Case History: The Estrondo Landslide Stabilization in Encino, California

Daniel Pradel, Ph.D., P.E., D.GE, G.E., F.ASCE[1]

[1]Professor of Practice, Dept. of Civil, Environmental, and Geodetic Engineering, Ohio State Univ., 491C Hitchcock Hall, 2070 Neil Ave., Columbus, OH 43210. E-mail: pradel.1@osu.edu

Abstract

During the 1998 El Niño rains, a landslide damaged a 1.5:1 (horizontal to vertical ratio) fill slope approximately 34 m (110-ft) in height that was graded around 1960. The slope was built in an urban neighborhood using low strength compacted fill derived mostly from the local diatomaceous bedrock. The slope had previously experienced several deep failures, which had been locally repaired by removal and recompaction, only to fail again. By 2003, the landslide had moved significantly and there were significant concerns about the safety of neighboring homes. This article describes: (1) the design and implementation of a long term repair to stabilize the landslide using a buttress, two rows of drilled shafts, and tiebacks; (2) the conventional slope stability and numerical modeling analyses used to determine the critical mode of failure for design; (3) the implementation of the landslide stabilization; and most importantly (4) how combining different structural elements resulted in an unanticipated critical failure mode.

INTRODUCTION

During the 1998 El Niño rains, a landslide damaged the lower 23 meters (75-feet) of a 1.5:1 (horizontal to vertical ratio) fill slope approximately 34 meters (110-feet) in height in Encino, California. The slope instability was approximately 85 meters (280 feet) wide, and had a basal plane slightly under 10 meters (30 feet) in depth (Figures 1 and 2). The landslide occurred at the south end of a canyon that was filled around 1960 in order to build a street across it and residential lots. We understand that the lower slope experienced at least three failures at the same location prior to 1998, and that previous failures were locally repaired by removal and recompaction. The exact dimensions of these previous failures and their repairs is not well known as limited records from the failures that occurred between 1969 and 1983 and their repairs have survived. Similarly, few records about the original grading were available for review.

Figure 1. Aerial photographs from www.Pictometry.com showing the failed slope covered with plastic (top), the repaired slope (middle), and the slope recently (bottom).

