SLOPE HAZARDS INVESTIGATION
510 KULI‘OU‘OU ROAD
TMK: 3-8-010:004

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INTRODUCTION

Purpose

This report presents the results of a slope hazard study conducted by Applied Geosciences, LLC for the property at 510 Kuli‘ou‘ou Road, located in Kuli‘ou‘ou Valley, Oahu. The purpose is to address the need for an engineering slope hazard analysis, as defined in the Revised Ordinances of Honolulu, Section 14-13.3 and 14-14.2(d), and as required for areas within the Oahu Hillside Interim Development Control Area per Ordinances 89-150 and 91-90.

The property is located on the western side of the valley in an area that has been affected by landsliding in the past. One end of the Kuli‘ou‘ou landslide crosses the middle section of the lot. Numerous previous investigations have focused on this particular slide, and as a result there exists an extensive body of literature to examine implications for land subdivision and development. In addition to sliding, creep and rockfall hazards are also addressed in this report.

Scope of Work

The scope of work included obtaining and reviewing information from past studies and evaluating the site-specific geology, soil morphology and hydrologic conditions. A series of field reconnaissance visits were conducted from October 2009 through January 2010. Engineering studies that were carried out included topographic modeling, slope stability modeling, rockfall analysis, and preparation of a set of recommendations for development. A sufficient number of subsurface borings, laboratory soil tests and slope monitoring results are available for reliable assessment of slope hazards without the need for further such efforts. The available information provides a clear and consistent description of the predominant slope processes, geologic and soil conditions, and rates of displacement. Further field investigations, beyond those undertaken as part of this study, are unlikely to provide relevant information that could impact the findings that are presented herein. It should be noted that monitoring of the landslide has resumed in the last couple of years through periodic Lidar measurements. These surveys agree with our findings, as is discussed below.

This report addresses specifically the development of Area A (see Figure 13), and generally addresses the hazards of Areas B and C. Additional investigation and analysis is required if Areas B and C were to be developed.

References consulted for this study are listed at the end. The content of each of these references was carefully evaluated and only relevant and credible information was used for this study, as discussed throughout this report.

Historical Background

Slope movements in the Kuli‘ou‘ou-Papahehi area have long been recognized. Damage to housing is reported to have begun at least as far back as 1986. Sliding accelerated dramatically as a result of the New Year’s Eve storm of 1987-1988. Many residences along Papahehi Road were damaged beyond repair and were eventually demolished. Beginning in 1988, the area became the focus of numerous investigations. The United States Geological Service began a systematic study of slow-moving landslides in Honolulu, including the one at Kuli‘ou‘ou, in order to understand their relationship to earth materials, rainfall, topography, and morphology (Ellen et al., 1995). Harding Lawson Associates (HLA) mapped the extent of the landslide following the 1987-1988 storm and began monitoring slope movements with street monuments and inclinometers. These monitoring efforts were continued by HLA and other consultants until as recently as 2007. According to the State Housing, Finance &

Development Corporation, the monitoring was discontinued when the agency sold the land and the contract to monitor the site was discontinued. It is worth noting that as of 2005 landslide movement rates had ceased almost completely and therefore the landslide had attained a new state of static equilibrium.

Following HLA's initial landslide investigation, Woodward-Clyde Consultants (WCC) began a detailed and comprehensive study of the Kuli‘ou‘ou landslide complex in order to make recommendations for stabilization alternatives. A number of other limited geotechnical investigations have also been completed since, but their extent and focus are typically limited to individual properties. The work by WCC and HLA stands out in terms of the level of detail, breath and significance. Following the report by WCC in 1990, the State's Housing, Finance and Development Corporation (HFDC) approved repairs and expanded the set of drainage ditches and culverts in and around the landslide area in an effort to capture surface runoff and prevent ground infiltration from destabilizing the slope. The set of drainage ditches that is in place today is shown in Figure 1. The condition of the system of ditches was examined as part of this study.

The boundaries of the landslide reported by WCC and HLA were judged against our observations from a series of field reconnaissance visits during the time period October 2009-January 2010. We are in general agreement with the mapping provided by WCC/HLA and our results are summarized in Figure 1. Of course, there will always be some level of uncertainty in the precise extent of the landslide and past patterns of movement may not always be replicated in the future. Figure 1 shows that the southeast end of the landslide crosses through the middle of the property at 510 Kuli‘ou‘ou Road, leaving other portions of the lot unaffected.

It should be noted that some grading has taken place in the area of the toe of the slide to either side of the property at 510 Kuli‘ou‘ou Road. WCC describes a small amount of filling on the area immediately to the south (Figure 1 in WCC 1990 report), while grading is also apparent at 518 and 520 Kuli‘ou‘ou Road, right beneath Papahehi Place.

SITE DESCRIPTION

Geology

Kuli‘ou‘ou Valley is one of several deeply eroded valleys that cut into the Koolau mountain range in East Honolulu. It has formed over the last 1.8 to 2.6 million years, after the main stage of volcanic growth ceased on Oahu. The ridges that flank the valley, and indeed most of the Koolau volcanic edifice, are composed of layer upon layer of pahoehoe and a‘a lava flows dipping at about 20^{o} toward the ocean. Fractured and weathered basalt rock and layers of clinker rubble of variable cementation characterize the exposed face of the steep hillsides (Stearns, 1985; Macdonald et al., 1983). Soil cover in the Kuli‘ou‘ou-Papahehi area disappears almost entirely above elevation 160 feet. Numerous outcrops of rocks and boulders are visible on the ridge that ascends above this elevation.

A gentler talus extends between elevations of 160 feet and 30 feet. The landslide and the lot at 510 Kuli‘ou‘ou Road are located on this apron. The talus merges with a similar fan that extends down from a side canyon located northwest of the property, which is aligned roughly parallel to Kuli‘ou‘ou Valley (see Figure 1). Whereas slopes on the flank of the ridge are typically in the 30^{o} to 60^{o} range, and sometimes steeper, they decrease to about 9^{o}, on average, on the talus apron. Soil materials are distinctly different from those found on the ridge and the valley floor. They are composed primarily of colluvium and some alluvium, which originate from higher elevations on the ridge. These materials have accumulated above the original basalt basement and have continued to weather with time. The colluvium is comprised of numerous boulders and cobbles, with sizes up to several feet in diameter, within a matrix of dark gray, high-plasticity sandy clay (CH in the Unified Soil Classification System). Surface soils on the talus belong to the Lualualei soil series (Soil Conservation Service, 1972). In general, these soils are expansive and susceptible to strength loss and creep. The colluvium transitions to weathered bedrock at depths that increase from zero at elevation 160 feet to several tens of feet near the valley floor. The transition from apron deposits to bedrock is gradual, which indicates significant weathering of the parent lava flows that make up the bedrock.

Rainfall, Surface Water and Groundwater Conditions

Rainfall

There are no rainfall records for the Kuli‘ou‘ou-Papahehi area, although partial records do exist for nearby locations. Many of these records are incomplete or do not go back far enough for meaningful interpretation. The closest complete record is for rain gauge station 723.4 (NCDC 517540) on Paiko Drive, near the entrance to Kuli‘ou‘ou Valley (Figure 2). Beginning in 1976, there are several months of large precipitation stand out. Among these is the December 1987-January 1988 New Year’s Eve storm that followed on the heels of a very wet month of December 1987, and which produced accelerated movement of the Kuli‘ou‘ou landslide responsible for much of the damage to residences and other structures. Since that time, there have been three other notable events: in March of 1991, in November of 1996, and in March of 2006. Monitoring of the landslide was in effect for the first two and the results are discussed below.

Surface Water

Virtually all the studies conducted on the slow-moving landslides in East Honolulu have concluded that the main mechanism for initiation or reactivation of large-scale sliding is associated with prolonged and intense rainfall. The reason for this is that rainwater infiltrates the colluvium through cracks in the ground surface, reducing effective stresses and soil strength (Peck, 1959; Ellen et al., 1995; Brandes and Tsui, 2001). Therefore, preventing such infiltration with collection and conveyance structures can be an effective remediation technique. Ditches 1, 3 and 4 shown in Figure 1 existed at the time of the sliding in 1987-1988 (WCC Report Figure 5), although indications are that they had been damaged. In the early 1990s, following the recommendations by WCC, Ditch 2 and a collector basin where added, as shown in Figure 1. Water that flows from upland areas into ditches 3 and 4 is now directed along ditch 2 to a storm drain culvert where ditches 1 and 2 come together. Other repairs and upgrades, including improvements to the storm drain system along Papahehi Place, were also approved by the HFDC and completed in the 1990s.

Groundwater Conditions

The Kuli‘ou‘ou-Papahehi area is located well above sea level and at sufficient distant from major water impoundments such as lakes, reservoirs, streams or water-bearing dike formations. Therefore, any groundwater observed within and immediately beneath the landslide must be perched and of a temporary nature. The predominant source of groundwater is rainfall that infiltrates into the subsurface through cracks that form as a result of previous sliding, or cracks that have resulted from shrinking of the CH clay during extended dry periods. It is possible, but far from clear, that deeper more permeable zones may exist that could convey groundwater from areas further uphill, although the relative amount of seepage transported in this manner would appear to be rather small. Piezometric data from HLA and WCC indicates that water is present at some locations but not others, and that generally the water levels fluctuate in response to rainfall. In summary, previous studies suggest that groundwater within the Kuli‘ou‘ou-Papahehi area is perched, lacking significant recharge from distant sources, and that it exists primarily in the form of discrete water lenses of limited extent. Prolonged and substantial rainfall recharges such lenses through cracks and fissures, likely extending the size of such lenses, and in extreme cases resulting in a more or less continuous transient water table throughout much of the talus apron. Such a condition can result in a reduction of resistance to sliding, as is discussed in the next section.

LANDSLIDE HAZARD

Past Landslide Activity

There are no accurate records of landslide activity prior to 1986. Major efforts to study and monitor the area began after the rains of December 1987-January 1988. Based on observed distress within the new Kuliʻouʻou subdivision, WCC estimated that horizontal ground surface displacements may have been as much as 6 feet across portions of the landslide in the period from 1987 through 1990, most of it during and immediately after the New Year's Eve storm of 1987-1988. Systematic monitoring of movements began in August of 1988. A series of inclinometers and surface monuments were installed throughout the area and were monitored on and off until 2007. For the most part, these instruments captured movements subsequent to the initial large displacements resulting from the New Year's Eve storm of 1987-1988. The observed style of movement is similar to that of other slow-moving landslides in East Honolulu, with the upper portion of the landslide undergoing extension, the lower portion compression, and the flanks shearing. The general direction of movement is toward the southeast. There are some internal portions of the landslide that may be undergoing close to pure translation; i.e. they are moving without much internal deformation. The portion of the landslide that crosses the property at 510 Kuliʻouʻou Road is characterized by overall compression, with shearing taking place along the upper and lower boundaries (Figure 1). The middle portion may be translating with little shearing of the ground surface. Of course, more intense shearing has occurred at depths corresponding to the main surface of sliding, which is reflected in the form of slickensides in some of the borings.

