LANDSLIDE IN AN URBAN ENVRIONMENT

Timothy D. Stark
Professor of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
205 N. Mathews Ave.
Urbana, IL 61801
(217) 333-7394
(217) 333-9464 Fax
tstark@illinois.edu

and

Erik J. Newman
Project Engineer
URS Corporation
1333 Broadway, Suite 800
Oakland, CA 94612
(510) 874-3296
erik_newman@urscorp.com

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By: Timothy D. Stark and Erik J. Newman

ABSTRACT: The causation of distress in two housing developments located downslope of a large development is discussed. The investigation shows that distress in the housing developments was caused by a large, deep bedrock landslide triggered by upslope fill placement. A large fill was placed to create a visual barrier between the existing housing developments and the large upslope structure and to balance the cut and fill quantities of the upslope development. This case history illustrates some of the ramifications of fill placement on natural slopes surrounded by urban areas such as, overstressing underlying weak bedrock material that may exist below the depth of subsurface investigations that are typically conducted for single family residences, the importance of surface and subsurface information in complicated geologic settings, and the effect of natural and man-made changes to a slope, such as rainfall, surficial grading, home construction, and fill placement, on slope stability. This case history also illustrates the importance of locating the critical cross-section before construction and designing the slope to ensure that this cross-section remains stable, the proper use of back-analyses in landslide investigations, the use of the critical cross-section in back-analyses, and the importance of installing a number of slope inclinometers shortly after distress is reported.

Keywords: Soil Mechanics, Landslides, Clays, Shear strength, Slope stability, Subsurface Investigation.

Introduction

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

A factor complicating hillside development is the usually significant cost of disposing of excess cut or excavated material from the project site. Environmental regulations usually make disposal of large amounts of cut material at an offsite location expensive. As a result, there is usually a significant cost incentive to "balance the site", which requires balancing the amount of cut material and the amount of fill material for the hillside development. If the site is "balanced" no fill would need to be imported or exported from the site.

The goal of 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. The details of the large fill and some surficial grading that occurred at the top and bottom of the slope, respectively, in this case history are presented herein. This case history highlights the need for adequate subsurface investigation and stability analyses to assess the stability of a natural hillside in an urban subjected to a large fill and surficial grading. The case history also illustrates the responsibility of an engineer in foreseeing the magnitude of future upslope development to guide the design of downslope developments. This foreseeability requirement can impact the conservatism that an engineer should adopt for the downslope development.

Landslide Chronology

Between 1988 and 1989, a housing development with about 50 units was completed on an undeveloped hillside near Novato, California and is referred herein as the Knolls. Novato, California is located about 30 miles north of the Golden Gate Bridge in San Francisco. An 11 unit housing development was constructed upslope of the Knolls and is referred to herein as the Vista. Only 7 of the 11 Vista lots were developed at the time of the 1996 landslide. Figure 1 presents an aerial view of these housing developments, 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 development are shown in Figure 1.

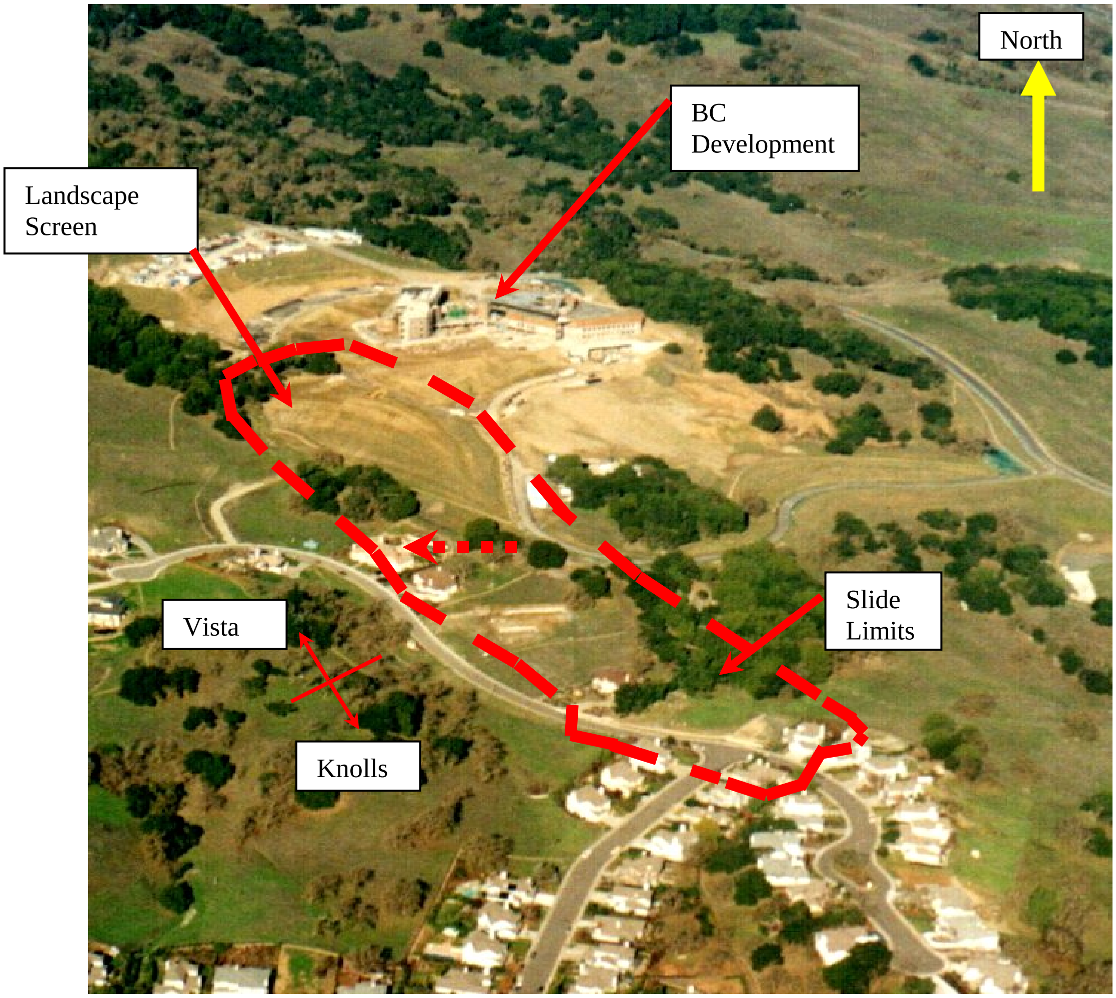

Fig. 1. Aerial view of housing developments, BC Development, and an outline of the slide mass

The Vista and Knolls housing developments did not experience any complaints of distress until September 1996. In September, 1996, the O'Rourke residence (see dotted arrow in Figure 1) experienced significant drywall cracking in the kitchen. In December, 1996, homeowners along the western edge of the slide mass in the Knolls development started experiencing distress in December, 1996. In January, 1997, homeowners along the toe of the slide mass in the Knolls started experiencing distress and damage. This damage chronology is significant because it suggests that slide movement occurred from the top of slope to the bottom of the slope instead of from bottom of the slope to the top. However, the upslope development would claim that the slide was caused by excavation at the toe of the slope and the slide progressed upslope to the BC development.

Shortly after January, 1997, homeowners mobilized a lawsuit against the BC Development. The BC Development subsequently sued the Knolls developer on the theory that the surficial grading performed for the Knolls development removed toe support and allowed a lower landslide to develop which triggered an upper slide that undermined the BC development. This undermining resulted in a large fill created for the BC development undergoing downslope movement.

