Bud Peck Slide, Interstate 15 near Malad, Idaho

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In May 1983, the two northbound lanes on Interstate 15, approximately 8.6 mi north of Malad, Idaho, were destroyed; a case history of the embankment failure is described. Details about the site investigation, dewatering, and final permanent repair are discussed as is the monitoring program that has spanned 7 years. The failure, known as the Bud Peck slide, followed an abnormally wet season. The damaged highway slumped an average of about 20 ft over a 500-ft length. A temporary diversion allowed traffic to pass around the failed area while studies were being conducted to investigate the cause of the failure and possible remediation plans. A site investigation consisting of 17 borings and shallow test pits and the installation of five slope inclinometers, which also served as groundwater observation wells, was quickly performed to determine the mechanisms that might have caused the embankment failure. The data from the borings indicated that the embankment, constructed in 1970, was placed on a 2- to 3-ft sand blanket overlying a deep residual clay layer. The slope indicator data identified a slip plane 3 to 5 ft into the clay layer. It appears that the buildup of pore water pressures in the embankment, due to an inadequate drainage layer, may have caused failure of the slope. On the basis of the available information, several alternatives, including a rock buttress, a sand and gravel fill, and a lightweight fill, were considered as a permanent solution. To lower the groundwater levels, horizontal drains were installed in 1983, and the final repairs used a lightweight pumice fill for embankment reconstruction in 1985. The use of the pumice fill provides a calculated factor of safety against slope failure of 1.3.

In July 1982, cracking was noted in the pavement of a 500-ft section of the two northbound lanes of Interstate 15, 8 mi north of Malad, Idaho (see Figure 1). This distress occurred in 1982 after an unusually wet season in which precipitation amounts were about 100 percent above normal. The pavement continued to crack and slump during 1982 and after another abnormally wet spring in 1983, it collapsed in early May 1983. Figure 2 shows a view, looking to the south, of the failed section of the embankment, as observed near Station 297 + 00 in May 1983. After the failure, the pavement for the two lanes dropped approximately 15 and 25 ft at the north and south ends, respectively.

At this location, I-15 is at an elevation of about 5,430 ft and ascends gradually toward Malad Summit (Elevation 5576). The highway is constructed of an 8-in. layer of continuously reinforced concrete pavement overlying a 4-in.-thick layer of cement-treated base over an embankment of silty clay fill. The embankment averages a height of approximately 45 ft and has slopes angled at 2:1 (horizontal:vertical). The northbound lanes are located on the highest part of the embankment.

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Because both northbound lanes of this vital link between Salt Lake City and Pocatello were disrupted by the cracking in 1982, the Idaho Transportation Department quickly prepared a plan for the design of remedial works. To allow northbound travel, a temporary paved roadway was quickly constructed in the central median area. At the same time a site investigation was being performed to determine the cause of the cracking and slumping.

SITE INVESTIGATIONS

The site investigation program was started in August 1982. It consisted of 17 borings and shallow test pits and the initial installation of five slope indicators to monitor the behavior of the embankment. The location of the test borings and slope indicator locations is shown in the sketches presented in Figures 3 and 4. Additionally, to investigate ground movements, three survey lines were established to supplement the slope indicator data. There was also some concern about the stability of the entire area and therefore eight additional slope indicators were installed to monitor the movement of a much larger portion of the hillside.

As part of the site investigation program, six siphon wells were also installed upslope from the distressed embankment to lower the groundwater levels. Such siphons, consisting of PVC pipe inserted into a shallow boring, provide an effective and economic method to lower the groundwater level 10 to 15 ft as long as there is an adequate elevational drop. However, these siphons must be initially primed to start the flow of water and do require regular monitoring to ensure that the exit pipes are operating properly. In order to monitor the operation of the siphons, two observation wells (OW-1 and OW-2) were drilled, west of the southbound lanes. All other borings were also used as open standpipes for monitoring groundwater levels at the site.

SOIL CONDITIONS

The site investigations provided information concerning the subsoil profile and the groundwater conditions. Representative soil samples were also tested in the laboratory to determine typical strength and compressibility parameters.

