Landslide Stabilization Using High Strength Aggregate-Cement Slurry

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ABSTRACT: Most large landslides are repaired via some combination of mass grading, structural restraint systems, and/or dewatering. A deep seated landslide occurred in Dana Point, California, in 2005 that resulted in the destruction of one home and the red-tagging of three others. Several attempts were undertaken to completely repair the landslide over a five-year period, but all attempts failed due to a lack of funds and concerns over landslide enlargement into adjacent properties. During preliminary design, a repair scheme was developed involving conventional grading and stabilization methods. To mitigate the potential for off-site landslide enlargement, an expensive caisson and tieback active restraint system needed to be integrated into the repair, along with a large toe surcharge and buttress for stability. However, this system resulted in the repair scheme being economically and logistically unviable. A unique repair scheme was then developed in conjunction with the contractor that utilized a high strength aggregate-cement slurry (ACS) keyway design in combination with a multi-faceted construction phasing scheme. The ACS consisted of a pourable mixture of aggregate rock, cement, and water. Construction phasing consisted of two phases: a global component and a keyway component. Design, construction methods, and testing will be discussed.

INTRODUCTION

The landslide is located on a slope behind four residential homes along Philemon Drive in Dana Point, California (Figure 1 – Location Map). The failed slope is an approximately 60-foot-high, 2:1 (vertical to horizontal) fill slope constructed in 1984 using typical state of the practice grading techniques (i.e., keyway at the toe, benching into natural materials, backdrain system, etc.). The fill slope is keyed into a natural, approximately 3:1 slope existing at the base of the fill slope (Figure 2 – Pre-Failure Cross-Section).

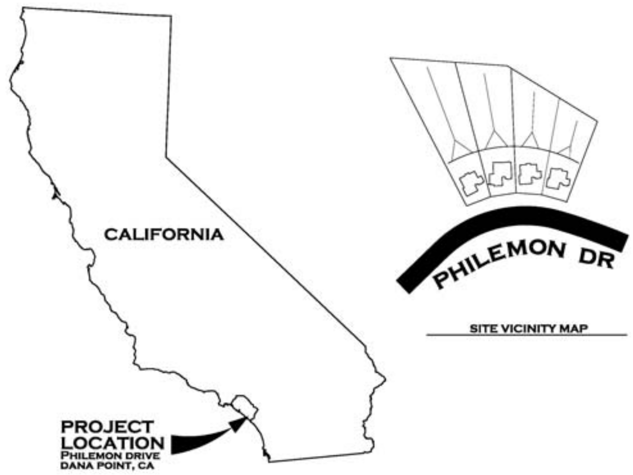

FIG. 1. Location Map

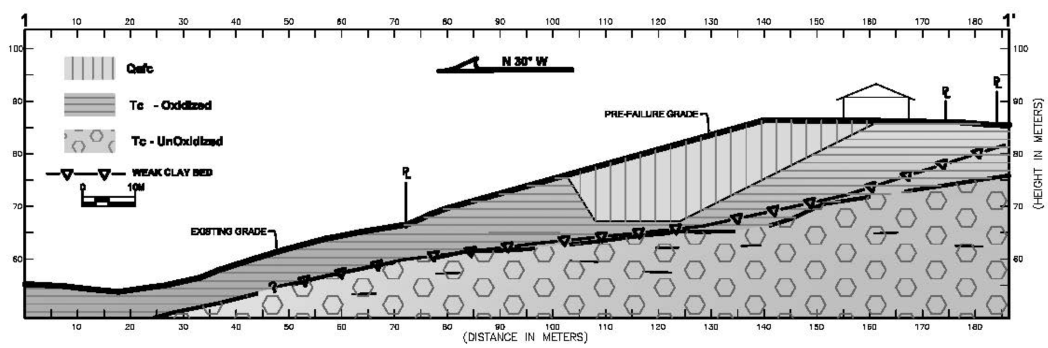

FIG. 2. Cross Section of Pre-failure Conditions

The slope remained in service with no recorded problems until the early 2000's when some repairs were completed to address what was thought to be at the time "slope creep issues". In the spring of 2005, after record heavy rainfall, the slope grossly failed (Figure 3). The resulting damage included demolition of one home, emergency underpinning of a second home, and significant distress to the rear yards of two other homes.