Table 1. Cumulative Displacements Recorded in Kuliʻouʻou-Papahehi Landslide Area

	Date of Initial Reading	Date of Last Reading	Cumulative Displacement (in)
Inclinometer:			
I-3	8/17/1988	1/5/2005	3.17
I-7	8/17/1988	1/5/2005	0.97
I-9	8/17/1988	1/5/2005	0.15
I-10	8/17/1988	1/5/2005	0.09
N-1	3/27/1997	1/5/2005	0.61
N-2	3/27/1997	1/5/2005	1.89
N-3	3/27/1997	1/5/2005	1.26
Survey Monument:			
1	8/20/1988	1/5/2005	0.27
2	8/20/1988	1/5/2005	0.43
4	8/20/1988	1/5/2005	56.73
5	8/20/1988	1/5/2005	60.30
6	8/20/1988	1/5/2005	1.39
3A	3/27/1997	1/5/2005	1.72
7A	3/27/1997	1/5/2005	3.05
8A	3/27/1997	1/5/2005	2.48
10	3/27/1997	1/5/2005	0.23
11	3/27/1997	1/5/2005	1.18

Source: MACTEC (2005)

Displacement data for inclinometers and surface monuments that were in operation in 2005 are summarized in Table 1 (MACTEC, 2005). Inclinometer I-3 and monument 4 are located on or near Papahehi Road, closest to the lot at 510 Kuliʻouʻou Road (Figure 1). Shown also in Figure 1 is the direction of movement observed from the two instruments. Plots of displacement as a function of time are shown in Figures 3 through 5. The results indicate that soon after the monitoring began, the rate of movement decreased substantially. From August 1988 through January 2005, most instruments indicate cumulative displacements of about 3 inches or less. The exception is for monuments 4 and 5, for which there is a sharp jump in deformation, on the order of about 35 to 40 inches, during the first year of monitoring (i.e. August 1988 through August 1989). It is not clear why this is the case, although both monuments are located on pavement or concrete connected to Papahehi Road. Perhaps the asphalt and/or concrete moved relative to the underlying colluvium early on. In any case, displacements in all the instruments are seen to dampen quite a bit with time. Since 2001, displacements amount to less than 0.4 inches per year. Such remnant deformations are due mostly to creep.

The level of displacements indicated in Table 1 and Figures 3 through 5 agrees with our surveys from December 2009 through 2010 of structures and other features in and around the landslide. Limited deformations were observed in the system of concrete ditches, mainly at joints in the concrete linings. Ditch 1 revealed the largest amount of movement, primarily in the first portion of the structure, near to where it crosses the upper boundary of the landslide. Gaps have opened up at joints in the concrete lining, suggesting downslope displacements in line with the axis of the ditch. Some vertical settling was also noted. Shearing, which would be expected as a result of overall landslide movement in a southeasterly direction, was very limited. It appears that at least some of the displacements may have been due to localized sliding and settling of individual concrete liner sections and therefore do not necessarily reflect global sliding. It is also not clear whether the displacements in Ditch 1 predate the New Year's Eve Storm of 1987-1988 or not. In any case, the maximum inline gap measured in Ditch 1 was 3 inches and the maximum relative vertical offset was 1.5 inches. Shearing in the cross-ditch direction was 1 inch or less. Deformations in the newer Ditch 2 were even less. Only at one joint was there any noticeable deformation, which amounted to 0.5 inches and 0.25 inches in the cross-ditch and down-ditch directions, respectively.

It is clear that there has been no reactivation of large-scale sliding since the New Year's Eve storm of 1987-1988. It should be noted that this is true despite several notable rain events since that time (Figure 2). In particular, heavy rainfall during March of 1991 and November of 1996 did not result in an acceleration of movement in any of the instruments. Also, the '40 days of rain' period of March-April 2006, while occurring after monitoring ceased, does not appear to have caused substantial additional sliding. Our field observations did not reveal structural distress in the ditches, culverts, and along Papehehi Road that can be attributed to that event. It has been reported that several of the ditches were filled with runoff during that period, indicating that the new and repaired storm structures were indeed effective in reducing potentially damaging infiltration of water into the body of the landslide.

WCC estimates that the depth to the failure surface is as much as 40 feet in some places, but is generally shallower than that. Inclinometer I-3 (Figure 3) indicates a depth of about 15 feet. It is estimated that the depth of sliding within the property at 510 Kuliʻouʻou Road ranges from 0 feet at the toe to perhaps as much as 20 feet near the middle of the lot.

Slope Stability Analysis

The previous discussion suggests that whereas slope failure did occur as a result of the New Year's Eve storm of 1987-1988, there has been no additional large-scale sliding since that time. This means that the Kuli'ou'ou landslide has regained a state of overall equilibrium, with smaller displacements since attributed mainly to slow creep. Here we examine current slope stability for the property at 510 Kuli'ou'ou Road in the context of the larger regional landslide.

An important consequence of prior sliding in colluvium soils of the type found at the property is that the strength of such materials is often reduced from a peak to a residual state because of concentrated shearing along the surface of failure. The reduction in strength is quite significant. As a result, reactivation of sliding almost always occurs along the original failure surface with a greatly reduced frictional resistance.

Slope stability can be evaluated using any number of methods, ranging from simple slope stability charts to numerical techniques such as finite element, finite difference or limit equilibrium. Slope stability charts are useful for preliminary estimates or to check numerical computations, but they are limited in terms of geometries and soil distributions that they can handle. Finite element and finite difference methods are very powerful, yet they typically require the value of numerous soil properties and knowledge of how they vary in the field. This information is rarely available. Limit equilibrium represents the usual consensus for most slope stability analyses since it balances a relatively simple set of strength parameters and soil weight with powerful mathematical algorithms that allow for a reasonably detailed characterization of the geometry and stress conditions of the slope. This is also the method used by HLA and WCC. Our stability analyses were performed using the computer program SLOPE/W, which is a code widely used in geotechnical engineering practice.

Topographic Modeling

Although of direct interest is the lot at 510 Kuli'ou'ou Road, it is necessary to consider the wider talus apron from ridge flank to valley floor since any potential sliding is not limited by property boundaries. We have considered several cross sections that cut across the landslide and the property. Initial calculations showed that cross section B-B in Figure 1, which is the line of steepest descent across the lot, represents the critical cross section for slope stability analysis. This cross section is similar to section B-B' analyzed by WCC in their report. In order to develop the surface profile of this cross section, we combined detailed topographic surveying of the lot by Wesley Tengan Land Surveyor with Digital Elevation Map (DEM) data developed by the USGS for the Kuli'ou'ou area. The two data sets were merged to a common coordinate system with elevations referenced to Mean Sea Level (Figure 6).

Subsurface Geometry and Soil Properties

Conventional limit equilibrium analyses were carried out using subsurface conditions compiled from a review of available borings, inclinometer data, and visible topographic features. Our findings are similar to those reported by WCC. The main sliding surface occurs within the colluvium soil, along the portion of profile B-B that crosses the body of the landslide (Figure 1). The basal sliding surface is probably of finite thickness, although indications are that it is no more than a few feet across. We modeled this zone as a distinct soil layer with strength reduced to a residual state. This implies a low residual friction angle and no cohesion. The colluvium above the basal surface has been subjected to far less shearing and therefore retains appreciable frictional strength. The cross section used for slope stability computations is shown in Figure 7 and is generally similar to the geologic profile shown for cross section B-B' in Figure 2 of the WCC report.

For our analyses, materials were divided into five classes. WCC conducted a comprehensive review of available soil parameters from various studies and multiple sets of laboratory tests. We reviewed these results in light of our own test results on similar materials from other slow-moving landslide areas. We conclude that the soil parameters used by WCC for their analyses are largely reasonable. Of critical importance is the residual strength of the CH colluvium matrix along the basal sliding surface. WCC suggests a residual friction angle of 10^o to 12^o. Our own laboratory tests on similar CH soils, along with preliminary back-analysis of the Kuli‘ou‘ou landslide, suggest a value closer to 13^o. In summary, the following soil parameters were used for slope stability computations:

Table 2. Material properties used in slope stability analyses

Material	ϕ’ (Degrees)	c’ (psf)	γ_{tot} (pcf)
Colluvium	20	500	110
Shear Zone	13	0	110
Weathered Basalt	39	500	125
Hard Basalt	0	10,000	140

<u>*Limit Equilibrium Modeling Results and Discussion*</u>
The limit equilibrium method is based on an analysis of force and moment equilibrium for the slope. It computes a factor of safety, which is an indication of the overall state of equilibrium. A safety factor larger than 1.0 generally implies that stable slope conditions exist. In contrast, a value less than 1.0 indicates that a particular slope may be unstable. However, there is always a certain amount of uncertainty associated with any computed safety factor, which can arise due to imprecise soil properties and soil layering, simplifying assumptions made by the particular analysis technique, and computational errors. For this reason, it is standard practice to require a minimum factor of safety of at least 1.5 for design purposes.

A number of analytical limit equilibrium techniques for computing the factor of safety are available. Spencer’s method of slices was selected for our study. This is one of the most rigorous methods and insures equilibrium of both forces and moments. All five soil materials were modeled using the Mohr-Coulomb strength criteria.

Of paramount concern in assessing landslide hazard is the current state of slope stability, as well as potential states of stability corresponding to periods of elevated groundwater. As discussed earlier, there is considerable uncertainty and temporal variability regarding the subsurface groundwater conditions. We have made simplifying but representative assumptions to account for water conditions that in reality are rather complex. Nonetheless, these assumptions are considered reasonable and they account for the most important effects of groundwater within the slope.

There have always been extended periods of time when rainfall in the area has been no more than average, and often less than that. Piezometer data suggests that during much of this time there may not have been an effective groundwater table across the slope that would lower the safety factor to any significant extent. We believe that this is more or less the current state as of March 2011. The result of our slope stability analysis under these conditions yields a calculated factor of safety along line B-B of 1.01 (Figure 8). This indicates a stable slope at present time. It should be noted that a very large number of trial failure surfaces were analyzed. Invariably, the critical surface occurred within the confines of the remolded shear layer. This is because of the low residual strength of the soil in this zone. In particular, trial failure surfaces concentrated on the lower and upper portions of cross section B-B (and hence the property) yield much larger factors of safety, all well in excess of 1.5.

To investigate the effect of a future rainfall event of sufficient duration and intensity to elevate the groundwater table above the level of the remolded sliding zone, we repeated the calculations with a perched groundwater table at various elevations. The results are also summarized in Figure 8. As long as a continuous water table remains below the remolded shear zone, the factor of safety does not change. The stability of the slope starts to decrease if the water table rises above the shear zone and into the overlying colluvium since effective stresses along the weaker sliding surface decrease, and this in turn reduces the shear strength. The slope becomes increasingly less stable as the location of the water table rises closer to the surface. The factor of safety crosses the 1.0 threshold when the water table is about 3 feet above what is shown as the 'baseline' water table in Figure 8. We believe that such an elevated transient water table developed during the New Year's Eve storm of 1987-1988, causing mobilization of the landslide. Such a condition has not been repeated at any time since.

