Partial Landslide Repair by Buttress Fill

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Two residences in Orinda, California, were threatened by the enlargement of an existing landslide area. Published geologic literature described the characteristics of landsliding in the geologic materials present in the Orinda formation. Site specific studies were performed using aerial photographs, small and large diameter exploratory borings, and laboratory testing. The studies indicated that the previous landslide was massive in character and that complete repair was probably economically impossible. However, buttress-fill grading could be performed at the rear of the two properties to stabilize those areas and leave the remaining portions of the landslide unchanged.

Two residences in Orinda, California, are immediately adjacent to an area that experienced significant landsliding more than 20 years ago. The rear of one residence was within 10 ft of the lateral scarp of the landslide, and the improvements behind the second home were about 30 ft from the lateral scarp. A slow retreat of this scarp threatened both properties. The owners of the homes had access to a significant amount of soil that could be used for grading to help stabilize the rear of the properties. Therefore, geotechnical studies were performed on the portion of the landslide in the rear yards of the two properties. These studies were intended to characterize the landslide materials and identify a reasonable engineering solution to protect the homes and their associated improvements. The general location of the Orinda area is shown in Figure 1.

INTRODUCTION

In the years following World War II, significant development pressures to provide housing opportunities occurred in the suburbs of the San Francisco Bay Area. Access to the suburban Orinda area had recently been improved by constructing a tunnel (completed shortly before the war began) through the hills that bound the immediate Bay Area. The state highway serving the Orinda area had significantly easier access using the Broadway tunnel than with the previous winding alignment across the top of the hills or through the earlier narrow tunnel. However, the geologic environment in the Orinda area posed many problems regarding landsliding. For example, a major landslide occurred in December 1950 that buried the highway just east of the Broadway tunnel. This landslide was more than 300 ft wide and extended up the hill adjacent to the highway for approximately 800 ft. Because previous instabilities had been observed on the hillside an extensive network of horizontal drains was installed to help stabilize the area (*1*).

The Federal Housing Authority (FHA) sponsored a significant portion of the subdivision construction in the Orinda area during the 1950s. Because of the geologic instabilities that were observed, the FHA contracted with the U.S. Geological Survey (USGS) to perform an engineering geology investigation of the area in Orinda where the Warford Mesa subdivision was proposed. The results of this evaluation were published as a USGS open-file report (*2*). The results concluded that the area was generally underlain by interbedded conglomerates, sands, and clays of the Orinda formation. These materials were described as chiefly continental, lacustrine, and fluviatile deposits, formed during the Pliocene epoch. X-ray diffraction evaluations of the clay materials indicated that they contained 35 to 95 percent montmorillonite, averaging 45 to 50 percent. Free-swell tests performed on this montmorillonite material indicated that the clays expanded from 20 to 100 percent, with a typical value of 30 percent expansion. The study indicated that there were widespread landslides in the proposed subdivision area and that the landsliding was not restricted to dip-slope environments. It also concluded that significant subsurface water could be carried by conglomerate or sand beds to the less pervious and weaker clay beds where slippage occurred. Furthermore, it concluded that joint planes in the bedrock appeared to be planes of greater weakness than the bedding planes. The report then provided some conclusions regarding the engineering criteria to be used in the grading and development of the proposed subdivision.

Continued landsliding in the general area led to more-detailed studies of the landslide problems by USGS. These further studies were published in an open-file report which studied an area of approximately 6 m^2 around Orinda (*3*). To evaluate the general characteristics of the study area, the authors collected data on approximately 195 landslides. They noted that many more landslides were present, but picked these landslides as representative of the largest and most clearly defined landslides in the area. All of the landslides were then tabulated and described with regard to their form, type of material present, geologic setting, vegetation present, and any observed damage to manmade structures. The authors concluded that rainfall was the most common triggering mechanism in the landslide areas. They further concluded that landslides often occurred because of combinations of geologic setting and site exposure. One of the most interesting findings was that landslides appeared to be more prevalent on slopes on which the bedding dipped in a direction opposite to the hillslope rather than on a dip-slope environment. They also pointed out the common relationship between the grading activities used to develop the subdivisions and the subsequent landslide movements.