An important event in the slide chronology is the placement of a large fill by the BC development just above the Vista development in June 1996. The reported purpose of the large fill was to create a visual barrier between the BC development and the Vista development. Another purpose, albeit possibly an indirect purpose, of the large fill was to balance the cut and fill quantities of the BC site. As a result, the large fill is referred as a landscape screen herein. Fill placement for the landscape screen ceased in late December 1996 with the onset of homeowner complaints even though the fill had not reached full height. The final height of the landscape screen is not known and it may have been related to the amount of cut material that had to be disposed. Figure 1 shows the landscape screen at the upslope end of the limits of the landslide. The surface area of the landscape screen is approximately 61,000 square meters and the estimated volume of the landscape screen is 76,600 cubic meters. The estimated volume of the landslide mass is 2.0 million cubic meters.

In summary, there was no significant change in slope geometry after completion of the Knolls housing development and the construction to date 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.

Landslide Factors

In general, a critical combination of the following three factors are required to initiate a landslide: (1) a weak layer underlying the site, (2) subsurface water, and (3) some driving force.

When a critical combination of these three factors coalesces at a site, a landslide can, and probably will, occur.

Weak Layer

The site, on the east side of Mount Burdell, exhibits 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 observed to occur in the underlying bedrock based on slope inclinometer data.

The predominant bedrock units underlying the near surface soils are the Tertiary Volcanics and the Franciscan Complex. The volcanic rocks overlie the Franciscan Complex at the upper portion of the Vista development shown in Figure 1. The volcanic rocks generally consist of hard andesitic rocks and a weaker agglomerate of ash and block flow rocks. Slope inclinometer data show that the landsliding occurred below the volcanic rocks and thus in the Franciscan Complex. The Jurassic-Cretaceous rocks of the Franciscan Complex include sandstones, claystones, mudstones, shale, conglomerates, and serpentinite. The Franciscan Complex is frequently referred as a mélange, or mixture, because the deposit was formed near the forward edge of a subduction plate boundary (Goodman 1993). The California Coastal Range, of which Mount Burdell is part, was created by an east-dipping subduction zone between the Pacific and North American tectonic plates (Wakabayashi 1999). The intense mixing and deformation of the bedrock materials is explained by the overriding North American plate scraping sediment and rock off the subducting Pacific plate. This results in a jumbled mix of highly sheared and deformed bedrock (Scholl et al. 1980). Over time this highly sheared and deformed rock can accumulate enough volume in a small area to create the California Coastal Range.

The jumbled nature of the Franciscan Complex/mélange presents a difficult challenge for engineers because it is not possible to predict the engineering properties of the rock that would be encountered at a particular site without a substantial amount of subsurface exploration and testing (Goodman 1993). Given the increasing tendency of clients to limit or even reduce subsurface investigation costs, a site underlain by the Franciscan Complex creates an extreme challenge for the design of hillside developments.

In cases where the client is receptive to a substantial amount of subsurface exploration and testing, the design engineer should determine the depth of influence of the cut and fill activities and design a subsurface investigation to sample and test the material that would be impacted by the development. For example, the maximum depth of influence of the landscape screen constructed for the BC Development is estimated to be about 120 meters using Boussinesq stress distribution theory for an inclined embankment loading (Holtz and Kovacs 1981). However, the various geotechnical engineers employed for the BC Development drilled almost 80 boring across the site and none of the borings exceeded a depth of about 15 m within the slide limits shown in Figure 1. The slope inclinometers installed in the BC Development site after homeowner complaints show the depth of sliding to be 24 to 30 m. Thus, none of the borings drilled within the slide limits for the BC development are deep enough to reach the problematic serpentinite (discussed below). As a result, the designers may not have been aware of the weak layer underlying the site although the serpentinite is outcropping at numerous locations across the project site.

In summary, stress distribution analyses should be performed to assess the depth of influence for fill operations and the subsurface investigation should sample and investigate the materials within this zone of influence to determine if a weak layer exists. If a weak layer does exist, stability analyses should be conducted to assess the change in the factor of safety of the slope caused by the fill placement.

One material frequently found in the Franciscan Complex/mélange that usually presents a severe siting and slope stability hazard is serpentinite. The serpentinite at the BC Development site consists of large intact rocks surrounded by a high plasticity clay matrix and thus the rock is referred as a block-in-matrix rock (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 the intact rock. The clay matrix is usually fully softened to highly sheared and thus exhibits a shear strength at or below the fully softened strength (Skempton 1970 and 1977). Even the fully softened strength of the clay matrix can be extremely low because it usually consists of highly plastic clay minerals such as montmorillonite (Stark and Eid 1997).

Testing of the serpentinite conducted during this study shows a liquid limit from 83 to 95, a plasticity index from 60 to 68, and a clay-size fraction (% < 0.002 mm) of 55 to 60%. As a result, the clay matrix classifies as a high plasticity clay (CH) according to the Unified Soil Classification system. If a linear failure envelope is passed through the torsional ring shear test results generated according to ASTM D6467, the resulting secant residual friction angle for the serpentinite is only six degrees, which is in agreement with the empirical correlations presented by Stark et al. (2005). The fully softened friction angle, also measured using torsional ring shear tests and a linear failure envelope, corresponds to about 12 degrees, which is in agreement with the empirical correlations presented by Stark et al. (2005). Twelve degrees also is 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 have an average slope of 12 degrees (Goodman 1993).

A number of researchers, e.g., Dickinson 1966; Moiseyev 1970; Blake et al. 1974; Cowan and Mansfield 1970; Phipps 1984, have reported large landslides involving serpentinite. Table 1 shows a number of the long and wide serpentinite slides that have occurred. The length to width ratios of these slides range from 2.3 to 17.5. The length to width ratio of the current slide, 3.8, is also presented in Table 1 and is in agreement with previously reported serpentinite slides even though a large fill is involved.

Table 1: Length and width of serpentinite slides

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

A number of researchers, e.g., Berkland 1969; Blake et al. 1974; Rice 1975, have reported a number of large, deep-seated bedrock landslides that underlie the surficial colluvial slide deposits in the Mount Burdell area and there is evidence of serpentinite slides in roadcuts along the eastern edge of the project area. Investigation into possible prior landsliding in serpentinite throughout the San Francisco Bay area, and in particular in the Mount Burdell area, should have revealed the potential for deep-seated sliding with the placement of the large landscape screen.

In summary, engineers designing hillside developments should review local landslide history, assess the depth of influence of the fill/development operations, and investigate potentially weak material through the full depth of influence to ensure a weak layer is not overstressed and does not cause a slope failure.

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 previous 0.6 m diameter borings that had been drilled previously within the slide limits shown in Figure 1. Two of the new borings reached the serpentinite while the third boring could not pass through the hard volcanic material on the BC Development property. Figure 2 is a photograph taken by the first author in one of the 0.6 m diameter borings that reached the serpentinite. The top of the serpentinite is encountered at a depth of 13 m and extends to a depth of 23 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, inspection of the serpentinite and the water condition were the main objectives of the down-hole inspection.

The first important observation from Figure 2 is that the serpentinite caved in and thus the clay matrix in the serpentinite could not support the blocky material in the open boring. As a result, frequent splashes from serpentinite entering the water filled bottom of the boring could be heard. The serpentinite caved in such that a 2.0 to 2.5 m of material from the edge of the boring was removed as shown in the diagram to the right of the photograph in Figure 2. Also shown in the photograph in Figure 2 is the diameter of the boring returned to about 0.6 m after the 4 to 5 m thick zone of caving serpentinite. This is in agreement with sliding being observed through this zone of serpentinite in the adjacent slope inclinometer. Because the serpentinite caved in 2.0 to 2.5 m from the edge of the boring, samples of this serpentinite could not be obtained. As a result, the testing described previously was conducted on grab samples from the auger while the two 0.6m diameter borings progressed through this layer and on grab samples obtained from previous 0.6 m diameter borings. Above the location shown in Figure 2, the soil materials were able to support themselves and the boring maintained a diameter of about 0.6 m. This is in agreement with the adjacent inclinometer indicating a well defined movement plane below these stronger materials that overlie the serpentinite.