In summary, whereas the landslide is currently stable, it is conceivable that it might move again, but only under truly exceptional rainfall conditions. It appears that an event of the magnitude of the New Year's Eve storm of 1987-1988, which had a return period of 100 years or more, would be necessary to reactivate the landslide and cause large-scale sliding. Of course, a return period of 100 years represents a statistical average estimate. It does not imply that this sort of an event could not be repeated following a shorter time interval. Therefore, development within the landslide area should only proceed along with one or more stabilization measures that can be shown to increase the factor of safety to at least 1.5. The developer would have to provide a detailed and convincing rationale for insuring the safety of occupants and the wellbeing of neighboring properties and their residents. Thus additional studies and analysis will be required for Areas B and C prior to development.

Stabilization Alternatives

In their report, WCC review a series of alternatives to stabilize the landslide. Their discussion is straightforward and is not repeated here. Given that the property at 510 Kuliʻouʻou Road does not encompass the entire slide, most of the options offered by WCC are considered impractical and also much too expensive, although in principle several of them are technically feasible. As mentioned earlier, the HFDC did implement a series of improvements to the surface drainage system and there are indications that this has had a positive impact.

If in the future one or more landslide stabilization measures are to be relied upon as the sole means of making the slide area of the property suitable for development (Areas B and C), then they need to be shown to lead to a minimum factor of safety of 1.5.

CREEP HAZARD

Creep is an inherent property of all viscoplastic materials, including soils. It is most prevalent in fine-grained soils with clay fractions of high plasticity, such as smectite, montmorillonite and hydrated halloysite. These characteristics are associated with local CH 'adobe' soils (Tsui, Brandes and Nakayama, 2001). The sandy to clayey CH matrix within the upper colluvium and shear zone constitutes this type of material. Adobe soils are susceptible to large moisture changes, expansion and shrinkage. They are also known for their tendency to creep on moderately to steep slopes with a gradient greater than about 6H:1V (Lum, 1982). By creep here we mean slow downslope deformations that occur over long periods of time, but which are not associated with large-scale failures resulting from a lack of static equilibrium. In addition to creep due to material viscoplasticity, there is also a related phenomena whereby these types of soils experience downslope displacements over time due to repeated cycles of shrinking and swelling. Such phenomena in slopes can produce a 'ratcheting' style of slow sliding that in many respects is similar to viscoplastic creep. Downslope creep movements accelerate during wet periods and dampen during dry periods.

Good examples of creep are provided by the data in Figures 4 and 5. The early time portions of the curves show much higher rates of deformation than the later ones. Indeed, WCC report ground surface movements on the order of about 0.25 inches per month between April and November of 1989. Displacements accelerated to as much as 1.0 to 1.5 inches per month between December 1989 and April 1990 due to additional rainfall. These deformations presumably occurred during a period when the soil was quite wet. They also probably reflect a certain amount of residual mobilization during a period when the landslide was transitioning to a new state of equilibrium.

More indicative of pure creep are the displacements seen in Figures 4 and 5 after the first one to two years of measurements. As already pointed out, since 2001 the rate of deformation in all the instruments is less than about 0.4 inches per year. It is our assessment that this value can be taken as the average rate of downslope creep movement for areas within the boundary of the landslide under typical conditions of low to average long-term precipitation. During extended and significant rainfall periods, probably on the order of many days to weeks, creep rates may rise somewhat above this value for short periods of time.

ROCKFALL HAZARD

Numerous large boulders exist on the talus section of the lot, most of which descended by gravity from the ridge above over a period of thousands of years. A wide range of analytical computations were carried out to assess the threat posed by descending boulders. We focused on a cross section of the slope that cuts through the property and ascends to the top of the ridge, as shown in Figure 9 (line D-D). This cross section represents the line of steepest slope and is therefore considered the critical path for computational purposes. Again, we relied on a combination of site-specific topographic survey information and area-wide USGS DEM data to develop the cross section.

Assessment of rockfall hazards is a difficult proposition. In general, the slope geometry is highly variable, the location where rocks begin is often unknown, the slope material can be highly variable, and the relevant ground properties may not be known accurately. Even worse, calculations used to simulate rockfalls are sensitive to small changes in these variables. This makes characterization of rockfall hazard in terms of a single factor of safety an unrealistic proposition. Instead, rockfall analyses typically rely on probability and statistics.

Calculations were carried out using the widely used program RocFall (Version 4). For modeling purposes, we made a distinction in terms of materials located on the upper rocky flank versus those found on the lower colluvium talus, with the transition occurring at the upper boundary of the property. A wide range of possible slope roughnesses, rock masses, ground restitution coefficients, and source areas of detachment were considered. In addition, each of these parameters was specified in terms of a normal distribution to allow for statistical variability. The following values were used:

Table 3. Parameters for Rockfall Analyses

	Upper Rocky Flank		Lower Colluvium Talus	
	Mean	**Std. Dev**	**Mean**	**Std. Dev**
Coef. Of Normal Restitution	0.35	0.04	0.32	0.04
Coef. of Tangential Restitution	0.90	0.04	0.85	0.04
Friction Angle (Deg)	30	2	30	2
Slope Roughness (Deg)	-	4	-	3
Rock Mass (tons)	4	1	4	1

In order to yield reasonable probabilistic results, each analysis consisted of releasing 1000 boulders in all. The area of detachment was assumed near the top of the ridge.

Critical rockfall trajectories are shown in Figure 10. As indicated, a fair number of boulders can be expected to cross the upper boundary of the property and come to rest near the top of the lot. None are seen to travel to the middle or lower portions. This agrees with our site reconnaissance, which identified large boulders in the upper portions of the property that had clearly descended from the ridge above at some point in the past.

A common means of protecting against destructive boulders is to install a flexible rockfall barrier or rock fence. Such a fence needs to be tall and strong enough to cut off the path of descending boulders. We believe that the most reasonable place to erect such a barrier is at or near the upper boundary of the lot. Computations indicate that maximum bounce height at that location is on the order of about 5 feet (Figures 11 and 12). Also, the maximum expected kinetic energy is on the order of 1,000,000 ft-lb (1,355 kJ).

In our opinion, it would be prudent to erect a rockfall fence along the length of the upper boundary of the property with a height of at least 6 feet and a minimum rating of 2,000 kJ. Design and installation of such a barrier should be under the supervision of a qualified engineer to insure that it performs as intended.

DISCUSSION

The property at 510 Kuli‘ou‘ou Road is located in an area that has been subjected to sliding in the past. Outright failure of a large portion of the talus deposits in the Kuli‘ou‘ou–Papahehi area last occurred as a result of the New Year’s Eve storm of 1987-1988. That event had a return period of at least 100 years. We believe that a storm of this magnitude would be necessary to cause renewed large-scale sliding. It is conceivable, but not likely, that such an unusual event would occur again within the conventional 50-year design lifespan of a typical residential structure. If the landslide were to reactivate as a result of such a large rainstorm, it is possible that global sliding of up to perhaps 5 feet or so may occur again. It is notable to point out again that significant rainstorm events in 1991, 1996 and 2006 did not cause any significant movements of the landslide. It is likely that the surface drainage improvements following the 1987-1988 event, which included an improved and expanded system of ditches in and around the landslide, have had a tangible stabilizing effect.

These observations do not account for any grading or repairs that have been made since 1988. Although small grading changes took place on properties adjacent to 510 Kuli‘ou‘ou Road, we do not believe that they have had a significant impact on the overall stability of the area. It is also presumed that existing underground sewer and storm drain structures are performing adequately at present and are not leaking. We saw no indications of any major leaks during our reconnaissance. If sewer or storm water structures were to rupture in the future, they could add water to the body of the landslide and have negative consequences. However, their location on Papahehi Place is sufficiently close to the lower portions of the landslide so that even if uncontrolled leaking were to occur for a limited period of time, it is unlikely that this would lead to reactivation of the entire landslide in the absence of other causes. Of course, localized problems could still occur, but we would expect them to be confined to Papahehi Place.

The body of the landslide is quite large and massive. This means that the weight of typical residential structures resting on it is not sufficient to affect overall stability. The only conceivable manner in which residential development in the slide area could have a detrimental effect on neighboring properties or structures is if dedicated water or sewer utilities were to rupture and water were allowed to seep into the ground for extended periods of time. Specific recommendations to deal with this potential problem are discussed in the next section.

Creep deformations have occurred and may continue to take place within the confines of the landslide due to the highly plastic nature of the colluvium soil. This is especially the case during extended periods of rainfall when the moisture of the soil increases. However, creep displacements are typically one to two orders of magnitude less than displacements associated with large-scale sliding. As mentioned earlier, we would expect no more than about 0.4 inches of creep per year under normal precipitation conditions. This level of creep displacements can be accounted for in the design phase of any new structures. Creep mitigation measures may include limited excavation and replacement of soil in critical areas surrounding house foundations, retaining walls, pavements and utility corridors. Regarding the latter, all water and sewer utilities can be designed such that they remain accessible for quick inspection and are outfitted with shutoff valves.

The rockfall hazard that exists as a result of the rocky ridge behind the property can be minimized by installation of a rock fence. Other remediation techniques such as scaling, meshing, or installing rock anchors would appear to be impractical due to the size of the hillside.

RECOMMENDATIONS

Not all portions of the property at 510 Kuli‘ou‘ou Road are subject to the same hazards. Whereas the landslide crosses the middle portion of the lot, it has left areas above and below it unaffected. For purposes of discussion, it is useful to identify three general areas within the property, as shown in Figure 13:

Area A: This portion of the lot is quite flat and is not susceptible to sliding, creep or rockfall hazards. The average grade of this section is less than 1V:6H. Past experience with similar slope environments in East Honolulu indicates that such slopes remain stable and are not susceptible to damaging creep deformations. This is confirmed by our slope stability analysis, which indicates safety factors well in excess of 1.5. Indeed, housing exists to either side of Area A, some of it dating back several decades. These structures do not appear to have suffered any major damages. No continued slope monitoring is necessary for development of this area. Construction within Area A should be able to proceed without any special slope hazard considerations. Of course, expansive soils appear to be present and normal design precautions regarding expansive/contractive soils need to be followed. Pools or other water-retaining structures should be acceptable within this area. A 10-foot setback should be observed extending downslope from the edge of the landslide, i.e. the boundary between areas A and C, and this is shown in Figure 13. This setback applies to the construction of principal residential structures and pools, but not to yard areas, retaining walls not exceeding 6 feet in height, driveways, walkways, sheds, or any other non-living secondary structures. A stability factor above 1.5 is estimated for the setback area, even when a transient shallow water table is assumed.

Area B: This area is located above the upper boundary of the landslide and there is no evidence from the WCC study, or from our own study, that this area has undergone landsliding either during the 1987-1988 event or thereafter. The safety factor for deep-seated sliding for this area is estimated to be in excess of 1.5. We believe that the probability of future sliding within this zone is negligible and that construction of housing and associated structures is feasible. Because of a certain degree of uncertainty regarding the precise location of the edge of the landslide, it is recommended that critical structures, such as residences and large retaining walls, be located closer to the upper boundary of the property. A rock fence, as discussed previously, is needed to handle the risk of stray boulders crossing into the property. Given the large number of boulders on the surface, some of which may be loose, site preparation will likely involve removal of many of these. Creep movements are a minor concern and they can be accommodated in the design phase as suggested elsewhere in this report. Pools and other water-retaining structures should be discouraged within this area. Additional drainage structures should be considered so that development does not result in increased amounts of water flow toward downhill areas. Drainage provisions may consist, for example, of lined surface drainage ditches that run from side to side, roughly parallel to topographic contours, and discharging either into Ditch 1 or to street level along Papahehi Place.