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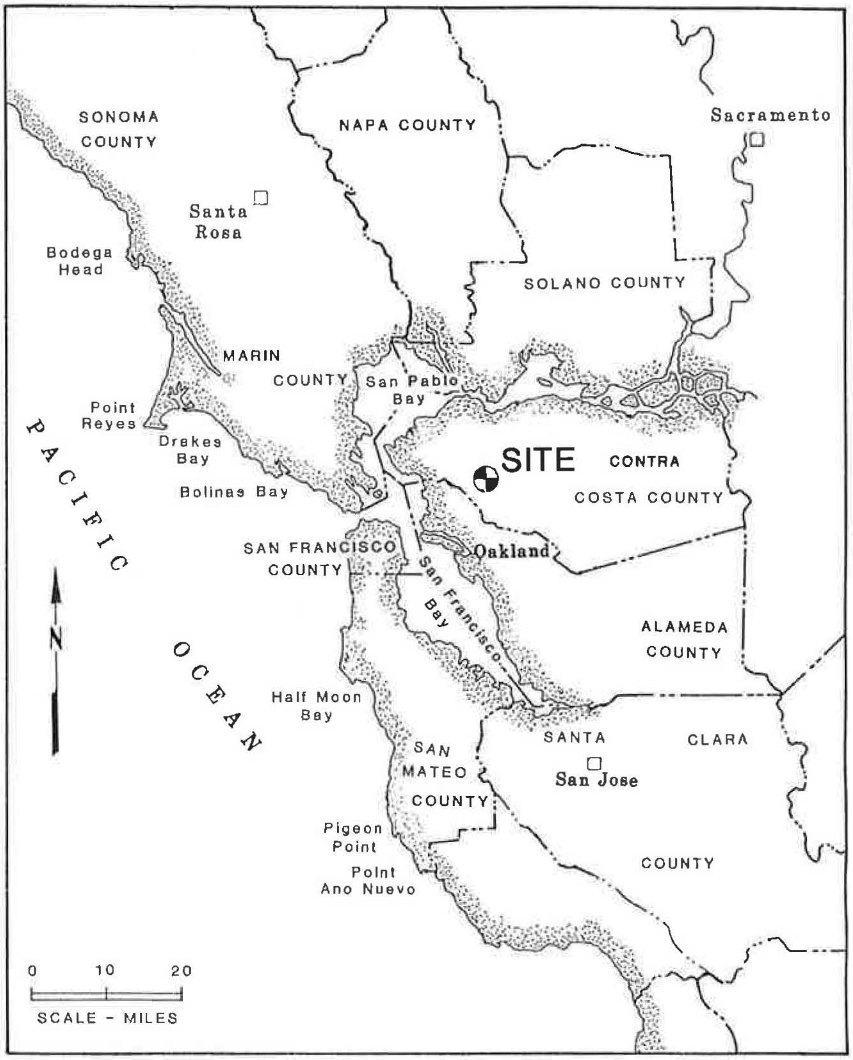

FIGURE 1 Vicinity map.

SITE HISTORY

The two Orinda lots that contained the homes threatened by the landslide evaluated in this study were located at the end of Silverwood Court (see Figure 2 for the configuration of the lots in the vicinity). Silverwood Court is a short cul-de-sac road that extends westerly off the main thoroughfare of the area, Tahos Road. An evaluation of a series of historic aerial photographs and records from the Contra Costa County Grading Department were reviewed to develop the historic site conditions. Aerial photographs taken through the mid-1950s indicated that this area was generally vacant except for a narrow unpaved road, which extended along the top of a major ridge line where Tahos Road was later constructed. At this time, the area where Silverwood Court was later built was a secondary ridge extending off of the main ridge line. The rear of the two lots studied extended into a swale area that extended downhill from Tahos Road in a westerly direction and ended in a canyon about 1,000 ft from the road. The general conditions around the secondary ridge where Silverwood Court was built appeared to be relatively stable, but the swale feature contained sparse vegetation and hummocky typography.

