Case Study

Fill Placement on Slopes Underlain by Franciscan Mélange

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Abstract: Distress in two housing developments was caused by a deep bedrock landslide triggered by placement of a large upslope fill. The large fill was placed to create a visual barrier between the upslope development and downslope housing developments and to balance the cut and fill quantities for the upslope site. This case history presents some of the ramifications of fill placement on natural slopes surrounded by urban areas, such as overstressing underlying weak material that may exist below the depth of borings typically conducted for single family residences and office complexes; the importance of surface and subsurface information in a formation known locally as Franciscan complex/mélange (which is a block-in-matrix rock formation common to the area); the shear behavior of serpentinite, which is part of the Franciscan complex; and the importance of natural and man-made changes to a slope, such as rainfall, surficial grading, home construction, and fill placement. It also illustrates the importance of locating the critical slope cross section before construction and the proper use of back-analyses in a landslide investigation. **DOI:** 10.1061/(ASCE)GT.1943-5606.0000394.

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Introduction

Cut and fill operations are routinely required to facilitate hillside development. Because these operations can affect hillside stability, the design process should address the potential impact of these operations on the surrounding landscape and developments. This involves considering the effect of hillside development on structures upslope and downslope of the proposed development, because frequently hillside site investigations only consider the impact of the cut and/or fill to that project site.

A factor complicating hillside development is the usually significant cost of disposing of excess cut or excavated material from the project. Environmental regulations usually make offsite disposal of large amounts of cut material difficult and expensive. As a result, there is usually an incentive to "balance the site," which means balancing the amount of cut material with the amount of fill material required for the development. Balancing a site can lead to placement of a large amount of fill at a single location on a natural slope, as occurred in this case.

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Purpose and Scope

The purpose of this paper is to (1) describe the factors that caused a bedrock landslide in a natural slope underlain by the Franciscan complex/mélange that distressed two downslope housing developments; and (2) present suggestions for avoiding such a situation in the future. Figure 1 presents an aerial view of the two housing developments (Vista and Knolls), the subsequent upslope development referred to as the BC development, and an outline of the slide mass. Only a portion of the housing units in the Knolls and Vista developments are shown in Fig. 1.

The details of the large fill and some surficial grading that occurred at the top and bottom of the slope, respectively, are presented herein and illustrate the need for (1) adequate surface and subsurface investigation in a variable and problematic formation like the Franciscan complex/mélange; (2) proper material characterization of the materials influenced or stressed by the proposed development; and (3) local and global stability analyses to evaluate fill placement and grading on hillside stability. This paper also presents laboratory shear strength data and back-analysis results for the serpentinite because of the limited shear strength information available and typical slide geometries for this block-in-matrix rock formation to guide future stability analyses for slopes in this formation.

Regional and Local Geology

The site is underlain by complex soil and bedrock conditions. The soils overlying the bedrock involve many surficial landslides, and thus there are a number of colluvial scarps and colluvial soil deposits in the housing developments and the BC development site. The colluvial deposits consist of unconsolidated clay, silt, sand, and some gravel derived from weathering of the underlying bedrock materials that have been transported by downslope movement. The colluvial slides have occurred and are occurring above the bedrock and thus are independent of the 1996–1997 slide movement, which was located in the underlying bedrock by slope inclinometer

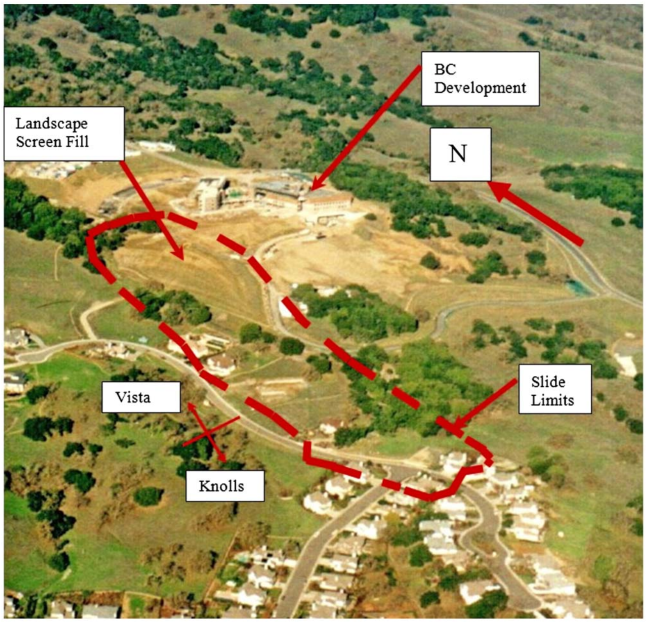

Fig. 1. Aerial view of housing developments, BC development, and an outline of the slide mass

data. Researchers, e.g., Berkland (1969), Blake et al. (1974), and Rice (1975), have also reported a number of large landslides in the area including deep-seated bedrock landslides that underlie the surficial colluvial slides.

The predominant bedrock units underlying the near surface soils are tertiary volcanics and the Franciscan complex/mélange. The volcanic rocks overlie the Franciscan complex in the upper portion of the slide limits shown in Fig. 1. The volcanic rocks generally consist of hard andesitic rocks and a weaker agglomerate of ash and block flow rocks. Slope inclinometer data shows that landsliding occurred below the volcanic rocks and in the Franciscan complex/mélange. The Jurassic–Cretaceous rocks of the Franciscan complex include sandstones, claystones, mudstones, shale, conglomerates, and serpentinite. The Franciscan complex is frequently referred to as a mélange, or mixture, because the deposit was formed near the forward edge of a subduction plate boundary (Goodman 1993). The hills in which this site is located were created by an east-dipping subduction zone between the Pacific and North American tectonic plates (Wakabayashi 1999). The intense mixing, shearing, and deformation of the bedrock materials is explained by the overriding North American Plate scraping sediment and rock off the subducting Pacific Plate. This resulted in a jumbled mix of highly sheared and deformed bedrock (Scholl et al. 1980). Over time, this highly sheared and deformed rock accumulated enough volume in a small area to create the hills along the California coast north of the Golden Gate Bridge and San Francisco Bay, where this site is located.