Area C: This area has experienced sliding in the past, although repairs and expansion of the system of ditches subsequent to the sliding of 1987-1988 should have had a measurable beneficial impact. Without any form of further stabilization, future sliding is conceivable but not very likely. On the other hand, the risk of personal injury is deemed virtually nonexistent since even in the worst case scenario the rate of movement would be quite slow. As already noted, further investigation and analysis would have to be carried out if development of Area C were to proceed. Normal precautions regarding expansive soils and creep would still need to be followed. Pools or other water-impoundment structures should not be placed within this area.

Residential construction in any of the three areas can be designed to have no effect on the current state of stability and thereby have no negative impact on neighboring properties. Special attention should be paid to the following issues:

Grading

Any cut or fill operation that involves replacing onsite materials with imported ones, or which results in a steepening of the grade, needs to be evaluated regarding its effect on slope stability and creep. Any adverse effects of creep can usually be overcome by replacing surface soils with materials that are more resistant to creep and by stabilizing the moisture of the surface soils. Grading should not lead to concentrated runoff. If anything, grading and drainage provisions should strive to reduce the extent of uncontrolled surface runoff and infiltration.

Drainage

In order to reduce runoff, we recommend that additional surface drainage structures be considered, particularly within Areas B and C. This may include new surface ditches placed across the property, more or less along selected elevation contours, which discharge into existing or new conveyance structures. Also, runoff from all roof areas should be collected by ample rain gutter systems and discharged to elevations well below the toe of the landslide.

Utilities

Of special concern are water and sewer utilities that would be necessary to service areas B and C. These utilities need to be designed with precautions to allow for ground movement of as much as 5 feet. Since it is difficult to conceive of utilities that can handle displacements of this magnitude, utilities should be accessible and outfitted with valves that are easy to operate. Installation of some utilities above ground is another option. On the other hand, gas lines should be discouraged in areas B and C.

Foundations

A number of options are available regarding foundation systems that take into consideration potential sliding and creep movements. These may include compliant, stiff or repairable footings, usually in combination with selective grading and filling beneath and adjacent to structural areas. Footings may consist of deep or shallow elements, or some combination thereof.

Retaining Structures

Because of low strength and the potential for creep, lateral pressures from the fine-grained soil can cause larger than usual lateral pressures and sliding of footings. This is of particular concern with large retaining walls. Retaining structures and footings need to be designed accordingly. In general, the onsite colluvium soil is unsuitable as backfill or base material.

Roadways, Walkways and Miscellaneous Structures

Since the possibility of future ground displacements cannot be discounted entirely everywhere within the property, structures should generally be conceived and designed with the potential for distress. This means that they should be repairable, replaceable, or deemed non-essential. This includes access roads, walkways, ditches, fences, etc.

Final Comments

We do not envision a threat to the health or safety of individuals as a result of residential development anywhere on the property because of the generally slow-moving character of the slope processes. Regardless of any property damage from sliding, rates of soil movement will always be slow enough so that residents can evacuate safely. The threat from rock falls can be managed by installing a fence as described previously. We further believe that the risk from slope hazards to neighboring properties can be managed to remain unchanged, or even be reduced, if prudent measures and improvements are implemented as discussed in this report.

Area A has been unaffected by slope processes and can be developed without any specific concerns regarding slope instability or rock fall hazards. Area B is also outside the recognized boundaries of the landslide and is therefore amenable to development, as long as reasonable precautions regarding rock fall hazards and creep are implemented. Area C may be developed, subject to the recognition that under extraordinary rainfall conditions some sliding may occur. Standard geotechnical soil investigations should be required for development of any of the three areas, in conjunction with specific development plans. These investigations should comply with the findings and recommendations presented herein. Plans for any of the three areas should be reviewed by Applied Geosciences, LLC for general conformance with the recommendations in this report and to verify that they do not exasperate existing hazards or pose an undue threat to surrounding areas.

LIMITATIONS

This study is based in part on a field investigation and the review of field conditions, monitoring data, borings and subsurface data developed by others. The analyses and recommendations presented in this report are based on the assumption that field conditions, including groundwater elevations, do not deviate appreciably from those used to arrive at our findings.

This report has been prepared in accordance with generally accepted practices regarding engineering slope hazard investigations, as they are followed within the geographical jurisdiction in which our firm provides services. No other warranties, expressed or implied, are made as to the professional advice included in this report and none should be inferred.

The report has been prepared to address the requirement for a slope hazard investigation for development of the property at 510 Kuli‘ou‘ou Road. It is intended for an evaluation of hazards and risks and is not meant to provide information for bidding or any other purposes. Additional geotechnical and civil engineering planning and analyses will be required for final design. The report was commissioned by 510 Kuliouou LLC and shall not be transferred to other private parties without the written consent of Applied Geosciences, LLC.

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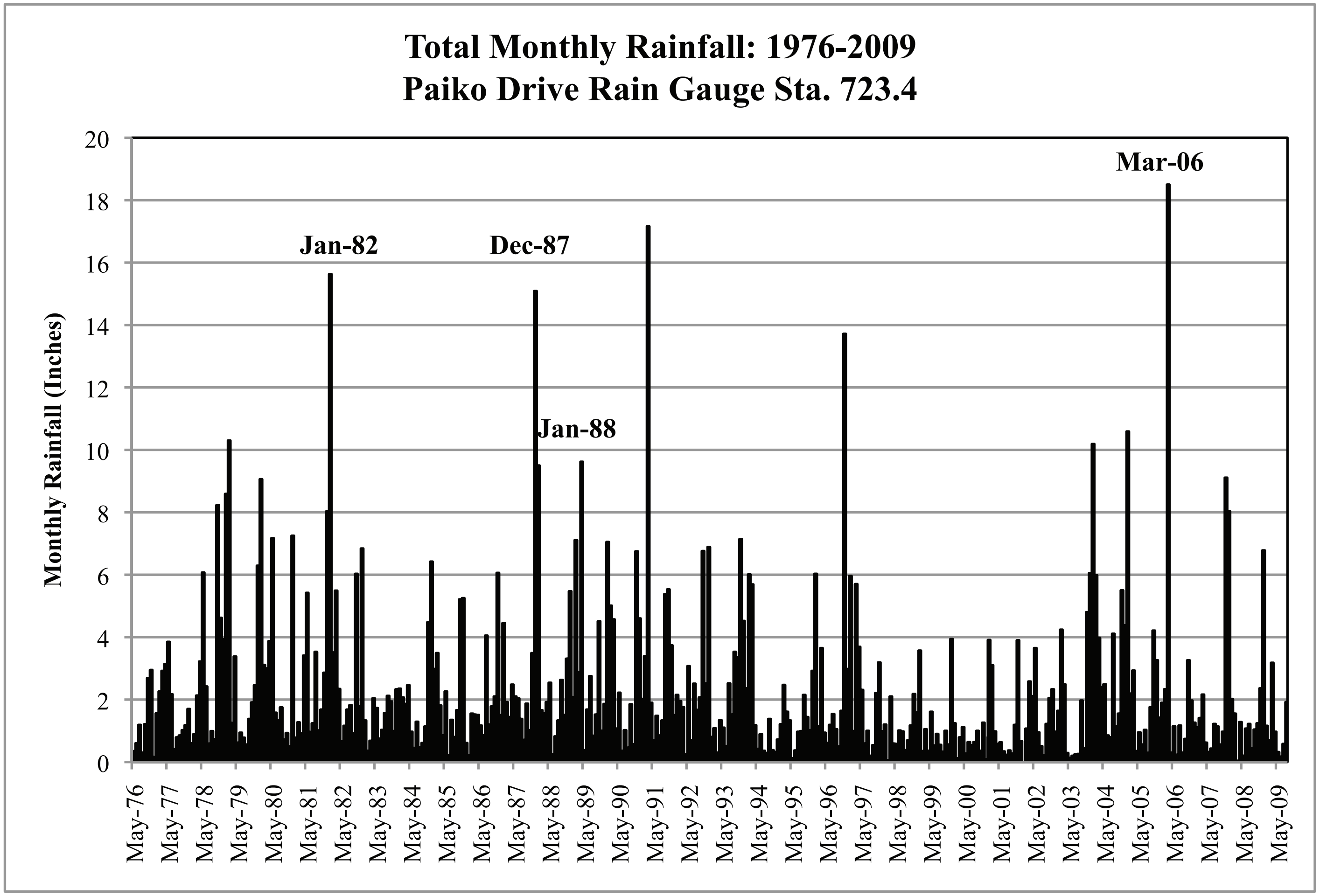
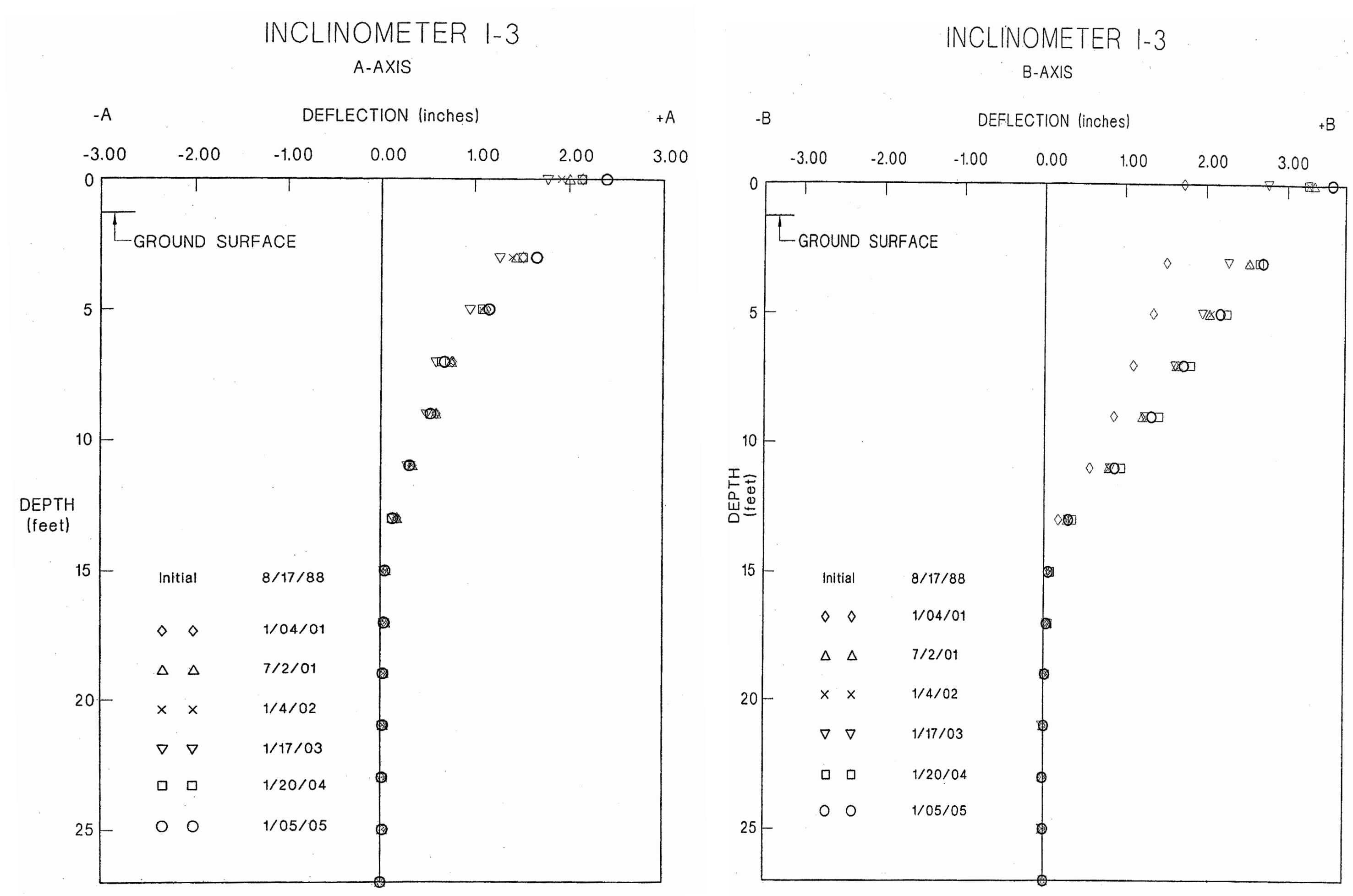
Project: 510 Kuliʻouʻou Road Date: January 31, 2010 **Applied Geosciences, LLC**	Kuliʻouʻou-Papahehi Area	Figure No: 1