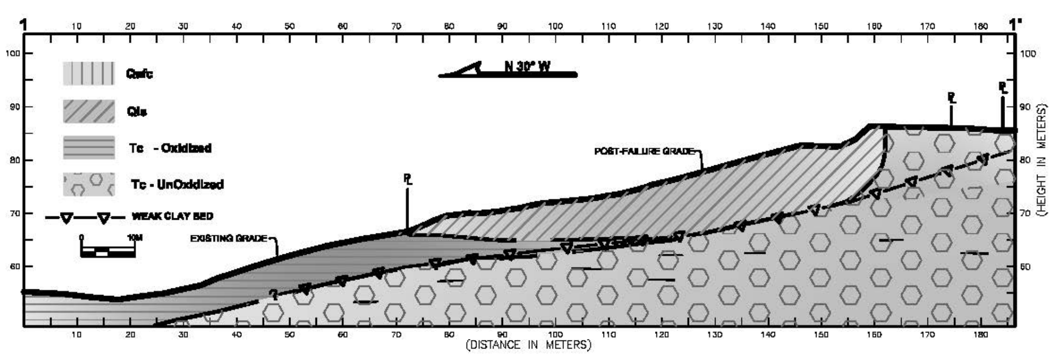

FIG. 3. Cross Section of Post-failure Conditions

Geotechnical investigations by several consultants were undertaken as part of subsequent litigation. In general, the investigations concluded that the landslide occurred due to a build-up of groundwater and adverse geologic conditions within the Capistrano Formation. However, these investigations did not yield a comprehensive and viable repair for the entire failure. In 2009, three of the lots (i.e., the main lots above the landslide) were purchased by a private developer/contractor for the purpose of repairing the landslide and redeveloping the lots.

During the winter of 2009 and 2010, minor ground cracking was observed along the limits of the landslide, indicating that movement was slowly occurring (i.e., creeping). A winterization plan was developed to minimize the potential for future movements until a permanent repair could be implemented. The plan consisted of three components: 1) the installation of horizontal drains; 2) re-grading of the slope face; and, 3) surface drainage improvements. Following the implementation of winterization, it was noted via visual observations that the rate of landslide movement was significantly decreased, but some movement was still occurring. In addition, the horizontal drains were producing small outflows of water. Two slope inclinometers installed by previous consultants (Figure 4 – Geotechnical Map) were monitored during and following the winterization. These inclinometers indicated that the movements were limited to the landslide mass.