The jumbled and sheared nature of the Franciscan complex/mélange presents a challenge for engineers because it is difficult to predict the engineering properties of the material that would be encountered at a particular site without a substantial amount of subsurface exploration and testing (Goodman 1993). With an increasing tendency to reduce subsurface investigation costs, a site underlain by the Franciscan complex presents a challenge for hillside development design, as shown by the following case history.

One material frequently found in the Franciscan complex/mélange that usually presents a design and/or slope stability problem in this area is serpentinite. The serpentinite at the BC development site consists of large intact rocks surrounded by a clay matrix.

Table 1. Length and Width of Serpentinite Landslides

Length (m)	Width (m)	Length/width ratio	Reference
1615	460–700	3.5–2.3	Dickinson (1966)
1070	60–155	17.5–7	Dickinson (1966)
610	120–215	5–2.8	Dickinson (1966)
1525	305–610	5–2.5	Dickinson (1966)
700	250	2.8	Phipps (1984)
460	120	3.8	Subject slide

As a result, serpentinite is referred to as a block-in-matrix rock formation (Goodman and Ahlgren 2000). Frequently, the percentage of the clay matrix is such that the engineering properties of the serpentinite are controlled by the clay matrix instead of by the intact blocks. A number of researchers, e.g., Dickinson (1966), Moiseyev (1970), Blake et al. (1974), Cowan and Mansfield (1970), and Phipps (1984), have reported large landslides involving serpentinite. Table 1 shows that the length-to-width ratios of prior serpentinite landslides range from 2.3 to 17.5. The length-to-width ratio of the slide described in this paper—3.8—is also presented in Table 1 and is in agreement with previously reported serpentinite slides, so such a slide is not completely unexpected. In addition, there was visible evidence of serpentinite slides in roadcuts along the eastern edge of the project area during this investigation. Investigation of prior landsliding in serpentinite throughout the San Francisco Bay area, and in particular, in the project area, is recommended to provide better understanding of the potential for landsliding prior to developing at a site.

Depth of Influence

In general, a designer should determine the depth of influence of the cut, fill, and other construction activities proposed at a site and design the subsurface investigation to sample and test the material that will be impacted by the development. For example, the maximum depth of influence of the landscape screen fill shown in Fig. 1, created to provide a visual barrier between the BC development and the downslope developments, is estimated to be about 110 m, using Boussinesq stress distribution theory for an inclined embankment loading (Holtz and Kovacs 1981). Using two triangularly loaded areas of limited length and the law of superposition, the depth that experiences at least 10% of the applied vertical stress of the landscape screen fill is about 110 m. The two triangularly loaded areas used to model the fill had a width and length of 126.5 m and 55 m, respectively.

However. none of the almost 80 borings drilled across the BC development exceed a depth of about 13 m, even though the depth of influence of the landscape screen fill is about 110 m. The slope inclinometers installed after homeowner complaints started in December 1996 show a depth of sliding of 35–40 m. Thus, none of the borings drilled within the slide limits were deep enough to reach the problematic layer, i.e., serpentinite (discussed subsequently). As a result, the serpentinite layer stability analyses did not consider this layer.

Landslide Chronology

Between 1988 and 1989, a housing development with about 50 single-family units was completed on an undeveloped hillside near Novato, California and is referred to in this paper as the Knolls (see Fig. 1). Novato is located about 40 km north of the Golden Gate Bridge near San Francisco. An 11-unit housing development was constructed upslope of the Knolls and is referred to here as the Vista development. Only seven of the 11 Vista lots were developed at the time of the 1996 landslide.

Neither housing development experienced any distress until late December 1996. In June of 1996, a large fill was started for the BC development just above the Vista development. The reported purpose of the large fill was to create a visual barrier or landscape screen between the BC development and the downslope developments. Another possible benefit of the large fill was to balance the cut and fill quantities of the BC development site. Fill placement for the landscape screen fill ceased in late December 1996 with the onset of homeowner complaints of damage, even though the fill had not reached full height. The final height of the landscape screen fill at the time of the landslide is not known and was probably related to the cut material that was generated by the development. Figure 1 shows the landscape screen fill at the upper end of the limits of the landslide. The surface area of the landscape screen fill was approximately $61,000\ m^2$, and the estimated volume of the landscape screen fill was $76,600\ m^3$. The estimated volume of the landslide mass was about 2.0 million m^3.

There was no change in the slope geometry after completion of the Knolls housing development in 1988 and the seven units in the Vista development until the BC development commenced in late 1995. In particular, there was no significant fill placement related to the BC development until June 1996 with the start of the landscape screen fill.

Causation Investigation

A causation investigation was initiated by the BC and Knolls developments to determine the cause of the landslide because affected homeowners sued BC development, and BC development sued the Knolls development. With damages reaching about $20 million, a more extensive investigation was conducted for litigation purposes. The main steps in the causation investigation were: (1) evaluate the impact of rainfall; (2) develop a number of cross sections to understand the variability and geometry of the subsurface materials; (3) determine the failure mechanism or failure surface from surface observations, subsurface information, and slope inclinometer results; (4) develop relevant material properties; (5) develop groundwater levels at the time of sliding; (6) perform a back-analysis to locate the critical cross section; (7) use the critical cross section and back-analysis to estimate the mobilized shear strength of the weak layer and compare it with laboratory test results, empirical correlations, and field observations to ensure accuracy of material properties; and (8) conduct stability analyses to determine the impact of various activities on slope stability, such as surficial grading and landscape screen fill.