Project: 510 Kuli‘ou‘ou Road Date: January 31, 2010 ***Applied Geosciences, LLC***	Kuli‘ou‘ou Rainfall Record		Figure No: 2



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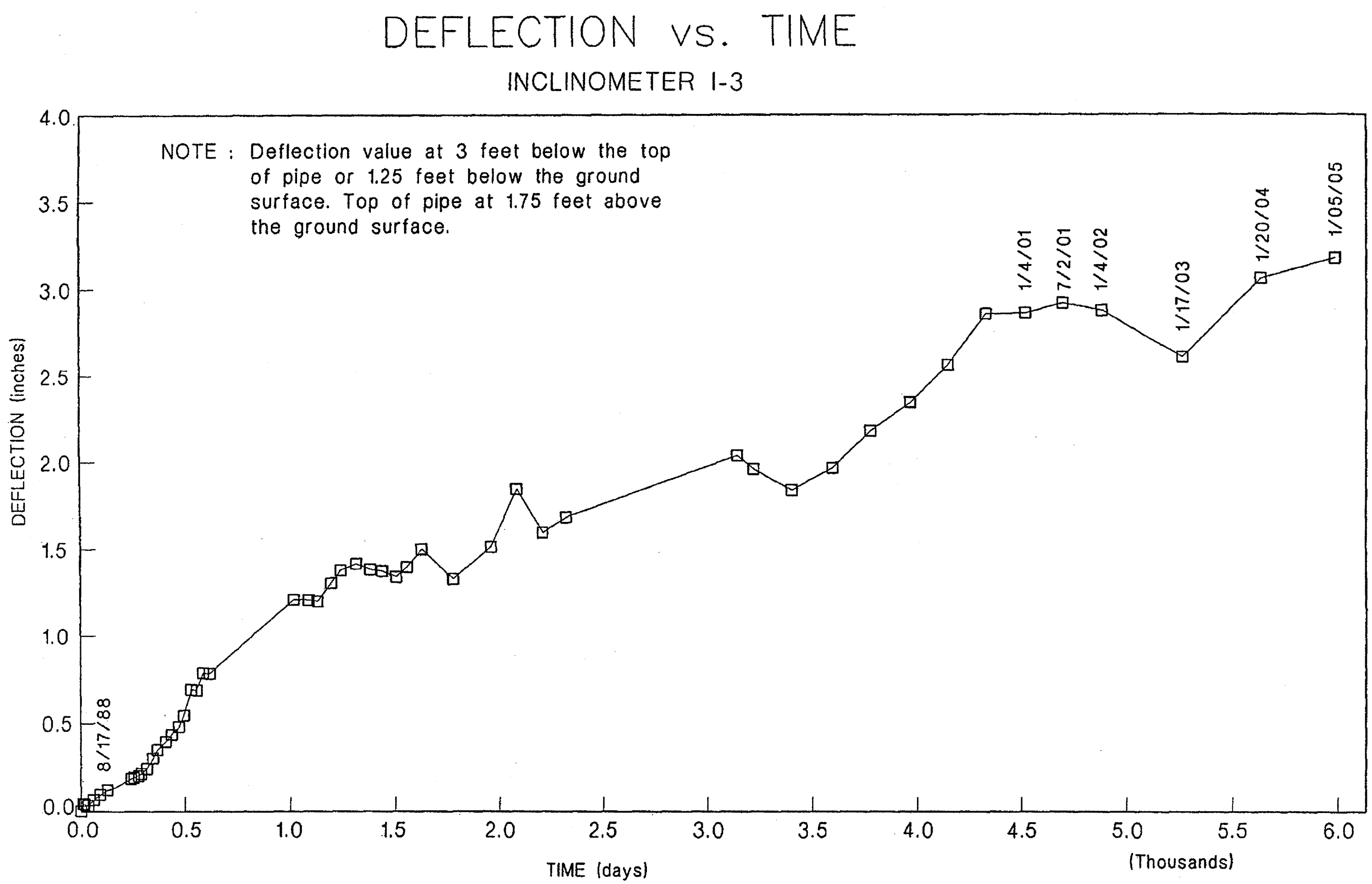
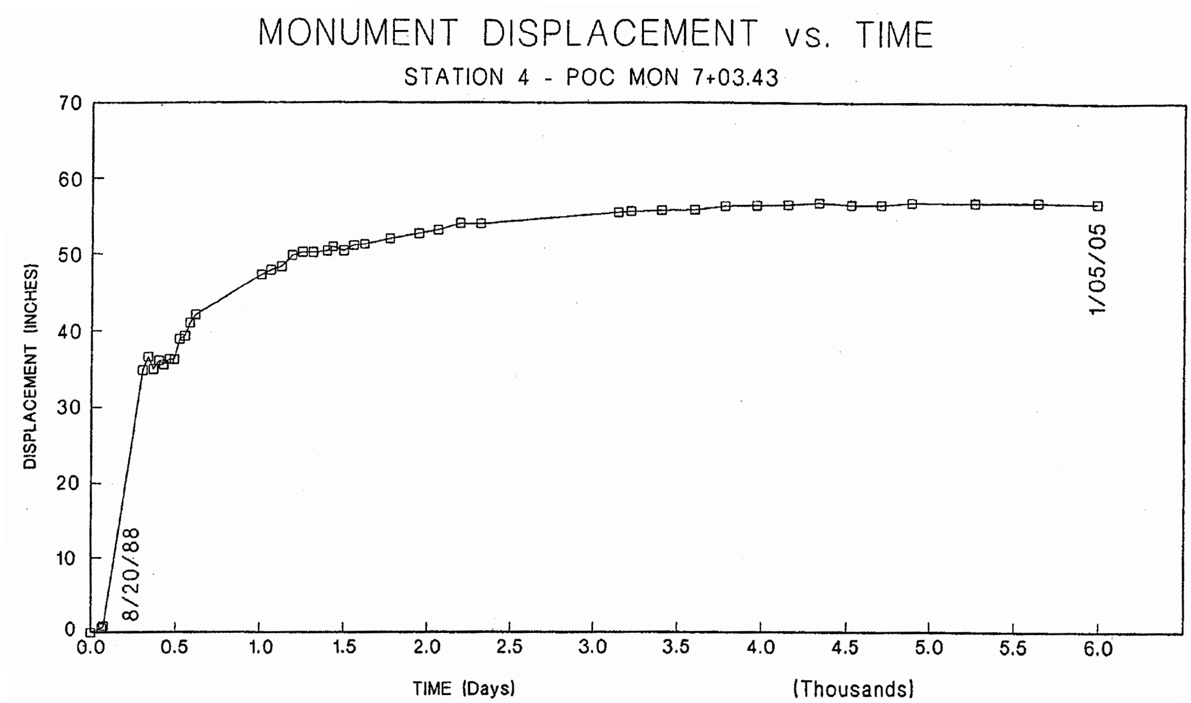
I-3 Inclinometer Downhole Record: 1988-2005 (MACTEC, 2005)