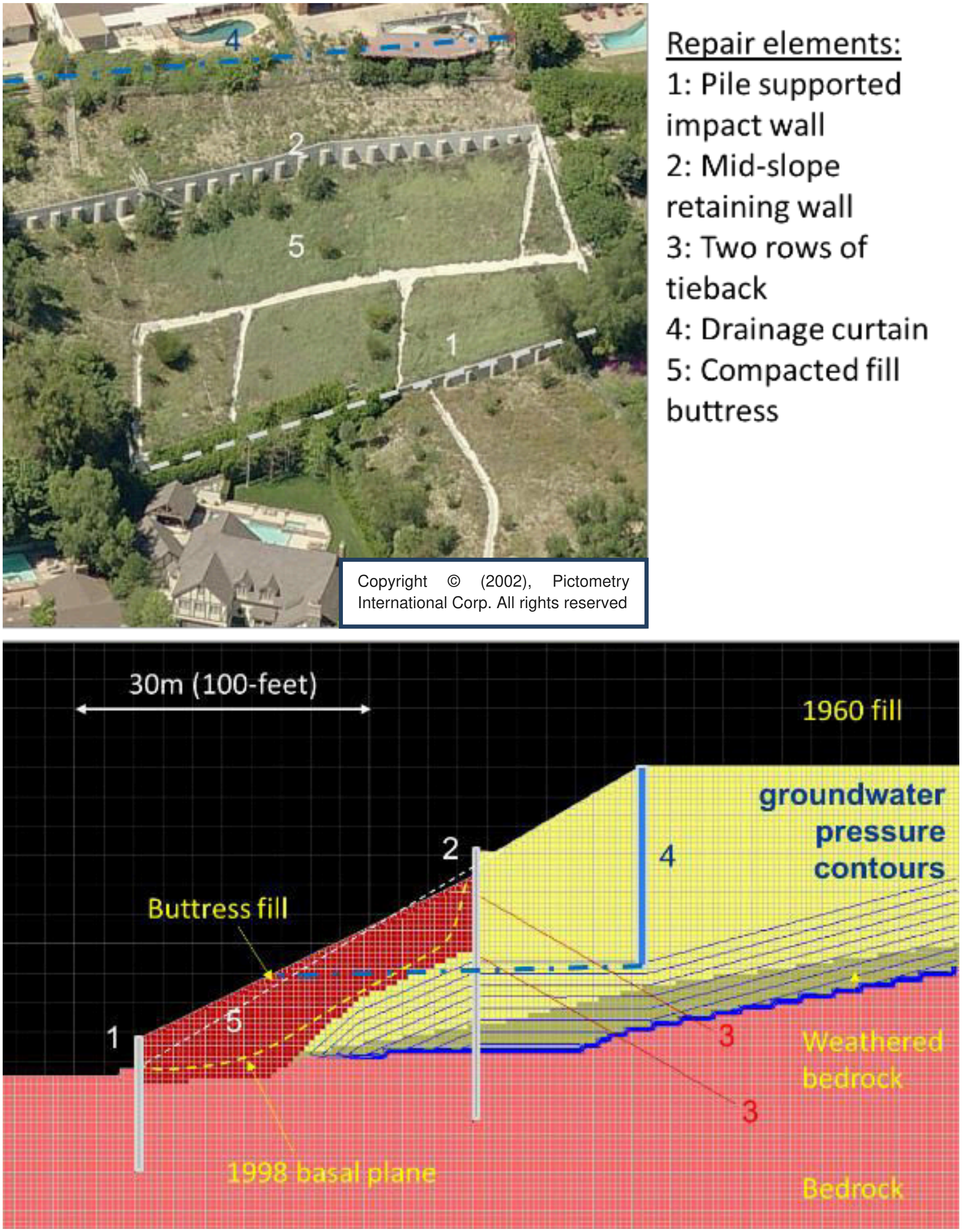

Figure 2. Elements of slope stabilization (aerial photograph from www.Pictometry.com).

The landslide, and the conditions that facilitated its occurrence were the subject of several geologic, geotechnical and forensic investigations, performed by consultants retained by property owners, as well as insurance companies and attorneys on their behalves. After several years of litigation, a settlement was reached and the author's firm was hired in 2003 by all the parties, for the specific purpose of designing and implementing a long term repair of the slope that would be acceptable to the controlling agencies. Hence, although the author was not involved in any of the prior litigation, he benefited from the many subsurface investigations of the landslide, laboratory testing, and instrumentation readings that were performed between 1998 and 2003 and shared with him. These investigations included, a total of 16 deep borings, where previous consultants collected samples, installed slope inclinometers, and/or performed downhole logging (to observe and sample the slide plane). In the author's experience this is an unusually large number of borings for landslides of similar size in Southern California.

NATURE AND CONDITIONS OF THE SITE

Although the landslide may have mobilized earlier, the Estrondo landslide became evident during the intense storms of February 1998, when concrete drainage swales were buckled and sheared, and a noticeable scarp had developed about two thirds up the slope (Figures 1 and 2). The 1998 storms, which were caused by the cyclic weather phenomena known as El Niño, delivered in a single month roughly an average year of Los Angeles rainfall (about 350 mm or 14-inches). Importantly, El Niño-fueled rainstorms began striking Los Angeles in late December, 1997, and intensified in January. The relentless string of storms resulted in deep saturation of the ground, washed away roads and railroad tracks, overflowed flood control channels, resulted in numerous debris flows and landslides, caused 17 deaths, and resulted in more than half a billion dollars in damage. Reportedly, the February, 1998, rainfall in Los Angeles was the strongest on record and many began referring to this rainy season as "The Great El Niño of 1997-98".