Rainfall

Rainfall records prior to and during the year that slope movement was first reported were reviewed to determine if rainfall contributed significantly to the slide. In general, the depth to groundwater is related to the amount of precipitation, i.e., high levels of precipitation usually result in higher levels of groundwater being measured and vice versa. The rainfall records from the nearby Petaluma Fire Station are summarized in Fig. 2. The city of Petaluma is located about 18 km north of Novato, California. Figure 2 presents the yearly rainfall total from July 1 through June 30. The Knolls housing development, which suffered the most damage, was completed between 1988 and 1989. Between 1989 and the 1992–1993 rainy season, the area received below-average rainfall. The average rainfall for the Petaluma Fire Station from 1948–2001 was 64 cm. In the 1992–1993 rainy season, 77 cm of rainfall, or 13 cm above the average rainfall, occurred. The area also experienced above-average rainfall in 1994–1995 (113.2 cm) and 1995–1996 (80.7 cm) without any reports of distress, even though the 1994–1995 rainfall exceeded the 53 year average by 49.2 cm. The reports of distress started in late December 1996 during a year of essentially average rainfall (63.8 cm).

Before the reported distress in late December 1996, more rainfall occurred in three prior years and no landslide occurred. The major difference between these years and 1996–1997, when slope movement did occur, is that the landscape screen fill had not been constructed. In the 1996–1997 rainy season, less rainfall occurred than in several previous years, but a landslide occurred. Thus, it was postulated that rainfall alone was not the landslide trigger and additional investigation was undertaken.

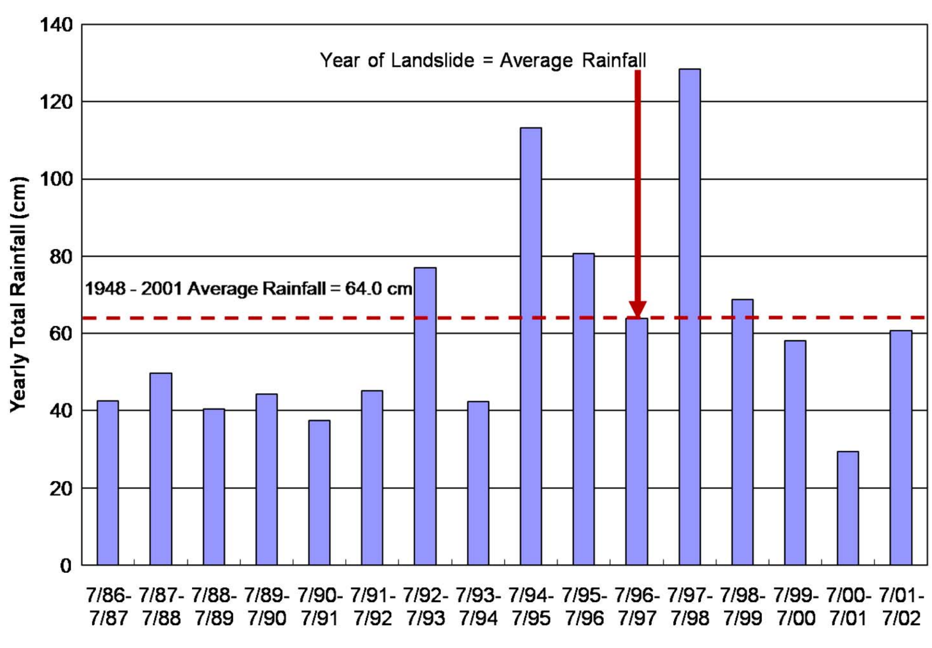

Fig. 2. Annual rainfall data from nearby fire station

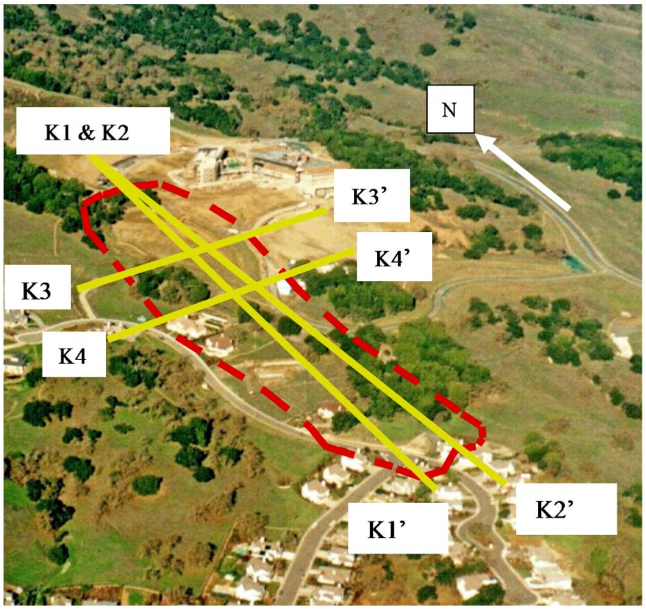

Fig. 3. Aerial view of the landslide illustrating four cross sections considered

It was considered that the rainfalls of 1994–1995 and 1995–1996 could have contributed to the slope movement that started in October 1996. Piezometers installed after the slide in February 1997 show a drop in groundwater levels during the dry summer months of June through September. More importantly, inclinometer data in the toe area of the slide after removal of the landscape screen fill shows a restart of movement during the months of high rainfall in 1998–1999, 1999–2000, 2000–2001, and 2001–2002 (see Fig. 2). During the dry summer months, movement was not observed in these inclinometers even though slightly above average rainfall was measured in 1998–1999. In addition, inclinometers in the middle and upper portions of the slide mass did not show an increase in movement during the high rainfall months in 1998–1999, 1999–2000, 2000–2001, and 2001–2002 as was observed in the toe area because the shear surface is at a depth of 35–40 m. Thus, the shallow depth of the shear surface in the toe area made it more susceptible to rainfall-induced movement, but this slide movement stopped within two or three months after the rainy season ended. This reinforces the conclusion that rainfall alone was not the trigger of the slide and that the rainfalls of 1994–1995 and 1995–1996 did not contribute to the slope movement that started in October 1996.

