32°. Based on the direct shear interface tests, the geotextile/geomembrane interface was characterised by peak and 100 mm displacement effective stress shear strength parameters of adhesion of 0 kPa and δ_P of 28° and δ_{100mm} of 25°, respectively.

5.2. Estimation of ground water level

The average head on a geomembrane in a cover layer can be calculated using the following equation

$$h_{avg} = \frac{k_c * L \cos\beta}{k_d * \sin\beta} \tag{1}$$

where k_c is the hydraulic conductivity of the cover soil, L is the length of the slope parallel to the slope, β is the slope angle, and k_d is the hydraulic conductivity of the drain (Soong and Koerner 1997). This calculation assumed that the cover soil was saturated, and thus any infiltrating water percolated through into the drain layer, and that the precipitation rate was greater than the hydraulic conductivity of the cover soil, so the rate of infiltration was controlled only by the hydraulic conductivity of the cover soil and not by the amount of precipitation falling on the slope.

To verify the infiltration assumption, the infiltration rate can be estimated. The hourly precipitation recorded at a National Climactic Data Center (NCDC) recording station in Greeley, CO, for the days leading up to the initial slope failure is shown in Figure 10. The data show that the peak

Table 1. Summary of input parameters for back-analysis of initial failure

Material	Unit weight (kN/m^3)	Friction angle (degrees)	Adhesion (kPa)	Reference
Riprap	21.2	45	0	Consultant report
Liner cover soil	18.9	30	0	Consultant report
Failed PVC/GT interface	N/A	28	0	Laboratory tests on exhumed materials
Subgrade soil	18.9	30	0	Consultant report

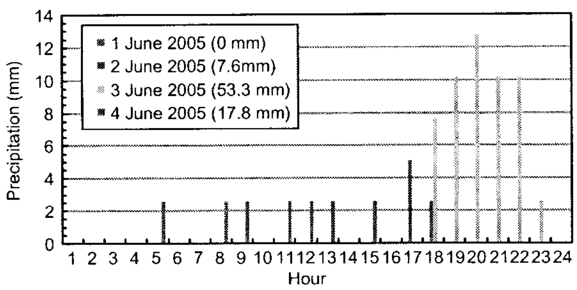


Figure 10. Hourly precipitation from NCDC site in Greeley, CO, for the days preceding the initial instability. Total precipitation for each day is listed in the legend

precipitation rate on 3 June 2005 was 12.7 mm/h (3.5×10^{-4} cm/s) and the 6-h average was 8.9 mm/h (2.5×10^{-4} cm/s). For comparison, the 100-year, 6-h design storm for the site is 14.5 mm/h (4.0×10^{-2} cm/s) (Miller *et al.* 1973). To verify the infiltration rate was controlled by the hydraulic conductivity of the cover soil, the potential infiltration from precipitation was estimated as $P(1 - RC)$, where P is the precipitation rate and RC is a Soil Conservation Society (SCS) runoff curve number. Using the 6-h average recorded precipitation rate of 2.5×10^{-4} cm/s and a runoff curve number of 0.40 corresponding to a fair stand of grass (Qian *et al.* 2002), the potential infiltration rate is 1.5×10^{-4} cm/s. This rate is greater than the hydraulic conductivity of a silty clay loam or a silty loam (4.2×10^{-5} cm/s or 1.9×10^{-4} cm/s, respectively (Qian *et al.* 2002)). Thus, the hydraulic conductivity of the cover soil probably controlled the infiltration rate.

Using $k_c = 4.2 \times 10^{-5}$ cm/s for a silty clay loam, $k_d = 10^{-2}$ cm/s for the sand overlaying the geomembrane (Qian *et al.* 2002), $\beta = 22°$, and a slope length of 2 m (corresponding to the portion of the slope between the pond level and the upper end of the geomembrane), the average head on the geomembrane was 21 mm. If the cover is more permeable than a silty loam and precipitation controls the infiltration rate, then the estimated infiltration rate from the precipitation replaces the cover soil hydraulic conductivity in the calculation, yielding an average head on the geomembrane of 75 mm. This calculation does not consider the effects of the drainage boundary conditions at the toe of the slope, which were likely to be significant for a slope only 2 m in length. Thus, an estimate of 21–75 mm of hydraulic head on the geomembrane may be low.

To account for the uncertainty in the hydraulic head, three different piezometric surfaces in the cover soil above the pond level were used to capture the effect of the 75 mm rainfall event. The lowest piezometric level corresponded to the pond level and the geotextile/geomembrane interface above the pond level. The intermediate piezometric level is shown in Figure 9 and corresponded to the pond level and a minimum of 75 mm above the geotextile/geomembrane interface above the pond level. The highest piezometric level corresponded to the pond level and top of the cover soil above the pond level and represented the worst case scenario. It is doubtful that the piezometric level in the cover reached to the top of the cover soil, and thus the intermediate water level probably provided the best representation of the seepage conditions in the cover soil above the pond level. The intermediate pond level also provided the closest agreement with field observations of the failure as shown subsequently.