At the subject site, seeps and/or saturated conditions were observed in roughly the lower two thirds of the slope, after the major storms in 1998 and by the author in later years. The timing of the failure and seepage observations indicate that groundwater played a predominant role in the 1998 failure. Since few records from the original grading survived, it is not known whether subdrains were installed during construction to drain the slope; nevertheless, none were located during the 2004-2005 grading of the buttress described below, nor during the 1998 to 2003 subsurface investigations. If subdrains were originally installed they were obviously insufficient, were destroyed by failures, and/or had become clogged by fines.

By 2003, the landslide shown in Figure 2 had moved significantly and there were significant concerns about its short term effect on nearby residences; hence, the necessity of urgently implementing a slope repair.

Concerns included the following:

- At the toe of slope, the landslide was pushing several rear yard property line walls. These walls had been designed and constructed as tall, free standing walls that were obviously not designed to laterally restrain a landslide; hence, could collapse.
- Although the slope had been covered with plastic on several occasions, it tended to quickly deteriorate (as shown in Figure 1), exposing landslide debris to incident rainfall. Hence, there was the concern that a debris flow could occur which would endanger the lower slope residents and their properties.
- Deformations and significant tilting were noticed in several homes overlooking the slope. The most noticeable movement was measured in a home at west end of Estrondo Dr. By 2003, this home had experienced a vertical differential across its floors in the excess of 25 cm (10 inches), as well as a reversal of the flow direction in several of its sewer lines. Hence, although no scarp was present, there was a concern that the landslide could rapidly enlarge and that new scarps would appear in the upper third of the slope and/or under the upper residences.

METHOD OF REPAIR

Since the slope had failed repeatedly in the past, a grading solution was not considered, additionally, the reluctance of neighbors to allow construction on their properties resulted in minimal space for stockpiling and equipment storage. Hence, for the long term repair of the slope, the author's firm designed a geotechnical repair where most of the operations were confined to the slopes and, listed in order of installation, consisted of:

- A toe of slope wall with 0.75 meter (2.5 feet) diameter drilled shafts (up to 13 meters deep), which served as an impact/protection wall during the earliest stages of construction, and, later, would laterally retain the lower end of the fill buttress (Figure 2).
- A wall at mid-slope with 1.4 meter (4.5 feet) diameter drilled shafts, up to 30 meters (100 feet) deep, strengthened by two rows of tiebacks (Figures 2 and 3). To avoid the difficulties associated with assembly of rebar cages, we opted to reinforce them with large steel beams instead. The beams were delivered in up to 15 meter (50 feet) segments and welded on-site (Figure 3).
- A 20 meter (65 feet) deep drainage curtain along the top of slope, to lower the groundwater pressures that lead to the previous failures of the slope (Figures 2 and 4). The drainage curtain was built using conventional drilling equipment of the type used to construct belled drilled shafts. After connectivity between the bells was verified, the holes were filled with gravel. Drilling was performed such that base of the bells had a gentle slope, and at the lowest bell elevation, an outlet pipe was installed by directional

drilling (Figure 4). A secondary pipe was installed from the slope to the middle of the drainage curtain for backup an maintenance purposes.

- Two rows of approximately 30 meter (100 feet) long tiebacks (Figure 5). The upper and lower row of tiebacks had ultimate capacities of 520 kN/m (36 kips/ft) and 1,230 kN/m (85 kips/ft), respectively.
- A compacted fill buttress that was placed between the two retaining walls (Figure 2). The finished slope of the buttress was graded at a 2:1 (horizontal to vertical ratio) gradient and reinforced with geogrid within 2 m (6 feet) of the final surface to prevent surficial failures.

Figure 3. Elements of the slope stabilization.