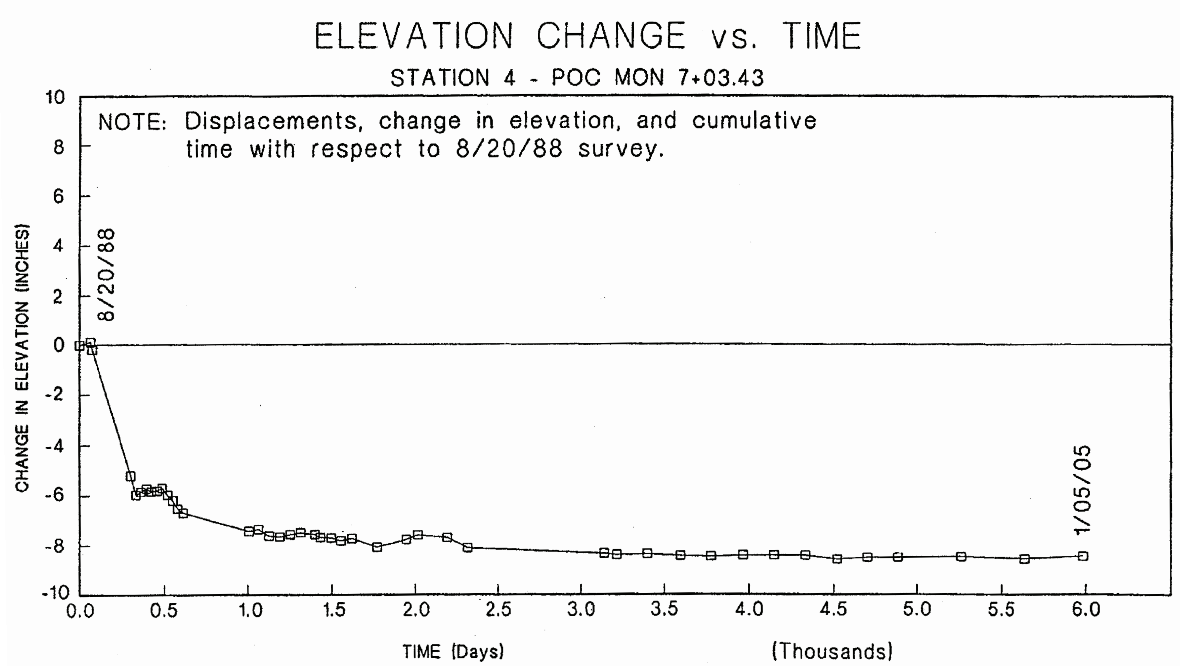
Figure No: 3



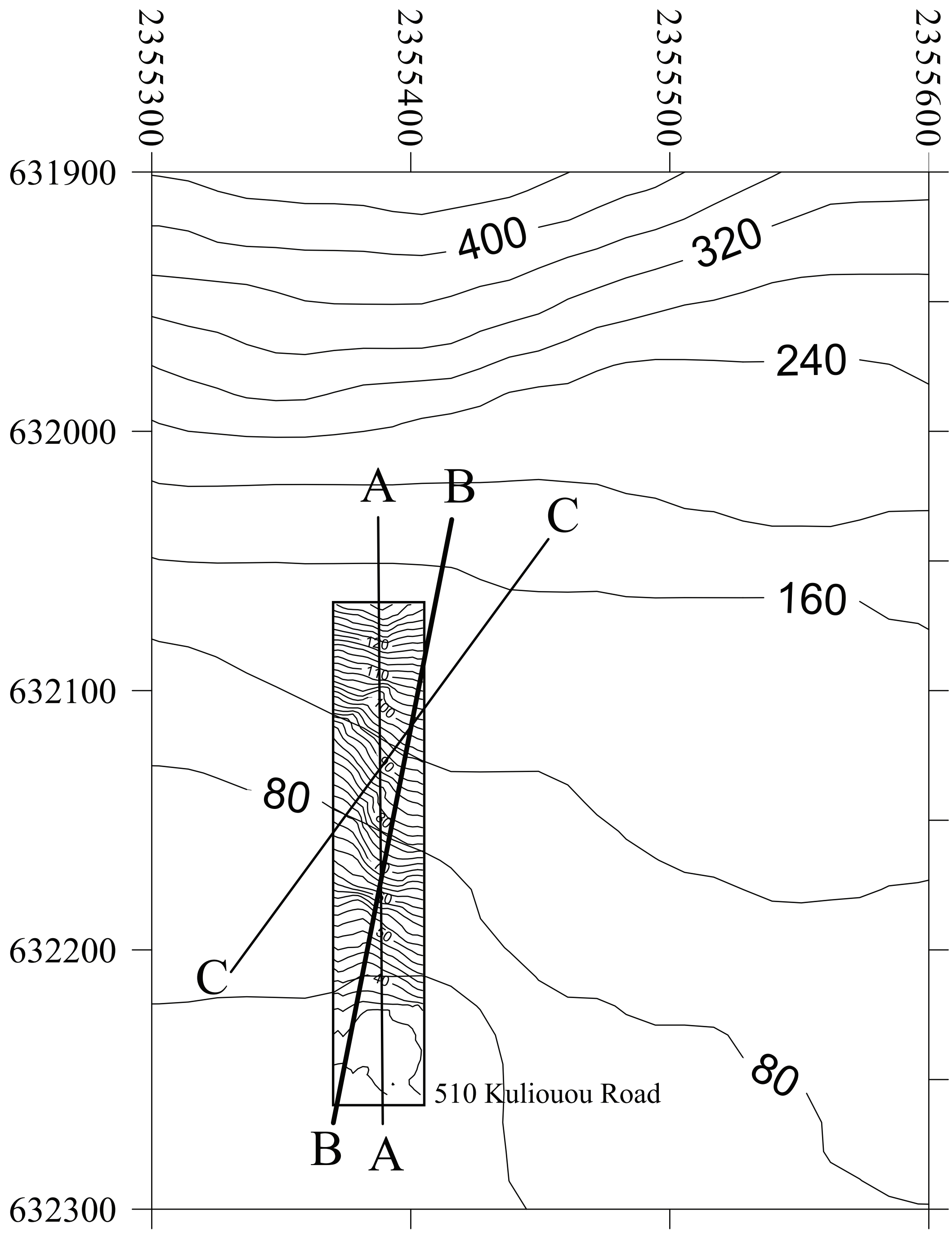
Project: 510 Kuli'ou'ou Road Date: January 31, 2010 **Applied Geosciences, LLC**	I-3 Inclinometer Horizontal Record: 1988-2005 (MACTEC, 2005)		Figure No: 4

Time Period: 1988 - 2005





Project: 510 Kuli‘ou‘ou Road Date: January 31, 2010 ***Applied Geosciences, LLC***	Monument 4 Displacements (MACTEC 2005)	Figure No: 5



UTM Coordinates
Elevations in feet above MSL

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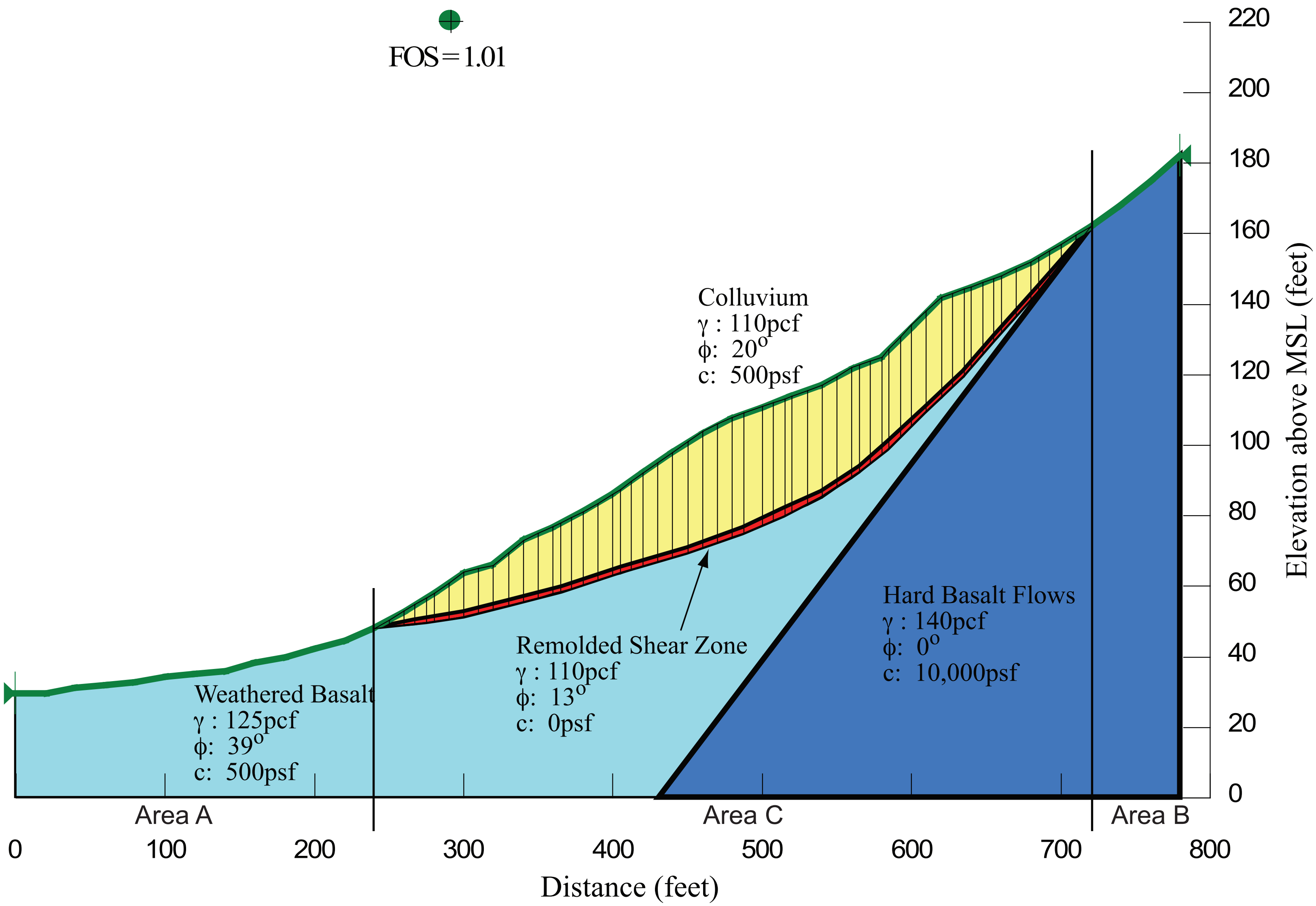
Applied Geosciences, LLC