5.3. Back-analysis of initial slide

The factors of safety for the various slope angles and piezometric levels considered and a peak interface strength for the geotextile/PVC geomembrane interface are shown in Table 2. The data show that the best agreement with field observations was the intermediate water level and a slope inclination of 2H : 1V. This is possible, even though the measured slope inclination was 2.5H : 1V after the slide; that is, it corresponds to the post-failure inclination of the cover soil from immediately

Table 2. Factors of safety for various slope angles and cover soil water levels using the peak geotextile/PVC geomembrane interface friction angle (28°)

Slope	High piezometric level	Intermediate piezometric level	Low piezometric level
3H : 1V	1.12	1.21	1.25
2.5H : 1V	1.06	1.14	1.17
2H : 1V	0.91	0.97	1.00

downslope of the riprap in the vicinity of the initial slide. This survey of the cover soil probably underestimated the inclination of the excavated native subgrade because the cover soil below the riprap had already moved to the bottom of the pond creating a flatter, more stable configuration for the slope. The results in Table 2 do not, however, account for the placement of the cover soil in an unbuttressed manner or the excavated native soil being steeper than the cover soil below the riprap after sliding.

Stark and Choi (2004) showed that a large displacement shear strength is less likely to develop in a veneer stability situation than a composite liner situation because detrimental shear displacements are less likely to occur in a veneer situation. Dixon *et al.* (2006) used numerical analyses to confirm that residual interface strengths can develop on landfill liner sideslopes because detrimental shear displacements usually occur as suggested by Stark and Poeppel (1994). Detrimental shear displacements are less likely for the veneer situation because of the presence of low shear stresses, low normal stresses (which limits detrimental, that is, damage-inducing, shear displacements to a geosynthetic interface), smaller shear displacements were required for stress transfer in cover soil than municipal solid waste (MSW), and smaller settlement of the compacted soil veneer in comparison with MSW. A large displacement shear strength can, however, develop when cover soil is placed from the top to the bottom of the slope, when there are traffic loadings on a slope, and if the slope angle of the veneer system is greater than the peak interface shear strength of the weakest interface (Stark and Choi 2004). In the present case history, some detrimental shear displacements may have been introduced to the geotextile/PVC geomembrane interface by placement and spreading of unsupported cover soil (Figure 2(b)) and the slope angle locally probably approached, if not exceeded, the peak interface friction angle of the geotextile/geomembrane interface. Factors of safety for the various slope angles, water levels considered, and the 100 mm displacement geotextile/PVC geomembrane interface strength are shown in Table 3.

The factors of safety presented in Table 3 provide better agreement with field observations than the factors of safety in Table 2. For example, the computed factor of safety for the intermediate water surface and a 2.5H : 1V slope is approximately unity, whereas the factor of safety for the 2H : 1V slope is less than unity. This suggests that the actual slope inclination is between 2.5H : 1V and 2H : 1V, which is probably the actual situation instead of 2H : 1V. The factor of safety values for the high water surface are not representative of field conditions because the factor of safety for a 3H : 1V slope is below unity, and most of the south slope which has an inclination of 3H : 1V did not fail.

The factors of safety for the low water surface are in agreement with the intermediate water level scenario and indicate a slope inclination that is slightly closer to 2H : 1V. The better agreement of the factors of safety in Table 3 with field observations also suggests that the geotextile/PVC geomembrane interface exhibited a large displacement shear strength and not a peak strength. This reinforces the importance of ensuring little, if any, detrimental shear displacement occurs prior to complete buttressing of the critical interface. This is especially important if the earthwork contractor has not worked with geosynthetics previously.

In summary, the best representation of the field conditions at the time of the initial slide shown in Figure 3 was probably the intermediate water surface and a slope slightly steeper than 2.5H : 1V, even though the scenario with a peak interface friction angle and either elevated piezometric levels or a steeper slope cannot be ruled out.

5.4. Analysis of operating conditions

The following three stages of pond operation were also investigated herein.

1. End of construction, namely no water in the pond.
2. The water is at full design depth.
3. The pond level is drawn down to the pond bottom but the water does not drain from the cover soil.

Table 4 presents the factors of safety calculated for these three operating conditions. Another investigator reported that the factors of safety for cases 1, 2, and 3 were all below unity (1.0), indicating the slope should have failed for each case. Field observations, however, show that the slopes were stable at the end of construction on 7 February 2005 (case 1), and thus the computed factor of safety of 0.9 does not agree with field observations. In other words, the pond should have failed during construction with a factor of safety of 0.9 and it did not. In addition, the pond slopes were stable after the pond was filled on 17 May 2005 (case 2), and thus the computed factor of safety of 0.9 for case 2 also does not agree with field observations after construction. In general, the factor of safety should increase when the pond is full because of the buttressing effect of the water.

