Effect of Toe Excavation on a Deep Bedrock Landslide

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Abstract: Observations, data, and analyses that were used to investigate the cause of distress to a single-family residence located adjacent to a major highway cutslope are presented herein. The investigation revealed that the distress in the single-family residence was caused by a deep, large excavation-induced landslide. The excavation, which was made to widen an existing highway, helped trigger the landslide by exposing geologic structures on the cutslope and by unloading the toe of the slope. This case history illustrates some of the ramifications of large highway excavations in natural slopes surrounded by urban areas, e.g., exposing significant geologic features such as shear zones, faults, and folds; the importance of investigating and explaining signs of movement at both the top and toe of a slope; the impact of rainfall on the movement of a large slide mass; and that large slide masses can undergo slow, episodic movement instead of sudden, large movement.

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Introduction

Major slope excavations are routinely required to facilitate construction of highways. Because these excavations affect the stability of the slopes on which they are made, the design process must carefully address the potential impact of excavations on the surrounding landscape. Details of a case history are presented herein in which a large highway excavation caused distress to a nearby single-family property developed with two homes over a 40 year period. This case history highlights the need for adequate subsurface investigation for the purposes of assessing postexcavation stability of highway cutslopes.

The distressed single-family residence is located near San Francisco, Calif. and sits atop a ridge upslope of an approximately 70 m (230 ft) high cutslope for a major east–west state highway. The cutslope has an average inclination of $2H:1V$ (horizontal:vertical) and was created by two different excavations. The initial excavation began in 1955 and was completed in 1957. This excavation allowed the initial construction of the highway. In 1967, a second excavation was undertaken to widen the highway and make space for the railroad tracks of the Bay Area Rapid Transit (BART) system. This excavation was completed in 1970 and resulted in a deepening and widening of the existing cutslope.

Since 1965, two different single-family residences have been constructed near the top of the cutslope. The first residence occupied the site from 1965 until 1985 and was located in essentially the same building footprint as the current (second) distressed residence, which was constructed in 1988. Fig. 1 presents an aerial photograph from 1999 that shows the cutslope and the second distressed residence. Both distressed residences are underlain by claystone of the Orinda Formation. The materials of the Orinda Formation are highly prone to landsliding as indicated by the 195 landslides that occurred in an 8.5 mi.^{2} area of the Orinda Formation in a 2 year period (Radbruch and Weiler 1963). The Orinda Formation is of Pliocene age and includes conglomerate, sandstone, siltstone, and claystone beds. The predominant geologic structure within this formation is a group of northwest trending parallel folds, which are offset by several faults (Radbruch and Weiler 1963). In most locations the weathered rocks of the Orinda Formation are exposed on hillsides and this weathering extends to a depth of 0.9–6.1 m (3–20 ft). These weathered materials have very low shear strength and are therefore susceptible to surficial landsliding (Duncan and Stark 1992). Although they contain rock fragments, these materials behave like soils, and the surficial landslides that occur within them are similar to shallow slides that occur in soil slopes. An interesting aspect of the landslide causing the residence distress is that it is not surficial, which is common in the Orinda Formation. Instead, the landslide is a deep bedrock slide and the surficial soils are intact but are being displaced by movement occurring in the underlying bedrock.

Chronology of Residences and Cutslope Movement

The data collection process focused on explaining the: (1) type of movement, e.g., slow and episodic movement; (2) depth of and

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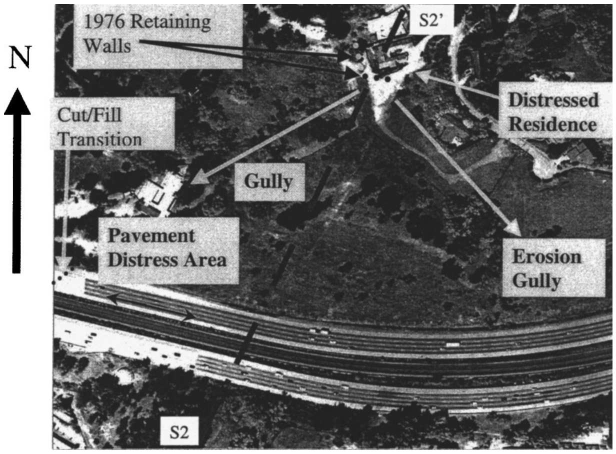

Fig. 1. Aerial photograph of cutslope and distressed residence in 1999

materials involved in the movement, e.g., typical surficial sliding versus deep-seated movement in bedrock; (3) repeated triggering of the movement, e.g., rainfall; and (4) cause of the movement, e.g., the highway excavation. To explain the movement, the history of distress at the residences and cutslope is described in the following sections. Table 1 provides a summary of the major movements at the residences and cutslope and the annual rainfall amount.

Performance of First Residence from 1965 to 1978

The first residence was constructed in 1965, which is eight years after completion of the first highway excavation and 5 years before the completion of the second excavation. The first residence was constructed on a level building pad created by cutting approximately 1.5 m (5 ft) into the existing slope and placing fill wedges, 1.5–2.5 m thick (5–8 ft thick) on the south and west downslope sides of the building pad. The fill wedges consist of on-site surficial material excavated and recompacted to create the fill wedges. The residence and subsequent addition were built across the cut/fill transition. The northern half of the residence was supported on shallow piers embedded approximately 1 m (3 ft) into rock while the southern half of the residence was supported on shallow piers approximately 1 m (3 ft) into fill. Between 1965 and 1970, there was no reported damage in the residence and no evidence of bedrock movement. Hence, the first highway excavation does not appear to have adversely affected the first residence.

In 1975, 5 years after completion of the second highway excavation, the first sign of distress to the first residence was documented. Pilecki (1975) states that the "residence started experiencing distress several years ago." This distress (cracking in the southwest corner of the residence and lateral separation of exterior improvements) corresponds with the occurrence of heavy rainfall. In 1967 and 1969, the total rainfall was approximately 99 and 94 cm (39 and 37 in.), respectively. A main finding of this current investigation is that an annual rainfall amount of greater than about 89 cm (35 in.) usually resulted in distress in both of the residences (discussed subsequently). However, by 1969 the second excavation had not been completed and, thus, the slope toe had not been fully unloaded. Therefore, distress was not observed in the first residence in 1967 or 1969 even though the rainfall amounts exceeded the postulated threshold value of 89 cm/year (35 in./year).

The rainfall in 1968, 1970, 1971, and 1972 is below 76 cm/year (30 in./year) and no movement was observed in the residence (see Table 1). By 1973, the second excavation was complete and the rainfall reached 99 cm (39 in.). In 1974, the rainfall was slightly less (93 cm or 37 in.) but above the threshold value for movement (89 cm/year or 35 in./year). The statement by Pilecki (1975) suggests that the distress might have begun during 1973 and 1974. The occurrence of movement after the second highway excavation during years of heavy rainfall resulted in the first suspicion that there might be a correlation between the second excavation, rainfall, and distress in the first residence. The timing of the movement is important because it is the first time that the new cutslope material was exposed to a relatively large amount of rainfall after being daylighted or exposed by the second excavation. Completion of the highway widening resulted in the cumulative removal (sum of the two excavations) of about 630,500 m^3 (823,000 yd^3) or about 680,400 kg (1.5 million t) of stabilizing material that was buttressing potential weak layers confined below the ground surface. These facts suggest that the second excavation reduced the stability of the cutslope to a point that rainfall amounts larger than a certain threshold value could cause the slope to move.

