Case Histories of Landslide Stabilization Using Drilled-Shaft Walls

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The Manning Canyon shale formation underlies many of the slopes adjacent to the Wasatch mountain range near Provo, Utah. The climate in Utah is relatively dry, so the strength of the shale is normally sufficient to prevent slope instability. During wet periods, however, the shale exhibits a significant decrease in strength that has led to a number of landslides. One method that has been employed to stabilize some of the slopes is the installation of closely spaced drilled shafts. Successful application of this stabilization procedure requires (a) accurate evaluations of the geometry and strength of the materials composing the slope, (b) reasonable evaluations of the location of potential failure surfaces, (c) estimations of the horizontal force required to increase the factor of safety against sliding to a suitable value, and (d) design and construction of drilled shafts capable of resisting the required horizontal force. The application of the method is illustrated with the use of several case histories for slopes on which slides have developed. Continued sliding threatened homes upslope and closed roadways to traffic. The success of the stabilization technique was recently proved in one of the cases in which the slope behind the drilled-shaft wall became wet. Although a slide developed in the slope immediately adjacent to the wall, the slope behind the wall has remained stable. Damage to the homes and roadways because of the movement of the slides has been arrested since the walls were constructed.

During the past 15 years, considerable development has taken place on the western slopes of the Wasatch mountain range in northern Utah. Unfortunately, many of these slopes east of Provo, Utah, are underlain by the Manning Canyon shale formation. This formation is known to exhibit significant decreases in strength following wetting, and it is frequently associated with landslides. Because the climate in Utah is relatively dry (annual precipitation is typically less than 20 in./year) the strength of the shale is normally high enough to prevent slope instability. During wet periods, however, the decrease in strength has frequently led to landslides. For example, during the period from 1981 to 1984, precipitation ranged from about 150 to 200 percent of the normal (*1*) and many landslides developed in the foothills east of Provo. In most cases, the Manning Canyon shale or residual soils derived from the shale were found to be responsible for the sliding.

One method that has been used to stabilize some of the slopes is the installation of closely spaced drilled shafts. The successful application of this procedure has been described by several investigators (*2,3*). In contrast to conventional concrete cantilever and counterfort walls, drilled shafts are capable of more economically resisting large lateral forces. Drilled shafts can easily be passed through the failure surface, and they can be constructed without excavating the toe of the slope and exacerbating the sliding problems. Recent cost comparisons between drilled-shaft walls and conventional walls at sites in Utah indicate that a savings between 40 and 50 percent can be expected.

In order for the drilled-shaft stabilization procedure to be successfully employed, it is necessary to (a) make accurate evaluations of the geometry and strength of the materials comprising the slope, (b) make reasonable evaluations of the location of potential failure surfaces, (c) make estimations of the horizontal force required to increase the factor of safety against sliding to a suitable value, and (d) design and construct drilled shafts capable of resisting the required horizontal force.

The principles and methods used in the analysis will be discussed in general terms, and then the application of the procedure will be illustrated with several case histories. Two of the case histories involve slopes on which shallow failure surfaces developed. The sliding caused significant damage to homes upslope from the slides; continued sliding threatened to destroy the homes. The slides also moved into adjacent roadways and caused disruption of traffic. Drilled-shaft walls were installed along with drainage systems and the homes and roadways were repaired. Two additional case histories involve much larger landslides, which damaged roadways but did not directly affect houses. Drilled-shaft walls have been designed, along with other measures, to stabilize these slopes.

GENERAL PROCEDURE

Evaluation of Geometry and Strength of Materials Composing Slope

The profile of the slope before failure was initially approximated on the basis of available topographical maps. However, in some cases cutting and filling operations associated with construction of the houses had changed the geometry significantly. In these cases it was necessary to use old photographs, construction plans, and eye-witness accounts to reconstruct the slope profile. Profiles of the failed slopes were determined using conventional surveying techniques.

The subsurface profile along the length of the slide was determined by drilling a number of boreholes that extended through the failure surface and into the more intact shale formation at depth. The shale boundary was generally found to be rather irregular, and this irregularity may be a result of previous landslides or normal faulting on the Wasatch fault in the immediate vicinity. Undisturbed samples of the cohesive overburden material were obtained using 2.5-in. diameter

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thin-walled shelby tubes. Miniature vane shear tests were performed on each sample in the field and unconfined compression tests were conducted in the laboratory.