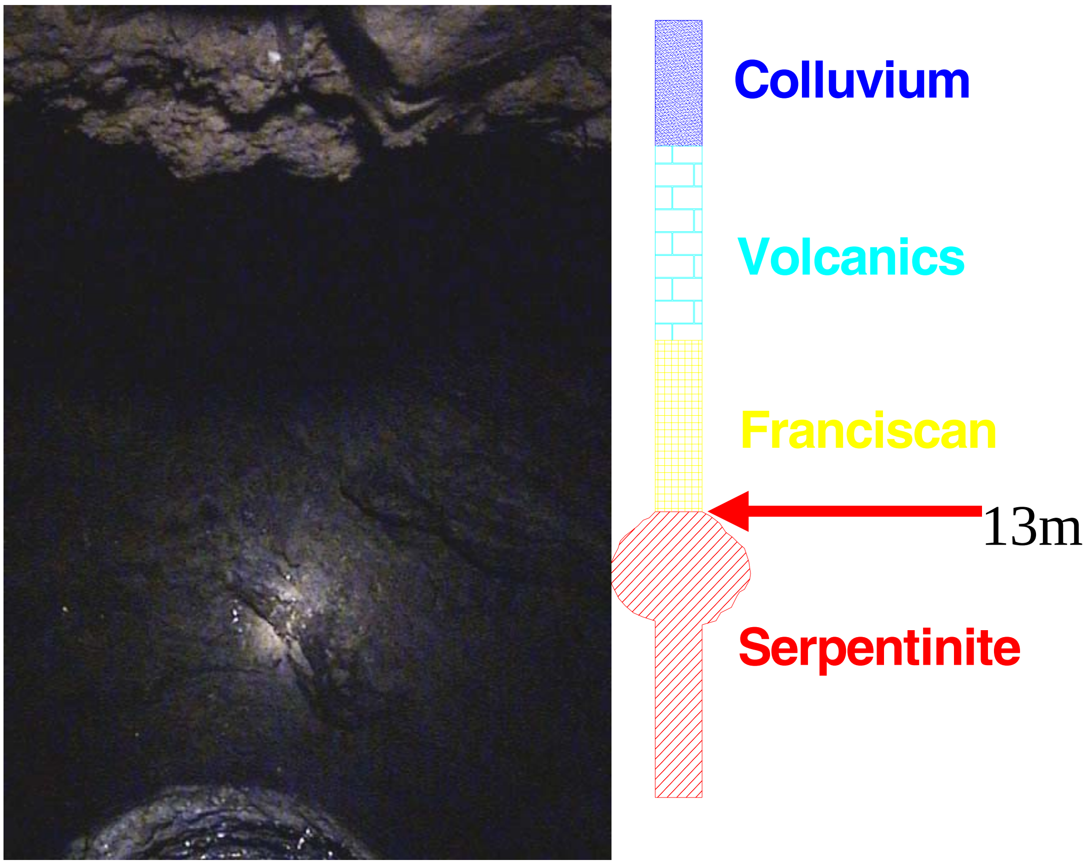

Fig. 2. View of caving serpentinite at a depth of 13 to 15 m in a 0.6 m diameter boring

In summary, the serpentinite is naturally occurring at the site and has been lurking below the project site for millions of years waiting for human development. Thus, the discussion of the three landslide factors turns to the second factor required for slope instability, which is water.

Water

The 0.6 m diameter borings also revealed a groundwater level above the top of the serpentinite. This water had to be pumped out before the boring could be entered and inspected. While suspended in the boring, 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 constructed in early April 2003 and thus the rainy season was nearing an end.

In general, the depth to groundwater is related to the amount of precipitation. High levels of precipitation usually result in higher levels of groundwater and vice versa. Thus, it is important to compare rainfall records prior to and during the year that movement is first reported to determine if rainfall is the trigger of the landsliding. To investigate the impact of rainfall on the causation of the landslide, the rainfall records from the nearby Petaluma Fire Station are summarized in Figure 3. Petaluma is the next town north of Novato, California. The yearly rainfall total from July 1 to June 30 of each year in presented in Figure 3. The Knolls housing development, which suffered the most damage, was completed between 1988 and 1989. Between 1989 and 1992-1993 rainy season, the area received below average rainfall. The fifty-three year average rainfall for the Petaluma Fire Station from 1948 to 2001 is 64.0 cm. In the 1992-1993 rainy season, 77 cm of rainfall or 13 cm above average rainfall occurred. The area also experienced above average rainfall in the 1994-1995 (113.2 cm) and 1995-1996 (80.7 cm) rainy seasons 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) see Figure 3.

In summary, prior to 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 landsliding did occur is the landscape screen had not been constructed. In the 1996-1997 rainy season, less rainfall occurred than in several previous years and a landslide occurred. As a result, it can be concluded that rainfall alone did not trigger this landslide and thus the investigation focused on the third factor, driving force.

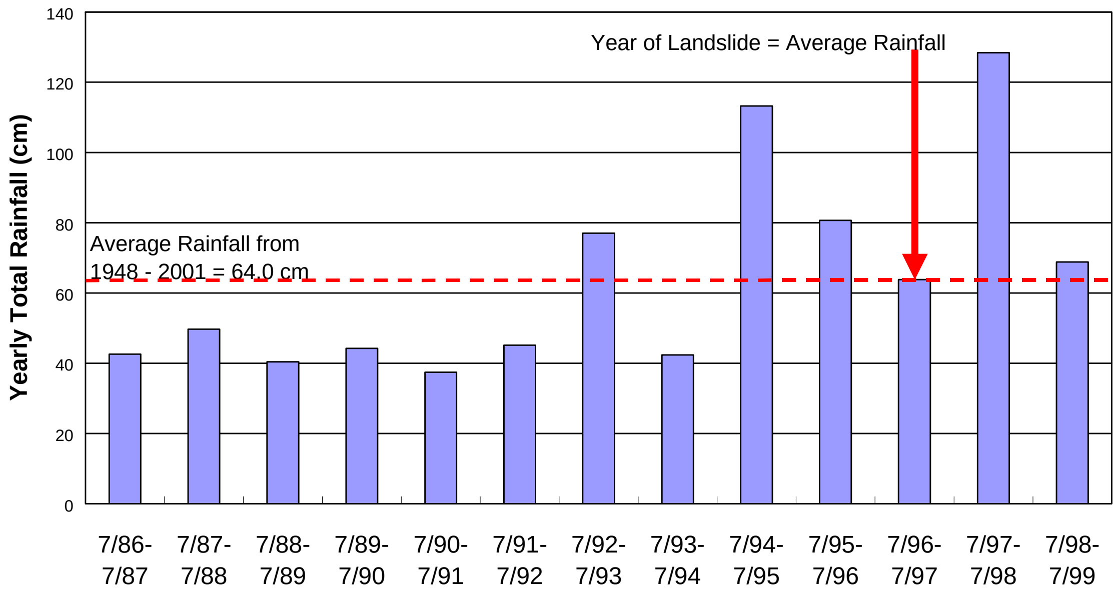

Fig. 3. Annual rainfall data from nearby fire station

Driving Force

The natural slope of the hillside shown in Figure 1 was not sufficient to initiate a deep landslide even with the above average rainfall that occurred in the 1992, 1994, and 1995 rainy seasons because homeowner complaints at the slope toe did not begin until late December 1996. In addition, the slope geometry or slope angle did not change significantly from 1989 until June 1996 when construction of the landscape screen commenced. As a result, this investigation focuses on two other sources of driving force, which are, in chronologic order, surficial grading for the Knolls development and placement of the landscape screen for the BC Development.