Aerial photographs taken in 1957 indicated that improvements had been made to Tahos Road. The road had been widened, and a number of areas of sloughing were observed along the shoulders of the widened road. These failures were most likely due to heavy rainfalls that occurred in the area during the winter of 1957 (shortly before the photographs were taken). Photographs and county records indicate that a major fill embankment was placed into the swale area along the west side of Tahos Road to develop two level building pads along Tahos Road for home construction. Homes were subsequently built on these two lots and soon after Silverwood Court and the building pads along this roadway were graded. A home was built on Lot A; Lot B remained vacant.

A major landslide occurred in the former swale area during the heavy winter rains of 1967 as shown in Figure 2. The homes previously on the two lots next to Tahos Road were either destroyed or removed from the property. The lateral scarp along

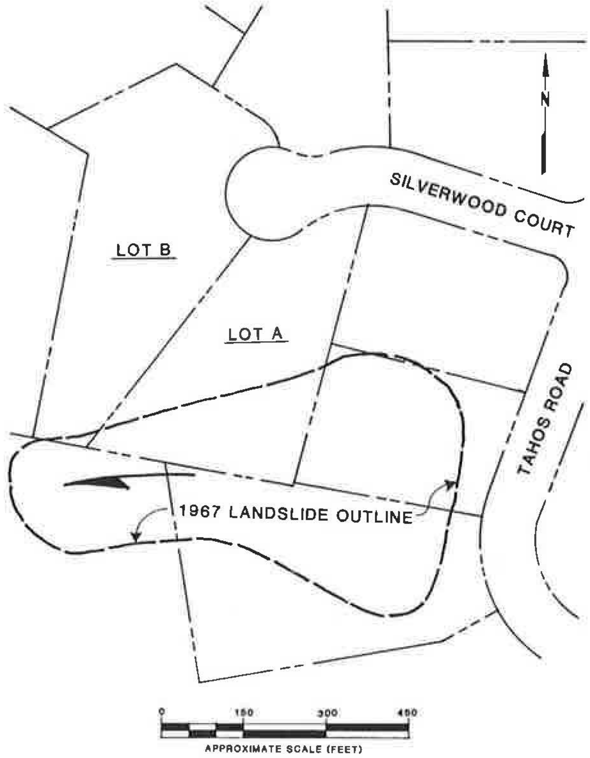

FIGURE 2 Lot plan.

the northern edge of the slide extended very close to the rear of the home on Lot A. This lateral scarp was about 10 ft in height, and the main head scarp adjacent to Tahos Road was approximately 20 to 25 ft in height. The overall dimensions of the landslide were approximately 250 by 400 ft.

The home on Lot B was built in the late 1970s. From 1967 to the late 1980s, there was a slow sloughing and lateral migration of the head and lateral scarps of the landslide, which led to the concern of the two homeowners on Silverwood Court that their houses or improvements might ultimately be threatened by future landslide movement or continued sloughing of the landslide margins.

SUBSURFACE EVALUATION

The mapping by Kachadoorian (*2*) indicated that the area of Silverwood Court and Tahos Road was clearly underlain by Orinda formation materials. It indicated that bedding in the area generally trended in an east-west direction and that a number of conglomerate beds crossed through the area. These beds seem to form the secondary ridge feature where Silverwood Court was constructed and another secondary ridge feature along the southern limits of the swale. There was an absence of conglomerate beds shown within the swale, so the conglomerate beds seemed to form a structural control along the northern and southern boundaries of the swale feature.