Disturbed samples were obtained in cohesionless overburden using a standard split-spoon sampler. In addition to standard penetration testing, testing in these materials was limited to grain-size analysis. The boreholes were generally extended 10 to 20 ft into the intact shale, and core samples of the shale were obtained for compression testing. The range of undrained strengths typically involved for the various material types encountered are shown in Figure 1. Companion tests on wet and dry samples of the residual clays indicate that the undrained shear strength at saturation is only about 30 percent of the strength at its normal moisture content.

Evaluation of Location of Potential Failure Surfaces

Because all cases involved active landslides, the locations of the head and toe of the slide were known. During drilling operations an engineering geologist was present to log the hole and estimate, if possible, when the failure surface was located. Based on field observations, the subsurface profile, and the shear strength test data, it was normally possible to make a reasonable estimate of the location of the failure plane. For the larger landslides, the failure plane appeared to follow the boundary of the shale layer; for the smaller slides the failure was located within the residual clays above the shale boundary.

The average strength on the failure surface was evaluated by back-calculating the strength necessary to produce a factor of safety of 1.0 using Spencer's method of stability analysis, which satisfies both force and moment equilibrium. The computer program SSTAB1 was used to perform the analyses (*4*). Both circular and noncircular failure surfaces were used in the analyses. The critical failure surface was generally in good agreement with field observations, and the back-calculated strength compared favorably with the low end of the range of laboratory test data.

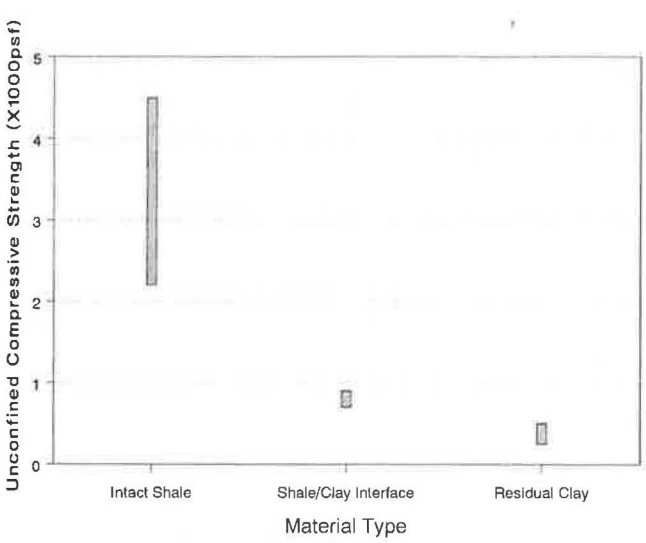

FIGURE 1 Comparison of undrained strength of various materials under saturated conditions.

Determination of Horizontal Force Required To Increase Factor of Safety Against Sliding

Once the soil profile and strength properties had been defined, the force required to increase the factor of safety against sliding to acceptable levels was determined. Horizontal forces were applied to the slope at locations where the drilled-shaft walls were to be located and slope stability analyses were performed. The analyses were performed again for potential circular and noncircular failure surfaces. Variations in the reconstructed slope angle and horizontal force were made until the computed minimum factor of safety for static conditions was between 1.4 and 1.5.

Although the sites are near the Wasatch fault, a formal evaluation of the wall under seismic loading was not considered necessary for several reasons. First, the probability of an earthquake on the Wasatch fault is less than 10 percent for a 50-year period, and—considering the dry climate of Utah—the likelihood is very small that the soil would be at a critical degree of saturation at the same time an earthquake occurred. Second, none of the soils involved were susceptible to strength loss because of earthquake shaking and therefore, a flow slide was not possible. Finally, the horizontal force on the wall already included a safety factor of 1.5, and this force was subsequently multiplied by a load factor in designing the shaft as will be described. Thus, a significant seismic force could be tolerated before any sizable deformations would be expected.

Design and Construction of Drilled Shafts Capable of Resisting Required Horizontal Force

The required horizontal force per foot of length was multiplied by the center-to-center spacing to provide the horizontal force acting on each shaft. On the basis of the required force, the necessary embedment below the failure surface and the maximum moment in the shaft were computed using Broms' method for free-headed piles (*5,6*). The shaft was then designed to resist the computed maximum shear force and bending moment multiplied by a load factor of 1.6. This load factor was necessary because the concrete design used was based on the ultimate strength method, which follows American Concrete Institute (ACI) code requirements. Computations for the required reinforcement were simplified by treating the shafts as if they were square beams that fit entirely within the diameter of the shaft.