Subsoils

The subsoil at the site consists of silty clay fill material and the 2- to 3-ft-thick sand layer from the original embankment

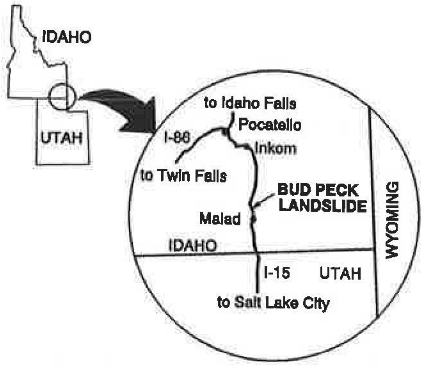

FIGURE 1 Site plan for Bud Peck slide, I-15, near Malad, Idaho.

FIGURE 2 View of failed embankment, near Station 297+00, May 1983.

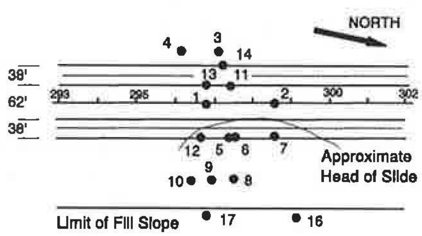

FIGURE 3 Location of test holes for site investigations.

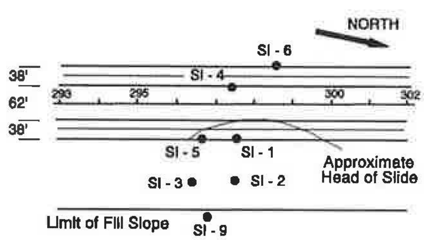

FIGURE 4 Location of slope indicators for site investigations.

overlying the natural clay soils as shown in Figure 5. The silty clay fill was used to construct the embankment in 1970 and had an approximate unit weight of 120 lb/ft^3 and estimated effective shear strength parameters of $\phi' = 25$ degrees and $c' = 0$.

The underlying natural material consists of weathered rhyolite that has decomposed to a gravelly clay residual soil near the surface. With increasing depth, these surface materials are less weathered and show consistent increases in strength and unit weights. The water content of the natural clayey soils in the upper 10 ft ranged between 27.7 and 32.4 percent. The Atterberg limits for the fine fraction of these soils generally ranged from 40 to 56 for the liquid limit, and the plasticity index varied between 22 and 33. Direct shear test revealed that the natural soils had the following range of shear strength parameters:

	ϕ' *(degrees)*	c' *(ksf)*
Peak strength	2–17	0.6–1.2
Residual strength	1–17	0.0–0.7

Groundwater Conditions

The groundwater measurements as expected, revealed the effects of the higher than normal precipitation. The groundwater levels measured at the end of September 1982, are shown in the subsurface profile shown in Figure 5. Some temporary artesian conditions were also encountered for wells in the sand blanket under the southbound lanes. This condition was probably due to the inability of the sand blanket to adequately drain the infiltrating water. Thus water backed up in the slope, and its presence was confirmed by the artesian conditions found at the end of September 1982.

Slope Indicator Data

For the locations shown in Figure 4, the slope indicator data for readings obtained during November 1982 are shown in Figures 6 and 7. The data for locations SI-1 and SI-5 at the crest of the slide (Figure 6) indicate the existence of a potential failure plane below the sand-fill blanket. It appears that there is considerable movement 50 to 60 ft below the surface, or 5 to 10 feet below the sand blanket. At location SI-1 and SI-5 movements of up to 2 in. were measured between November 17 and 26, 1982. The increasing rate of movement for SI-1 and SI-5, as shown in Figure 6, suggests that the slope mass was unstable and appeared to be moving rapidly. Supporting evidence regarding the location of the failure plane was also provided by the slope indicator data for SI-2 and SI-3, located near the toe of the embankment, as shown in Figure 7. Unfortunately, the observations made at slope indicator sites, SI-2 and SI-3, were small and generally inconsistent as the base of the slope indicator does not appear to have been anchored into a stable zone.