The slope repair was started in mid-2004 and completed in late 2005. To monitor displacements, the site was regularly surveyed, previously installed slope inclinometers were regularly read, and three of the mid-slope drilled shafts were instrumented. These measures allowed our engineers to evaluate the potential for adverse deformations above the mid-slope wall, and the safety of the slope below during excavations and tieback installations.

The instrumentation became particularly useful during the 2005 rainy season, when at the most critical stage of excavation the site experienced another major El Niño. Although the 2005 storms were very destructive in Southern California (e.g., Pradel, 2014), the installation of the drainage gallery and mid-slope structure prior to the rainy season, proved to be adequate to resist the hydrostatic pressures imposed on the slope by the 2005 El Niño storms.

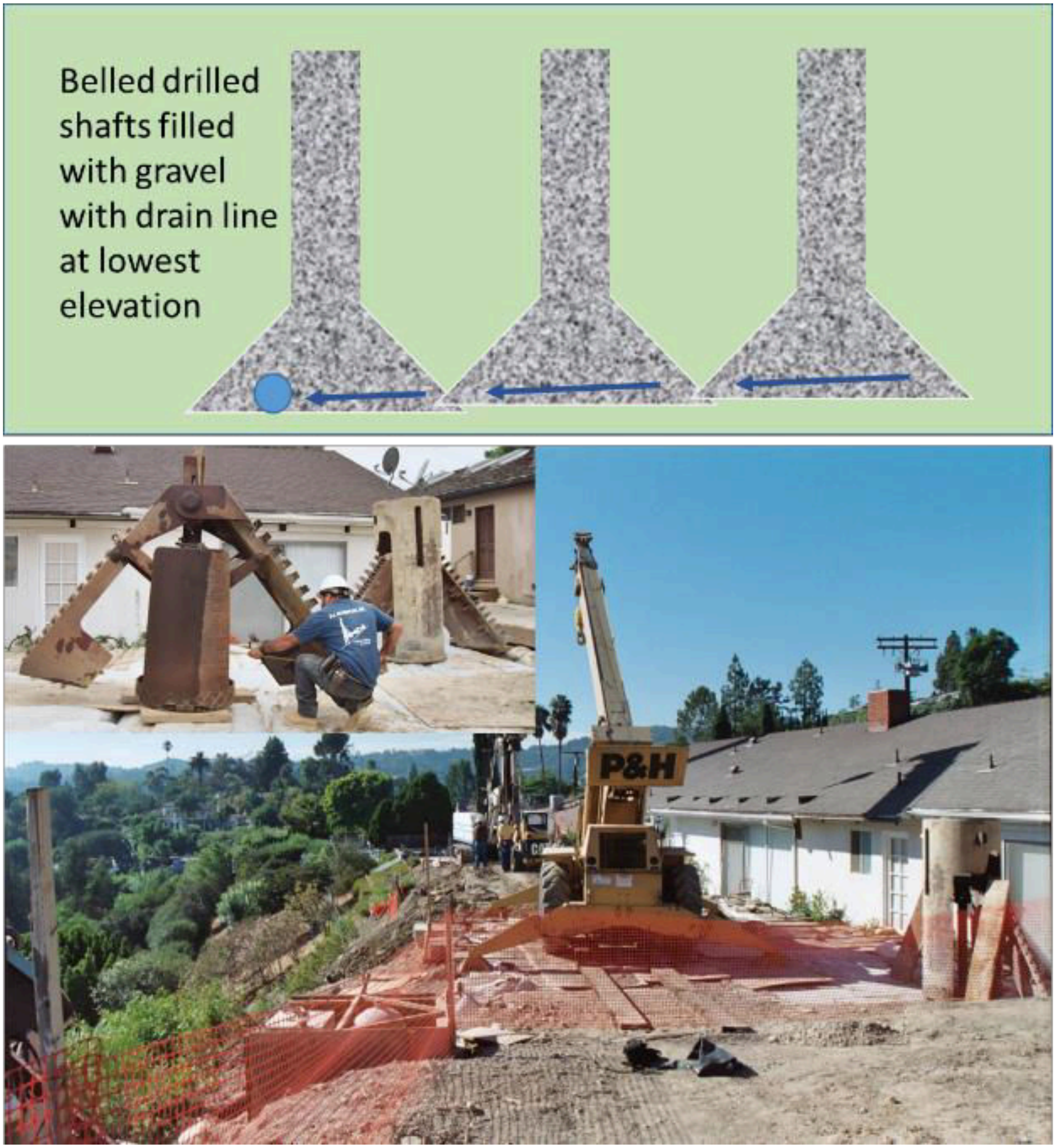

Figure 4. Construction of the drainage curtain.

Figure 5. Excavation for tieback installation and buttress construction.

MATERIALS

The various materials encountered under the subject slope are depicted in Figure 6. The soils for the original fill was derived from the underlying diatomaceous bedrock and near surface colluvial soils. The diatoms in the bedrock consist of fossilized skeletons of microscopic marine life which create lattice like structures with large internal empty spaces/voids. Typical dry densities of the local bedrock are very low, and at our site typically ranged from 8.8 to 11.9 kN/m^3 (56 to 76 pcf). Compacted fills derived from these diatomaceous rocks are low plasticity silts and clays (ML to CL) that have low drained shear strength, and low permeability. The unusual combination of low dry density and low permeability results in dry bedrock fragments that often float in water for several minutes.

Because, the microscopic skeleton structures tend to slowly absorb moisture and trap water bubbles within their lattice configuration, these materials are very difficult to dry after they have been wetted, and may often appear dry at degrees of saturation above 80%. Compaction curves often have more than one peak, due to crushing of the diatoms and release of trapped moisture. Field compaction can be particularly challenging for grading contractors, as the structural collapse of diatoms can suddenly release large amounts of trapped water which results in a nearly compacted lift suddenly turning into mud.

Using the shear tests from all the geotechnical consultants, a conservative composite strength envelope was derived for the different materials; the shear strengths adopted in our analyses are summarized in the table included within Figure 6.