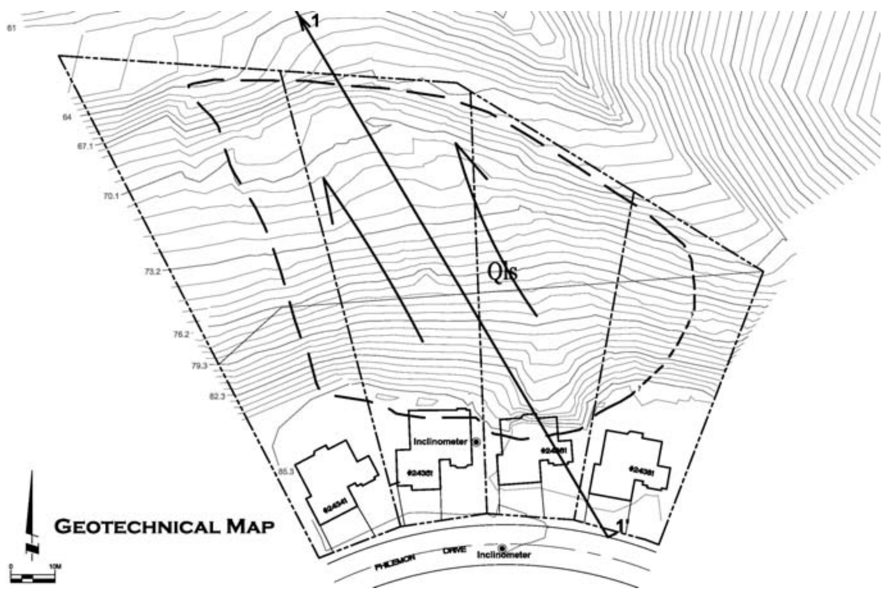

FIG. 4. Geotechnical Map

In order to design a permanent repair for the landslide and residential lots, investigation and analyses were conducted to prepare a repair plan. As discussed in subsequent sections, several options were considered prior to the chosen aggregate-cement slurry (ACS) design. The choice of this design required detailed construction phasing including both global and keyway components. The global component involved determining, via stability analyses, which section of the landslide could be repaired and in what order. The keyway component involved cutting phased narrow construction slots within each global component down below the landslide plane and then filling the excavation with the ACS material that was mixed on-site. The chosen repair design and methods were implemented in late 2010 and 2011 to stabilize the landslide and reconstruct the damaged lots.

LANDSLIDE CONDITIONS

The landslide that occurred at the site in 2005 was comprised of engineered fill materials and oxidized bedrock of the Capistrano Formation. As discussed below, no evidence of a pre-existing landslide mass was found during the subsurface investigation.

Geology

The materials underlying the site prior to failure included engineered fill and bedrock of the Capistrano Formation. The engineered fill, as discussed above, was placed during grading of the residential tract in 1984 and was derived from the underlying Capistrano Formation bedrock. The Capistrano Formation underlying the existing engineered fill is comprised of gently folded siltstones and claystones with rare, thin, highly plastic clay beds that are typically prone to failure when saturated and/or adversely oriented. In addition, the oxidized portion of the Capistrano Formation, observed to range from about 30 to 50 feet in thickness below the site, tends to be significantly weaker and prone to failure. In general, the Capistrano Formation bedrock found underlying most of south Orange County tends to fail within the oxidized portions of the unit, particularly when continuous adversely oriented clay beds are present. Failures within unoxidized portions of the formation are rare, likely due to the sharp increase in shear strength between the oxidized and unoxidized portions.

Groundwater

Eyewitness accounts and geotechnical exploration performed shortly after the landslide occurred indicate that the lower portion of the oxidized bedrock was likely saturated at the time of failure. Exploration and observations made just prior to and during on-site winterization and construction indicated that groundwater was perched on top of the unoxidized bedrock and water was within the landslide mass. Groundwater was not observed within the unoxidized bedrock. These groundwater conditions were considered to be a critical factor in the failure of the slope in 2005.

Landslide Mechanics

The failure that occurred in 2005 was likely due to adverse geologic structure exacerbated by saturated soil and bedrock conditions. A thin, highly plastic, adversely oriented clay bed was observed at the base of the failure just above the unoxidized bedrock (see Figure 2). The elevation of this clay bed places it below the original keyway constructed in 1984, causing it to be ineffective in preventing the failure (Figure 3). In addition, the general geologic structure underlying the site is a synclinal structure with an axis oriented in the downslope direction, creating a gently curving bowl-like shape, which likely contributed to the perched water condition and subsequent failure of the material along the saturated weak clay bed. As discussed above, this type of geologic scenario within the oxidized Capistrano Formation is highly prone to failure.

Further complicating the failure and subsequent repair were the conditions surrounding the landslide, predominately the existing residences and the City street upslope of the landslide. The clay bed that was determined to be the failure plane continued to the south (upslope) of the landslide, as noted in the boring drilled within the residential front yards (see Figures 3 and 4). Temporary stability of these adjacent properties was considered critical and could have easily been compromised by reactivation of the landslide mass or propagation of the headscarp upslope of the existing failure area.