The shafts were typically 2 to 3 ft in diameter, and the free space between adjacent shafts was generally 4 to 6 ft. The shafts were designed to penetrate into the intact shale layer at a depth where the undrained strength of the shale was relatively high. To construct the shafts, holes were excavated through the slide material using conventional drilling, the reinforcing steel was placed, and the shaft was back-filled with concrete. After construction of the drilled-shaft walls, the slope was reconstructed to a specified inclination.

CASE HISTORY 1—WINDSOR DRIVE SLIDE 1

Slide Description

A plan view of the slide mass in relation to the house and Windsor Drive is shown in Figure 2. The house is on a hillside approximately 20 to 25 ft above the elevation of Windsor Drive. A steep slope extends from the front of the house to the east side of Windsor Drive. Above-normal precipitation and poor drainage allowed the subsurface soils to become wet, and a slide occurred in the slope immediately in front of the home. The head of the slide in front of the house was well defined and the toe of the slide intersected the asphalt paving on the east side of Windsor Drive. As a result of the slide, the southwest corner of the house settled several inches and caused a large crack in the basement floor. The outside edge of the front porch settled and pulled away from the house. The driveway, which extended diagonally up the hill in front of the house, was displaced and broken, and a cinder block retaining wall was sheared in two and carried down the hillside.

Subsurface Soil and Water Conditions

Four boreholes were drilled into the bedrock at locations shown in Figure 2. A cross section through Boreholes 1, 2 and 4 is shown in Figure 3. The subsurface profile generally consisted of a brown to black clay that graded into a dark brown to black Manning Canyon shale. The clay material above the shale has principally been derived from the weathering of the parent material. The clay classified as a CL material using the unified soil classification system, and the plastic index ranged from about 15 to 30 percent. No static water table was encountered in any of the test holes; however, the clays were very moist, and experience has shown that seeps frequently occur in the fractures of the Manning Canyon shale.

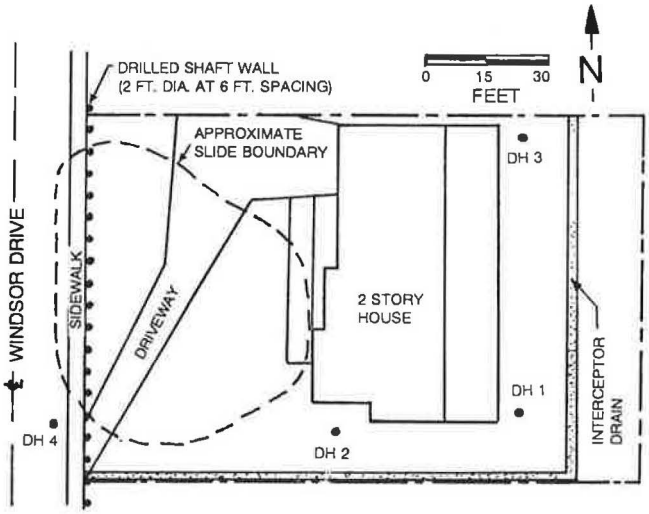

FIGURE 2 Plan view of Windsor Drive Slide 1 and drillhole locations.

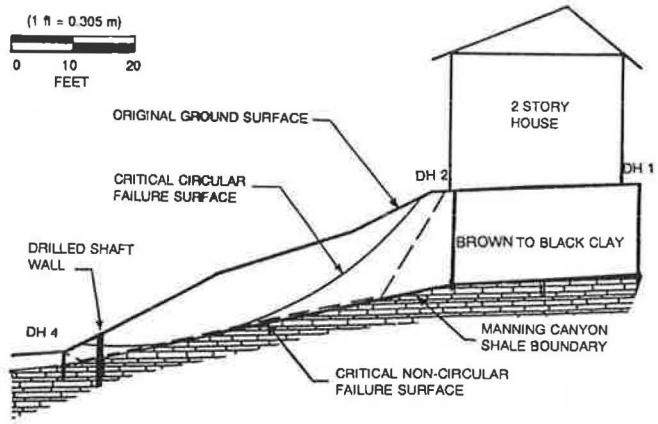

FIGURE 3 Profile through Windsor Drive Slide 1.

Stability Analyses

The shear strength of the shale layers consistently exceeded 2,500 psf, the strength of the clay was fairly erratic and varied from 200 to 2,000 psf. Because the failure surface was approximately known, it was possible to estimate the shear strength corresponding to a factor of safety of 1.0. For an assumed shear strength, the critical factor of safety was determined for potential circular surfaces passing through the toe of the slide. The safety factor became equal to 1.0 when an undrained shear strength of 270 psf was specified.