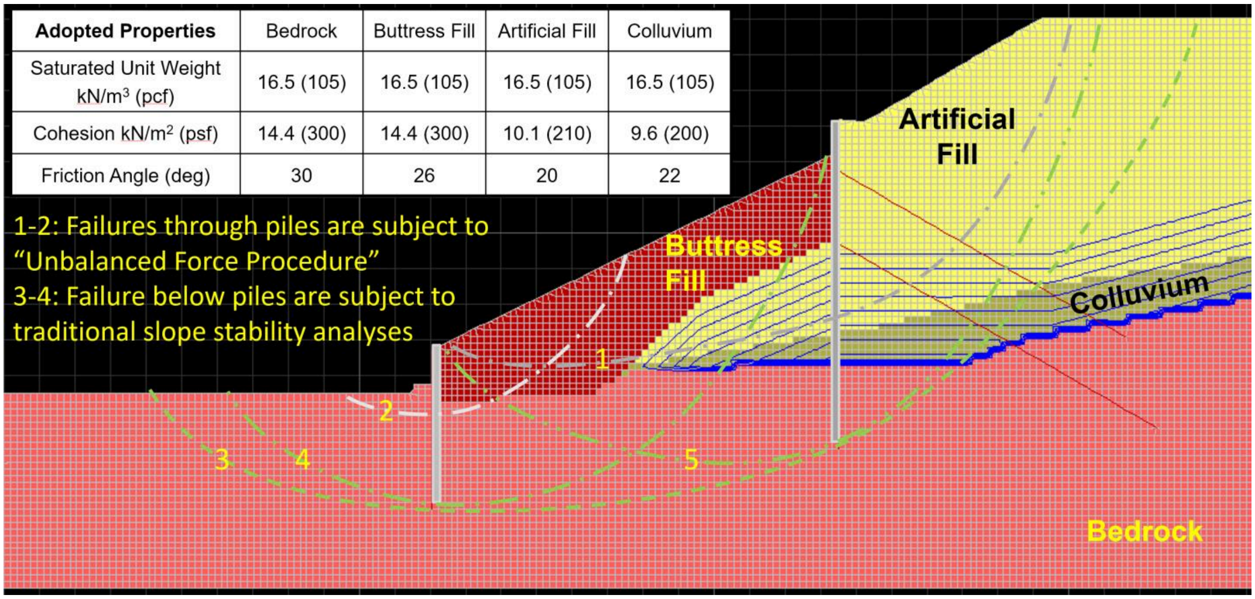

Figure 6. Excavation for tieback installation and buttress construction.

CONVENTIONAL SLOPE STABILITY ANALYSES

The regulatory agency for the Estrondo landslide stabilization was the City of Los Angeles, Dept. of Building and Safety. This agency has its own building code, as well as, numerous written ("Rules of General Application," or RGAs) and unwritten rules that specify how geotechnical investigations and analyses should be performed, and even which soil strengths may be used.

Although more advanced method of design are available (e.g., Boeckmann and Loehr, 2013, Pradel et al. 2010), the City required that landslide stabilizing piles be designed using the "Unbalanced Force Procedure" which is used in combination with traditional slope stability techniques, such as method of slices. This procedure is exemplified in NAVFAC (1986) and involves adding a lateral force into the slope stability model that replaces the stabilizing pile, and provides an equivalent reaction force. Thus, in this procedure the magnitude of the equivalent force becomes a function of the desired factor of safety. Several geotechnical software companies have incorporated the "Unbalanced Force Procedure" in their computer programs, although their implementations tend to differ on "where" and in "which direction" the equivalent force is acting. Although more advanced methods of design are available (e.g., Pradel et al., 2010, and Boeckmann and Loehr, 2013), in the author's experience, the unbalanced force analyses is still widely used in practice and required by many regulatory agencies.

The City also mandates that gross stability be verified, which requires adequate factors of safety be demonstrated for failure surfaces below the tip of the stabilizing piles. For gross stability and unbalanced force analyses, regulatory agencies in Southern California generally require factors of safety of 1.25 for temporary stability (e.g., during the stabilization phase) and 1.5 for permanent conditions. Occasionally, controlling agencies issue variances, which allow lower factors of safety, e.g., when the landslide and shear strengths are particularly well know through major investigations and testing. Figure 6 exemplifies the types of surfaces required for projects such as the Estrondo Landslide stabilization, and consequently the only modes of failure that are generally considered by local geotechnical engineers.

GEOMECHANICAL NUMERICAL ANALYSES

The author performed a series of Finite Difference analyses using the program FLAC by Itasca (2017). Some FLAC analyses where originally performed as a sanity check of the conventional slope stability analyses mandated by the City; later these analyses were refined to better understand the behavior of the slope stabilization.

The critical mode of failure and structural demands, shown in Figure 7, were obtained by the "Strength Reduction Method (SRM)" (Griffith and Lane, 1999). To incorporate stabilizing piles, the original SRM methodology was slightly modified using the procedure described in Pradel et al. (2010). The main advantage of using the SRM is that no assumption about the mode of failure is needed before hand. By contrast, traditional slope stability methods require the engineer to make "educated guesses" of potential critical failure surfaces. With the SRM slopes fail naturally as the strength is systematically reduced; hence, the critical mode of failure becomes apparent.

For the repaired slope, the critical mode of failure predicted by FLAC is shown in Figure 7. It involves the development of one plastic hinge in the lower piles, and two in the mid-slope piles. Figure 7 shows that, at failure, three large soil blocks are formed, that experience large shear strains where they contact each other. The shearing in this mode of failure is of a fashion that is not easily predicted or anticipated in traditional slope stability analyses using methods of slices.