For the current investigation, both small- and large-diameter subsurface borings were extended into the landslide area within the two subject properties. Twelve small-diameter borings ranging from about 3 to 8 in. across were drilled using various types of drilling equipment, depending on access requirements. Selected samples from these borings were taken to the laboratory for testing. These borings were generally extended to depths of 10 to 60 ft. To further define the subsurface geologic environment, five 24-in. borings were drilled and downhole-logged by the engineering geologist. These borings generally extended to depths of approximately 15 to 30 ft. The locations of both the small- and large-diameter borings are shown on the site plan in Figure 3.

The upper materials encountered in the borings generally consisted of silty and sandy clays, which included fill materials, natural residual soils, and old landslide debris. The clays contained abundant angular rock fragments, and often thin layers of soft, saturated clays were encountered in the thicker layers of stiff to very stiff clays. Various slickenside planes were noted in the softer clay materials. At the base of the fill embankment placed in the swale to create the building pads on Tahos Road, a layer of top soil was encountered at a depth of approximately 25 ft. Generally, the clayey soils were underlain by 5 to 10 ft of highly weathered, soft-hardness claystone, siltstone, and sandstone bedrock. Some slide planes or slickensided surfaces were observed in these bedrock materials. These materials were underlain by moderately weathered bedrock and contained no obvious evidence of past shearing.

Groundwater was encountered in several of the borings and caving of the large-diameter holes limited the logging by the engineering geologist. In several cases, groundwater rose to within 10 to 15 ft of the ground surface. Soil inclinometer casings were installed in five of the small-diameter borings, so longer-term groundwater monitoring was not possible in these holes. These casings were installed to provide future slope movement monitoring during the anticipated grading operations and subsequent heavy winter rainfall events.

LABORATORY TESTING

As noted above, three general categories of materials were encountered in the borings. These included clay soils (both native materials in-place and native materials placed as fill—most of which became landslide debris), highly weathered bedrock materials, and moderately weathered underlying bedrock. Typical properties obtained from laboratory testing and standard penetration resistance values, which were obtained during the sampling process, are presented in Table 1.

In addition to the laboratory test data presented in Table 1, one direct shear test and one multistage, U-U triaxial test were performed on clay soils at field moisture content levels. The results of these additional strength tests are presented in the following table:

	Apparent Cohesion ($kips/ft^2$)	*Apparent Friction Angle (degrees)*
Direct shear	0.27	16
Triaxial (U-U)	1.06	10

ANALYSIS

The landslide that occurred in 1967 appeared to conform to the pattern of landslides in the area described by previous