Cross Sections

Knowing that the landslide is underlain by the highly variable Franciscan complex/mélange, four cross sections were drawn to try to understand the materials present, the variability of the materials, and the presence of unusual and varying subsurface features, such as a buried sandstone ridge under the eastern portion of the slide mass. These four cross sections are shown in Fig. 3 and are labeled K1–K1' through K4–K4'. The stability analyses revealed that K1–K1' in the western portion of the slide is the critical cross section, i.e., it yields the lowest factors of safety. As a result, the causation analysis described subsequently focuses on K1–K1'. Cross sections K3–K3' and K4–K4' revealed the presence of a buried sandstone ridge that increases from a depth of about 40 m on the western portion of the slide mass to a depth of only about 18 m on the eastern portion of the slide mass in the vicinity of K2–K2'. The presence of the sandstone ridge provided some resistance, which increased the stability along cross section K2–K2' and contributed to K1–K1' being the critical cross section.

Cross section K1–K1' is shown in Fig. 4 and was obtained from the results of 74 borings drilled in and around the slide mass shown in Fig. 1. Some of these borings were drilled before the landslide and some after the slide to install piezometers and inclinometers or to assess soil stratigraphy. Based on field observations, slope inclinometer results, and the stratigraphy developed from the results of the subsurface investigation, the 1996 failure surface passes through the tertiary volcanics upslope of the landscape screen fill at a steep inclination to the underlying weak and saturated serpentinite (Fig. 4). The failure surface continues along the serpentinite layer until the depth of overburden allows it to daylight in the Knolls development. The landslide also damaged Vista development homes along the western edge of the slide mass.

Therefore, the failure surface shown in Fig. 4 corresponds to a translational failure mechanism, although it is somewhat atypical in that it passes up and over a metasandstone ridge near the middle of the cross section. A similar process was used to develop cross section K2–K2' prior to the stability analyses.

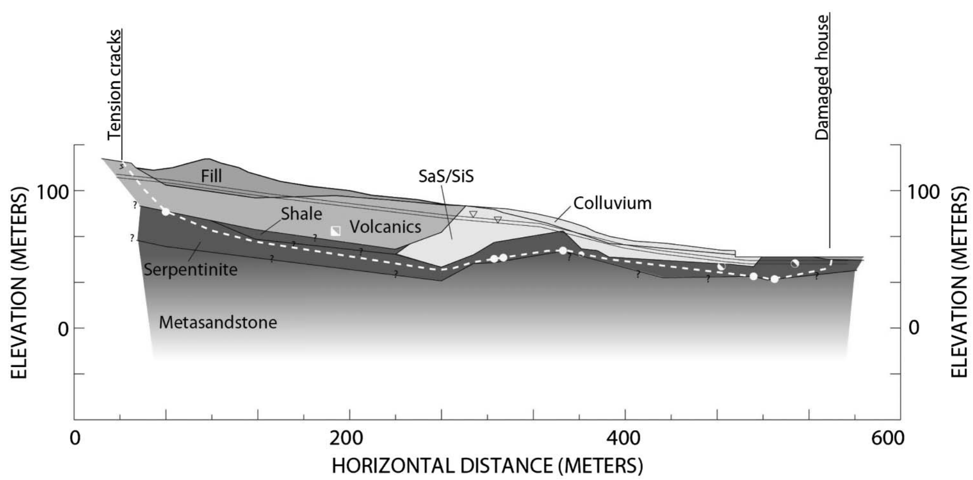

Fig. 4. Plan view showing location of 16 inclinometers installed

Failure Mechanism

The appearance of continuous and substantial tension cracks along the top of the slide limits, i.e., upslope of and around the landscape screen fill, indicated that landscape screen fill was pulling away from the upslope materials. This is an indication of a translational failure mechanism instead of a rotational failure mechanism (Cruden and Varnes 1996). The tension cracks upslope of the landscape screen fill continued to widen until the landscape screen fill was removed between December 1997 and April 1998. In addition, no vertical offset was associated with these tension cracks, which is expected for a translational slide that only experienced about 0.2–0.25 m of movement at the toe of the slide mass. This magnitude of movement is based on the observed housing distress and inclinometers installed prior to landscape screen fill removal in December 1997. Eighteen inclinometers were installed in and around the slide mass, and the locations are shown in Fig. 5. The first set of inclinometers was installed in late February 1997 by the project engineer, and they are shown as squares in Fig. 5. The second set of inclinometers was installed in mid-September 1997 by another engineering firm. They are shown as circles in Fig. 5.

The inclinometers show a total of only about 0.1 m of shear movement because they were installed after movement started and thus did not capture all the movement. The first set of inclinometers show a rate of movement of 12.7–15.2 mm per month. After the landscape screen fill was removed between December 1997 and April 1998, the rate of movement decreased to 0.25–0.5 mm per year in the toe area.

Nine of the 18 slope inclinometers installed after the initial report of distress provided useful information, whereas the other nine were either too shallow or outside the slide limits and did not provide direct slide movement information. Each of the nine useful inclinometers showed only one slide plane at depths ranging from 5 m near the landslide toe to about 40 m near the middle of the slide mass. The depth of movement, or the total depth of the inclinometers, is plotted on cross section K1–K1' in Fig. 4 using various symbols. A solid circle corresponds to an inclinometer that is within 30 m of cross section K1–K1' and exhibits a distinct shear displacement; a partially shaded circle corresponds to an inclinometer that is not within 30 m of K1–K1', but shear displacement was observed in the inclinometer. The failure surface in Fig. 4 (see dashed line) was developed by connecting the location of shear movement in the inclinometers, following the various material types, and passing the failure surface through the cracks observed at the top of the landslide and distress observed at the landslide toe.