GEOTECHNICAL ENGINEERING PARAMETERS

The Capistrano Formation bedrock materials underlying the site range from sandy lean clays to highly plastic, expansive clays. Liquid limit (LL) values for the bedrock excluding the “clay bed” rupture surface ranged from 30 to 55 while Plasticity Index (PI) values range from 16 to 32. The materials are largely fine grained materials (i.e., 90 percent to 95 percent passing the number 200 sieve) with 2 micron clay contents ranging from about 25 percent to approximately 40 percent. The clay bed rupture surface had LL and PI values of 68 and 47 respectively with a 2 micron clay content of 57 percent.

Shear strength values for the Capistrano Formation materials were determined in general conformance with Recommended Procedures for Implementation of DMG Special Publication 117 – Guidelines for Analyzing and Mitigating Landslide Hazards in California (2002). The strengths were developed from direct shear testing, strength correlations (Stark, Et. Al., 2005) and local experience. As shown in Table 1, the strength model contained strengths for landslide debris (i.e., bedrock materials within the landslide mass), portions of the rupture surface that were along bedding as well as portions along a clay bed, intact oxidized and unoxidized bedrock materials below the landslide, as well as engineered fill to be used in the repair.

Table 1 – Capistrano Formation Material Shear Strengths

Material Description	φ (degrees)	c (kN/m^2)	γ (kN/m^3)
Capistrano Formation Landslide Mass/Debris	24	8.4	19
Capistrano Formation Rupture Surface/Clay Bed	Shear/Normal Function (12-14.5 degrees)	0	19
In-place - Capistrano Formation (unoxidized)	30	33	19.5
In-place - Capistrano Formation (oxidized)	23	23.9	19.5
Capistrano Formation Rupture Surface/Along Bedding	21	0	19
Engineered Fill (derived from Capistrano Formation)	24	14.4	10.5

To verify the Capistrano Formation strengths, a back-calculation was performed. The back-calculation was aided by the groundwater and stability conditions noted following winterization. These conditions enabled the use of the following assumptions: 1) groundwater levels could be assumed to be at the horizontal drain elevations; and, 2) the landslide was slowly moving so that a safety factor of 1.0 could be reasonably assumed. The results of the back-analysis verified the above strengths.

INITIAL REPAIR OPTIONS

Two types of repairs were initially evaluated: 1) grading repair only; and, 2) grading repair with structural restraint for temporary stability.

Grading types of repairs are normally significantly less costly than structural repairs; therefore, a grading solution was sought first. Adequate factors of safety (as required by the reviewing agency, the City of Dana Point) of 1.5 static and 1.1 pseudo-static for the grading repair could only be achieved with a fairly large buttress and considerable toe-surcharge. The relatively low design shear strengths for the engineered fill to be placed within the keyway were the primary cause of the low factors of safety obtained. The resulting large buttress and surcharge resulted in both a logistics issue as well as a cost issue. The logistics issue centered on traffic concerns relating to trucking imported soil to the site through City residential streets. The cost issue included cost of the import as well as costs associated with street damage. These issues could likely have been solved; however, the most significant design constraint involved how to excavate the main keyway to the depth required in a manner that did not induce movement of the landslide material left in place, potentially putting at risk existing homes and the City street. This rendered the option unviable, as keyway slots that would likely not induce significant upslope movements would be too small to obtain adequate compaction.

Based on the problems associated with the grading-only option, additional stabilization methods were evaluated to maintain stability during grading and construction above the landslide. These methods included the utilization of caissons and tiebacks to supplement the grading-only repair design. One of the schemes developed is shown on Figure 5. This scheme resulted in a smaller keyway and less toe-surcharge, but the cost of the structural system with the grading was still too expensive and consequently deemed to also be unviable. It was further concluded that a stand-alone structural system would also be unviable from a cost perspective.