Two slope inclinometers, SI-4 and SI-6, were also installed to monitor movements below the southbound lanes. Data collected at these two locations are shown in Figures 8 and 9. The data from SI-4 (Figure 8) indicate small movements that suggest a sliding plane approximately 32 ft below the

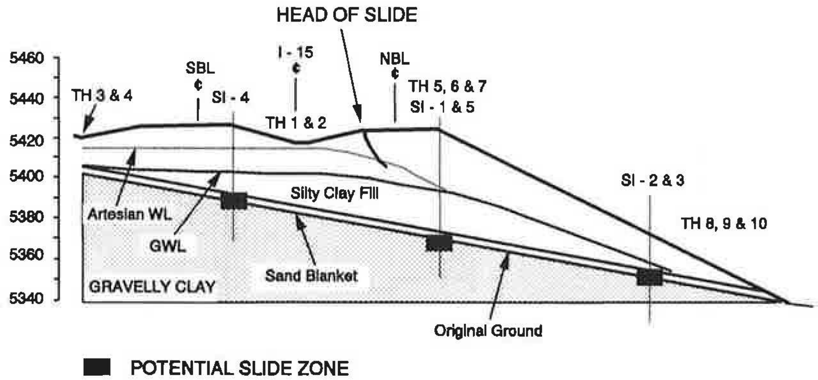

FIGURE 5 Subsoil profile at Station 297+00 (NBL = northbound lane; SBL = southbound lane; GWL = groundwater level; WL = water level; and TH = test hole).

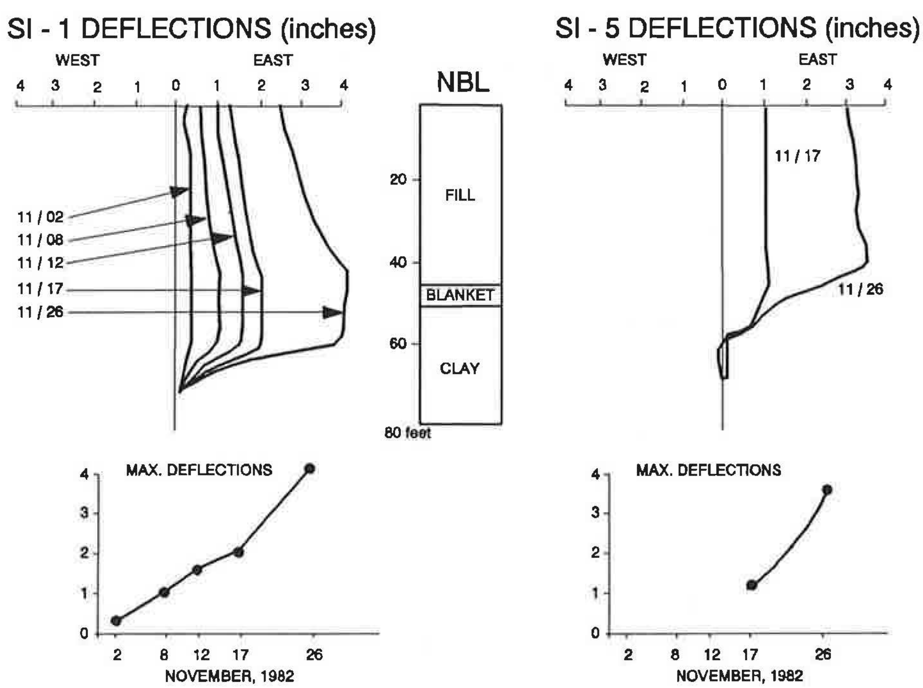

FIGURE 6 Slope indicator data for SI-1 and SI-5 at crest of embankment.

pavement surface. The data from SI-6 (Figure 9) indicated a movement of about 1.3 in. at the surface in early 1983, but the movement appeared to have stabilized after that initial slip. However, the actual movements at the slip plane are less than ½ in. because again, the slope indicators were not anchored in a stable zone. Overall, these movements were small and the southbound lanes were considered to be fairly stable.

The slope indicators were originally installed to monitor the movements of the highway embankment rather than the hillside slope. Unfortunately, the entire hillside slope was found to be actively moving, on the basis of the slope indicator data. This was confirmed by independent observations made using two survey lines. However, because a slip plane could be readily interpreted from the existing slope indicator data, additional slope indicators installed to a much greater depth were not used for studying the behavior of the hillside slope.