Another anomaly with the other reported factors of safety is that the first slope failure occurred on 4 or 5 June 2005 after an intense rainfall. The other analyses did not incorporate this rainfall event. Finally, case 3 corresponds

Table 3. Factors of safety for various slope angles and cover soil water levels using the 100 mm shear displacement geotextile/PVC geomembrane friction angle (25°)

Slope	High water surface	Intermediate water surface	Low water surface
3H : 1V	0.98	1.07	1.10
2.5H : 1V	0.94	1.01	1.04
2H : 1V	0.81	0.86	0.89

Table 4. Factors of safety for various operating conditions, a 2.5H : 1V slope, intermediate water level, and the large displacement geotextile/PVC geomembrane interface strength

	End of construction	Pond full	Pond drawn down
Author's factors of safety	1.12	1.19	Not applicable
Other factors of safety	0.9	0.9	0.7

to the full drawdown condition, which did not occur until after the initial failure occurred and thus is irrelevant.

By comparison, the analyses performed herein show a factor of safety greater than unity for case 1 (factor of safety = 1.12) and case 2 (factor of safety = 1.19). These two factors of safety are in agreement with field observations of the slope being stable after construction and pond filling (see Figure 1). The factor of safety is higher for case 2 (pond full) than case 1 (end of construction) because the water provides some buttressing effect to the slope. The slope became unstable with the pond full and the occurrence of a significant rainfall that was modelled using the intermediate water level.

Finally, the pond did not experience a complete drawdown between the pond being full on 17 May 2005 and 4 or 5 June 2005 so this condition was not analysed during this study.

In summary, field observations concerning the behaviour of a slope are important and should guide a causation analysis. For example, a factor of safety less than unity is not representative of field conditions unless the slope failed.

6. CAUSATION

Based on the field observations, direct shear test results and stability analyses described herein, it is concluded that sliding occurred at the geotextile/PVC geomembrane interface because the excavated slope was steeper than 2.5H : 1V, which exceeded the design slope of 3H : 1V, a water level close to the intermediate water level developed in the cover soil above the pond level due to excessive rainfall, the unsupported placement of cover soil, traffic loadings on the slope during and after construction, and the slope angle being at or near the peak strength of the geotextile/PVC geomembrane interface, which mobilised a large displacement interface friction angle ($\delta_{100mm} = 25°$).

7. REMEDIAL MEASURES

The pond riprap, cover soil, and liner system were completely excavated. The pond was relined with a 1.0 mm textured polyethylene geomembrane, non-woven geotextile, cover soil, and riprap liner system. The oversteepened north slope was not flattened prior to installation of the new liner system because of the proximity of the irrigation canal. The subsequent cover soil was placed from the bottom to the top of the slope and the textured geomembrane and non-woven geotextile developed a Velcro bond that increased slope stability.

8. CONCLUSIONS

This paper describes a veneer slide in a non-potable water pond. To accomplish this objective the subsurface investigation, possible failure mechanisms, engineering properties of the geotextile/PVC geomembrane, back-analysis of the slide, and lessons learned are presented. The pond lining system consisted of a geosynthetic liner system designed for a native subgrade that was to be prepared at a 3H : 1V slope. The main lessons from this case history are listed here.

1. Engineers should not 'recycle' designs from prior sites and assume that the design will be suitable for another site even if the design has been previously successful. Each site should be considered independently and the necessary design steps, such as geosynthetic interface testing, should be repeated.
2. Interface tests on site-specific material should be performed to determine placement orientation of the geosynthetics; for example, whether the smooth or faille side of a PVC geomembrane should be facing up, whether or not the critical interface exhibits sufficient shear resistance, and the post-peak strength loss characteristics of the critical interface.
3. Veneer slopes, such as the one involved in this case, should be designed for a static factor of safety of 1.5 because of the many variables such as variations in geosynthetic interface strengths, uncertainty in the intensity of the design storm event, variations in slope geometry and unexpected construction operations, such as different access scenarios, traffic loadings, unsupported cover soil placement and larger than expected riprap, that were encountered in the design and construction processes.
4. Field inspection should ensure that the constructed subgrade inclination does not exceed the design slope angle. The slope angle should be confirmed prior to installation of the geosynthetics.
5. Placement of unsupported cover soil can lead to mobilisation of a post-peak interface shear resistance even in a veneer situation, so it is important that cover soil be rolled, not pushed, from the bottom to the top of the slope.
6. Interface shear tests conducted for causation analyses should utilise the geosynthetics actually installed and the actual orientation of the geosynthetics in the field and not use a random combination.
7. Causation analyses should carefully observe the conditions under which the slope failure occurred

and allow these observations to guide the necessary stability analyses.

NOTATION

Basic SI units are given in parentheses.

- k_c hydraulic conductivity of the cover soil (degrees)
- k_d hydraulic conductivity of the drain (degrees)
- L length of the slope parallel to the slope (m)
- P precipitation rate (m/s)
- RC SCS runoff curve number (dimensionless)
- β slope angle (degrees)

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