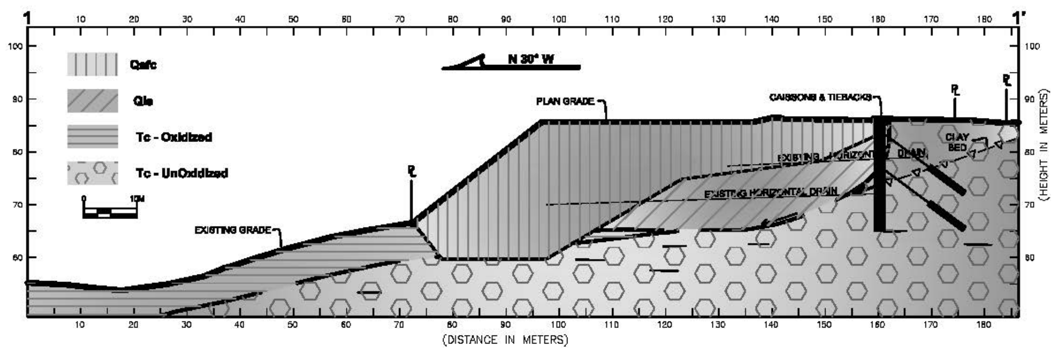

FIG. 5. Cross Section of Keyway/Toe-Surcharge Repair Option

AGGREGATE CEMENT SLURRY (ACS) REPAIR

Following unviable cost and logistics evaluations of the "initial" repairs, additional alternatives for repair were evaluated. A repair had to be developed that was both cost effective and required lesser amounts of imported earth materials and maintained stability during construction. To address these issues, the use of high strength materials placed into narrow width slots (i.e., that could be narrow enough to maintain temporary stability of the landslide mass) was investigated (Figure 6 – ACS Repair). The owner/contractor suggested that because of the required slot depth, the high strength material could be poured into place, which would streamline construction and result in a significant cost savings.

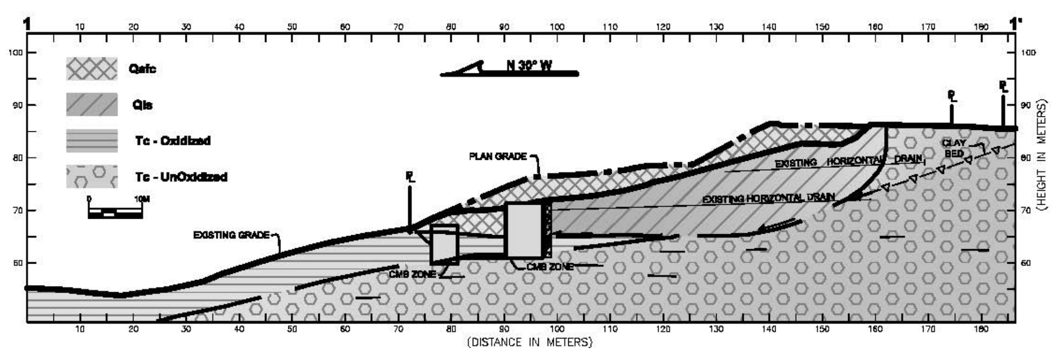

FIG. 6. Cross Section of ACS Repair

Strength Testing, Results, and Field Slots

To determine the viability of the pourable slurry, strength testing was conducted using poured mixes of Crushed Miscellaneous Base (Report by GMU, 2010) mixed with water and cement. The selection of trial slurry mix proportions was selected based upon: 1) the ease of mixing during construction; 2) cost; and 3) predicted strength. Strength testing results for the trial slurry mix designs are shown in Table 2.

Table 2 – Design ACS Strength Results

Cylinder Number	CMB Aggregate Weight (kg)	Cement Weight (kg)	Water Weight (kg)	7-Day Compressive Strength (kN/m^2)
A1	17	1.5	3.0	296.5
A2	17	1.5	3.0	248.3
B1	17	1.2	2.5	324.1
B2	17	1.2	2.5	296.5
C1	17	1.5	1.75	1,655.0
D1	17	2.25	2.0	2,110.2

The percent moisture content was calculated based upon the weight of the dry aggregate plus the weight of the cement. The percent cement content was based upon the weight of the dry aggregate.