The rainfall from 1975 through 1977 was below 76 cm/year (30 in./year) and no residence distress was documented. In January 1976, two retaining walls (west wall and south wall, see Fig. 1) were installed to reduce the distress to the first residence, which at the time was thought to be caused by movement of the 1.5–2.5 m thick (5–8 ft thick) fill wedges underlying the residence (Pilecki 1975). The west wall was installed to resist movement towards the western erosion gully (see Fig. 1) and the southern wall was installed because the observed distress suggested a component of movement in the southern direction. The west retaining wall is 9.1 m (30 ft) long and consists of six 61 cm (24 in.) diameter concrete piers that are 7.6 m (25 ft) deep with WF10 $\times$ 25 steel beams or piles. The piers are spaced 1.8 m (6 ft) apart and are installed through the fill and into bedrock. The south retaining wall is reportedly about 11 m (36 ft) long and consists of a row of similarly reinforced concrete piers installed to a depth of 6.1–7.6 m (20–25 ft). However, there is no above-grade projection of the south wall so the dimensions were not confirmed by inspection. The walls were designed for fill wedge movement believed to be occurring between 1.5 and 4.0 m (5 and 13 ft) deep near the crest of the fill slopes.

Performance of Cutslope from 1958 to 1978

In 1958 and 1959, at least three surficial slides occurred on the cutslope due to rainfall. During this time, joints in the rock units exposed in the upper two thirds of the cutslope opened due to stress relief from the first excavation, which facilitated infiltration of water into the joints. In 1965, horizontal drains were installed along the toe of the slope in an effort to reduce the pore water pressures in the cutslope and, therefore, increase its stability. In 1970, a major surficial slide occurred in the southwest corner of the cutslope. This slide was repaired in 1974 by excavating approximately 13,000 m^3 (17,000 y^3) of slide debris and replacing it with compacted fill. Prior to the slide repair, a state geologist (Heyes 1982) identified the surficial geologic features in the southwest corner of the cutslope. The most important geologic feature identified is a syncline that the second highway excavation

Table 1. Summary of Major Events or Movement at Residence and Cutslope

Year	Rainfall (cm/in.)	Event/movement observed	
		Residence	Cutslope
1955	51/20.0	Not constructed	First highway excavation
1956	99.1/39.0	Not constructed	First highway excavation
1957	57.9/22.8	Not constructed	First highway excavation completed
1958	116.6/45.9	Not constructed	Three surficial slides on cutslope
1959	36.8/14.5	Not constructed	Pavement heave below three surficial cutslope slides
1960	57.7/22.7	Not constructed	—
1961	41.7/16.4	Not constructed	—
1962	69.1/27.2	Not constructed	—
1963	90.4/35.6	Not constructed	Surficial slides on cutslope and pavement repair below previous three surficial slides
1964	48.8/19.2	Not constructed	—
1965	86.4/34.0	First residence constructed	Horizontal drain installed and 20,700 m^3 (27,075 yd^3) of soil at base of cutslope
1966	52.8/20.8	—	—
1967	99.1/39.0	—	Construction of Bay Area Rapid Transport (BART) widening (second excavation) begins
1968	51.3/20.2	—	BART widening
1969	94.2/37.1	Addition added at NW corner of first residence	BART widening and pavement patch triples in size
1970	75.9/29.9	—	BART widening completed and major slide appear at SW corner of cutslope
1971	69.9/27.5	—	—
1972	37.9/14.9	Drought	—
1973	98.6/38.8	First residence showing distress/cracks	Open rock joints appear 2/3 up SW corner of cutslope (see Fig. 2)
1974	93.0/36.6	First residence starts shows distress/cracks	Major repair of 1970 slide at SW corner of cutslope—12,900 m^3 (16,873 yd^3) of slide debris removed and open rock joints sealed —additional surficial slide occurs and two horizontal drains installed at toe
1975	67.3/26.5	—	—
1976	26.9/10.6	Two shear key walls installed at first residence and drought	—
1977	31.8/12.5	Drought	—
1978	106.7/42.0	—	1974 horizontal drains covered over by slide debris
1979	61.7/24.3	—	—
1980	86.4/34.0	—	Pavement heave below 1974 slide repair in SW corner
1981	45.7/18.0	Drought	—
1982	128.0/50.4	Severe distress in first residence—excavation shows well-defined failure plane inclined at 45 degrees toward SW corner of cutslope	Pavement heaves 17.8 cm (7 in.) below 1974 slide repair—highway inclinometer installed
1983	139.7/55.0	Severe distress in first residence	Highway inclinometer closed after 15.2 cm (6 in.) of shear displacement at 8.8 m (29 ft) depth
1984	69.3/27.3	Damage irreparable	—
1985	66.6/26.2	First residence sold for salvage and burns down	Severe heave observed in four westbound lanes
1986	102.1/40.2	—	Roadway heave reduced and patched
1987	43.2/17.0	Drought	Pavement heave and patch enlarged to 400 m (1,312 ft)
1988	45.7/18.0	Second residence completed—drought	—
1989	52.1/20.5	Drought	—
1990	49.3/19.4	Drought	—
1991	45.2/17.8	Drought	—
1992	59.4/23.4	Drought	—

Table 1. (*Continued.*)

Year	Rainfall (cm/in.)	Event/movement observed: Residence	Event/movement observed: Cutslope
1993	88.4/34.8	—	—
1994	47.5/18.7	Drought	—
1995	108.7/42.8	1.0–5.0 cm (0.5–2 in.) of settlement in SW corner of second residence	Pavement heave in four westbound lanes
1996	91.4/36.0	2.5–3.0 cm (1–1.2 in.) of settlement in SW corner of second residence—inclinometers installed	Pavement heave in four westbound lanes
1997	85.3/33.6	Significant movement measured in inclinometers, bucket auger borings confirm deep-seated (11.6 m, 38 ft) movement and extensive repair begins	Pavement heave in four westbound lanes
1998	120.9/47.6	Repair completed	Pavement heave in four westbound lanes
1999	72.4/28.5	—	Shoulder graded below SW corner

daylighted. The heavy dashed line in Fig. 2 parallels the axis of the syncline and the contact between the claystone below and the siltstone/sandstone unit above. The claystone below this dashed line represents the rock unit at the center of the syncline and the bedrock units to the north of this axis represent one side of the syncline. The axis of this syncline was reported by Heyes (1982) as "a potential natural failure plane." A comparison of aerial photographs from 1959 and 1973 shows that the syncline was not exposed after completion of the first excavation. As a result, one of the main effects of the second excavation was to expose the syncline and daylight the weak claystone of the Orinda Formation, which provided a potential deep-seated failure surface.

As shown in Fig. 2, there are also several shear zones present in the southwest corner of the cutslope. One large shear zone extends almost vertically and truncates the syncline axis. Another shear zone extends across the face of the cutslope as indicated by the dotted lines in Fig. 2. The presence of three claystone layers with different strike and dip orientations was reported by Heyes (1982). All of this evidence indicates a zone of complex shearing and fault movement in the southwest portion of the cutslope, which is the area that exhibited deep-seated movement. Both shear zones consisted of highly sheared claystone/gouge.

In 1974, the southwest corner of the cutslope experienced additional surficial sliding and one horizontal drain was installed on each side of the 1974 repaired slide mass. In 1978, these horizontal drains were reported buried due to additional surficial sliding. Because the surficial sliding is confined to the cutslope, it is not considered to be a cause of the residence distress.