EMBANKMENT ANALYSIS

The stability of the embankment slope was investigated using the slope indicator and groundwater level data. With knowledge of the location of the slip plane and groundwater conditions, a limiting equilibrium analysis was used to find the failure mechanism and the residual shear strength along the potential failure plane. The results of the analysis are shown in Figure 10. By analyzing failure surfaces passing through the shear zone indicated by the slope indicator data, the shear strength parameters of the residual soil were back-calculated as $\phi' = 10.5$ degrees and $c' = 325$ psf for the critical surface shown in Figure 10. These parameters appear to be in reasonable agreement with the laboratory test results.

In reviewing the available data, it was apparent that the buildup of water within the embankment had led to exces-

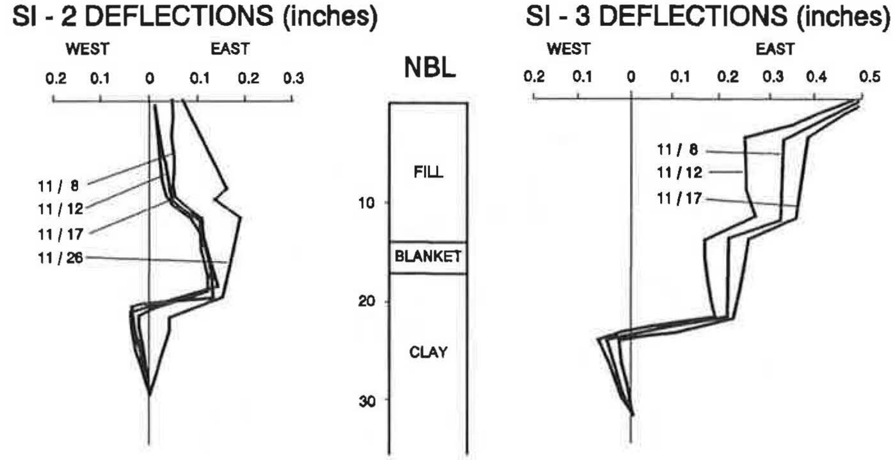

FIGURE 7 Slope indicator data for SI-2 and SI-3 at toe of embankment.

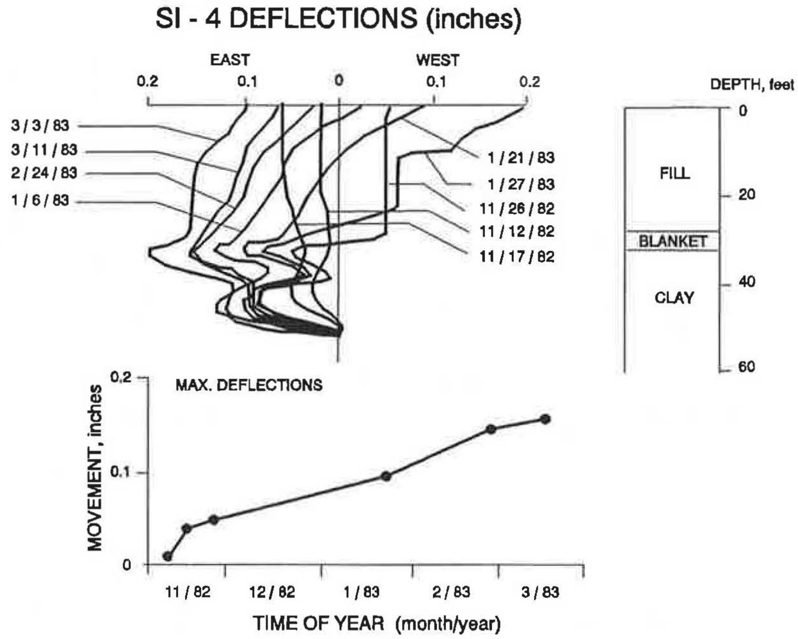

FIGURE 8 Slope indicator data for SI-4, southbound lane.

sively high pore water pressures that probably caused the failure. Thus, it was important that such high pore water pressures be reduced if the slope was to be effectively stabilized. The slope was found to have an adequate factor of safety (FS) against failure providing that the groundwater level could be maintained below the top of the underlying sand blanket. However, because the embankment also needed to be repaired, an additional increase in the FS of the newly constructed slope could be achieved using materials other than the displaced, original silty clay fill.