Cross Sections For Slope Stability Analysis

Figure No: 6

Cross Section B-B



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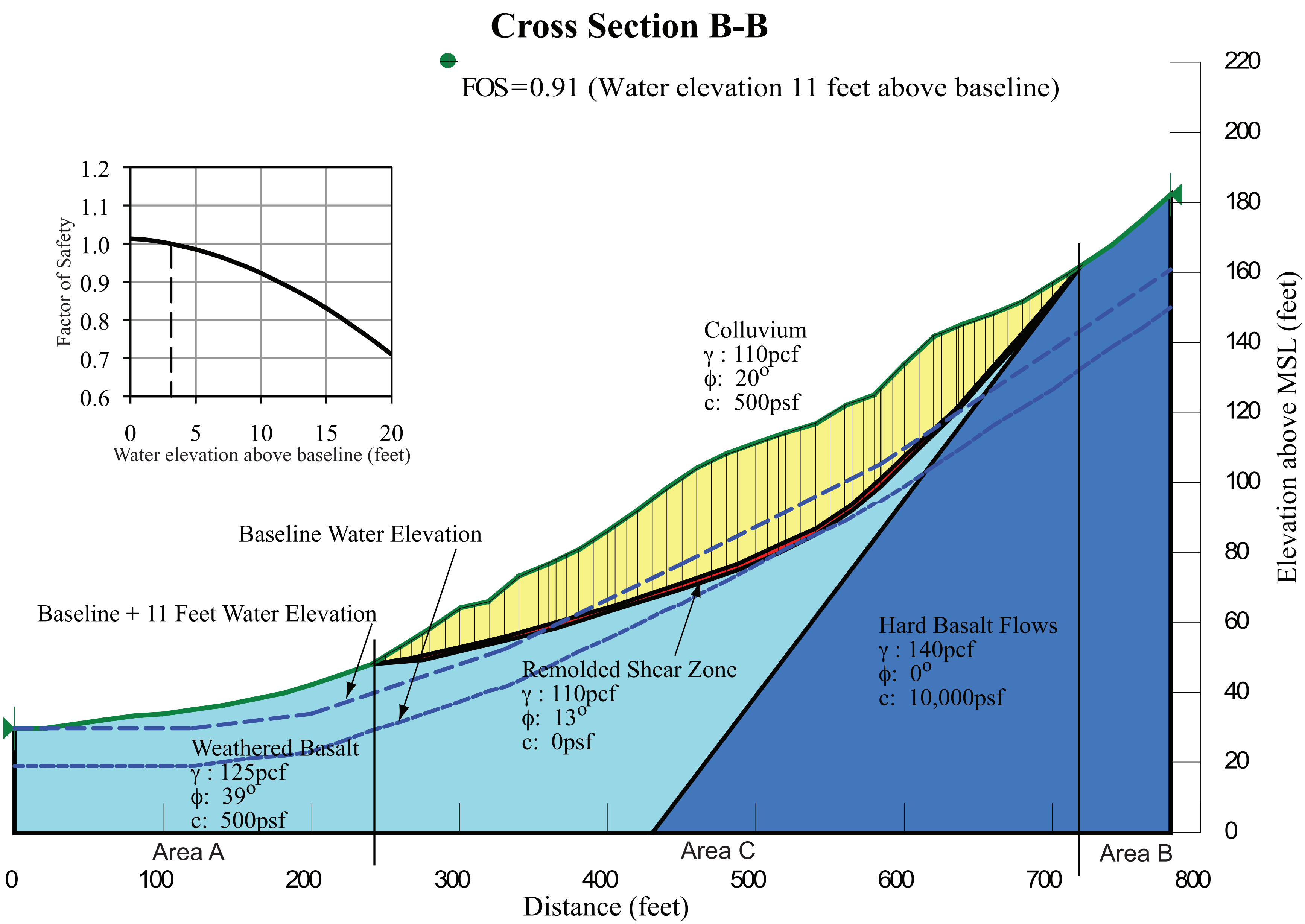
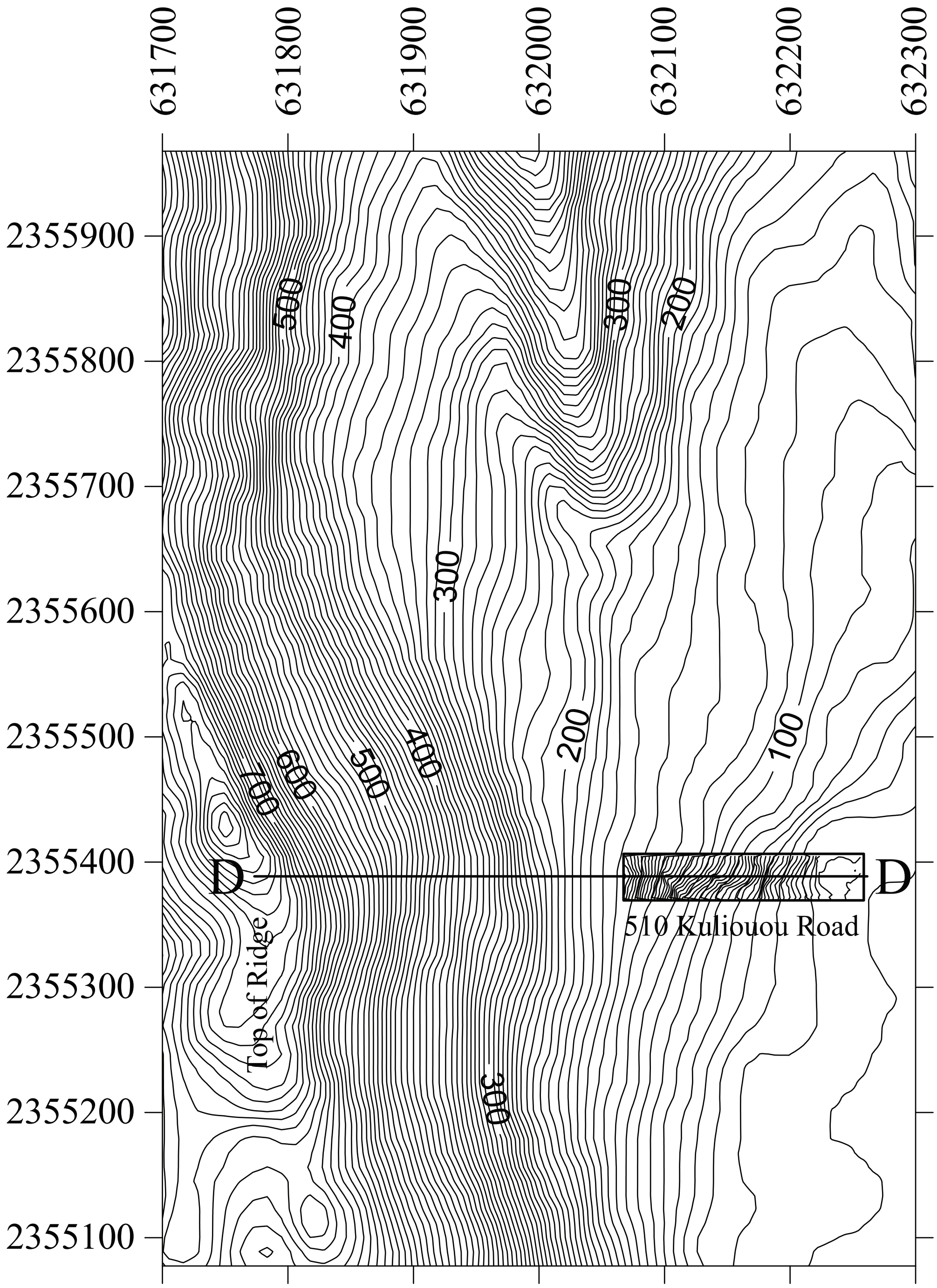
Current State of Slope Stabilty Without Groundwater

Figure No: 7



Project: 510 Kuli'ou'ou Road Date: January 31, 2010 *Applied Geosciences, LLC*	Effect of Groundwater on Current State of Stability		Figure No: 8



UTM Coordinates
Elevations in feet above MSL

Project: 510 Kuli‘ou‘ou Road
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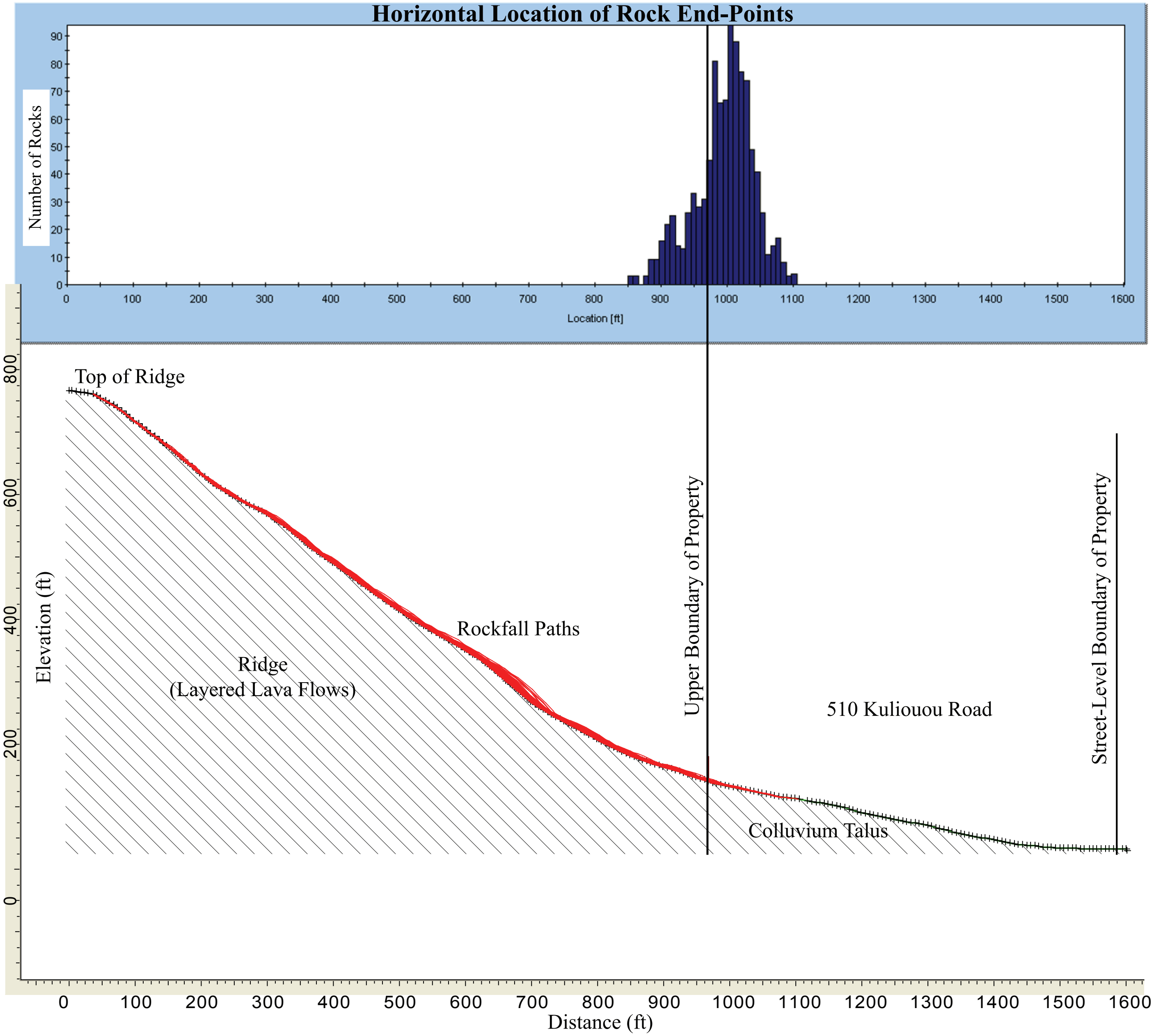
Cross Section For Rockfall Analysis