For the same strength values, a search was made for the critical noncircular failure surface, assuming that the failure occurred along the clay-shale interface. The search procedure employed the method proposed by Celestino and Duncan (*7*). The safety factor for the noncircular failure case was 20 percent higher than that for the circular failure case, so it was concluded that a circular failure mode was more critical. This finding also agrees with the shape of the slide.

Stabilization Measures

Additional stability analyses were conducted for potential failure surfaces through the reconstructed slope to determine the horizontal force necessary to stabilize the slope. It was determined that a horizontal force of 7,000 lb/ft of wall applied at the location shown in Figure 2 would bring the factor of safety up from 1.0 to 1.5. Based on Broms' method (*5,6*) it was determined that the shafts should extend to a depth of 15 ft in the intact shale a center-to-center spacing of 6 ft. On the basis of shear and moment requirements, 2-ft-diameter shafts and four #10 steel reinforcing bars were specified. The locations of the drilled-shafts in plan and profile are shown in Figures 2 and 3. The cost of the wall was estimated at about $160/ft of wall, a total cost of about $14,400. Because the cost of the wall was a small fraction of the total value of the house and few other alternatives for stabilization were available, the drilled-shaft wall was constructed.

After construction of the drilled-shaft walls, the slope was reconstructed to its original inclination and thin concrete panels were placed between the shafts above ground level. These panels, which did not exceed a height of about 4 ft, were

FIGURE 4 View of completed drilled-shaft wall with landslide developing immediately adjacent to it.

FIGURE 5 View of slide mass with drilled-shaft wall in background.

intended to prevent surface erosion that might not be controlled by soil arching. In addition to the shaft wall, a 10-ft-deep interceptor drain was constructed across the back side of the house as shown in Figure 2. A perforated 4-in. PVC drainpipe was placed at the bottom of the trench and extended to the roadway at the base of the slope. A photograph of the completed wall is shown in Figure 4.

The success of the stabilization procedure was recently proved when the slope behind the drilled-shaft wall became wet because of heavy spring rains and a leaky sewage line that had not been completely repaired in the reconstruction of the hillside. A slide similar in magnitude to the original slide developed in the slope immediately adjacent to the wall and moved onto the street. The stark contrast in the hillside performance at the interface between the slide mass and the wall is shown in Figure 5. Clearly, without the presence of the shaft wall, the slope in front of the home would have once again failed. Although the soil behind the wall became nearly saturated, the slope has remained stable and there are no signs of distress in the shafts. Structural damage to the home because of movement of the slide has been arrested since construction of the shafts.

CASE HISTORY 2—WINDSOR DRIVE SLIDE 2

Slide Description

Within a year of the original slide described in Case History 1, a similar slide began to move in front of another house several hundred ft away. A plan view of the slide mass in relation to the existing house is shown in Figure 6. The house consists of a basement and two levels above the ground surface. The topography in this area slopes steeply toward the west; a level site for the facility was developed by cutting into the hillside. A profile showing the original ground surface and the ground surface prior to slope failure is shown in Figure 7.

The intersection of the slip plane with the ground surface appears to be located a few feet east of the western side of the house. Slip plane traces could be seen on both the north and south side. A major north-south crack developed along the basement floor directly behind the western wall and exhibited considerable vertical displacement. Differential movement between the north and south corners at the first-floor level was more than 3 in., and cracking of the basement walls was also evident. Cracking of the brickwork was also observed in front of the house and on the north and south sides. Slide movements in front of the house had significantly disrupted sidewalks, steps, and a retaining wall made from wood railroad ties. Bulging of the asphalt pavement on Windsor Drive directly adjacent to the easterly curb of the street suggested that the slide terminated in this area.

Site Characterization and Stability

To evaluate the stability of the site, four test borings were drilled to depths between 20 and 60 ft, as shown in Figure 6. The locations of these holes in profile and the soil stratigraphy