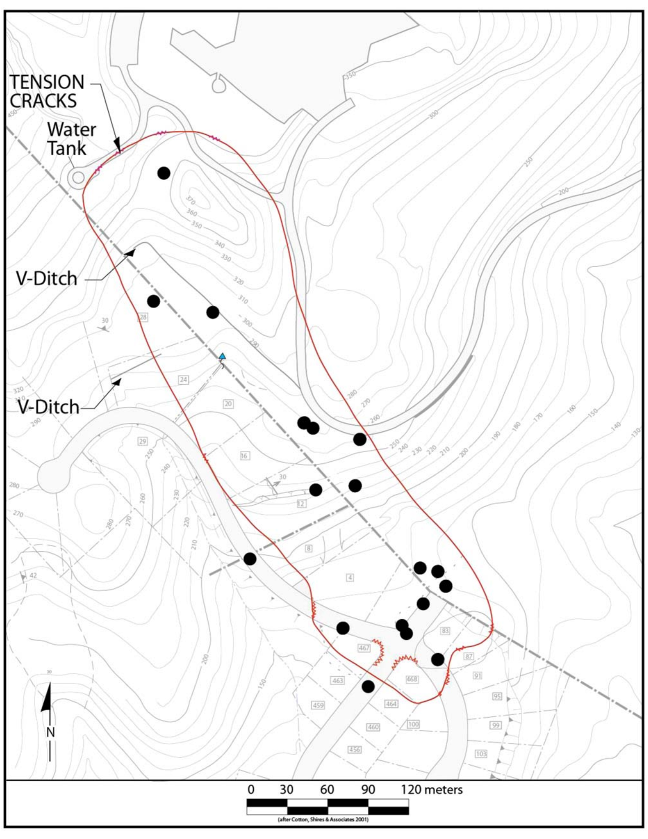

Fig. 5. Cross section K1–K1' through western portion of landslide after surficial grading and placement of landscape screen fill

Serpentinite and Other Material Properties

Frequently, the percentage of serpentinite clay matrix is such that the engineering properties of the serpentinite are controlled by the clay matrix instead of the intact rock. The clay matrix usually consists of fully softened to highly sheared clay and thus exhibits a shear strength at or below the fully softened strength (Skempton 1970, 1977). Even the fully softened strength of the clay matrix can be extremely low because the matrix usually consists of highly plastic clay minerals, such as montmorillonite (Stark and Eid 1994, 1997).

The size and frequency of hard blocks in the serpentinite clay matrix control the engineering behavior because the blocks are much stronger than the clay matrix. Medley and Goodman (1994) indicate that volumetric block proportions of about 25–75% control the overall strength of mélanges. Thus, if the serpentinite has less than 25% blocks, the clay matrix will control the shear behavior, which could have been the case along the observed failure surface in this slide.

A Bromhead torsional ring shear device (Stark and Eid 1993) was used to test the clay matrix because both residual and fully softened strengths were applicable to this case. A direct shear device can provide an adequate estimate of the fully softened strength, but limited continuous shear displacement in the direct shear device prevents an estimate of the drained residual strength (Stark and Eid 1992). To eliminate device-related issues, it was decided to use the ring shear device for both strengths.

The index properties of the serpentinite clay matrix are a liquid limit of 83–95, a plasticity index of 60–68, and a clay-size fraction ($\% < 0.002$ mm) of 55–60%. As a result, the clay matrix classifies as a high plasticity clay (CH), according to the Unified Soil Classification System (USCS). If a linear failure envelope is passed through the torsional ring shear test results generated using ASTM D6467 (ASTM 1999), the resulting secant residual friction angle is only 6°, which is in agreement with the stress-dependent empirical correlations presented by Stark et al. (2005b). The fully softened friction angle, also measured using torsional ring shear tests and a linear failure envelope, corresponds to about 12°, which is in agreement with stress dependent empirical correlations presented by Stark et al. (2005b). Twelve degrees (12°) is also in agreement with field observations of marginally stable serpentinite landslides in the area, such as the landslide at Land's End in the Golden Gate National Recreation Area, which has an average slope of 12° (Goodman 1993).

To investigate the serpentinite in this study, three 0.6-m-diameter borings were drilled to view and sample the serpentinite and to supplement three 0.6-m-diameter borings that had been previously drilled within the slide limits. Two of the new 0.6-m-diameter borings reached the serpentinite, whereas the third boring could not pass through the hard overlying volcanic material on the

BC development property. In one of the 0.6-m-diameter borings that reached the serpentinite, the top of the serpentinite was encountered at about 13 m. The depth of shear movement observed in a slope inclinometer within 4 m of the boring is about 16 m, which indicates that shearing or sliding was occurring in the serpentinite. Knowing that shear movement was occurring in the serpentinite, inspections of the serpentinite and the groundwater conditions were the main objectives of the downhole inspection in these two borings.

The first important downhole observation was that the serpentinite caved into the boring, and thus the clay matrix in the serpentinite could not support the blocky material in the open boring. The serpentinite caved in such that the boring diameter increased from 0.6 to 2.0–2.5 m. This outcome is in agreement with sliding observed through this zone of the serpentinite in the nearby slope inclinometer. Some of this caving may have been caused by dewatering near the depth of the serpentinite that occurred prior to downhole inspection. Because the serpentinite caved in 2.0–2.5 m from the edge of the boring, samples of this serpentinite could not be obtained from the wall of the boring at the slide plane depth during the downhole inspection. As a result, the testing described previously was conducted on grab samples from the auger while the two 0.6-m-diameter borings progressed through this layer and on grab samples obtained from previous 0.6-m-diameter borings. Above the serpentinite, the soil materials were able to support themselves, and the boring maintained a diameter of about 0.6 m, which facilitated downhole inspection.

One of the uncertainties in the stability analyses is the shear strength of the serpentinite. A back-analysis, described subsequently, was conducted to estimate the mobilized shear strength of the serpentinite. A literature search yielded limited information about shear strength for serpentinite in this area.

Groundwater Levels

The 0.6-m-diameter borings also revealed a groundwater level at or slightly above the top of the serpentinite. This groundwater had to be pumped out before the downhole inspection could be done. While the downhole inspection was being performed, water could be seen flowing into the boring through the serpentinite layer and filling the bottom of the boring. The three 0.6-m-diameter borings drilled for this study were completed in early April 2003, as the rainy season was nearing an end.