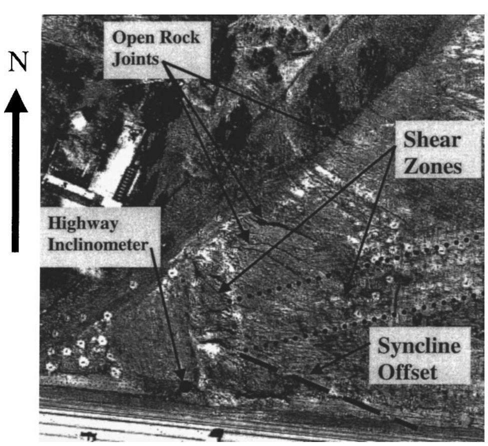

Fig. 2. Aerial photograph of southwest corner of cutslope that shows discontinuities in bedrock units in 1973

Performance of First Residence and Cutslope from 1978 to 1986

In 1978, approximately 107 cm (42 in.) of rainfall occurred but there was no reported movement at the residence, perhaps due to the exceptionally low rainfall the previous 2 years. Between 1978 and the 1982 winter, the annual rainfall was less than 89 cm (35 in.) and no distress was documented in the residence. The winters of 1982 and 1983 were unprecedented in the previous 30 years with annual rainfalls of approximately 128 and 140 cm (50 and 55 in.), respectively. As expected, there was severe distress in the first residence. In response to this distress, the contractor that installed the two retaining walls in January 1976 was summoned to explain the new movement in the residence. The contractor hand-excavated a 7.5 m (25 ft) deep, large diameter boring near the southwest corner of the residence to investigate the feasibility of underpinning the residence. The contractor's geotechnical engineer found a well-defined shear zone inclined at 45° toward the southwest corner of the cutslope in the large diameter boring (Hillebrandt 1982). The contractor also observed a vertical scarp in the crawl space underneath the residence (see Fig. 3). The contractor's geotechnical engineer concluded that a deep-seated bedrock landslide was causing the distress in the residence and the periodic distress in the residence was primarily due to the second highway excavation and rainfall. Therefore, the residence was not underpinned because underpinning would be ineffective against future deep-seated movement.

During the winters of 1982 and 1983, pavement heave was observed in the outboard highway lanes of the westbound direction near the southwest corner of the cutslope. (The outboard lanes correspond to the two northernmost lanes of the four westbound lanes of the highway and the inboard lanes correspond to the two southernmost lanes of the four westbound lanes and they are just north of the BART tracks.) No heave was observed in the westbound inboard lanes or the BART tracks located in the highway median. In fact, the BART system has never lost service in the area of the landslide. Fig. 1 shows the location of the heaved pavement, which is the area of a subsequent patch that was placed to remediate this pavement heave. The maximum heave is esti-

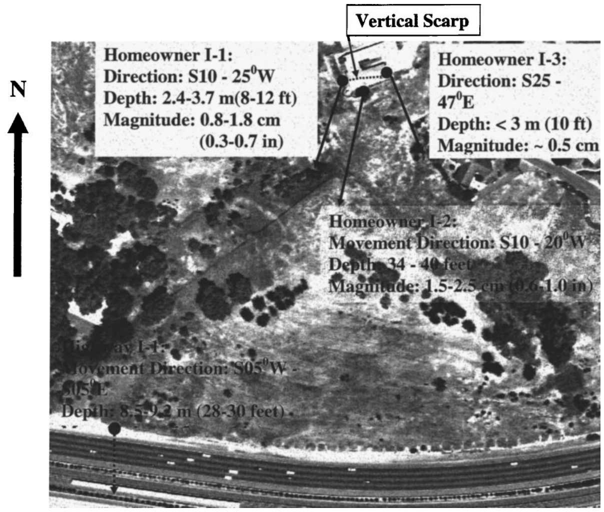

Fig. 3. Slope inclinometer data at highway and residence

mated to be 18 cm (7 in.) (Cassinelli 1985). More importantly, the magnitude and width of heave was the greatest at the toe of the cutslope and tapered out towards the south. The shape of the heave resembles a tongue shape and the heave terminates before reaching the westbound inboard lanes. This evidence suggests that a deep-seated landslide is daylighting in the westbound outboard lanes, causing these lanes and the shoulder to experience heave.

The drilling and sampling for installation of a slope inclinometer and two piezometers at the slope toe, described in the next paragraph, revealed that the claystone contains expansive clay minerals. The presence of expansive soils provided a possible explanation for the observed pavement heave, but was dismissed by the current investigation for the following reasons: (1) claystone heave probably would have been concentrated along the northern edges of the highway pavement where the claystone is exposed and where infiltration of water is the greatest and would not result in a tongue-shaped feature; (2) no heave was observed along the northern unpaved shoulder edge of the pavement; (3) there was no reduction in the pavement heave during the dry summer months; (4) the heave only occurred during years of substantial rainfall (greater than 89 cm or 35 in.); (5) swelling of the claystone would result in an increase in water content and a corresponding decrease in bearing capacity but no pavement distress due to a decrease in bearing capacity has been observed; and (6) installation of horizontal drains in the slope above the heaved pavement area implies the presence of a near surface source of water to initiate heave, yet no pavement distress had been previously documented. Therefore, the occurrence of pavement heave only during years of heavy rainfall is not likely caused by the presence of expansive claystone.

The pavement heave was located just below the area of the 1970 slide (see Fig. 1) described previously. Therefore, it can be inferred that either the repair for the 1970 slide was not effective or that a deeper landslide was present below the 1970 slide. To clarify the situation, the State Highway Department installed a 21 m (70 ft) deep slope inclinometer in the northern shoulder of the highway just below the 1970 slide area (see Fig. 2). The slope inclinometer was installed in July 1982 and little movement was observed in this inclinometer through December 1982. By February 3, 1983, 3.8 cm (1.5 in.) of abrupt shear displacement had occurred at a depth of 8.5–9.2 m (28–30 ft) and the groundwater level in the inclinometer was located 0.28 m (0.9 ft) below the ground surface. By March 18, 1983, 15 cm (6 in.) of shear displacement had occurred at the same depth and the water level in the inclinometer was located 0.18 m (0.6 ft) below the ground surface. It was possible to measure 15 cm of displacement with the inclinometer because the inclinometer has an inside diameter of 20 cm (8 in.). The increase in abrupt movement at the same depth is a manifestation of the presence of a well-defined failure surface. On April 29, 1983, the Highway Department reported that the inclinometer was blocked and thus inoperable. The inclinometer data are extremely significant because they show that deep-seated bedrock sliding is occurring underneath the cutslope. In addition, the data indicated that the deep-seated sliding was daylighting in the westbound outboard lanes instead of at the slope toe.

From the winter of 1983 until 1986, the annual rainfall did not exceed 89 cm (35 in.) and no further distress was observed in the first residence. In 1985, the first residence and property were slated for decommissioning by the owner's insurance company because the repair cost was deemed too great (Buller 1985). The residence was subsequently sold for salvage and then destroyed by a wildfire that summer before it could be moved offsite.

Performance of Second Residence and Cutslope from 1986 to 1999

In 1986, there was approximately 102 cm (40 in.) of rainfall. Because no residence was present in 1986, no distress was reported even though rainfall exceeded 89 cm (35 in.). However, the State Highway Department reported severe heave in the westbound outboard highway lanes (Cassinelli 1985). The pavement was lowered and a large pavement overlay was installed over the four westbound lanes in 1986. The longest portion of the overlay was placed in the northernmost westbound lane and the overlay tapered in towards the westbound inboard lanes.