The surficial grading removed some material in the vicinity of the landslide toe, reducing the stability of the slope in two ways. The first way is the reduction in buttressing force caused by the removal of soil and rock from the landslide toe. This reduction increases the difference between the driving force imposed by the slope angle and the landscape screen at the top of the slope and the resisting force caused by the soil at the slope toe. Increasing the difference between the driving and resisting forces causes a decrease in the factor of safety.

The second way the stability of the slope is reduced is by the reduction in available shear strength along the failure surface. The available shear strength increases as the normal stress applied to the soil increases. The normal stress is related to the thickness of soil and/or rock above the failure surface and thus is reduced by removal of soil and/or rock from the slope toe.

Figure 4 presents a plan view of the landslide area with the slide limits from Figure 1 superimposed. In the eastern portion of the landslide toe the maximum depth of material removed during the surficial grading is approximately 6.5 m. The area corresponding to a removal of more than 6.1 m of material is indicated by the small ellipsoidal area in the eastern portion of the landslide toe. This maximum depth of excavation is due to a small hill that was situated at that location and had to be removed to create a level building pad for the house that would be constructed on the pad. Figure 4 also shows a larger area that corresponds to a depth of excavation exceeding 3.5 m. This area again is in the eastern portion of the landslide toe which is significant because subsequent stability analyses will show that the critical cross-section is located in the western portion of the landslide toe and outside both shaded areas in Figure 4. If the removal of the small hill had caused the landslide by reducing the buttressing effect and lowering the normal stress, most, if not all, of the landslide toe would have occurred through the point of maximum excavation. However, Figure 4 shows that a large portion of the landslide toe is the western portion where the removal of material is less than 2.5m. The maximum depth of excavation to create building lots in the western portion of the slide is about 2.5 m.

In summary, the surficial grading conducted to facilitate the construction of the Knolls development did not trigger the landslide in 1996 because (1) the landslide occurred 7 to 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 and thus did not impact the triggering of the landslide, (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 excavation is 6.5 m and is insignificant compared to a depth of landsliding of 36 m measured in two slope inclinometers because shallow excavations usually do not trigger deep bedrock landslides.

The other change in the driving forces acting on the slope is the placement of the landscape screen. Figure 5 provides a comparison of the landscape screen to the homes in the Vista development. The landscape screen has a height of at least 22 m above the adjacent natural terrain and has a length and width of about 165 and 80 meters, respectively. The volume of the landscape screen when fill placement ceased is 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.

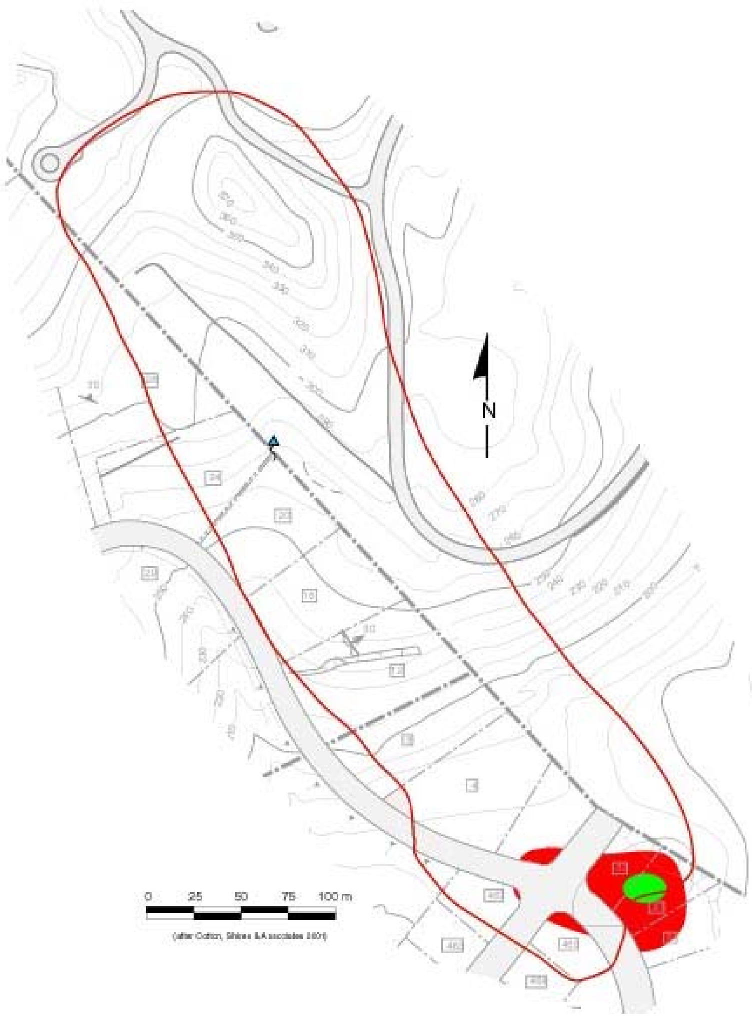

Fig. 4. Plan view illustrating the areas of greater than 3.5 m and 6.1 m (ellipsoidal area) of surface grading

In summary, the three landslide factors, weak layer, water, and driving force, coalesced in September and continued through December 1996 with the triggering event appearing to be the placement of the large landscape screen because (1) the weak layer had always been present, (2) the surficial grading had occurred 7 to 8 years before the reporting of distress, and (3) the site experienced years of greater rainfall after the grading than the year distress initiated. The following sections of the paper present the stability analyses used to quantify the impact of the surficial grading and the landscape screen on the stability of the slope.

Fig. 5. Aerial illustrating size of landscape screen in relation to a 375 m^2 single family residence (see arrow) and an excavator on the top of the landscape screen (see arrow)

Forensic Investigation

The main steps in the forensic investigation to determine the causation of the 1996 landslide are: (1) develop a number of cross-sections to understand the variability and geometry of the subsurface materials, (2) determine the failure mechanism or failure surface from surface observations and slope inclinometer results, (3) develop material properties for the materials involved and appropriate groundwater levels, (4) perform a back-analysis to locate the critical cross-section, (5) use the back-analysis to estimate the mobilized shear strength of the weak layer and compare it with laboratory test results and field observations to ensure consistency and thus accuracy, and (6) conduct stability analyses to determine the effect of surficial grading and placement of the landscape screen on the stability of the hillside.

Cross-Sections

Knowing that the landslide is underlain by the highly variable Franciscan Complex, six cross-sections were drawn to gain an understanding of the materials present, the variability of the materials, and the presence of unusual and varying subsurface features, such as the buried sandstone ridge under the eastern portion of the slide mass. These six cross-sections are shown in Figure 6 and not only extend the length of the slide mass but also traverse the slide mass to determine material variability and distribution of the weak layer. The cross-sections are labeled TDS1 through TDS6 and stability analyses were performed on all of the cross-sections that extend the length of the slide mass, i.e., are in the direction of sliding. Cross-sections TDS1 and TDS5 (see Figure 6) extend through the western portion of the landslide toe while TDS2 and TDS6 extend through the eastern portion of the landslide toe. The stability analyses reveal that TDS5 and TDS6 are the critical cross-sections, i.e., yield the lowest factors of safety, for the western and eastern portions of the landslide toe, respectively. As a result, the forensic analysis described subsequently provides a comparison of the results obtained using cross-sections TDS5 and TDS6. However, drawing of the six cross-sections provided an invaluable insight to the subsurface conditions underlying the landslide especially cross-sections TDS3 and TDS4 which traverse the slide mass. These cross-sections reveal the presence of a buried sandstone ridge that increases from a depth of about 40m on the western portion of the slide mass to a depth of about only 18m on the eastern portion of the slide mass in the vicinity of TDS4.