The groundwater level acting on the failure surface at the time of the initial movement in October–December 1996 was not known. As a result, a range of groundwater level was used in the analysis, with the high level corresponding to the rainy season and the low level corresponding to the dry season. These two groundwater levels were developed from water levels observed in small- and large-diameter borings, piezometers, and monitoring wells installed as part of the remedial measures and from water levels observed around the site.

Back-Analysis of Landslide

It is difficult to obtain a representative sample of block-in-matrix serpentinite and test it in laboratory shear devices because of the large blocks and variation in quantity and consistency of the clay matrix. To overcome this dilemma, a two-dimensional limit-equilibrium back-analysis of the slide was conducted to estimate the mobilized strength of the serpentinite. The back-analysis and stability analyses discussed subsequently utilize Spencer's (1967) two-dimensional, limit-equilibrium method because it satisfies all conditions of static equilibrium. The slope stability program *XSTABL Version 5* was used for the analyses.

Table 2. Material Properties Used in Stability Analyses

Material description	Moist unit weight (kN/m^3)	Shear strength parameters: Effective stress cohesion intercept (kPa)	Shear strength parameters: Effective stress friction angle (degrees)	Source
Volcanics	21.2	0	35	Laboratory testing
Sandstone and Siltstone	21.0	48	30	Laboratory testing
Metasandstone	21.2	145	30	Laboratory testing
Shale	20.4	0	12.5	Testing and Stark et al. (2005a)
Colluvium	19.7	0	25	Testing and Stark et al. (2005a)
Serpentinite	19.7	0	?	Back-analysis and Stark et al. (2005a)

In each back-analysis, the serpentinite friction angle was varied until a factor of safety of 0.99 was obtained (Duncan and Stark 1992). The effective stress cohesion of the serpentinite was assumed to be zero because of the fully softened and/or sheared nature of the serpentinite (Stark et al. 2005b). Engineering properties of the other materials involved in the stability analyses are presented in Table 2.

In a back-analysis, the failure surface that has undergone sufficient movement to result in the mobilization of an overall factor of safety of just less than unity, e.g., 0.99, must be analyzed. As a result, a search for the failure surface in a back-analysis is not appropriate because it may yield a failure surface that did not exhibit movement and thus a friction angle that is less than the back-calculated friction angle for the observed failure surface. For example, a back-analysis for the present case should not use a search with a circular failure surface to back-calculate the shear strength of the serpentinite because the observed failure mechanism is translational and has been identified at a number of locations. However, it is acceptable to vary the failure surface between known points, e.g., cracks at the top of the slide mass, shearing observed in inclinometers, and distress at the toe of the slide mass, to ensure that the minimum friction angle is back-calculated for the observed failure surface.

In summary, a forensic investigation differs from a design investigation because the failure surface is known, whereas in design, the failure surface is not known, and a search is conducted to locate the weakest or least stable portion of the hillside to ensure that it exhibits an acceptable factor of safety.

The results of the back-analysis of cross sections K1–K1' and K2–K2' are presented in Fig. 6. The back-calculated failure envelopes for K1–K1', i.e., friction angles of 9.9 and 9.5° for high and low water, respectively, are slightly below the fully softened failure envelope measured in the torsional ring shear tests. The back-calculated failure envelopes for K2–K2', i.e., friction angles of 7.7 and 7.3°, are well below the measured fully softened failure envelope and are in better agreement with the measured residual failure envelope. In engineering practice, it is more appropriate to round friction angles to the nearest one-half degree, but if this was done for the back-analysis, it would appear that there is no effect of the different water levels for K1–K1' because both back-calculated friction angles would be reported as 10.0°, instead of 9.9 and 9.5°, which would not make sense.

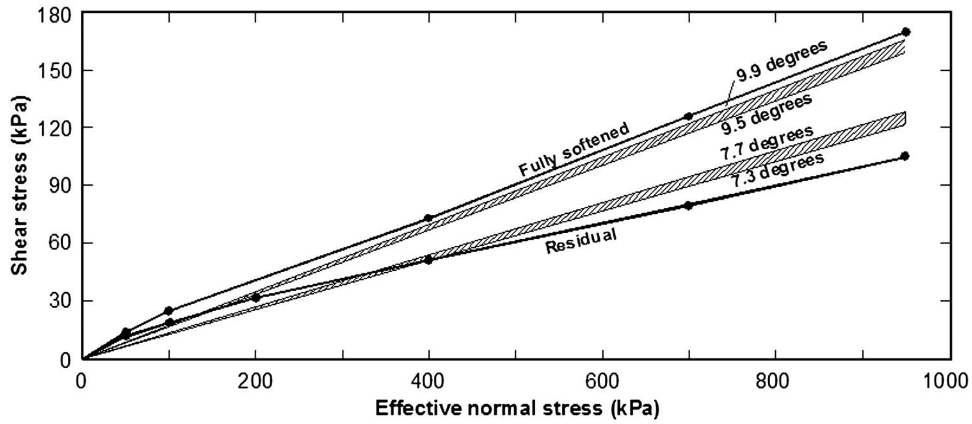

Fig. 6. Serpentinite failure envelopes derived from torsional ring shear tests and back-analyses

The cross section that yields the highest back-calculated friction angle is the critical cross section. Therefore, the western portion of the slide mass, i.e., K1–K1', is critical because it yielded higher back-calculated friction angles for both the high (rainy season) and low (dry season) groundwater levels considered than did K2–K2'. This is in agreement with distress being first reported in homes in the western portion of the landslide toe and the western portion of the Vista development. Therefore, it was concluded that shear movement started along cross section K1–K1' and induced movement along K2–K2'.