Two options were considered for repairing the failed embankment. The first option considered a granular-fill buttress at the toe of the existing embankment as shown in Figure 11. The second option required removal of the upper 20 ft of the existing, disturbed silty clay fill and restoration of the embankment to the original 2:1 slope with one of the following fill materials (a) sand and gravel, (b) pumice fill, and (c) wood fibers (Figure 12).

Stability calculations were performed for the three options, assuming that the groundwater level would be located in the sand blanket and strength parameters of $\phi' = 10.5$ degrees and $c' = 325$ psf for the underlying gravelly clay. The minimum FS for the buttress was estimated as 1.5. For the replacement fill alternatives, the following FS values were computed as

Material	ϕ' *(degrees)*	c' *(psf)*	*Compacted Unit Weight (pcf)*	*Factor of Safety*
Sand and gravel	39	0.0	130	1.14
Pumice	41	0.0	75	1.29
Wood fibers	37	0.0	55	1.35

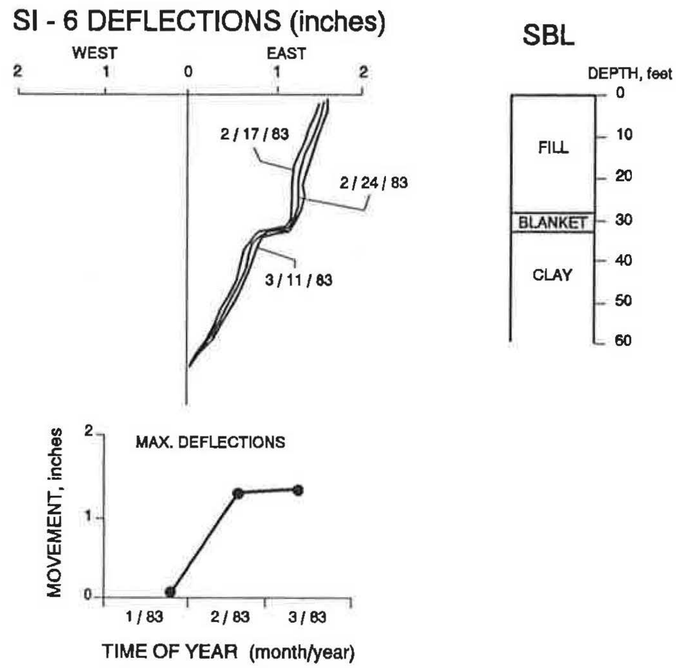

FIGURE 9 Slope indicator data for SI-6, southbound lane.

In reviewing the costs of construction, the buttress would have required at least 50 percent more material than the other alternatives, even though a higher FS would have been obtained for stability. However, after monitoring the site for about 12 months, there were signs that a much larger portion of the hillside was moving, perhaps because of a very deep seated failure plane. Thus the buttress option was discarded as a possible long-term solution in favor of extensive dewatering, to stabilize the hillside, and the use of a lightweight fill to repair the embankment. The lightweight fill would also have minimized the impact of an external load on a potential deep-seated failure surface.

In comparing the remaining options, the pumice fill was found to be the most desirable and could be readily obtained from a source 40 mi away. The strength of the compacted pumice at 95 percent of modified Proctor compaction was estimated as $\phi' = 41$ degrees from direct shear tests. The nearest source for the wood-fiber fill was about 60 mi from the site. However, the wood fibers would have required protection from water and air to avoid potential spontaneous combustion and decay. Also, concerns were expressed about the disposal of the slightly toxic and odorous leachate that would have drained from the wood-fiber fill. The sand and gravel fill would have been marginally cheaper, but the increased stability offered by the pumice fill was a deciding factor in the selection of the pumice for the restoration of the failed embankment.