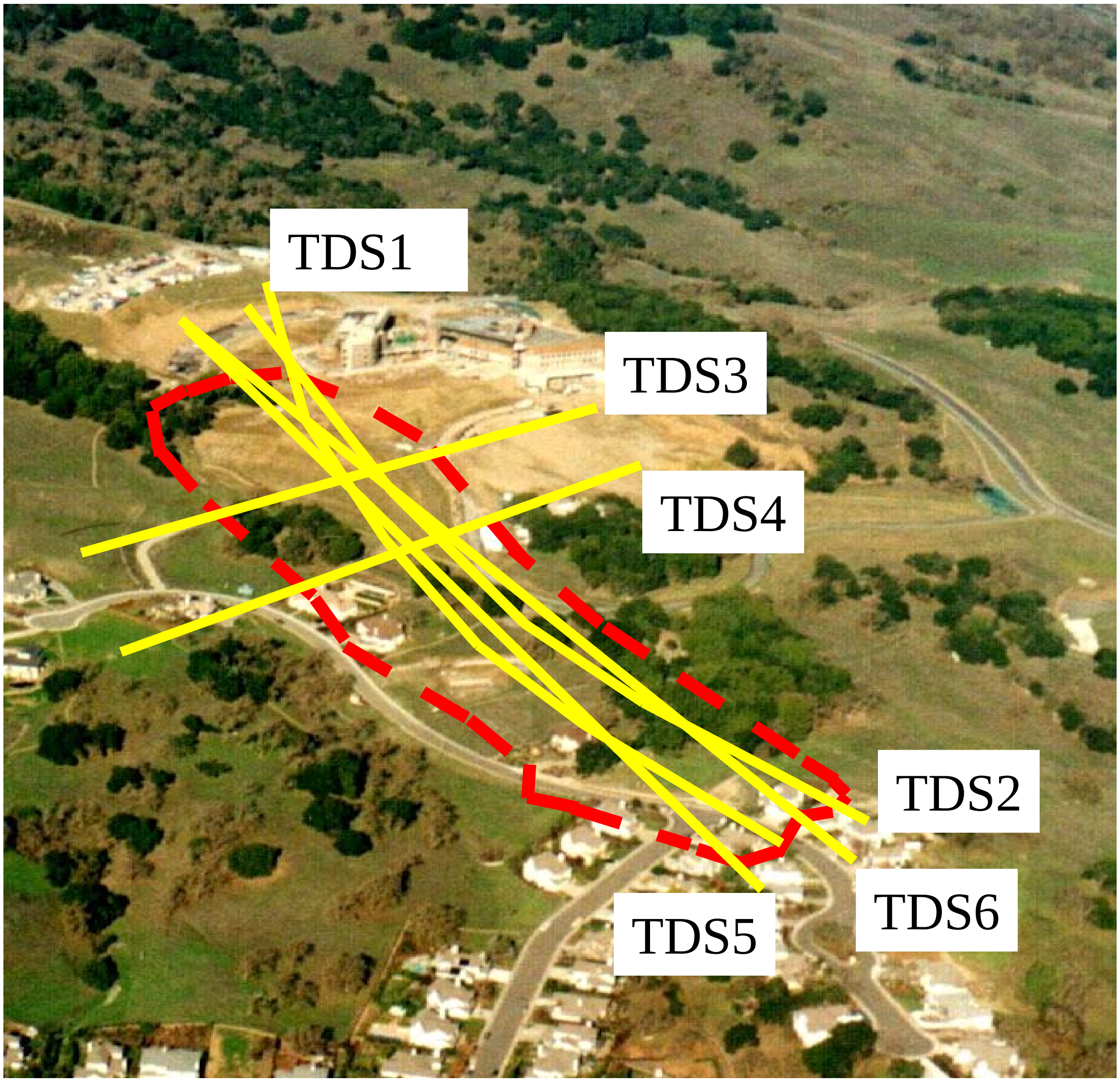

Fig. 6: Aerial view of the landslide illustrating the six cross-sections considered in the forensic study.

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, indicated that the screen area pulled away from the natural materials upslope of this area. This is an indication of a translational failure mechanism instead of a rotational failure mechanism (Cruden and Varnes 1996). In addition, no vertical offset was associated with these tension cracks, which is expected for a translational slide that has only undergone about 20 to 25 cm of deep-seated movement. In a rotational slide, as the slide mass rotates to reduce the driving force, a vertical offset would be associated with the cracking at the top of the slide mass. Vertical offset can be associated with a translational slide if a graben starts to develop. The tension cracks upslope of the landscape screen continued to widen until the landscape screen was completely removed in April 1997 and indicate that the slide mass was simply pulling away from the natural material upslope of the fill area.

Nine of the fifteen slope inclinometers installed after the initial report of distress provide useful information but the other six are either too shallow or outside the slide limits shown in Figure 1 and do not provide direct information on shear movement of the slide. Each of the nine useful inclinometers show only one slide plane at depths ranging from 5 m near the landslide toe to 40 m near the middle of the slide mass. The depth of movement from the inclinometers is plotted on cross-section TDS5 in Figure 7.

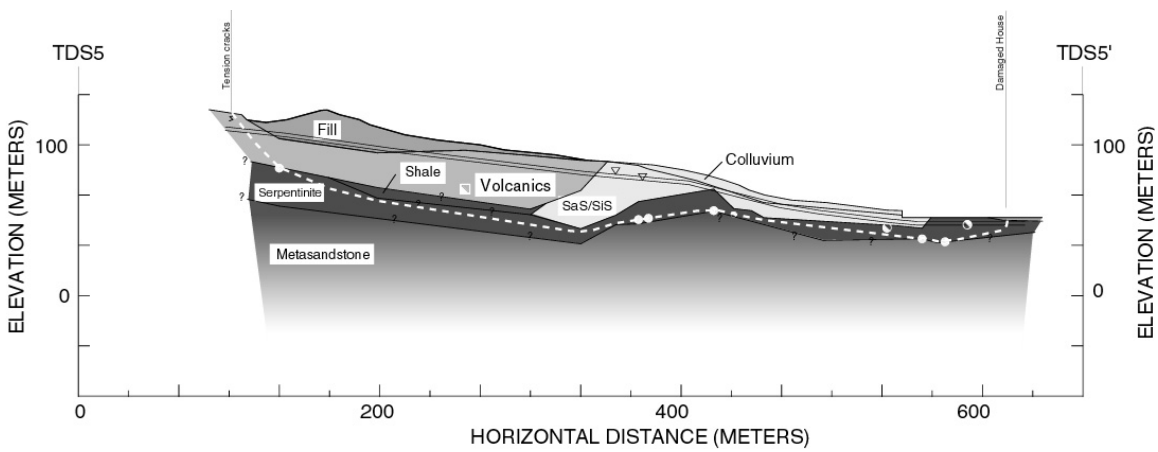

Fig. 7: Cross-section TDS 5 through the western portion of the landslide after surficial grading and placement of the landscape screen

Figure 7 presents the depth of movement or the total depth of the inclinometer in the fifteen inclinometers with various data symbols. The solid circle corresponds to an inclinometer that is within 30 m of cross-section TDS5 and distinct shear displacement is observed. The partially shaded circle corresponds to an inclinometer that that is not within 30 m of TDS5 but shear displacement is present, and a partially shaded square corresponds to an inclinometer that is not within 30 m of TDS5 and the inclinometer is too shallow and thus the failure surface should pass below this inclinometer. The failure surface shown in Figure 7 (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 the housing distress observed at the landslide toe.