From 1987 until the winter of 1993, the area experienced drought conditions with rainfall not exceeding 59 cm (23 in.) per year. In 1988, construction of the second residence was complete. The second residence was located in essentially the same building footprint as the first residence albeit with a different floor plan. The winter of 1993 produced a rainfall of approximately 88 cm (35 in.). This amount is right at the threshold value of 89 cm (35 in.) but it occurred after 6 drought years. As a result, no distress was reported in the second residence in 1993 and no severe pavement heave was observed. However, the winter of 1995 produced approximately 109 cm (43 in.) of rainfall. The second residence began experiencing significant distress during this rainy season. The distress was again concentrated in the southwest corner of the residence and approximately 1–5 cm (1/2–2 in.) of floor settlement occurred (Hallenbeck 1995), along with lateral displacement of the southwestern grade beam to the point that floor joists had less than 1.3 cm (0.5 in.) of bearing on the sill plate. Highway pavement heave also occurred in the same area as the 1986 pavement overlay.

The homeowner installed three slope inclinometers near the residence in January 1996 to investigate the depth and rate of movement. Fig. 3 shows the location of the three inclinometers and the direction, depth, and magnitude of movement observed in each of the inclinometers. The inclinometers were monitored from March 1996 until April 1997, at which time the residence was repaired (discussed subsequently) and the inclinometers decommissioned. The most important information derived from the inclinometers is the difference in the type, depth, and direction of movement observed at the western and eastern ends of the residence. Inclinometers I-1 and I-2, located near the southwestern corner of the residence, both show well-defined shear movement in the claystone bedrock in the southwesterly direction (see Fig. 3). The arrows in Fig. 3 correspond to the direction of movement in the inclinometers. I-1 shows a shallower shear movement (2.4–3.7 m) because it is closer to the vertical scarp under the second residence. This scarp is in the same location as the scarp observed under the first residence (see Fig. 3). The scarp exits the west side of the residence and extends to the west retaining wall that was constructed in 1976. However, I-2, which is located near the center of the scarp and is 6.1–7.6 m (20–25 ft) south of the scarp, shows a depth of shear movement of 10.4–12.2 m (34–40 ft). Because I-1 and I-2 show movement in the southwesterly direction (see Fig. 3), the scarp has to extend steeply into the bedrock, indicating that the movement is controlled by the claystone underlying the residence and not the surficial natural or fill soil underlying the residence.

The rainfall during the winter of 1996 was approximately 91 cm (36 in.). Pavement heave occurred again in the same area as the 1986 pavement overlay. The residence experienced additional distress and the homeowner drilled three 0.6 m (2 ft) diameter borings adjacent to the slope inclinometers. The large diameter borings revealed that the residence is underlain by highly sheared Orinda claystone. Well-defined shear planes in the claystone were observed and are in agreement with the depth of shear movement in I-1 and I-2. By following the shear plane from one side of boring BA-2 (adjacent to I-2) to the other side, the shear plane was found to be inclined at an angle of 45° to the southwest. This is in agreement with the 1976 findings of the contractor's geotechnical engineer and with I-1 and I-2 that the scarp extends steeply into the bedrock. The upslope projection of the steeply angled shear surface is coincident with the scarp location underneath the residence. This inclination of 45° was used in the stability analyses (described subsequently) to help locate and define the probable landslide failure surface causing the residence distress.

Another important piece of information obtained from the large diameter borings is the bedding planes in the claystone exhibited a dip/inclination to the southeast. This is significant because the movement direction (southwest) does not correspond to the bedding inclination (southeast). This indicates that the southwestern movement is being caused by external factors, such as the highway excavation and not natural bedding.

The rainfall during the winters of 1997 and 1998 was approximately 86 and 121 cm (34 and 48 in.), respectively. Pavement heave occurred again in the 1986 pavement overlay area. Additional movement of the residence was observed and the residence was deemed uninhabitable due to a maximum settlement of 15.2 cm (6 in.) in the southwest corner. At the urging of engineers and contractors, the homeowner undertook a large repair that involved the installation of 20 drilled piers to a depth of 12–15 m (40–50 ft) along the south side of the residence. The piers were connected with a 0.6 m (2 ft) thick integrating pier cap with tiebacks into the hillside to resist further movement of the vertical scarp. The residence has not experienced any major distress since completion of this repair in 1998 even though faulted cracks have developed in the slope outboard of the repair area, which is a scenario that was expected because the repair was only stabilizing the upper wedge of the steeply inclined failure mass.

Performance of West Retaining Wall from 1976 to 1999

The scarp observed under the second residence exited the west side of the residence, traversed the yard, and daylighted in the west retaining wall that was installed in 1976. A vertical offset of 2.5–5.0 cm (1–2 in.) is evident in the retaining wall. The location of the vertical offset in the wall is in agreement with the location of the scarp exiting the west side of the residence. More significantly, the southern three soldier piles translated to the southwest, causing a lateral extension of the wall and causing one of the lagging boards to fall out of the pile flanges. Because the soldier piles were not destroyed during the 1985 wildfire, they provide an indication of the direction and magnitude of movement since 1976. Displacement vectors estimated for the three southern solider piles indicate that extension of the retaining wall is occurring in the South 40° West direction. This direction is in excellent agreement with the I-1 and I-2 slope inclinometer data (see Fig. 3) and the observed distress. The magnitude of movement was evaluated by examining the wood lagging that separated from the flanges of the soldier pile between 1976 and 1998. This movement was estimated to be 10–15 cm (5–6 in.) assuming that the lagging was initially behind the flange and in near contact with the web of the southern solider pile.

The soldier piles were installed in concrete piers to a depth of 7.5 m (25 ft) and the beams are nearly vertical. Because the soldier piles are not tilted, this suggests that the southwesterly movement causing the extension of the wall occurs at a depth greater than 7.5 m (25 ft). This is in agreement with the slope inclinometer data and the fact that the southwestern portion of the residence was moving even though the south and west drilled pier walls were installed in 1976.

Summary of Data and Movement from 1958 to 1999

The collection of data and chronology of movement from 1958 to 1999 at the two residences and toe of the cutslope yield three main correlations. The first correlation is that the distress at the two residences is related to the distress observed in the westbound outboard lanes of the highway. The second correlation is that the residence and highway distress are related to periods of above average annual (89 cm/35 in.) rainfall. The third correlation is that the preceding two correlations are related to the completion of the second excavation that widened the highway and created space for the BART system. It will be shown in the stability analyses that the second excavation allowed a deep-seated failure surface to daylight in the westbound outboard lanes.

Stability Analyses to Verify Interpretations

Extensive slope stability analyses were conducted on three slope cross sections to verify the interpretations of the observed movements and evaluate the impact of the second excavation on the stability of a deep-seated failure surface. Because all three cross sections yield similar results, only the results from cross section $S2$-$S2'$ (see Fig. 1) are discussed herein. All of the cross sections were analyzed using Spencer's (1967) two-dimensional, limit equilibrium method because this method satisfies all conditions of static equilibrium. The slope stability program *XSTABL* Version 5 (Sharma 1996) was used to conduct all of the analyses.