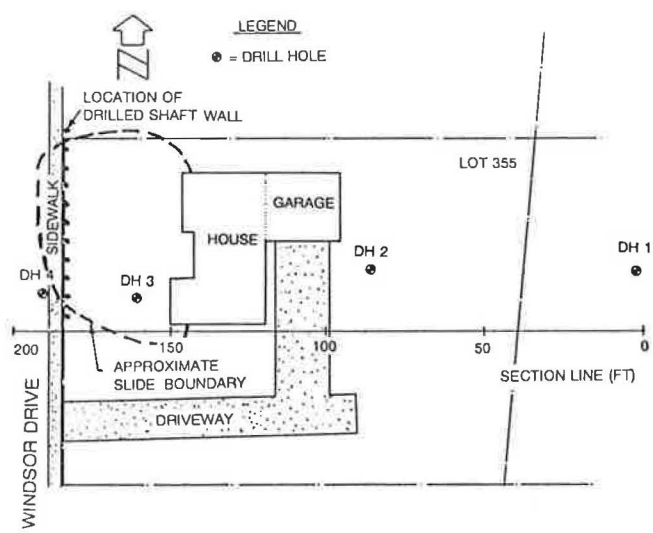

FIGURE 6 Plan view of Windsor Drive Slide 2 and drillhole locations.

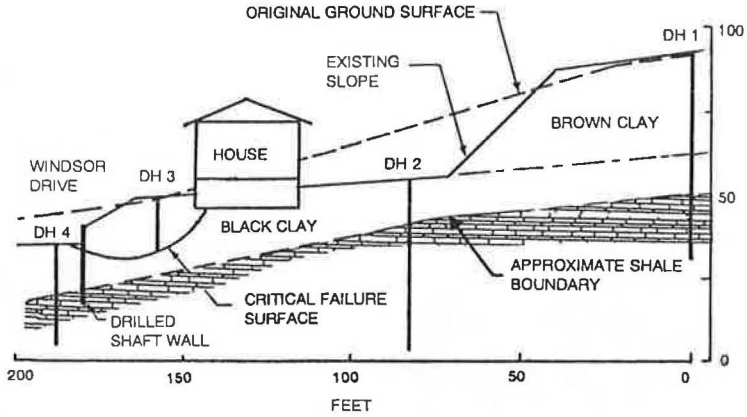

FIGURE 7 Profile through Windsor Drive Slide 2.

at the site are shown in Figure 7. The subsurface profile generally consisted of gray to black clay that graded into Manning Canyon shale. However, the steep slope behind the house was composed largely of a brown clay with a markedly lower plasticity index.

Although groundwater was not encountered in any of the boreholes, the black clay in the vicinity of the slide was 80 to 90 percent saturated. The undrained strength generally varied from around 400 psf near the ground surface to about 1,200 psf at a depth of 20 ft; however, samples obtained near the failure surface had undrained strengths of about 300 psf. Slope stability analyses determined that a factor of safety of 1.0 was obtained for an average shear strength of 300 psf. The location of the critical failure circle is shown in Figure 7; it may be seen that it closely approximates the observed failure surface. It is interesting to note that the undrained strength and failure mode in this case are very similar to the previous case history.

Stabilization Measures

The undrained cohesion of 300 psf was subsequently used in stability analyses to calculate the magnitude of the lateral force acting on the downhill face of the slide necessary to increase the factor of safety from 1.0 up to 1.5. It was determined that a force of 8,100 lb/linear-ft of wall would be required. On the basis of horizontal force, 2-ft-diameter shafts spaced at 6 ft on centers were specified. The shafts extended 12 ft into the subsurface profile at the location shown in Figure 7. This embedment depth was beyond the observed failure surface at the base of the slide. The investment in the drilled-shaft wall was considered worthwhile because it restored the original value of the property at a small fraction of the original cost of the structure.

In addition to the drilled-shaft wall, other measures to improve stability included channeling runoff from roof drains into pipes that were carried to the street level and the installation of a shallow interceptor drain along the front of the house. The house itself was then repaired, and no additional distress has been observed even though slides in the immediate vicinity of the house have occurred.

CASE HISTORY 3—MILE HIGH DRIVE–IMPERIAL WAY SLIDE

The previous case histories involved small shallow slides adjacent to structures, the following case histories involve somewhat larger landslides.

Slide Description

The general location of the slide area between Mile High Drive and Imperial Way is shown in Figure 8. The head of the slide intersects nearly all of Mile High Drive, and the toe of the slide is located on the easterly edge of Imperial Way. Both roadways were closed because of the slide. The topography of the slide area is also shown in Figure 8, and a profile through the center of the slide is shown in Figure 9. The location of two small drainage channels east of Mile High Drive, designated as "A" and "B," are also shown in Figure 8. Before construction of the roadway, water from these drainage channels flowed down a depressed area to the south of the slide and was carried downslope. With the construction of the roadway, the drainage path was interrupted and water from both drainages seeped into the subsurface material in the general slide area. Following heavy runoff the slide moved downslope about 15 ft.