Based on field observations, slope inclinometer results, and the stratigraphy developed from the results of the subsurface investigation, the failure surface passes through the Tertiary Volcanics upslope of the landscape screen at a steep inclination to the underlying weak and saturated serpentinite (Figure 7). The failure surface continues along the serpentinite layer until the depth of overburden allows it to daylight in the Knolls housing development and destroy four homes at the landslide toe. The landslide also damaged the homes in the Vista development that are at or near the western edge of the slide mass. The other homes did not show substantial landslide damage because they are located inside of the slide boundaries and are essentially "along for the ride".

In summary, the failure surface shown in Figure 7 corresponds to a translational failure mechanism although it is somewhat atypical in that it passes up and over a sandstone ridge near the middle of the cross-section.

Material Properties and Groundwater Levels

One of the uncertainties in the stability analyses is the shear strength of the serpentinite and thus a back-analysis, described subsequently, was conducted to estimate and/or confirm the mobilized shear strength of this material. The engineering properties of the other materials, i.e., unit weight and shear strength, involved in cross-section TDS5 are shown in Table 2.

The groundwater level acting on the failure surface at the time of the initial movement in October to December 1996 is not known. As a result, a range of groundwater level is used in the analysis with the high level corresponding to the rainy season and the low level corresponding to the dry season as shown in Figure 7. These two groundwater levels were developed from water levels observed in small and large diameter borings, monitoring wells installed as part of the remedial measures, and water levels used by other experts.

Table 2. Material properties used in stability analyses

Material Description	Moist Unit Weight (kN/m^3)	***SHEAR STRENGTH PARAMETERS*** Effective Stress Cohesion Intercept (kPa)	Effective Stress Friction Angle (degrees)	Source
Volcanics	21.2	0	35	Laboratory testing
Sandstone (SaS) & Siltstone (SlS)	21.0	48	30	Laboratory testing
Metasandstone	21.2	145	30	Laboratory testing
Shale	20.4	0	12.5	Testing and Stark et al. (2005)
Colluvium	19.7	0	25	Testing and Stark et al. (2005)
Serpentinite	19.7	0	?	Back-analysis & Stark et al. (2005)

Back-Analysis of the Landslide

One of the main uncertainties in the stability analysis is the mobilized shear strength of the serpentinite because it is difficult to obtain a representative sample of the block-in-matrix rock and prepare it for testing in a laboratory shear device. This is due to the large rocks in the serpentinite and the variation in the quantity and consistency of the clay matrix across the site. Thus, it is difficult to ascertain whether or not the clay matrix controls the engineering properties of the serpentinite. To overcome this dilemma a two-dimensional limit-equilibrium back-analysis of the landslide was conducted to investigate the shear behavior and overall shear strength of the serpentinite. The back-analysis and the stability analyses discussed subsequently utilize 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 for all of the analyses. A three-dimensional stability analysis was not conducted because of the large length of the slide which resulted in a length to width ratio of 3.8 (Stark and Arellano, 2000).

In each back-analysis, the friction angle of the serpentinite was varied until a factor of safety of 0.99, i.e., failure, was obtained. In each analysis the effective cohesion of the serpentinite was assumed to be zero because of the fully softened and/or sheared nature of the serpentinite (Stark et al. 2005). The engineering properties of the other materials used in the back analysis 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, should be analyzed. If sufficient movement has not occurred along the failure surface, the factor of safety is greater than unity and thus the back analysis would under estimate the shear strength of the weak layer because the factor of safety is greater than unity but the exact value is not known. In other words, the factor of safety could 1.1 to 1.5 and the slide mass would not be exhibiting movement. The magnitude of the under estimation is not known because the relationship between factor of safety and back-calculated friction angle of a material is not linear. As a result, a search for the failure surface in a back-analysis should not be performed but 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 the minimum friction angle is back-calculated. It is inappropriate to disregard the surface and subsurface features and search for a failure surface that may yield a friction angle that is less than the friction angle back-calculated for the observed failure surface. This is inappropriate because this is a new failure surface that has not undergone failure and thus the friction angle *should* be higher than the friction angle back-calculated for the observed failure surface. If the friction angle is greater than the back-calculated friction angle for the observed failure surface there is a flaw in the analysis because movement did not occur along the new failure surface.

In summary, a forensic investigation differs from a design investigation because the failure surface is known whereas in design the failure is not known and the engineer searches for the weakest or least stable portion of the hillside to ensure that it is safe. If the weakest portion of the hillside exhibits a suitable factor of safety in design, it is presumed that the remainder of the hillside would be stable. This design process is different from a forensic analysis and a search for the critical failure surface should not be conducted. Most importantly, the forensic analysis should utilize the failure mechanism (translational v. rotational) determined from the surface and subsurface observations and not conduct a back-analysis with a failure mechanism that is not present in the field. For example, a back-analysis for the present case should not use a search with circular failure surfaces to back-calculate the shear strength of the serpentinite because the failure mechanism is translational based on the location of the failure surface identified at a number of locations. However, it is prudent to vary the failure surface between these known locations to ensure that the minimum back-calculated friction angle is obtained.

The results of the back-analysis of cross-sections TDS1, TDS2, TDS5, and TDS6 are presented in Table 3. The cross-section that yields the highest back-calculated friction angle is the critical cross-section, i.e., the weakest or least stable part of the hillside, because the highest is required to achieve a factor of safety of unity. Therefore, in the western portion of the slide mass, i.e., TDS1 and TDS5, TDS5 is critical because it yields a higher back-calculated friction angles for both the high water (rainy season) and low water (dry season) cases than TDS1. In the eastern portion of the slide mass, TDS2 and TDS6 yield similar back-calculated friction angles ranging from 7.3 to 7.9 degrees for the low water (dry season) and high water (rainy season) cases.

Table 3. Back-calculated friction angles for serpentinite in cross-sections TDS1, TDS2, TDS5, and TDS6

	Back-calculated friction angle	
Cross-section	High water	Low water
TDS1	9.6	8.9
TDS2	7.9	7.6
TDS5	9.9	9.5
TDS6	7.7	7.3

A comparison of the back-calculated friction angles for cross-sections TDS5 and TDS6 reveals a significantly greater back-calculated friction angle for TDS5. Thus, the critical cross-section for the entire slide mass is TDS5 which is located through the western portion of the landslide toe. This is in agreement with distress being first reported in homes in the western portion of the Vista development and the western portion of the landslide toe. Therefore, it is concluded that shear movement started along or near cross-section TDS5 and induced movement along TDS6 to create the slide limits shown in Figure 1.