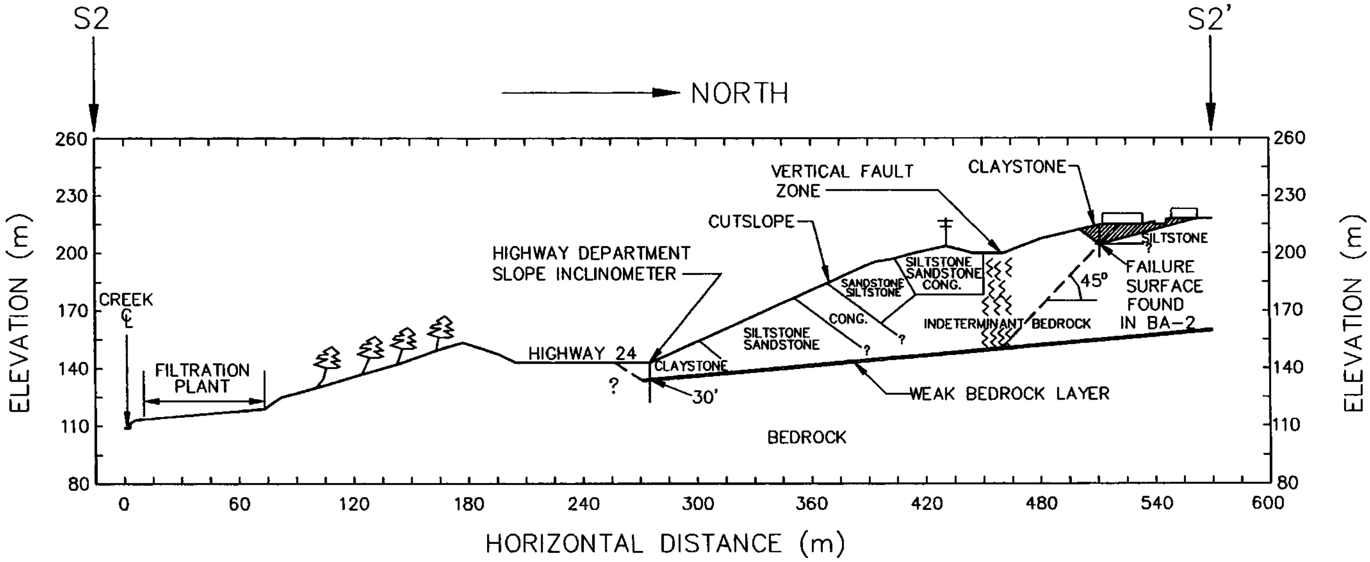

Fig. 4. Material profile used in stability analyses for cross section $S2$-$S2'$

Development of Cross Sections and Failure Mechanism

One of the main difficulties in determining the cause of the deep-seated movement that was impacting both residences was the lack of subsurface information between the residence and the toe of the cutslope. Only surficial (1–1.2 m or 3–4 ft deep) backhoe trenching was conducted at the top and along the cutslope by the State Highway Department, even though it was suggested by experts that bucket auger borings, rock corings, and slope inclinometers be installed to clarify the deep-seated movement. Based on surficial geologic mapping, topographic maps, aerial surveys, and subsurface information at the residence and the toe of the cutslope, the postexcavation cross section shown in Fig. 4 was developed for use in the stability analyses.

The next decision in evaluating the stability of the cutslope was the selection of the appropriate failure mechanism for the deep-seated movement. There are two main types of failure mechanisms: circular and translational. A circular failure mechanism occurs through deposits that are homogeneous. A translational failure mechanism occurs through a weak material that underlies a stronger material. The stronger material is essentially translating or sliding over this weaker layer. Based on the strong, dissimilar rock units exposed on the cutslope (see Fig. 4) and the presence of highly sheared and weak claystone under the residence and at the toe of the cutslope, it was concluded that the translational failure mechanism was applicable to this site.

Probable Failure Surface

Fig. 5 presents the probable translational failure surface, which extends through claystone to a depth of 12.2 m (40 ft) under the residence, into indeterminant bedrock below the claystone, into a deep, weak bedrock layer, and then along the weak layer until it daylights in the westbound outboard lanes. This failure surface

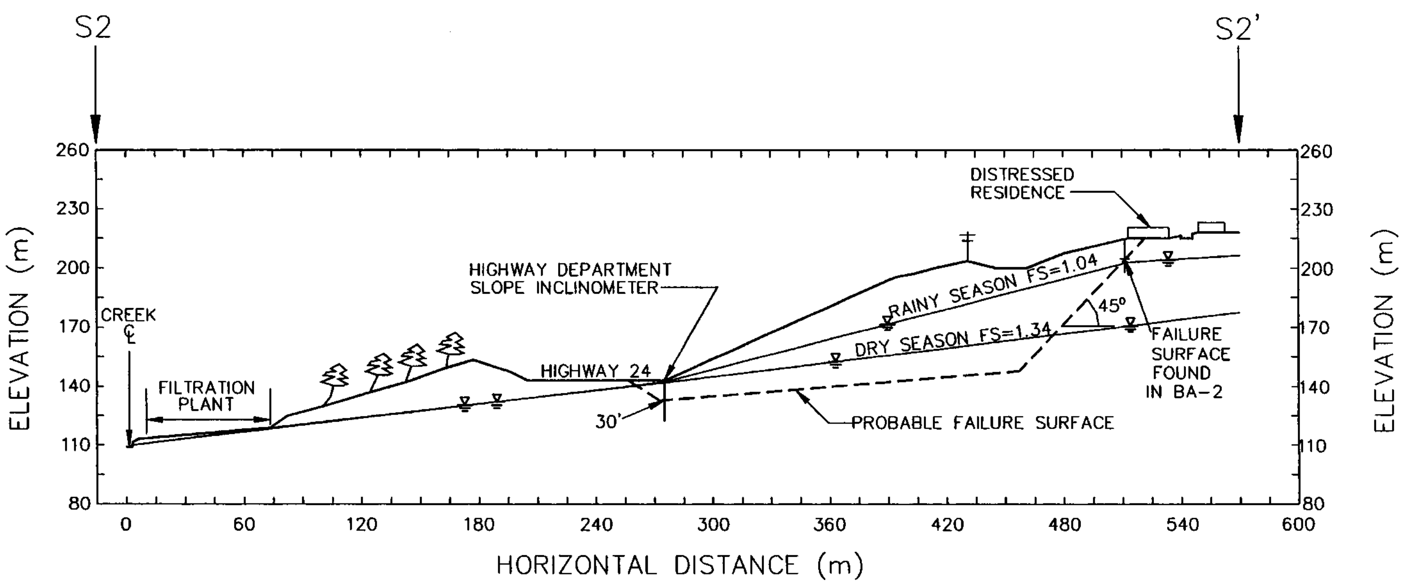

Fig. 5. Postexcavation stability analysis for cross section $S2$-$S2'$

accounts for all of the movements observed at the residence and toe of the cutslope because: (1) it passes 9.2 m (30 ft) below the northern highway shoulder to model the shear displacement observed in the State Highway Department's inclinometer; (2) it exits into the westbound outboard lanes to account for the observed periodic pavement heave; (3) it models the scarp observed under both residences and in the west retaining wall; (4) it corroborates the observed movements in the slope inclinometers installed at the residence; (5) it models the 45° angle of inclination of the shear surface observed in bucket auger boring BA-2; and (6) it models the location of the shear surface at a depth of approximately 12.2 m (40 ft) in BA-2.

For the stability analyses, this weak layer was assumed to be 1.5 m (5 ft) thick and exhibit the same material properties as the claystone. Of course, there is significant uncertainty in both the probable failure surface described above and use of the claystone material properties for the hypothesized weak layer because of the limited subsurface information. However, the presence of a syncline and the claystone underlying the dipping bedrock layer on the cutslope, suggest a complex and highly sheared geologic condition (see Figs. 2 and 4). For example, the syncline axis extending into the hill could be allowing the probable failure surface to reach the claystone in the cutslope area and may even connect with the claystone underneath the residence. As a result, the actual failure surface may differ from the probable failure surface even though the probable failure surface yields reasonable values for the factor of safety.

Stark and Arellano (2000) show that a three-dimensional (3D) analysis can result in a factor of safety that is 1.6 times greater than the two-dimensional (2D) value. This is based on a $1H:1V$ slope and a slide width to height ratio of 2. The difference in the 2D and 3D factors of safety are one half this difference for this case history because a large gully is present on the west side of the slide mass (see Fig. 1) and thus there is little, if any, side resistance along the west side of the slide mass. As a result, a 2D analysis was used in this investigation.