Site Characterization and Stability

The characteristics of the subsurface material throughout the slide area were defined by drilling four test holes to depths varying from 14 to 40 ft as shown in Figures 8 and 9. It should be noted that the Manning Canyon shale was encountered in each of these test borings at depths varying from 9 to 36 ft below the ground surface. The location of the overburden and shale materials are shown in Figure 9. The overburden generally consisted of gravelly sand and clayey gravel overlying a 6- to 8-ft-thick layer of residual clay above the shale. Groundwater was encountered in every boring. The water table was typically 25 ft above the shale interface near the

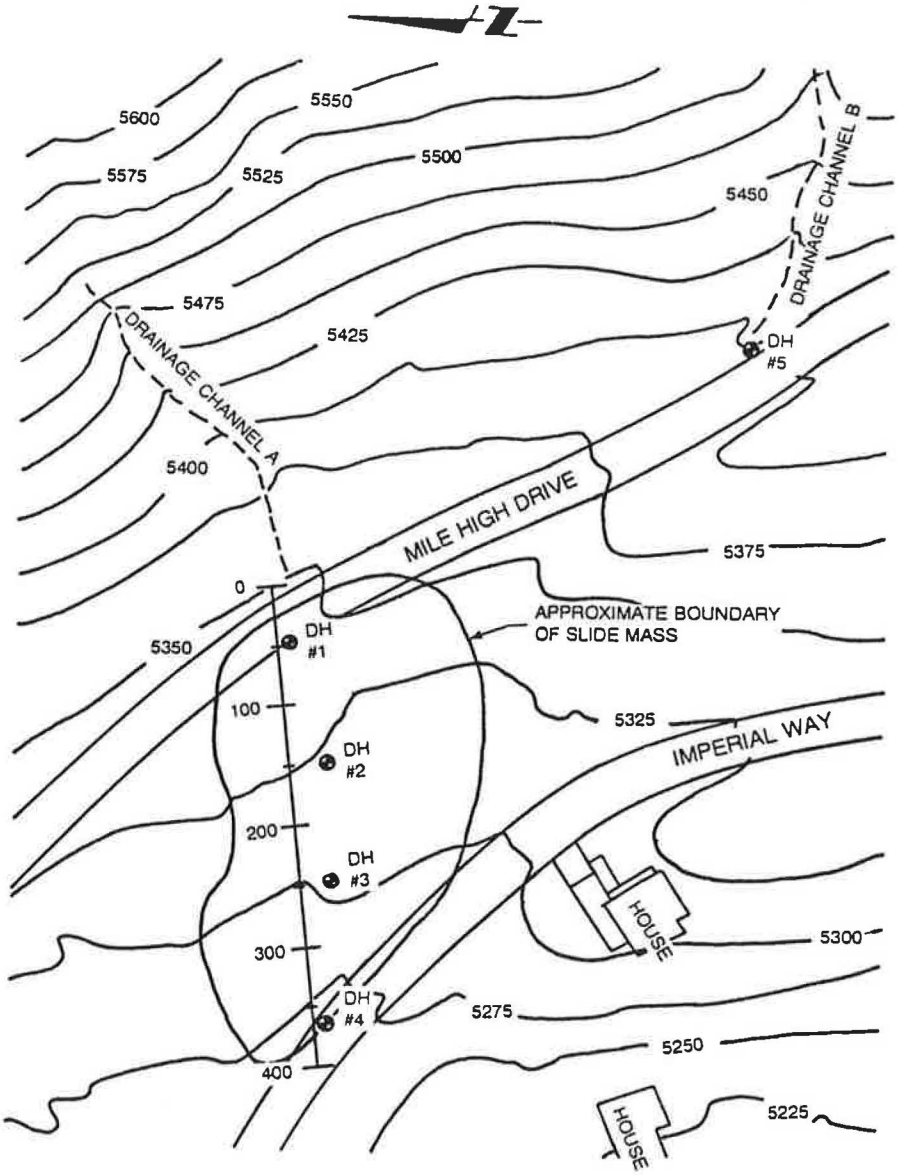

FIGURE 8 Plan view of Mile High Drive–Imperial Way landslide and drillhole locations.