A seven day compressive strength of 2,110 kN/m^2 was selected for design based on cost considerations and discussions with the contractor. To account for field placement variability, a safety factor of 3 was utilized for shear strength determination. Consequently, a design shear strength for the ACS was developed based on an unconfined compressive strength of 703 kN/m^2. The shear strength determined was phi = 35 degrees and cohesion = 144 kN/m^2.

Stability Based Design

Parametric stability analyses were performed using varying configurations of ACS filled keyways to obtain the required safety factors. Shear strengths utilized included those in Table 1 as well as the design shear strength determined from ACS strength results as discussed above. The parametric analyses also involved iterative discussions with the contractor in regards to constructability.

The final design consisted of three keyways cut through the toe portion of the failure (Figures 6 and 7). Keyway 1 is approximately 58 meters long and 6.1 meters in width. Keyway 2 is located downslope of Keyway 1 and is approximately 4.6 meters in width and approximately 61 meters long. Keyway 3 is an extension of Keyway 1 and is approximately 52 meters long and is also 6.1 meters in width. The depths of the keyways were controlled by the elevation of the unoxidized bedrock contact, which the keyways needed to extend into for stability. Consequently, the height of the keyways ranged from approximately 3.7 meters to 10.4 meters. In addition to the three keyways, a toe fill surcharge with a height of 4.6 meters was required to achieve the required slope safety factors.

ACS Slots and Construction Sequence

Once the final dimensions and locations of the ACS keyways were developed, the maximum width of the keyway slot needed to be determined. In other words, how large a portion of the keyway could be excavated and filled with the ACS without causing renewed movement of the landslide. The width of the keyway slot was also limited by the quantity of material that could be poured in one working day. After consulting the contractor, it was preliminarily decided to excavate the keyway in slots ranging from 15 to 20 feet. Field verification testing was then undertaken to verify that the slot widths would be stable at the 15- to 20-foot widths.

The sequence with which the keyways were to be excavated was also considered to be critical, primarily due to the temporary stability concerns for the adjacent properties and existing residences, and secondarily due to constructability within the small confines of the site. Keyway 1 (the largest keyway) was chosen to be excavated first, starting on the east side. This excavation would be within the central portion of the landslide and would provide stability of the bulk of the landslide mass for excavation of Keyway 2. Because the failure direction of the mass was towards the northwest along bedding, beginning the excavation of Keyway 1 on the west side was considered inadvisable due to concerns of reactivation of the mass when the keyway was open. Upon completion of Keyway 1, Keyway 2 was to be excavated in the same manner. Keyway 3 was to be excavated last, and was considered to potentially be the most difficult to excavate due to the steep existing slope, the existing residence at the top of the slope, and the large portion of landslide mass that would not yet be "locked in" place by Keyways 1 and 2. However, this portion of the repair was ultimately designed to be excavated last due to the potential for significant upslope propagation of the failure and damage to the existing residence should the entire landslide mass reactivate.

Prior to excavation of the slots, the contractor would be required to remove the upper portion of the landslide mass in the keyway area to create a construction bench. A steel form would be brought on site to provide sidewall stability to pour the keyway to the top ACS elevation. As shown on the Construction Schematic, Figure 7, ACS Keyway 1 was to be excavated via this method first, followed by ACS Keyway 2, and finally ACS Keyway 3. Upon completion of the keyway excavation, the keyway backdrain system was to be excavated and installed.