In summary, the actual failure surface may differ from the probable failure surface assumed herein. However, the probable failure surface does yield reasonable values of factors of safety and allows a comparison of the stability before and after the roadway excavations with and without rainfall.

Development of Groundwater Surface and Engineering Properties

The observations used to estimate the groundwater surfaces shown in Fig. 5 are the presence of a creek at the toe of the original slope, piezometric measurements made by the State Highway Department at the cutslope toe that show the water level to be near the ground surface, and water level measurements in the bucket auger borings at the residence that show water levels of 9.2–13.1 m (30–43 ft) below the ground surface. The groundwater surface during the dry season is based on the same observations described above except that the slope of the water surface between the creek and the cutslope is extended to the location of the residence.

The engineering properties of the subsurface materials, i.e., unit weight and shear strength, used in the stability analyses are presented in Table 2. It can be seen that two material types, bedrock and claystone, were used in the analyses. The bedrock material properties were assigned to all of the rock types except for the claystone and the weak bedrock layer shown in Fig. 4. The bedrock material properties were developed by the State Highway

Table 2. Material Properties Used in Stability Analyses

		Shear strength parameters		
Material description	Moist unit weight (kN/m^3/pcf)	Cohesion intercept (kPa/psf)	Friction angle (degrees)	References
Bedrock	22.0/140	12/250	45	State Highway Department
Claystone	20.4/130	Residual envelope (0)	Residual envelope (9–13)	Stark and Eid (1994)

Department. The claystone residual failure envelope was estimated using a liquid limit of 76% measured using samples obtained from the borings near the residence, a clay-size fraction greater than 50%, and the correlations presented by Stark and Eid (1994), which are described in the next paragraph. A residual shear strength, instead of a peak or fully softened shear strength, is used to model the claystone strength. The drained residual shear strength is applicable to slopes that have undergone shearing previously, such as old landslides or soliflucted slopes, bedding shears in folded strata, sheared joints or faults, and failed embankments. This differs from the peak or fully softened shear strength that is applicable to slopes that have not undergone previous shearing or landsliding. The extensive shearing, faulting, and landsliding clearly indicate that a residual strength should be used to model the shear strength of the highly sheared claystone underneath the residence and at the cutslope toe because of the presence of the syncline, shear zones visible on the cutslope, and prior sliding at the slope toe. A peak shear strength is used for the other bedrock materials because these materials do not exhibit a large difference between the peak and residual strength because of their granular nature. However, claystone can exhibit a residual strength that is significantly less than the peak value because of change/removal of the clay structure and orientation of the clay particles parallel to the direction of shear (Stark and Eid 1994).

The liquid limit of the sheared claystone obtained from the large diameter borings was measured by GeoFocus Geotechnical Engineers (1997) to be 76%. Using a liquid limit of 76% and a clay-size fraction of greater than 50%, the residual friction angle was estimated to vary from 13° at an effective normal stress of approximately 96 kPa (2,000 psf) to 9° at an effective normal stress of approximately 700 kPa (14,600 psf) using the relationship presented by Stark and Eid (1994). To accurately model the stress-dependent residual strength in the stability analyses, the entire residual failure envelope of the claystone, estimated using Stark and Eid (1994), was input into *XSTABL* as shown in Table 2.

Analysis of Probable Failure Surface

Fig. 5 presents the cross section for the analysis that represents the conditions during a rainy season, i.e., a high groundwater surface, for the probable translational failure mechanism. This factor of safety (FS) of 1.04 (see Table 3) indicates that the slope is metastable and it is in excellent agreement with the following field observations: (1) pavement heave and residence distress only occurring during or at the end of a rainy season; (2) the slope movement is slow; (3) the magnitude of movement at the residence is less than 5 cm (2 in.) per rainy season; and (4) the small movement does not result in a large scarp that is visible on the exposed ground surface. The FS for the same failure mechanism

Table 3. Results of Stability Analyses

Excavation condition	Season	Factor of safety
Post-Bay Area Rapid Transport	Rainy	1.04
(after 1970)	Dry	1.34
Preinitial excavation	Rainy	2.24
(before 1955)	Dry	2.52

with groundwater surface conditions during a dry season, i.e., low groundwater surface, is 1.34 (see Fig. 5). This FS is in excellent agreement with field observations that pavement heave and residence distress do not occur during the dry summer months.

The impact of the entire highway excavation on the probable failure surface is illustrated in Fig. 6. The shear surface presented in Fig. 6 is labeled critical shear surface because a search was conducted with the probable failure surface in Fig. 5 being the starting point for the search. The groundwater surface prior to excavation was estimated using the water surface during the rainy season but lowered slightly near the residence to reflect an increased amount of runoff prior to the excavation. It can be seen from Table 3 that the factors of safety for the pre-excavation condition during the rainy and dry seasons are 2.24 and 2.52, respectively. Therefore, the highway excavations had a destabilizing effect on the slope and both of the residences probably would have performed satisfactorily had it not been for the highway excavations. These factors of safety are significantly higher than the postexcavation condition because: (1) the probable translational failure surface must pass through significantly more material to exit the slope; (2) the additional material at the slope toe (see shaded area in Fig. 6) results in a higher effective stress on the failure surface, which increases shear resistance and stability; and (3) the presence of the additional material promoted more surface runoff, which probably resulted in a lower groundwater surface near the cutslope toe than that assumed in Fig. 6.

Evaluate Alternatives

Stability analyses were also conducted to evaluate alternate explanations for the observed movements at the cutslope toe and the residences. The only alternate hypothesis is that the movements at the cutslope toe and the residences are not related. Therefore, the possibility of the movement at the toe and top of the slope occurring independent of each other was investigated.

Independent Movement of Cutslope Toe

A number of explanations for the distress and pavement heave at the cutslope toe were considered including expansive soils, differential settlement caused by a transition from natural to fill material, and a landslide confined to the cutslope. As noted previously, the distribution and magnitude of the heave, occurrence of heave with only large rainfall events, no observed reduction in heave during dry seasons, and no observed reduction in pavement bearing capacity suggest that the pavement distress in the westbound lanes is not caused by expansive soils. In addition, the area of pavement heave is significantly east of the transition from natural to fill material along the highway alignment and, thus, the differential movement cannot be explained by the location of the cut/fill transition at the western edge of cutslope (see Fig. 1).

Extensive stability analyses were conducted to investigate the possibility of a landslide confined to the cutslope toe area to explain the observed pavement movements. Fig. 7 provides a summary of the stability analyses at the toe of the cutslope. The boundary conditions for these analyses include: (1) the toe of the failure surface daylighting or exiting in the westbound outboard highway lanes; (2) the failure surface passing through a depth of 9.2 m (30 ft) at the location of the inclinometer installed by the State Highway Department; and (3) the top of the failure surface exiting on the cutslope. It can be seen in Fig. 7 that these boundary conditions are satisfied using both circular (failure entirely in the claystone) and noncircular failure surfaces (heterogeneity of the materials in the slope toe area). The groundwater surface es-