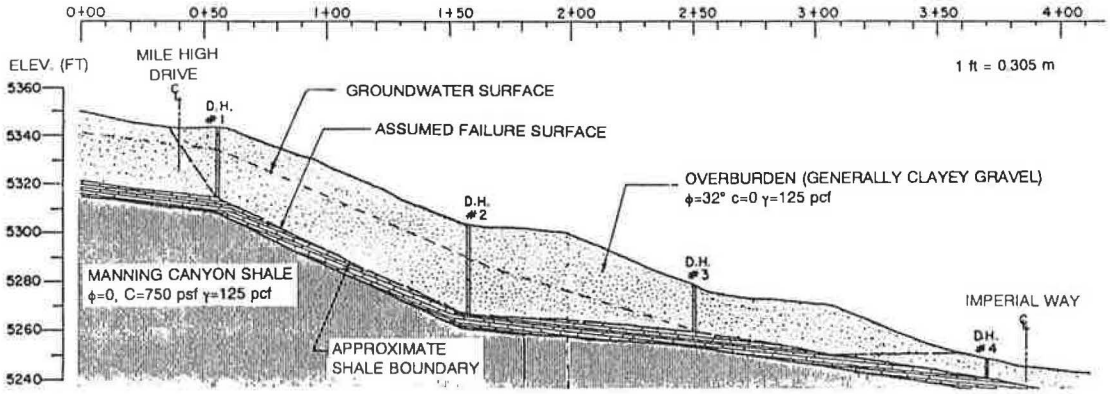

FIGURE 9 Profile through Mile High Drive–Imperial Way landslide.

head of the slide but just a few feet above the shale elevation at the base of the slide.

The location of the shale surface along with the groundwater conditions and the characteristics of the overburden material strongly suggest that the failure surface is in the vicinity of the interface between the overburden and the shale. Assuming that the failure surface lies near the clay-shale interface and that the factor of safety against sliding is equal to 1.0, a stability analysis was performed to determine the average shear strength along the failure surface. Assuming undrained conditions in the clay-shale, the results of the stability analyses indicate an undrained shear strength of about 750 psf. At the head and toe of the slide the failure surface passes through the overburden material. The groundwater level shown in Figure 9 was taken as the piezometric surface for the granular soils. The unit weight and strength parameters used for the overburden material are shown in Figure 9. The results were not highly sensitive to small variations in these properties.

Stabilization Measures

The stabilization measures for this slide called for (a) installation of a 24-in.-diameter storm drain to carry the flow from the drainage channels away from the slide, (b) a 3-ft-wide interceptor drain back-filled with coarse concrete aggregate to a depth of 10 ft, and (c) a line of drilled shafts extending across the slide zone on the west side of Mile High Drive. Stability analyses indicated that a drilled-shaft wall near the toe of the slide would produce an increase in the factor of safety of less than 10 percent for the entire slope; however, a drilled shaft wall at the top of the slope could provide significant resistance and protect the upper roadway from future sliding. Based on the back-calculated strength values, stability analyses indicated that the drilled shafts adjacent to the roadway would need to provide a resistance of 7,000 lb/linear-ft to provide a factor of safety of 1.5 against sliding of the upper wedge of the slide.

The shafts were designed to penetrate through approximately 30 ft of overburden and 15 ft into the shale itself. The concrete shafts were 2.5 ft in diameter and were spaced at 5 ft on centers. Reinforcing consisted of four #11 bars. Movements of the slide have been minor since the construction of the stabilization measures.

CASE HISTORY 4—OAK HILLS SLIDE

Slide Description

A plan view of the Oak Hills slide is shown in Figure 9. This slide is located at the toe of a large ancient landslide near the base of Provo Mountain. Although the topography next to the slide is shown in Figure 10, the topography of the slide zone immediately before the slide occurred is uncertain. Construction was under way to prepare a level pad for a house and an access road up to the house. On the basis of photographs, eyewitness accounts, and volume comparisons with the slide debris, the best estimate of the original profile through the slide is shown in Figure 11. The slide was about 400 ft long and 100 ft wide. The scarp formed by the slide was more than 40 ft high and the slide moved more than 70 ft laterally. The slide threatened to engulf an adjacent home before coming to equilibrium approximately 10 ft in front of it.

Site Characterization and Stability

To define the soil profile and the probable location of the failure surface, it was necessary to drill four borings at the locations shown in Figure 11. Three of the borings penetrated the intact Manning Canyon shale, and the fourth boring, located at the bottom of the slide, did not. On the basis of the borings, the approximate location of the shale boundary is shown in Figure 11. It should be noted that the shale boundary is not a simple linear feature because previous landslides have displaced the shale. The material above the shale generally consists of low-plasticity gravelly silt and silty gravel. In addition to the intact shale zone, photos taken by a geologists before the slide showed a mass of shale with granular material above and below it was exposed in one of the lower cuts. It is theorized that the failure surface followed this detached shale layer and then ran along the shale interface and finally ruptured through the overburden as shown in Figure 11. No groundwater was encountered in any of the holes, but the soils were approaching saturation.