Because the weight of pumice fill is considerably less than the original weight of the silty clay fill being replaced, the embankment was expected to experience negligible settle-

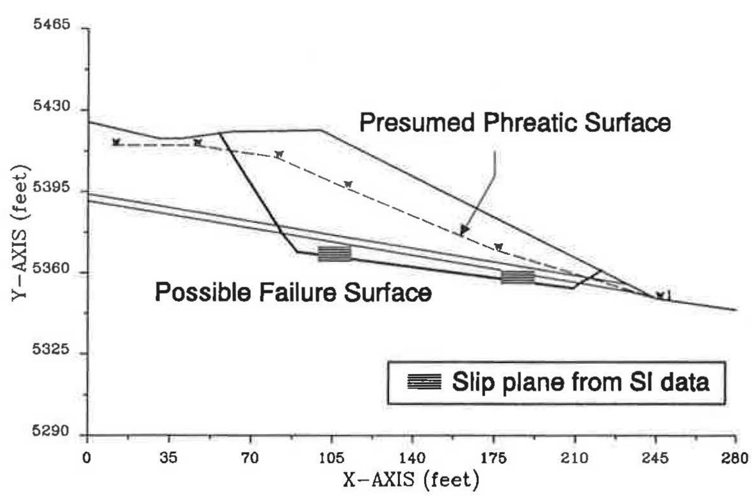

FIGURE 10 Potential failure surface for 1983 slide.

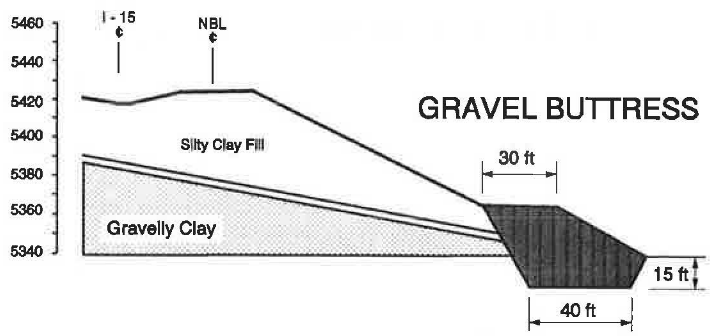

FIGURE 11 Repair of embankment with a granular buttress.

ments. Such settlements were expected to have occurred because of compaction of the pumice fill during construction.

CONSTRUCTION

The damaged embankment and northbound lanes were repaired between 1983 and 1985; work was done under two separate contracts. The initial contract concerned the dewatering of the affected embankment in order to prevent any further movement or failures. The second contract consisted of repairing the embankment and construction of the northbound lanes.

Horizontal Drains

The first contract was started in August 1983 and concentrated on dewatering the slope and embankment. This consisted of installing 42 horizontal drains from 4 separate pads near the toe of the embankment, as shown in Figure 13. Drainage was provided by slotted PVC pipe, 2 in. in inside diameter, laid at an approximate gradient of 5 percent in predrilled borings. The lengths of the drains ranged between 125 and 350 ft and were arranged to extensively penetrate the failed portion of the embankment. These drains were installed at a cost of $100,000.

Initial flows of about 200 gal/min quickly lowered the groundwater level approximately 20 ft below the southbound lanes to Elevation 5,388 ft. With groundwater levels at this depressed level, embankment movements were finally arrested as shown by the survey line data presented in Figure 14. These data clearly show the influence of the horizontal drainage in lowering the groundwater level at Observation Well OW-1 and the negligible movements measured from the two survey lines, SL-1 and SL-3.

Embankment

The embankment was repaired in the fall of 1985. The work consisted of removing the disturbed silty clay fill from the damaged embankment and replacing it with pumice. The slope was reconstructed to its original 2H:1V angle to match the adjacent surviving portions of the embankment. The pumice was placed in 8-in. lifts and compacted with a 12-ton single-drum vibratory roller to achieve a minimum 95 percent modified Proctor unit weight of 75 pcf at about 25 to 30 percent water content. The exposed slope was covered with a 12-in. layer of topsoil.

A considerable amount of water was exiting the embankment through the installed horizontal drains, so a French drain was constructed near the toe of the slope. This drain was intended to convey the exiting water away from the embankment and toward the natural drainage path of Devil Creek, about 750 ft to the east.

PERFORMANCE OF EMBANKMENT

The repaired embankment has performed satisfactorily since its reconstruction. The slope indicators and the groundwater observation wells are being monitored regularly. The slope indicators near the embankment have showed negligible

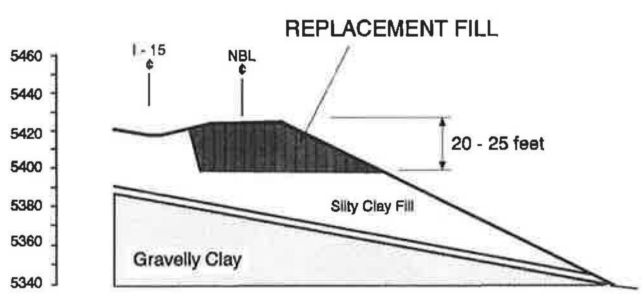

FIGURE 12 Repair of embankment with replacement fill.