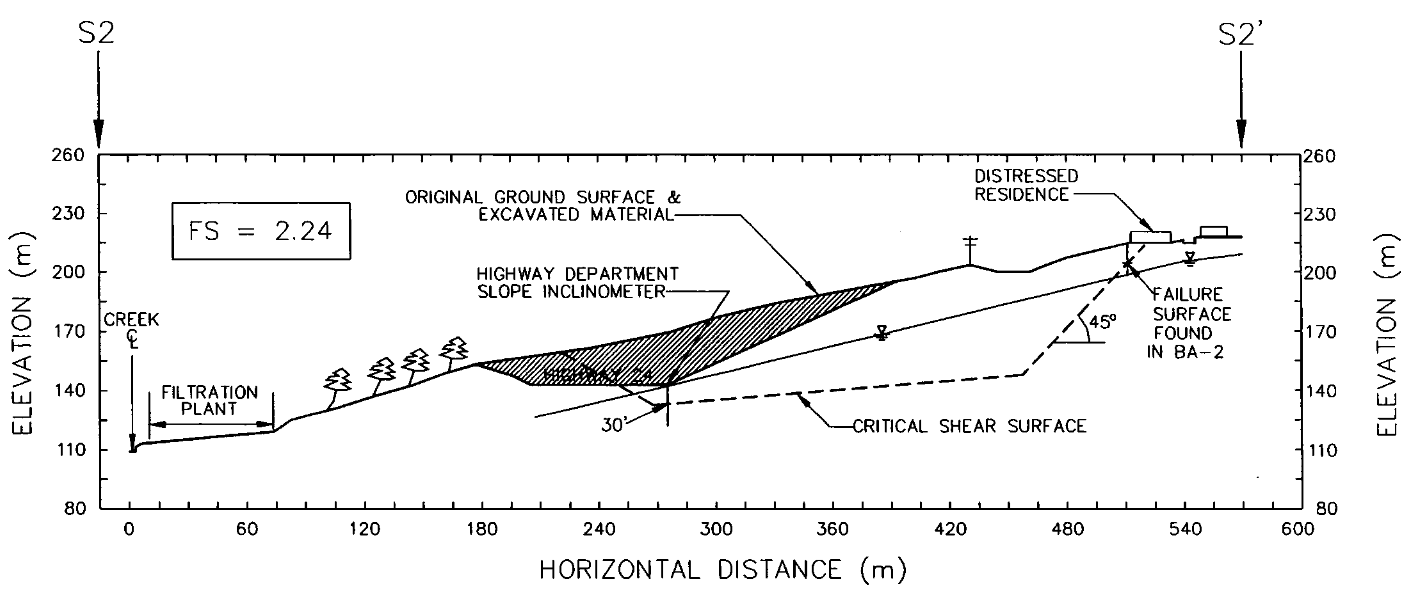

Fig. 6. Pre-excavation stability analysis for cross section $S2$-$S2'$

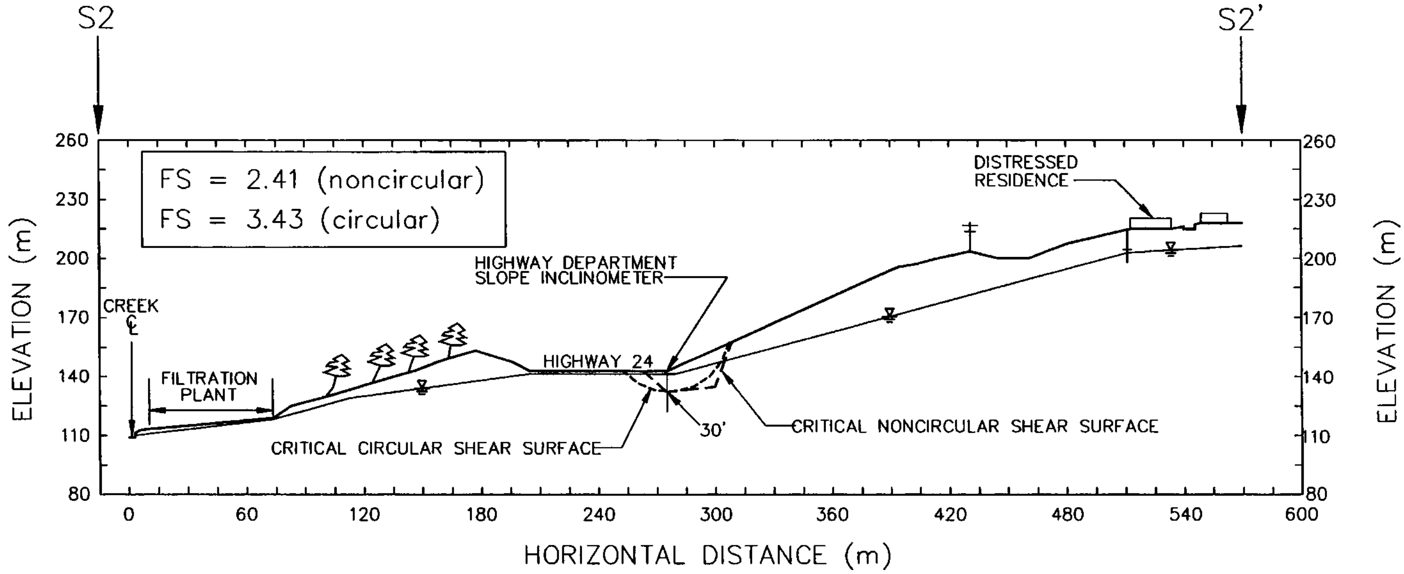

Fig. 7. Stability analyses at toe of cutslope for cross section $S2$-$S2'$

tablished for the rainy season (see Fig. 5) was used for these analyses. The material properties presented in Table 2 were also used in these analyses. It can be seen that the circular and noncircular failure surfaces yield factors of safety that are significantly greater than unity (3.43 and 2.41, respectively), even for the high groundwater surface condition. Therefore, a landslide limited to the cutslope area does not appear likely. The lack of a scarp on the cutslope confirms that these failure mechanisms are unlikely and thus do not explain the historical pavement heave.

If the residual failure envelope in Table 2 is assigned to all of the material at the cutslope toe, the resulting factors of safety for the critical circular and noncircular shear surfaces are 1.58 and 0.71, respectively. Therefore, a circular failure surface is not likely because the factor of safety is substantially greater than unity. The lack of a scarp on the cutslope and the absence of catastrophic movement suggest that a factor of safety of 0.71 for the noncircular shear surface implies that a noncircular landslide limited to the toe area is not likely.

Independent Movement at Top of Slope

A number of possible explanations for the movement at the residences were considered. These explanations include expansive soils underlying the residence, differential settlement within the 1.5–2.5 m thick (5–8 ft thick) compacted fill wedges constructed to create a level building pad, differential settlement between the bedrock and fill material underneath the residence, a landslide confined to the southeast or southwest erosional gullies (see Fig. 1), and failure of the 1976 shear key retaining walls.

Expansive soil was not considered a feasible cause because: (1) the second residence was founded on piers into bedrock and had a crawl space under the house; (2) there is a characteristic lateral component of movement in the documented house damages and a vertical and lateral offset of the scarp under the residences that are inconsistent with expansive soil; and (3) expansive soil cracking, i.e., alligator-type cracking, was absent in the crawl space. Finally, the second residence was surrounded by a perimeter subdrain and good drainage practices were incorporated in the second residence to maintain a uniform soil moisture content in the crawl space.

Differential settlement within the compacted fill material was ruled out as a cause because the maximum depth of fill is less than 1.5–1.8 m (5–6 ft) under the house. Because there is no differential settlement of the southeastern corner of the residence and slope inclinometer I-3 at the east side of the residence shows a maximum movement of approximately 0.5 cm (0.2 in.) at the ground surface, it may be assumed that fill settlement and/or lateral creep does not adversely impact the residence. The cut/fill transition underneath the residence also does not appear to be causing the detrimental differential settlement because the scarp is located to the north of this transition.