After conducting a back-analysis it is important to compare the back-calculated friction angles with the results of laboratory shear tests and empirical correlations to ensure the back-analysis yielded reasonable values of friction angle. The effective stress cohesion is assumed to be zero because of the fully softened nature of the serpentinite (Stark et al. 2005). The fully softened and residual failure envelopes estimated from torsional ring shear tests conducted on samples of serpentinite obtained from a 0.6 m diameter boring on the BC Development property are shown in Figure 8 and compared to the range of friction angle or failure envelopes for the high and low water conditions in cross-sections TDS5 and TDS6. The back-calculated failure envelopes for TDS5, i.e., friction angles of 9.9 to 9.5 degrees, are slightly below the fully softened failure envelope measured in the torsional ring shear tests. The back-calculated failure envelopes for TDS6, i.e., friction angles of 7.7 to 7.3 degrees, are well below the measured fully softened failure envelope and are in better agreement with the measured residual strength failure envelope.

The serpentinite has undergone shearing over geologic time but there is no evidence of prior landsliding along the failure surface shown in Figure 7 and thus a residual strength condition is probably not applicable to the observed failure surface for the causation of the landslide. However, a residual condition may be applicable for the design of remedial measures. In addition, if a residual strength condition had been mobilized along the failure surface shown in Figure 7 a small increase in the driving force or reduction in resistance, e.g., surficial grading, would have caused a reactivation of movement. The landscape screen is a large increase in driving force and the movements were increasing when the removal of screen commenced indicating the onset of post-peak behavior.

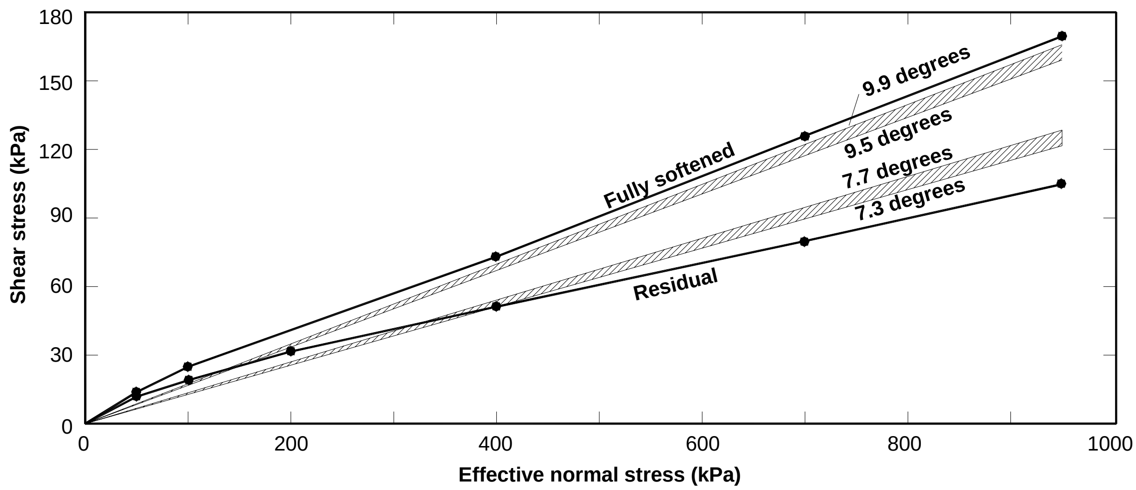

Fig. 8. Serpentinite failure envelopes derived from torsional ring shear tests and back-analyses

Analysis of a number of case histories by Stark and Eid (1997) and supplemented by Mesri and Shahien (2003) shows that it is reasonable to back-calculate friction angles below the full softened failure envelope for sites that have not undergone previous sliding. A strength below the fully softened value can be caused by progressive failure, which is discussed subsequently. Thus, the back-calculated friction angles for TDS5 and TDS6 falling between the fully softened and residual failure envelopes is in agreement with prior landslide observations and suggests consistency between field observations and the back-analysis. If the back-calculated friction angle plots above the fully softened failure envelope or below the residual failure envelope, then it is inconsistent with field observations and laboratory test results, which suggests a flaw or inconsistency in the back-analysis.

The back-calculated friction angles should also be compared with empirical correlations to ensure the back-analysis yields reasonable values of friction angle. Using a liquid limit of 83 to 95, a clay-size fraction (% < 0.002 mm) of 55 to 60%, and the fully softened and residual friction angle correlations presented by Stark et al. (2005), the back-calculated friction angles for TDS5 are in agreement with these empirical correlations that have been verified using field case histories.

In summary, the back-calculated friction angles for TDS5 are in agreement with laboratory test results, empirical correlations, and field observations of distress occurring first in the western portion of the landslide and sufficient deformation occurring to destroy the homes, which confirms a factor of safety near unity. Thus, it is reasonable to assume that the mobilized friction angle of the serpentinite at the on-set of movement in this case is about 9.9 degrees and can be used to study the impact of surficial grading and placement of the landscape screen on the stability of the hillside.

Effect of Surficial Grading and Landscape Screen

Using the critical cross-section, TDS5, the material properties in Table 2, and the a back-calculated friction angle of 9.9 degrees for the serpentinite, the impact of surficial grading and placement of the landscape screen on the stability of the hillside is presented in Table 4. Cross-section TDS5 was modified to reflect the four conditions shown in Table 4 because the cross-section shown in Figure 7 represents the slope geometry after surficial grading and after placement of the landscape screen, i.e., the circumstances under which the landslide was triggered. The cross-section was modified using the topography before surficial grading and before placement of the landscape screen that is available from the grading plans for the Knolls development and the BC Development, respectively.

Table 4 shows that the slide mass in Figure 1 exhibits a factor of safety between 1.10 and 1.15 before any surficial grading or fill placement occurred. (This slide mass is probably different than the slide mass used to illustrate a factor of safety of 1.5 or greater to obtain a building permit for the various projects.) After the surficial grading occurred in the vicinity of the western portion of the landslide toe, the factor of safety was still between 1.10 and 1.14 indicating a stable condition and in agreement with no homeowner complaints even though three years of heavy rainfall occurred before placement of the landscape screen. After surficial grading and placement of the landscape screen, the factor of safety decreased to between 0.99 and 1.03 indicating the slope was unstable regardless of the water level. This is in agreement with the observation of tension cracks occurring in the road that intersects the slide limits near the structures in the BC Development in October 1996 (see Figure 1). These tension cracks are located in a limited area and reappeared in the same location after the 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 the road cracking was observed in October 1996. In December 1996 and January 1997, 27.4 and 22.0 cm, respectively, of the 63.8 cm of rainfall that occurred during the 1996 and 1997 rainy season occurred. This is in agreement with homeowner complaints in the Knolls development starting in late December.

In summary, this analysis and field observations confirm that movement started at the landscape screen area as early as September to October 1996 and progressed down slope until a continuous failure surface was created from just upslope of the landscape screen to the Knolls development below. When the entire failure surface had been created or mobilized, the lower portion of the slide mass started to impact the Knolls development until the landscape screen was removed in April 1998. The start of movement was caused by the coalescence of the presence of a weak layer, water in the serpentinite layer, and enough fill placement for the landscape screen, i.e., driving force, to reduce the factor of safety to near unity even for the low water case. This is reinforced by the last analysis that shows the factor of safety is unchanged if the landscape screen is in place and no surficial grading has occurred for the Knolls development.

Table 4. Effect of surficial grading and landscape screen on the factor of safety for TDS5 using a serpentinite back-calculated friction angle of 9.9 degrees

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	0.99	1.03
Landscape screen and no surficial grading	0.99	1.03

Progressive Failure

Another possible mechanism alluded to earlier in the paper for the triggering of a landslide and considered during the investigation is progressive failure because the landslide occurred 7 to 8 years after the surficial grading and the delay in the slide may have been triggered by the time required for progressive failure to develop from the slope toe to the top of the slope (Mesri and Shahien 2003). Progressive failure occurs in slopes in which the driving force exceeds the mobilized strength of the weak layer, i.e., the slope angle exceeds the friction angle of the weak layer, at the excavation. If this occurs, soil at the location of the excavation or surficial grading becomes overstressed. If the local overstressing is large enough that the soil near the surficial grading yields, the applied shear stresses are transferred to the soil element just upslope of this overstressing. If the existing shear stresses and the transferred shear stresses are great enough to cause the upslope soil to yield, the overstressing would be transferred upslope again. This process can continue until enough soil is overstressed that a slope failure occurs. If the shear strength of the weak layer soil increases sufficiently upslope, initial progressive failure can be arrested.