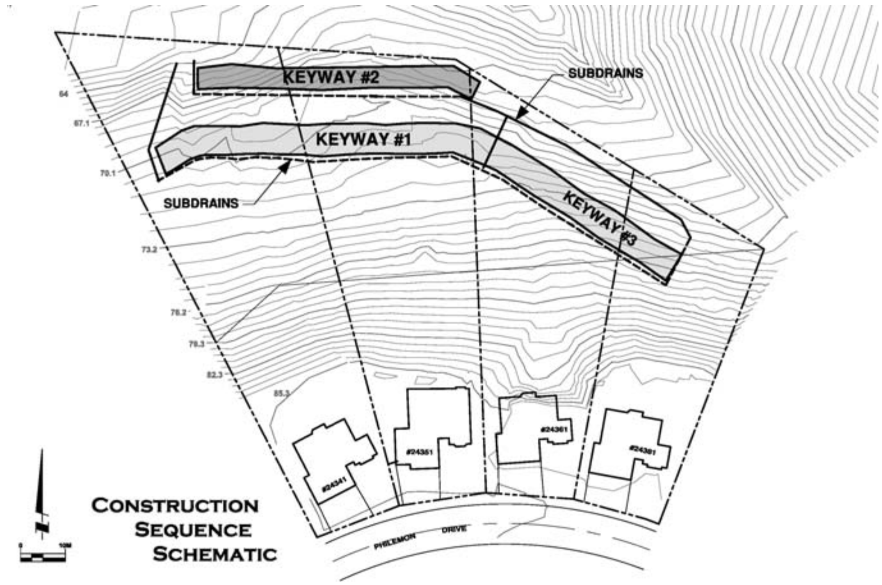

FIG. 7. ACS Keyway Locations and Construction Schematic

Back-drainage

In order to minimize hydrostatic pressure building up behind the ACS keyways, a keyway backdrain system was designed behind the ACS portions of the keyways. Given the projected difficulty in constructing a traditional backdrain with a pipe and gravel "burrito" with outlets at equally spaced intervals, a unique system was designed that included constructing a French-type drain with outlets at each end of the ACS keyways. The drain was designed to be excavated in slots against the back of ACS Keyways 1 and 3, filter fabric was designed to be placed against the entire trench, and the trench was then to be backfilled with aggregate to the top of the ACS keyway block. The backdrain for ACS Keyway 2 was designed as a typical keyway backdrain "burrito"-type system within the engineered fill behind the keyway.

Final Grading

After construction of the keyway and backdrain system was completed, remedial removals of the compressible portions of the landslide were to be excavated upslope of the keyway. The entire landslide mass was not to be removed due to the potential risk of damage to the upslope and adjacent properties. Exploration and laboratory testing indicated that the lower portions of the landslide consisted of relatively intact bedrock blocks that were considered to be adequate to leave in place for support of the design fill slope. Structural setbacks were required in the rear yards because this material was left in place. After removal of the compressible portions of the landslide mass, engineered fill was to be placed to design grade to re-build the original slope and residential rear yards. Surface drainage in the form of a terrace drain system was designed to control surface water and minimize future erosion and surficial failure.

CONSTRUCTION

Equipment

In order to excavate and pour the ACS keyways efficiently, a unique combination of equipment was utilized. The sloping nature of the site as well as the design depth and width of the keyway slots contributed to the complexity of the construction setup. The equipment used included a track-mounted grout plant, a Cat D6 dozer, a Cat 950 loader, and several excavators, including a Cat 345, a Cat 320 with a vibrating rod attached, and a modified Cat 350 with a side-mounted mixing and discharge hopper attached. This mixer-excavator was built by the contractor and utilized the excavator motor and hydraulic system to operate the hopper.