Figure No: 9



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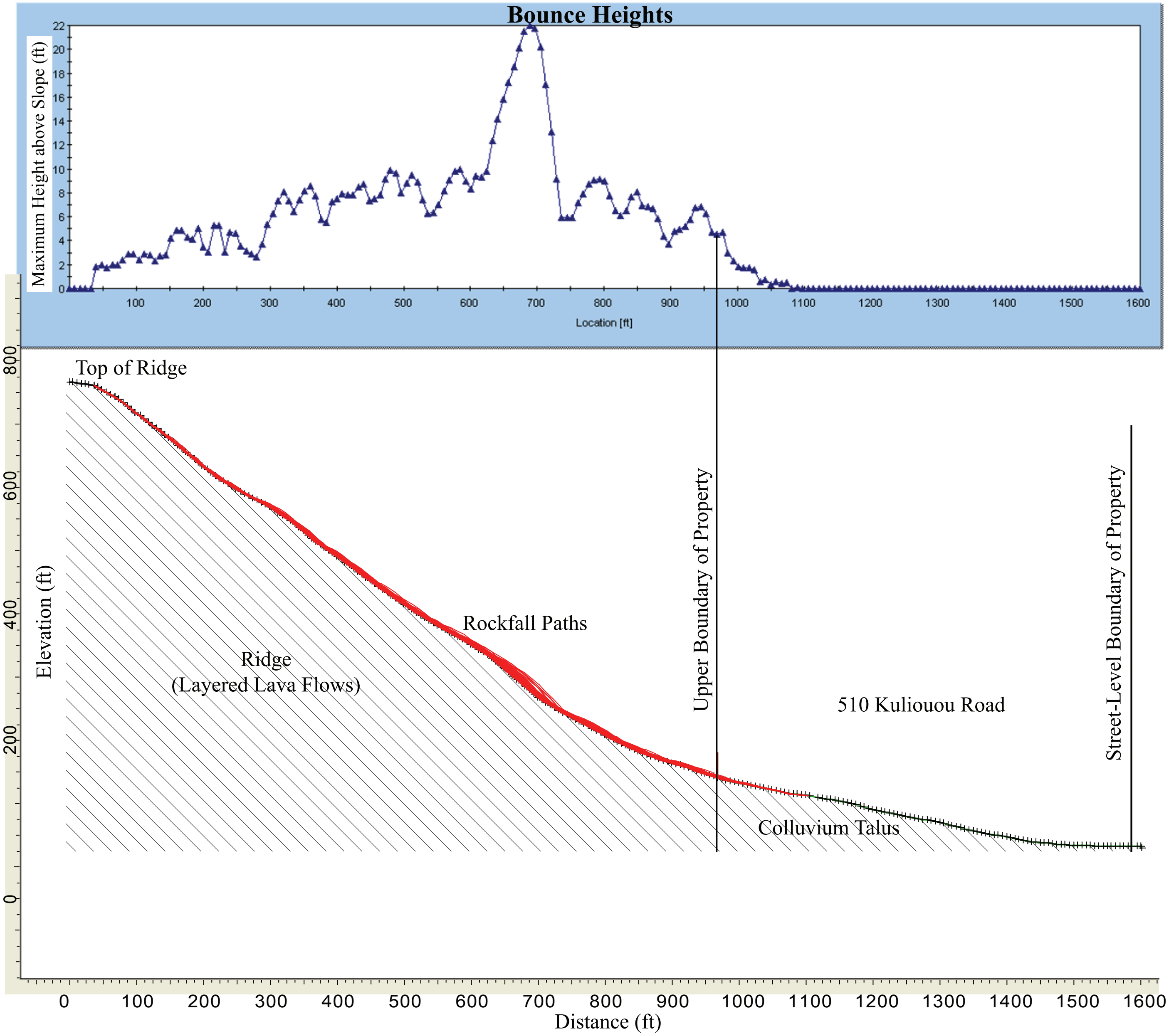
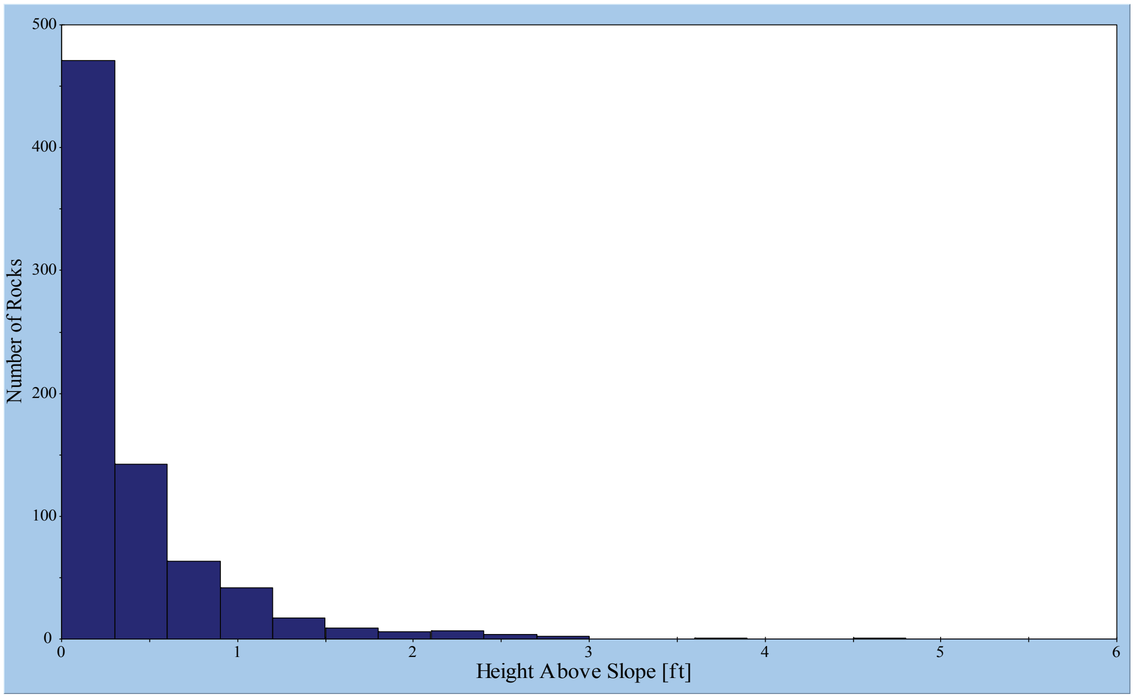
Rockfall Analysis: Location of Rock End-Points

Figure No: 10

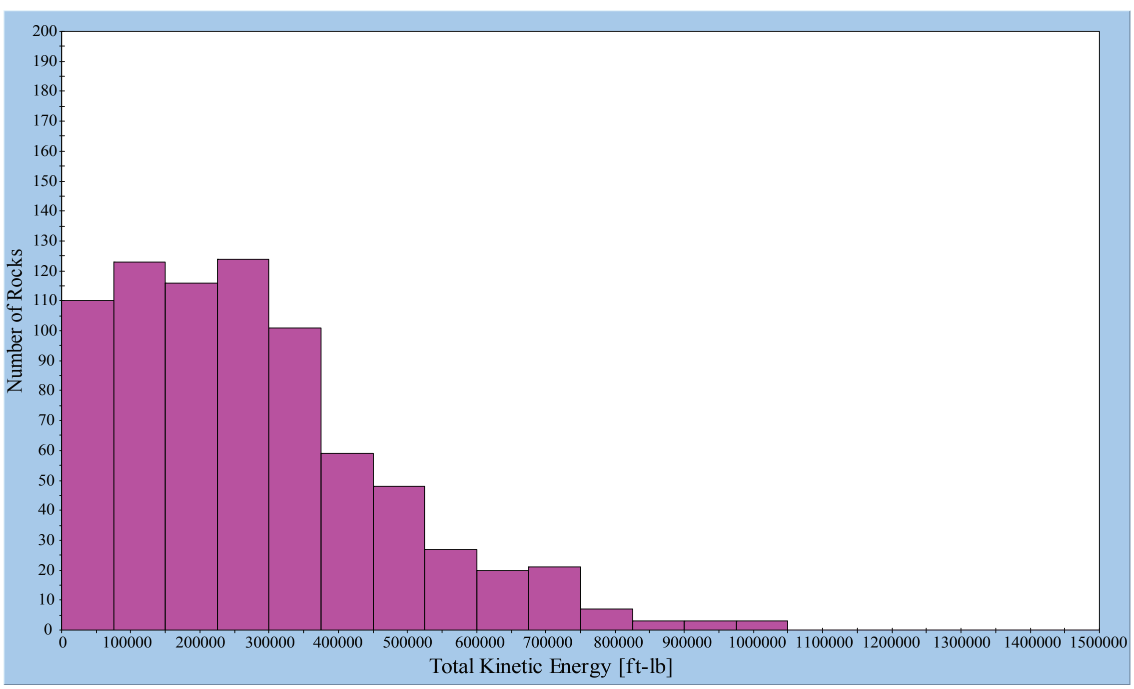


Project: 510 Kuli'ou'ou Road Date: January 31, 2010 ***Applied Geosciences, LLC***	Rockfall Analysis: Maximum Bounce Heights		Figure No: 11

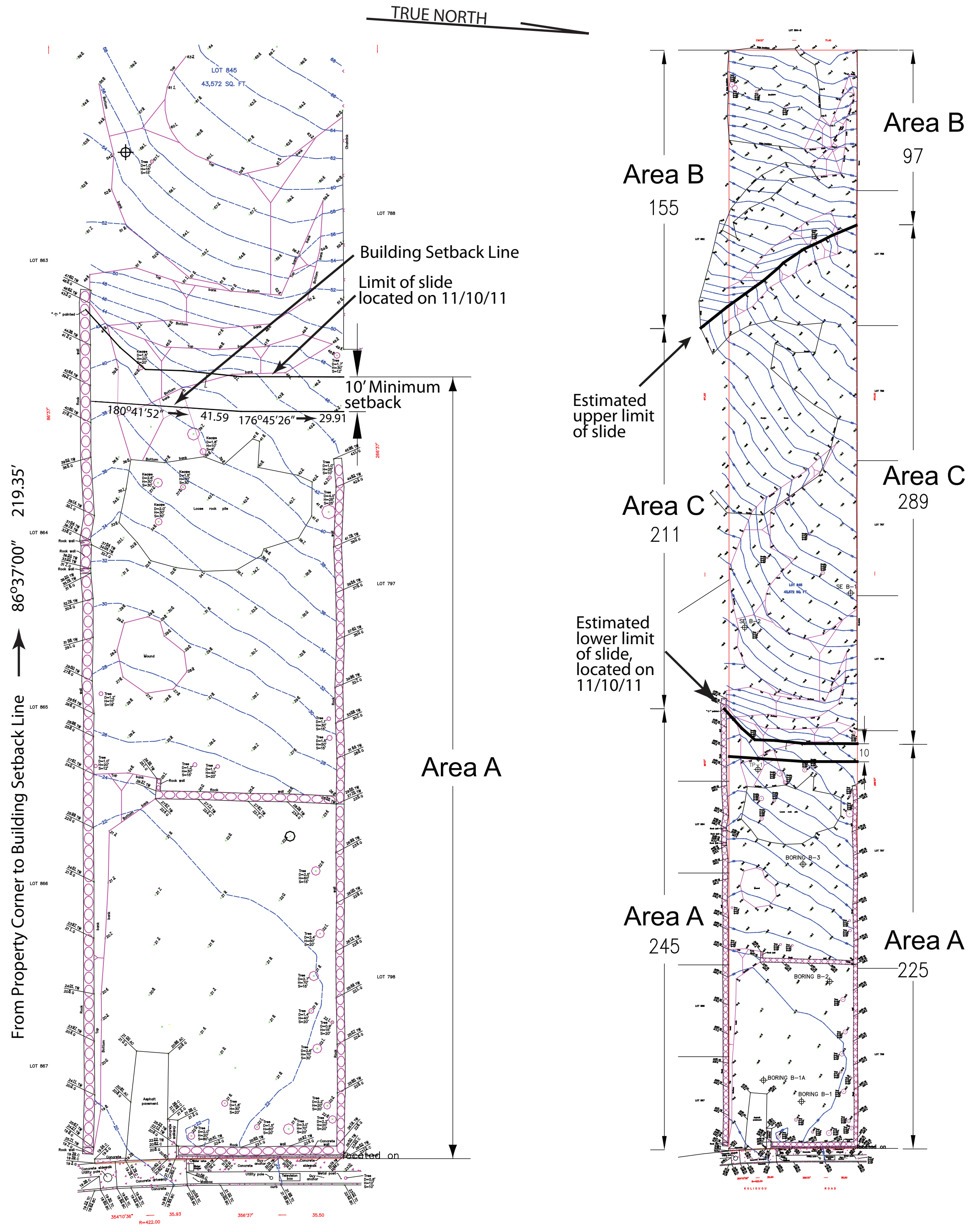
Bounce Height Distribution at Upper Boundary of Property



Total Kinetic Energy Distribution at Upper Boundary of Property



Project: 510 Kuli'ou'ou Road Date: January 31, 2010 **Applied Geosciences, LLC**	Rockfall Analysis Results at Upper Boundary of Property		Figure No: 12



Project: 510 Kuli'ou'ou Road Date: January 31, 2010 **Applied Geosciences, LLC**	Property Hazard Areas	Figure No: 13