In this case the back-calculated friction angle for the critical cross-section is 9.9 degrees and the slope angle estimated from survey measurements along the ground surface along cross-section TDS5 is 5.9 degrees. Thus, the mobilized friction angle along the observed failure surface is greater than the slope angle. It is concluded that the soil element at the slope toe did not yield and thus a progressive failure mechanism did not commence at this site. This is in agreement with no Knolls development homeowner reporting distress until late December 1996. If the initial soil element at the slope toe had yielded and then subsequent soil elements had yielded, some of the houses and associated sidewalks and pavements in the Knolls development would have exhibited movement but none was reported.

Legal Aspects of Landslides in an Urban Environment

In some respects a landslide in an urban environment is easier to investigate than a landslide in a remote area. It may be easier to investigate because there is more hardscape present which makes it easier to delineate the limits and shape of the slide mass. However, a landslide in an urban environment usually results in substantially higher damages and thus litigation. For example in the case described herein, the total damages awarded before legal fees and costs is about $15 million. In this case a four month jury trial was conducted and the jury voted 9 to 3 against the upslope development which resulted in the upslope development paying all of the damages and legal costs.

In a landslide case, a number of causes of action can be filed by plaintiffs, i.e., the damaged parties, to recover their loses. The causes of action range from negligence, public or private nuisance, breach of contract if there is privity of contract between the parties, and trespass. The easiest cause of action is trespass because the plaintiffs only have to show that the upslope property moved outside of its property boundaries and onto the downslope properties. This can be proven via property surveys that show the upslope property "physically invaded" the downslope properties. Thus, if a landslide occurs in an urban environment it is usually easy for plaintiffs to prove trespass which can create liability for many, including engineers, developers, homeowners, and contractors.

Negligence is difficult to prove because it requires a plaintiff to prove the following four elements against the defendant: (1) the defendant owed the plaintiff a duty of care, (2) the defendant breached the duty of care by not following local industry standards, (3) the breach of care caused the landslide, and (4) the amount of damage incurred by the landslide. Negligence is difficult for a plaintiff to prove because it requires expert testimony on the local industry standard of care. Thus, plaintiffs usually pursue a breach of contract claim before a negligence action but if there is no privity of contract between the damaged parties, e.g., downslope homeowners in this case, and the defendant, upslope development in this case, a breach of contract claim in not possible. As a result, another cause of action can be sought such as nuisance.

Nuisance can be plead against private or public entities. Nuisance is also difficult to prove because it requires a plaintiff to prove the following eight elements against the defendant: (1) plaintiff owns the property, (2) plaintiff's use and enjoyment of the property is affected in some way, (3) plaintiff suffered unreasonable interference with his/her use of the property, (4) defendant acted intentionally or negligently, (5) interference was caused by defendant's use of the land, (6) interference caused substantial harm to the property, and (7) the amount of damage incurred by the landslide.

In summary, a trespass or breach of contract action is usually pursued against developers, homeowners for various activities such as property irrigation or excavation, engineers, contractors, and building departments for a landslide in an urban environment.

Foreseeability of Future Development

In addition to determining whether natural events or construction activities would start a new landslide or reactivate an old landslide, the design engineer for a downslope housing development should be concerned about the level of foreseeability that is required for their design. In general, the size of structures usually decreases as development progresses up natural hillsides. Thus, is it foreseeable that a much larger and heavier development could occur above the single-family housing developments that the engineer is designing? If so, the housing development may have to be designed with large slope stabilization techniques to ensure stability during and after construction of the upslope development.

In this case the downslope design engineer did not know that the BC Development would occur and thus did not design stabilization techniques to resist the large landscape screen. It is recommended that design engineers clearly state their assumptions in the stability analyses in regards to future upslope development so engineers for future upslope development can identify the prior assumptions and perform stability analyses to assess the impact, if any, of a future upslope development on the existing downslope development. In this case, some of the design professionals involved in the Knolls development were also involved in the BC Development which made for some interesting foreseeability questions.

Conclusion

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 only mechanism that explains ALL of the surface and sub-surface movements observed in or near the two housing developments is a deep-seated translational failure surface that follows a layer of weak serpentinite and exits beneath the downslope housing development (see Figure 7). Movement along this translational failure surface was activated when fill placement for a large visual barrier was sufficient to reduce the factor of safety to near unity in September 1996 and the movement progressed from upslope to downslope until a continuous failure surface was created.

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 determining the relationship between a weak layer, groundwater, and static and seismic driving forces imposed on the slope. Most importantly, this case history illustrates the importance of understanding the depth of influence of the proposed development via a stress distribution analysis and conducting a subsurface investigation that extends through the depth of influence of the development. If this subsurface investigation is not conducted, it would be impossible to thoroughly assess the slope behavior and its effect on surrounding development because the presence or absence of potentially problematic layers would not be known.

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, should be analyzed. If sufficient movement has not occurred along the failure surface, the factor of safety is greater than unity and thus the back analysis would under estimate the shear strength of the weak layer because the factor of safety is greater than unity and the exact value is not known. The only time that the factor of safety is known is when movement occurs and thus the factor of safety along the observed failure surface is at or near unity. It is inappropriate to disregard surface and subsurface features that indicate a translational failure mechanism and search for a circular failure surface that may yield a friction angle that is less than the friction angle back-calculated for the observed failure surface. This is inappropriate because this new failure surface has not undergone failure and thus the friction angle should be higher than the friction angle back-calculated for the observed failure surface. If it is greater than the back-calculated friction angle there is probably a flaw in the analysis with the new failure surface. However, it is suitable to vary the failure surface between known points, e.g., cracks at the top of the slide mass, shear movement observed in inclinometers, and distress at the toe of the slide mass, to ensure the minimum friction angle is back-calculated for the observed failure surface.

Acknowledgments

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LANDSLIDE IN AN URBAN ENVRIONMENT

By: Timothy D. Stark and Erik J. Newman

Figure Captions:

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

Fig. 2. View of caving serpentinite at a depth of 13 to 15 m in a 0.6 m diameter boring

Fig. 3. Annual rainfall data from nearby fire station

Fig. 4. Plan view illustrating the areas of greater than 3.5 m and 6.1 m (ellipsoidal area) of surface grading

Fig. 5. Aerial illustrating size of landscape screen in relation to a 375 m^2 single family residence (see arrow) and an excavator on the top of the landscape screen (see arrow)

Fig. 6. Aerial view of the landslide illustrating the six cross-sections considered in the forensic study

Fig. 7. Cross-section TDS 5 through the western portion of the landslide after surficial grading and placement of the landscape screen

Fig. 8. Serpentinite failure envelopes derived from torsional ring shear tests and back-analyses

Table Captions:

Table 1. Length and width of serpentinite slides

Table 2. Material properties used in stability analyses

Table 3. Back-calculated friction angles for serpentinite in cross-sections TDS1, TDS2, TDS5, and TDS6

Table 4. Effect of surficial grading and landscape screen on the factor of safety for TDS5 using a serpentinite back-calculated friction angle of 9.9 degrees