FIG. 8. ACS slots

FIG. 9. Cement and water mixed in on-site grout plant

FIG. 10. Cement water mixture pumped through a hose

FIG. 11. Mixer-excavator with side-mounted hopper

FIG. 12. Inside of hopper

FIG. 13. Collecting aggregate to add to hopper

System

The Cat 345 excavator was set up on an equipment bench at the toe of the keyway, and was used to excavate the ACS slots (Figure 8). After excavation and verification that the slot had been excavated into unoxidized bedrock, cement and water were mixed in the grout plant per the design mix ratio (Figure 9). The grout plant was located at the top of the site, adjacent to the residential street, so that cement delivery trucks could easily access the plant. The mixture of cement and water was then pumped via hose (Figure 10) to the mixer-excavator and into the side-mounted hopper (Figure 11). The mixer-excavator was set up on an equipment bench upslope of the keyway slot that was close enough to swing the discharge chute over the slot such that the ACS would pour directly into the slot. As the hopper filled with the cement-water mix (Figure 12), a loader was used to add a set amount of aggregate to the hopper (Figure 13). Once the hopper had mixed the aggregate cement and water together forming the ACS to the satisfaction of the engineering technician, the discharge chute was opened and the ACS was poured into the slot (Figures 14 and 15). The Cat 320 excavator with the vibrating rod was used periodically throughout the pour (Figure 15). If required for the slot construction, a form would be set up to complete the ACS pour to the design elevation (Figure 16). The poured slot would be allowed to cure overnight, and the following workday the ACS would be adequately set to allow removal of the form and excavation of the adjacent slot with no adverse affects on the previously poured section (Figure 17).

FIG. 14. ACS is poured into slot

FIG. 15. Cat 320 excavator with vibrating rod

FIG. 16. A form was set up as needed to contain ACS pour

FIG. 17. Excavation of adjacent slot

ACS Strength Testing Verification

Representative samples of the ACS were collected for unconfined compressive strengths during the construction to confirm that the design strength of 2,110 kN/m^2 was achieved. At the beginning of construction every slot was tested. Strengths at 3, 7, 14, and 28 days were determined for each sample. The results indicated 28 day strengths ranging from 2800 kN/m^2 to 7186 kN/m^2 with an average of 4700 kN/m^2. The frequency of testing was reduced to one test per roughly every four slots based on the initial test results.

Construction Landslide Movement Monitoring

Movement monitoring during construction consisted of visual observations of the slopes, rear yards, and street areas as well as the monitoring of slope inclinometers installed behind the landslide mass and within Philemon Drive. Minor movements within the landslide mass were noted via visual observations during the excavation of Keyway 1. These movements ceased following the completion of Keyway 1. To check for movement outside the landslide mass, two slope inclinometers were read throughout construction in addition to visual observations. The slope inclinometers did not indicate movement at any time during construction and visual observations confirmed the lack of movement outside the landslide mass.

CONCLUSIONS

Use of the ACS keyways in conjunction with a detailed construction phasing plan provided a unique cost-effective solution to a long-standing unremediated landslide. Prior to the development of the repair procedure, no entity had been able to develop a global repair which was economically viable. Consequently, the City and the local residents were left with a condition that was visually unappealing, a safety hazard, and posed a risk to the City street and utilities. In addition, the presence of the landslide likely put downward pressure on real estate values of neighboring properties. Implementation of the repair enabled the failed area to meet City slope stability standards and enabled the re-use of the three previously red-tagged homes as well as the improvement of adjacent lots.

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REFERENCES

Committee Organized through the ASCE Los Angeles Section Geotechnical Group Document, (2002). “Recommended procedures for implementation of DMG special publication 117 – Guidelines for analyzing and mitigating landslide hazards in California.” Southern California Earthquake Center.

GMU Geotechnical, Inc., (2010). “Preliminary geotechnical investigation report for repair of slope at rear of 24341, 24351, 24361 and 24381 Philemon Drive, City of Dana Point, California.”

GMU Geotechnical, Inc., (2010). “Response to geotechnical review comments by the City of Dana Point, 24341, 24351, 24361 and 24381 Philemon Drive, City of Dana Point, California.”

Public Works Standards, Inc. (2012). *Standard Specifications for Public Works Construction.*

Stark, T.D., Choi, H., and McCone, S. (2005). “Drained shear strength parameters for analysis of landslides.” *Journal of Geotechnical and Environmental Engineering.* (Volume 131 (5)), 575-588.