After conducting a back-analysis, it is important to compare the back-calculated effective stress friction angles with laboratory shear test results and empirical correlations to ensure that the back-analysis yielded reasonable values of the effective stress friction angle. The fully softened and residual failure envelopes estimated from torsional ring shear tests conducted on samples of serpentinite clay matrix obtained from a 0.6-m-diameter boring on the BC development property are also shown in Fig. 6. They are compared with the range of the effective stress friction angle or failure envelope for high and low water conditions for back-analysis of cross sections K1–K1' and K2–K2'. The back-calculated failure envelopes for K1–K1', i.e., friction angles of 9.9 and 9.5°, are slightly below the fully softened failure envelope measured in the torsional ring shear tests (see Fig. 6). In general, the mobilized strength in first-time slides is at or above the fully softened value (Skempton 1970, 1977). However, the serpentinite was presheared during formation, so it is reasonable that the strength could be below the fully softened value. In addition, Stark and Eid (1997) and Mesri and Shahein (2003) show that the strength mobilized in first-time slides can be less than the fully softened value as a result of preshearing and/or progressive failure. Serpentinite is formed by the overriding North American Plate scraping sediment and rock off the subducting Pacific Plate, resulting in a jumbled mix of highly sheared and deformed material (Scholl et al. 1980) which could result in a postpeak strength being mobilized along at least a portion of the failure surface.

Thus, it is reasonable that the mobilized friction angle would be at or slightly below the fully softened failure envelope. Even though the serpentinite has undergone shearing over geologic time, there is no evidence of prior landsliding along the observed failure surface (see Fig. 4), so a residual strength condition may not be applicable to the observed failure surface. In addition, if a residual strength condition had been mobilized along the observed failure surface, a small increase in driving force or reduction in resisting force, e.g., surficial grading, could have caused reactivation of movement. The landscape screen fill is a large increase in driving force, and the movements were increasing when the removal of screen commenced, indicating the onset of postpeak behavior, or a strength greater than the residual value because the movements were increasing with a decrease in loading.

Thus, the back-calculated friction angles for K1–K1' and K2–K2' falling between the fully softened and residual failure envelopes were in agreement with prior landslide observations in the area and suggest consistency between field observations and the back-analysis. If the back-calculated friction angle plotted above the fully softened failure envelope or below the residual failure envelope, it would be inconsistent with field observations and laboratory test results.

The back-calculated friction angles also need to be compared with empirical correlations to ensure the back-analysis yields reasonable values of the effective stress friction angle. Using a liquid limit of 83–95, a clay-size fraction ($\% < 0.002$ mm) of 55–60%, and the fully softened and residual friction angle correlations presented by Stark et al. (2005b), the back-calculated friction angles for K1–K1' were in agreement with these empirical correlations that have been verified using field case histories.

The back-calculated friction angles for K1–K1' were in agreement with laboratory test results, empirical correlations, and field observations of house distress occurring first in the western portion of the landslide toe, which confirms a factor of safety near unity. It is reasonable to assume that the mobilized friction angle of the serpentinite clay matrix at the onset of movement is about 10° (9.9° from back-analysis), which can be used to study the impact of surficial grading and placement of the landscape screen fill on hillside stability. The agreement between the back-calculated and laboratory friction angles is good (12° versus ~10°) for this case, which also suggests a good estimate of the pore water pressures and engineering properties of the overlying materials. In this case, the slope geometry was established through a number of aerial surveys so that little uncertainty was introduced via slope geometry. In general, an agreement within a few degrees (~2–3°) between the back-calculated and laboratory friction angles is considered good agreement for landslides.

Causation Analysis

The natural hillside slope shown in Fig. 1 was not sufficient to initiate a deep-seated landslide, even with the above-average rainfall that occurred in the 1992, 1994, and 1995 rainy seasons, because homeowner complaints at the western slope toe did not begin until late December 1996. In addition, the slope geometry did not change significantly from 1989 until June 1996, when landscape screen fill construction commenced. As a result, the causation analysis focused on the effect of, in chronological order, surficial grading for the Knolls development, and the landscape screen fill.

Effect of Surficial Grading

The surficial grading for the Knolls development removed some material in the vicinity of the eastern landslide toe (near K2–K2'), reducing slope stability in two ways. The first way is the reduction in buttressing force caused by removal of soil and rock from the toe. This reduction decreases the buttressing force available to resist the driving force imposed by the landscape screen fill. The second is the reduction in available shear resistance along the failure surface. The available shear resistance decreases with a decrease in applied normal stress. The normal stress is related to the thickness of soil and/or rock above the failure surface, and thus it is reduced by removal by grading of soil and/or rock from the slope toe.

In the eastern portion of the landslide toe (near K2–K2'), the maximum depth of material removed during the surficial grading for two building pads in the Knolls development is approximately 6.5 m. This maximum depth of excavation was a result of a small hill situated at the location of the two homes in the eastern portion of the landslide toe (see Fig. 1). This small hill had to be removed to create a level building pad for the two houses. This area is in the eastern portion of the landslide toe, which is significant because prior stability analyses have shown that the critical cross section is located in the western portion of the landslide toe (K1–K1') and outside the area of maximum excavation in the Knolls development. If the removal of the small hill in the eastern portion of the slide mass had caused the landslide by reducing the buttressing effect and lowering the applied normal stress, most, if not all, of the landslide toe would have occurred in the area of maximum excavation. However, Fig. 3 shows that a large portion of the landslide toe (K1–K1') was in the western portion where the removal of material was less than 2.5 m.

The surficial grading conducted to facilitate the construction of the Knolls development probably did not trigger the landslide in 1996, because (1) the landslide occurred 7–8 years after the surficial grading and after several years of significantly above-average rainfall; (2) the maximum amount of excavation did not occur in the critical portion of the slope; (3) a large portion of the landslide toe occurs outside of the area of the largest surficial grading and if grading did destabilize the slope, the landslide toe would be concentrated at the point of the deepest excavation; and (4) the maximum depth of grading is 6.5 m in the eastern portion, which is insignificant compared to the depth of landsliding, i.e., 35–40 m, since shallow excavations do not usually trigger deep bedrock landslides (Stark et al. 2005a). However, there are cases of a large landslide being triggered by a shallow excavation, e.g., Clarke (1904).