Extensive stability analyses were conducted to investigate the possibility of a landslide confined to the southeast or southwest erosional gullies. Because inclinometers I-1 and I-2, floor surveys of both residences, bucket auger borings, the west retaining wall, and locations of scarps under the residences indicate movement in the southwest direction, cross section $S2$-$S2'$ was used for stability analyses in the vicinity of the residence. No evidence of sliding was observed in the southeast erosional gully.

Fig. 8 provides a summary of the stability analyses near the distressed residence. The boundary conditions for these analyses include: (1) the toe of the failure surface exiting in the southwest direction and before the cutslope because there is no evidence of a landslide toe in the upper portion of the cutslope; (2) the failure surface passing through a depth of 11.6–12.2 m (38–40 ft) at the location of BA-2; (3) the steeply inclined scarp existing under the residence; and (4) movement occurring in the southwest direction. It can be seen in Fig. 8 that these boundary conditions are satisfied and circular and noncircular shear surfaces were considered. The groundwater level established for the rainy season (Fig. 5) was also used for these analyses. A circular failure surface was considered because the failure surface might remain primarily in the claystone that was encountered at the residence. A noncircular shear surface was also considered because of the heterogeneity of the materials in this area of the slope (see Fig. 4). The material properties presented in Table 2 were used in these analyses; however, the majority of the critical shear surfaces are located in claystone.

Fig. 8 shows that the critical circular and noncircular shear

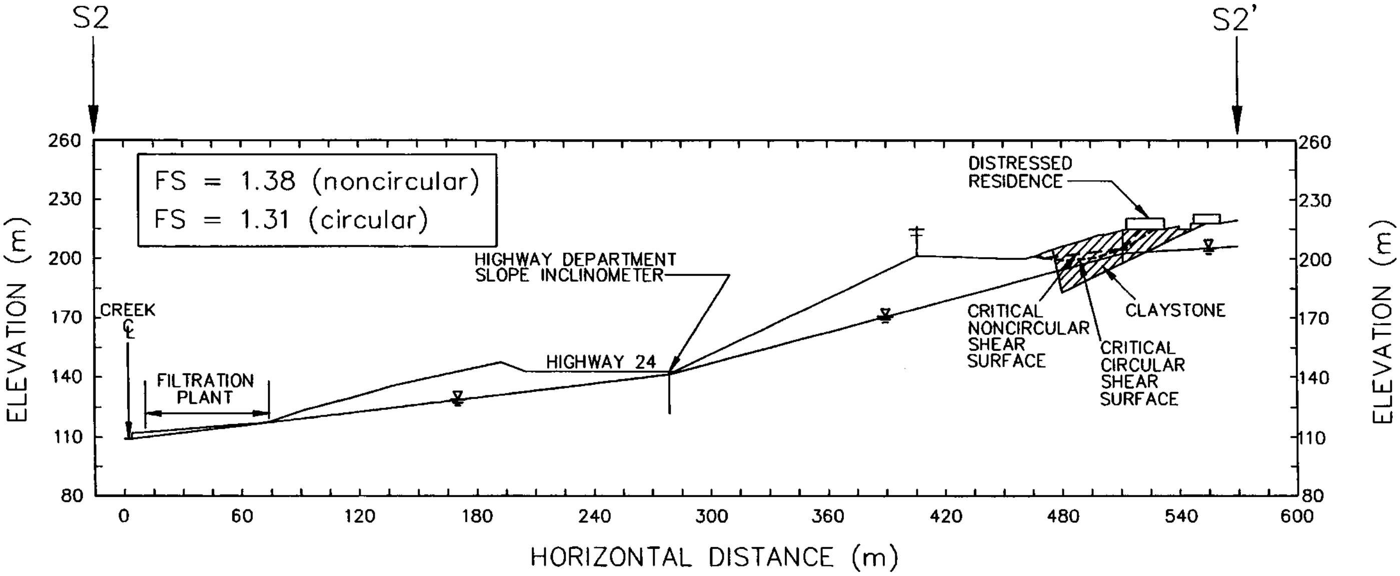

Fig. 8. Stability analyses near distressed residence for cross section *S*2-*S*2′

surfaces yield factors of safety that are greater than unity (1.31 and 1.38, respectively) for the high groundwater level condition. These factors of safety are too high to account for the movement observed in the residences and slope inclinometers. Similar stability analyses conducted with the groundwater level raised to the ground surface (a worst-case scenario) yield factors of safety significantly less than unity (0.75 and 0.77, respectively). These factors of safety are too low to explain the slow, episodic movement observed in the residence and slope inclinometers. Furthermore, at no time in any of the subsurface explorations (inclinometers, borings, bucket auger holes, and deep pier excavations for repairs) was groundwater encountered near the ground surface, which makes this an unreasonable scenario. Therefore, it is concluded that a small landslide in the vicinity of the current residence could not explain the slow and episodic distress observed in both residences.

Finally, failure of the south shear key wall was considered a possible cause of the damages in the first residence following the 1976 repairs and the damages observed in the second residence. The piers of the shear key were analyzed using the computer program *COM624*, which was developed to estimate the stresses and deflections in laterally loaded piles (Reese and Sullivan 1983). The soldier piles were analyzed for the worst case condition of fill wedge movement occurring at a depth of 4 m (13 ft) and no passive support existing on the downslope side of the soldier piles. The lateral load imposed by movement of the fill wedge was taken as an equivalent fluid pressure of 11.8 kN/m^3 (75 pcf), which corresponds to a material with a friction angle of 18° using Rankine's equation for active earth pressure. Under this worst-case condition, the tops of the soldier piles would undergo a lateral deflection of approximately 3.8 cm (1.5 in.). Considering the location of the shear key wall relative to the residence, approximately 4.6–6.1 m (15–20 ft) to the south, it appears unlikely that these small wall deflections would account for the lateral [5–7.5 cm (2–3 in.)] and vertical [2.5–5 cm (1–2 in.)] offsets observed in the scarp under the second residence. Rather, it is believed that the 1976 south shear key wall is located above the deep, steeply inclined failure surface observed and measured in inclinometer I-2 and bucket auger boring BA-2, and thus is moving along with the slide mass. Based on these evaluations and the depth and locations of the failure surface at the south side of the house, it is concluded that failure of the 1976 south shear key wall was not the cause of the episodic movements in the two residences.

Conclusions

Observations, data, and analyses used to investigate the cause of distress at a single-family residence in the San Francisco Bay area are presented. The only mechanism that explains all of the movements observed in or near the two residences that occupied the site and at the toe of the cutslope is a deep-seated translational failure surface that exists beneath the cutslope (see Fig. 5). The toe of the failure surface exits in the westbound outboard highway lanes and the scarp daylights or exits under the southwest corner of the residence. Movement along this translational failure surface is only activated when the factor of safety is at or near unity. A review of the observed movements and rainfall data suggests that the factor of safety is reduced to at or near unity in years that the annual rainfall exceeds 89 cm (35 in.). The slide mass is so large that the movement is slow and usually limited to less than 5 cm (2 in.) per year at the residence. The movement usually occurs near the end of a rainy season, because time is required for the groundwater surface to rise in response to the increase in rainfall.

This case history illustrates some of the ramifications of large highway excavations in natural slopes, e.g., exposing significant geologic features such as shear zones, faults, syncline, and folds, the importance of investigating signs of movement both at the top and toe of a slope, the importance of rainfall on the movement of a slide mass, and that large slide masses can undergo slow and limited episodic movement. This case history also illustrates the importance of an adequate subsurface investigation and consideration of the history of previous structures during the design phase to thoroughly assess anticipated slope behavior and its possible effects on surrounding structures.

Acknowledgments

The contents and views in this paper are those of the writers and do not necessarily reflect those of the State of California or the homeowner.

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