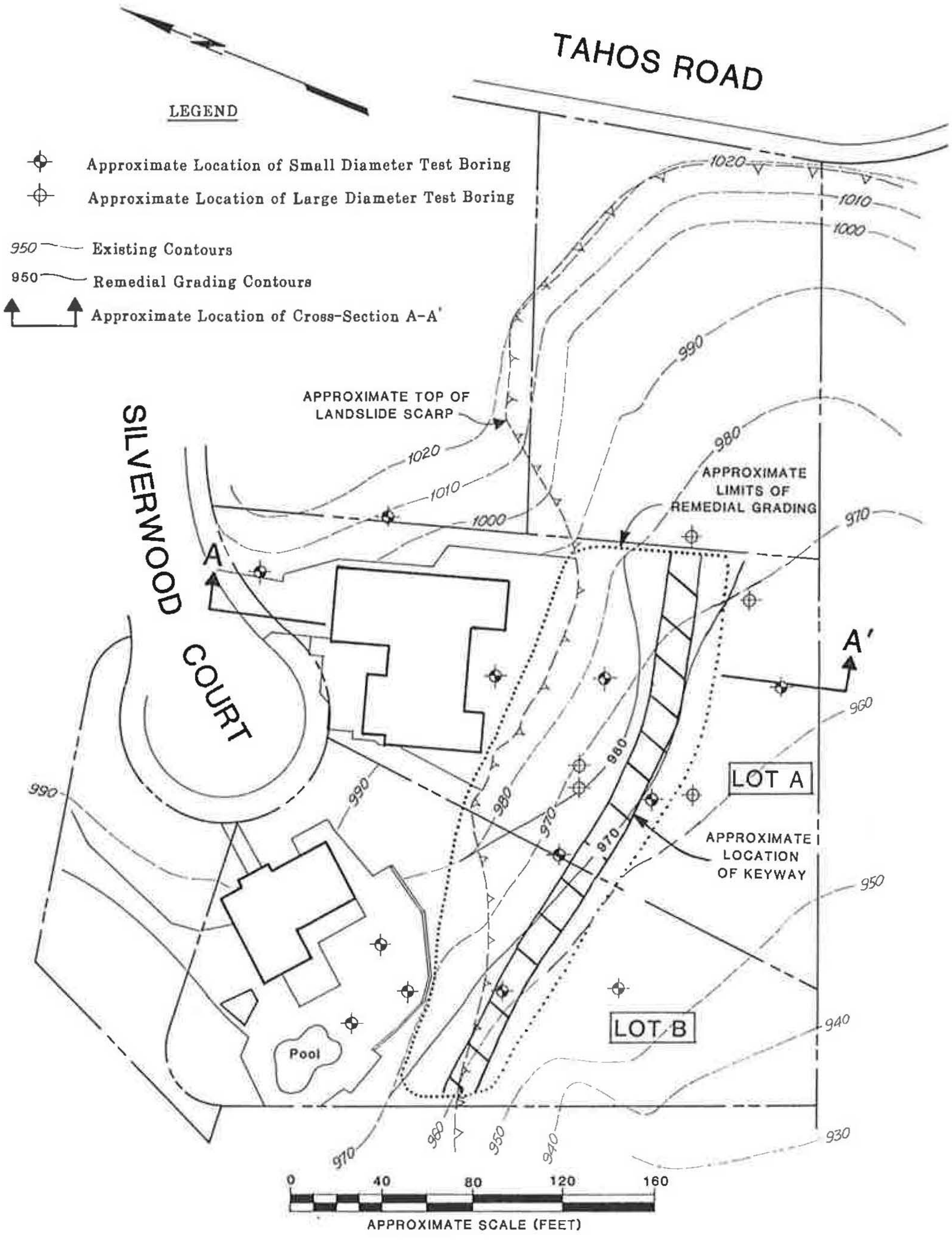

FIGURE 3 Site plan.

investigators. This landslide was apparently structurally controlled by the geologic framework present in the area. The published geologic maps indicated that conglomerate beds were present along the north and south borders of the swale area. No conglomerate material was encountered in any of the subsurface studies within the landslide. Therefore, it appeared that the lateral limits of the landslide were controlled by stronger secondary ridges that were underlain by conglomerate. Not only did the geologic structure limit the extent of the landsliding, but the presence of the conglomerate beds (and, to a lesser extent, the presence of the sandstone beds) provided a source for significant infiltration of subsurface water into the swale area during heavy winter rains. The water weakened the natural soils, the highly weathered bedrock underlying the fill embankment, and the clay soils in the embankment itself. The presence of slickensides within the natural clay soils and the highly weathered bedrock materials indicated that multiple slippage planes were present and the landslide may have been slowly developing over geologic time as the slippage surfaces became more defined and interconnected. With the added weight of the fill embankment, mass failure occurred.