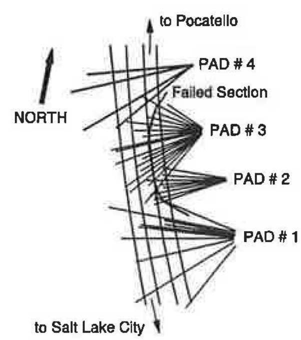

FIGURE 13 Location of horizontal drains.

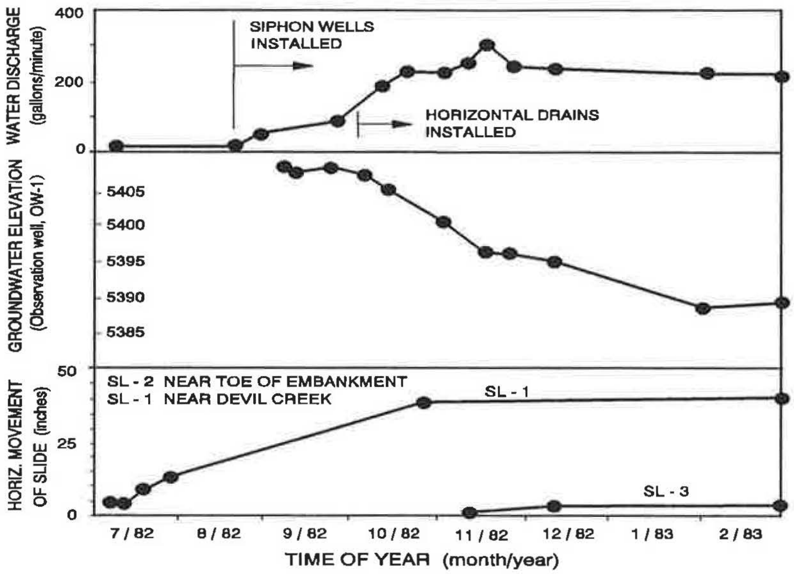

FIGURE 14 Slope monitoring data for seepage, groundwater levels, and movement.

movements, and the groundwater levels are currently at their lowest since 1983. However, some movement has been detected at slope indicators that were placed about 200 ft downslope (east) of the embankment. This movement is probably due to creep of the surficial soils over the entire hillside.

The seepage from the horizontal drains has been monitored to ensure proper operation. In September 1989, these drains were flowing at a rate of about 22 gal/min, which is considerably less than the 74 gal/min measured in April 1987 after the spring rains. The drains were cleaned and flushed in 1987 and 1988.

SUMMARY AND CONCLUSIONS

An embankment constructed in 1970 suffered a failure after two consecutive abnormally wet seasons in 1982 and 1983. The failure damaged a 500-ft section of the northbound lanes of I-15 near Malad, Idaho. Unfortunately, the 2- to 3-ft-thick sand blanket failed to discharge the large quantity of infiltrating groundwater, which resulted in a buildup of pore water pressures. In the opinion of the authors, this buildup of pore water pressures within the embankment was the primary cause of the failure.

While traffic was diverted over a temporary pavement installed in the central median, the slope was stabilized by dewatering in 1983 and final repairs were made to the embankment in 1985. The total cost for the installation of the horizontal drains and reconstruction of the damaged embankment was estimated at $770,000.

For future design and construction of silty clay embankments, the sand blanket should be designed as a positive drainage layer. This feature may be implemented by including slotted pipes within the sand blanket to provide a direct path for the flow of infiltrating groundwater. The relatively high permeability of the sand blanket cannot be relied on to prevent the buildup of high pore water pressures as encountered at the Bud Peck slide near Malad. Such a scheme can be specified at the construction stage for only a nominal increase in the overall costs. Also, a French drain should be used for all sideslope embankments to intercept groundwater flows above the embankment, thus limiting the chances of pore pressure buildup.

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