Effect of Landscape Screen Fill

The other change in driving force acting on the slope is the placement of the landscape screen fill. The landscape screen fill had a height of at least 22 m above the adjacent natural terrain and a length and width of about 165 and 80 m, respectively. The volume of the landscape screen fill when fill placement ceased was approximately 76,600 cubic meters, which corresponds to about 147 million kg of soil, assuming a soil unit weight of 18.8 kN/m^3.

Using critical cross section K1–K1', material properties in Table 2, and a back-calculated friction angle of 9.9° for the serpentinite, the impact of surficial grading and placement of the landscape screen fill on the stability of the hillside is presented in Table 3. Cross section K1–K1' was modified to reflect the four conditions shown in Table 3 because the cross section shown in Fig. 4 represents the slope geometry after surficial grading and after placement of the landscape screen fill, i.e., the circumstances under which the landslide was triggered.

Table 3. Effect of Surficial Grading and Landscape Screen Fill on Factor of Safety for K1–K1', Using a Serpentinite Back-Calculated Friction Angle of 9.9°

Condition	High water	Low water
Before surficial grading	1.10	1.15
After surficial grading	1.10	1.14
After surficial grading and landscape screen fill	0.99	1.03
Landscape screen fill and no surficial grading	0.99	1.03

Table 3 shows that the slide mass exhibits a factor of safety of 1.10–1.15 before any surficial grading or fill placement occurred for the observed failure surface. After the surficial grading occurred in the vicinity of the western portion of the landslide toe, the factor of safety was still 1.10–1.14, indicating a stable condition and in agreement with there being no homeowner complaints before placement of the landscape screen fill. After surficial grading and placement of the landscape screen fill, the factor of safety decreased to 0.99–1.03, indicating the slope was unstable regardless of groundwater level. This is in agreement with the observation of tension cracks occurring in the road that intersects the slide limits in the BC development (see Fig. 1) prior to the rainy season in October 1996. These tension cracks are located in a limited area and reappeared in the same location after this road was repaved. This is significant because only 2.5 cm of the annual rainfall had fallen at the Petaluma Fire Station at the time that the road cracking was observed in October 1996. In December 1996 and in January 1997, the area received 27.4 and 22.0 cm, respectively, of the 63.8 cm of rainfall that occurred during the 1996 and 1997 rainy season. This is in agreement with homeowner complaints of damage starting in late December 1996.

This analysis and field observations indicate that movement started near the landscape screen fill in October 1996 and progressed downslope until a continuous failure surface was created from just upslope of the landscape screen fill to the Knolls development below. When the entire failure surface had been created, the lower portion of the slide mass started to affect the Vista and Knolls developments, until the landscape screen fill was removed in April 1998. The start of movement was caused by the coalescence of (1) presence of a weak layer (serpentinite); (2) water in the serpentinite layer; and (3) enough fill placement for the landscape screen to reduce the factor of safety to near unity even for the low-groundwater case. This is reinforced by the last analysis in Table 3, which shows that the factor of safety was unchanged if the landscape screen fill was in place and no surficial grading had occurred for the Knolls development.

The results in Table 3 are also important because slope stability analyses are usually performed locally for a hillside development instead of globally. For example, stability analyses were performed for the shallow grading in the Knolls development using slip surfaces that did not exit the Knolls development property. These analyses show factors of safety (FS) greater than 1.5, but the actual failure surface mobilized in this case (see Fig. 4) extends off the Knolls property and was not analyzed even though the same consultant was involved in both the Knolls and BC developments, and this failure surface was marginally stable, i.e., FS = 1.1. In addition, no global or local stability analyses were performed before placement of the landscape screen fill because the decision to create the fill was made during construction.

The mobilized failure surface probably remained stable under prior static and seismic conditions because the factor of safety was 1.10 when the groundwater level was high and 1.15 the remainder of the time. It is doubtful that a large earthquake affected the slope during the brief rainy season, when the factor of safety could be 1.10, to induce slope movement. As a result, no reports of prior sliding along the observed failure surface during static and seismic conditions was not surprising. In addition, the groundwater levels may have been elevated at the time of the great 1906 San Francisco earthquake, which occurred on April 18 and was centered about 35 km west of this area. However, no reports of landslides have been found, which also could be because the site was undeveloped at the time.

Other failure mechanisms were considered in the causation analysis, including the possibility that Knolls development grading facilitated a slide that extended to about the boundary of the Knolls and Vista developments and undermined the upper portion of the slope and landscape screen fill. Analyses do not indicate that this mechanism is more likely than the top-down slide mechanism shown in Fig. 4, and the inclinometer data only identifies one failure surface near the boundary of the Knolls and Vista developments, not two. Other mechanisms considered include the surficial grading for the Vista development and the 2.5–3-m-high cut at the street intersection at the toe of the slide mass (see Fig. 1). In summary, the most plausible explanation for the observed landslide and associated distress is the placement of the landscape screen fill and movement occurring along the weak serpentinite layer.

Conclusions

Landslide observations, data, and analyses used to investigate the cause of distress in two housing developments downslope of a hillside development near Novato, California, are presented. The mechanism that explains all the surface and subsurface movements observed in or near the two housing developments was a deep-seated translational failure surface that follows a weak layer of serpentinite in the Francisan complex/mélange and exits beneath the downslope housing development.

This case history illustrates some of the ramifications of constructing a large fill on a natural hillside upslope of housing developments and the importance of understanding the depth of influence of the proposed fill and development via a stress distribution analysis and a subsurface investigation that extends through the depth of influence to identify the presence of potentially problematic layers. This case also illustrates the importance of: a thorough understanding of the underlying and surrounding geology, including past stability and instability of local and regional slopes in similar terrain and formation type, understanding the influence that a proposed development may have on a marginally stable terrain, and conducting local and global stability analyses to assess the effect of the proposed development on surrounding developments. This paper also provides some information on the shear resistance of serpentinite clay matrix (an effective stress friction angle of about 10°). However, the shear strength of a block-in-matrix rock formation varies with the size and frequency of the blocks. Therefore, the shear resistance estimated at this site has limited applicability to other sites.

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