Laboratory strength measurements of the clay materials probably gave strength properties higher than the strength of the materials present at the time of the failure. This is a result of significant weakening because of increased water content

TABLE 1 MATERIAL PROPERTIES

	SOIL		HIGHLY WEATHERED BEDROCK		MODERATELY WEATHERED BEDROCK	
	Range	Average	Range	Average	Range	Average
Standard Penetration Resistance (Blows/Foot)	4-30	14	27-88	49	70- >100	>100
Water Content (%)	12-25	18	12-20	17	10-18	15
Plasticity Index (%)	25-41	30	-	-	-	-
Liquid Limit (%)	42-59	49	-	-	-	-
Unconfined Compressive Strength (Kips/Square foot)	1.3-4.5	3.0	-	-	-	-

during heavy rainy winters and the slow decline toward residual strength on the slide planes. Previous analyses of the soil and bedrock failures in the Orinda formation were performed by Duncan (*4*). Using back analysis with charts by Taylor (*5*), he assumed a friction angle of 20 degrees and found that a fairly consistent cohesion intercept of 20 and 55 psf would be obtained for soil and bedrock at failure, respectively, in the Orinda formation. Using an assumed friction angle of 10 degrees in soil, he also found that the back-analyzed cohesion necessary for stability varied from about 75 psf for a slope 8 ft high to about 200 psf for a slope 25 ft high. It should be noted that these values are not the fundamental properties of the soil but simply empirical coefficients whose values reflect the behavior of the soil and bedrock under the particular climatic conditions of the area. A comparison of our laboratory test data to the back-analyzed strength values indicated that the laboratory test values provided a significant overestimation of the strength of the soil along the shear plane at the time of failure for the reasons mentioned earlier.

The focus of the investigation was to provide increased stability to the area immediately behind the two homes on Silverwood Court. It was clear that the stabilization of the entire landslide was not economically possible for the two landowners and that a complete repair would also involve the coordination and cooperation of multiple homeowners in the area. Therefore, the repair analysis concentrated solely on the portion of the landsliding within the two subject lots and recognized that the remainder of the landslide might reactivate. Using the back-analyzed strength values from Duncan (*4*), analyses indicated that the lateral margin of the landslide would continue to fail even if the main slide mass did not displace significantly. The scheme chosen involved installing a buttress fill along the rear of the two properties. Because of the relatively high strength of the moderately weathered bedrock materials, it was decided that adequate stability could be achieved by notching or keying and benching into these materials and installing subsurface drainage blankets to collect subsurface water flow. Analyses of the mass stability of a failure plane immediately behind and below the buttress fill and the stability of a potential failure plane through the buttress fill were conducted, and both yielded satisfactory factors of safety. An idealized cross section taken through Lot A is shown in Figure 4 and illustrates this buttress-fill concept.

PROJECT CONSTRUCTION

Grading to create the buttress fill begin in summer 1988. The initial excavation extending into moderately weathered bedrock was performed first. It was found that significant vari-

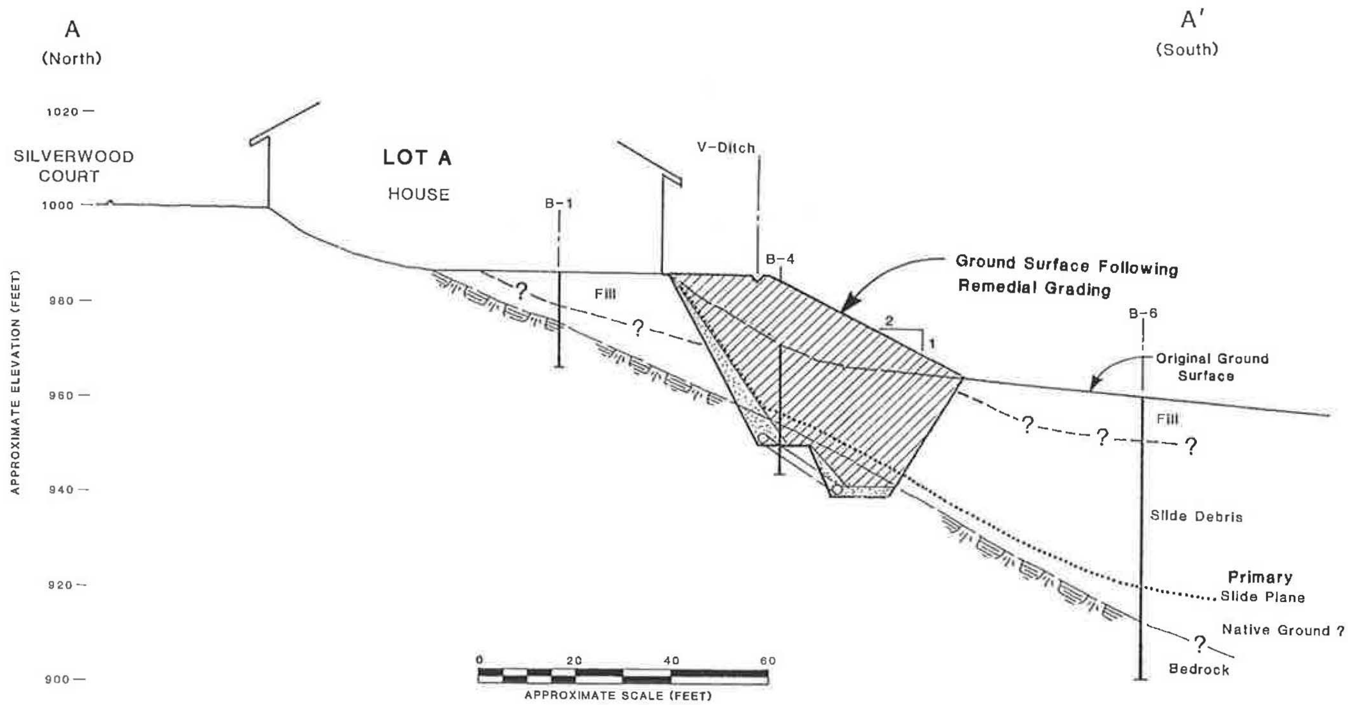

FIGURE 4 Idealized cross section.

ation in the depth of the bedrock occurred between boring locations. Whereas the moderately weathered bedrock was encountered in the keyway (lowest notch) at a depth of about 30 ft at the boring locations as predicted, the bedrock dipped to as deep as 50 ft below the existing ground surface between the borings. The excavation work was performed with bulldozers and front-end loaders that continued to shave the side slopes of the excavation back as it became progressively deeper. A drainage blanket was installed in the bottom of the keyway. This blanket consisted of a 4-in.-diameter ABS pipe surrounded by a 12-in. layer of ½- to ¼-in. clean gravel, which was encircled by a nonwoven, polyester geotextile. To provide gravity-flow discharge for water collected in the drainage blanket that was placed in the bottom of the keyway, hydrauger pipes were drilled from downslope to connect to these drainage blankets. Two additional notches or benches were made upslope of the keyway. The native soils and bedrock that were excavated at the site were stockpiled and moisture-conditioned for reuse as fill. These materials were placed in thin layers and compacted to at least 90 percent relative compaction (in accordance with ASTM 1557-78). The majority of the work was completed within 2 months. Surface drainage measures were also provided on top of the buttress fill, and erosion-resistant vegetation was planted.

The owners of the project did not notify the grading contractor of the presence of slope inclinometer pipes before commencement of grading. Therefore, the final monitoring of most of these inclinometer pipes occurred before they were destroyed by the major excavations, and no movement was recorded during that period. Subsequent financial concerns by the owners have eliminated any readings of the slope inclinometer following the completion of the grading work. Although the area has endured drought-type conditions since the grading work was completed, no obvious indications of movement have been observed in the buttress fill.

SUMMARY AND CONCLUSIONS

Stabilization of the rear slopes behind two residences became necessary when ongoing sloughing of the lateral scarp of a large landslide occurred. Literature about the geologic characteristics of landslides in the area provided general insights into the landsliding. Site-specific studies indicated that a buttress fill would stabilize the rear yards, even if new movements occurred in the rest of the landslide. The remedial grading work to install the buttress fill was completed in 1988 and has performed satisfactorily to date.

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