An examination of the cores in the shale indicated that they were highly fractured with zones of relatively hard shale interbedded with relatively soft layers. The plastic index of the shale ranged from 5 to 18, and the unconfined compressive strength varied from 1,500 to 4,900 psf. Thus, if the failure plane moved through the weakest zones, the shear strength would be about 750 psf. Stability analyses were conducted to determine the strength of the shale interface that was required to produce a factor of safety of 1.0. Assuming a reasonable range of values for the strength and unit weight of the granular zones above the shale (see Figure 11), the back-calculated strength of the shale interface was found to be between 700 and 900 psf. This strength is in good agreement with laboratory results and the interface strength that was determined for the Mile High Drive slide.

Stabilization Measures

The stabilization evaluation assumed that the slide mass would be removed from the roadway and that the slope would be cut back from the edge of the roadway to provide a stable slope. A number of alternatives were investigated and stability evaluations made for both circular and noncircular failure surfaces. The strength of the shale interface was assumed to have a value of 800 psf, which was based on back calculations and laboratory test results. For the case with the slope cut back to an inclination of about 1.75 H to 1.0 V, the factor of safety was 1.35. The addition of a horizontal force of 10,000 lb/linear-ft increased the factor of safety to 1.5. This force could be provided by using 3-ft-diameter shafts spaced at 6 ft on centers. The shafts would need to extend to a depth of 15 ft into the shale at the base of the slide. An interceptor drain was also recommended for the uphill side of the slide. It should be noted that because of the volume of the slide mass involved, the drilled-shaft wall, in this case, produced

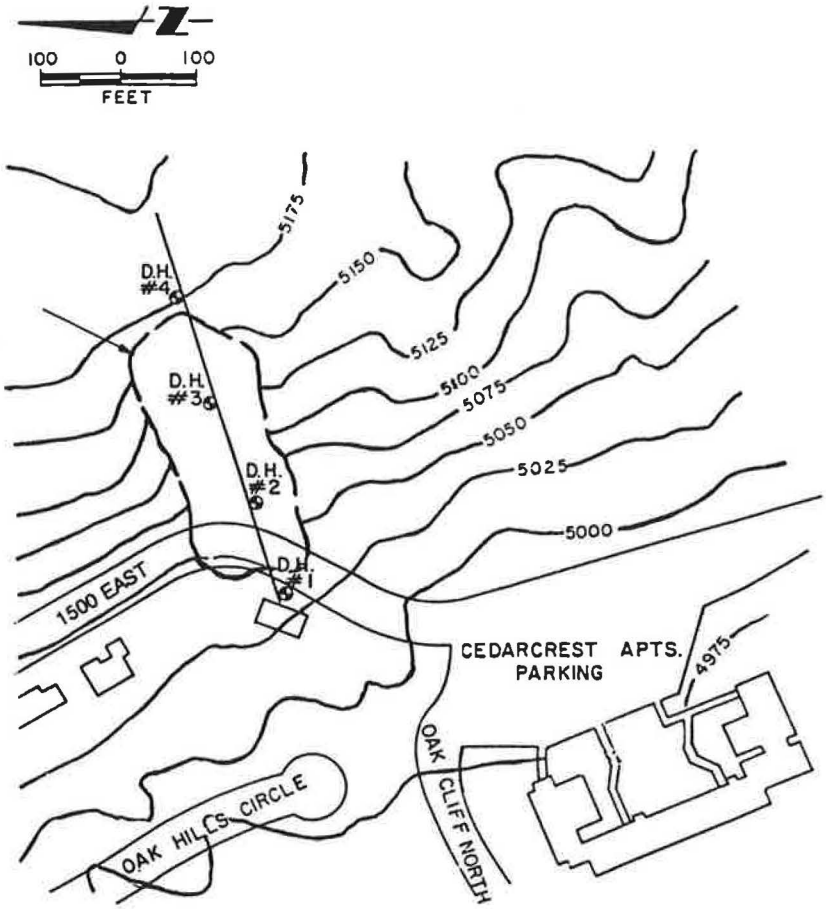

FIGURE 10 Plan view of Oak Hills slide and drillhole locations.

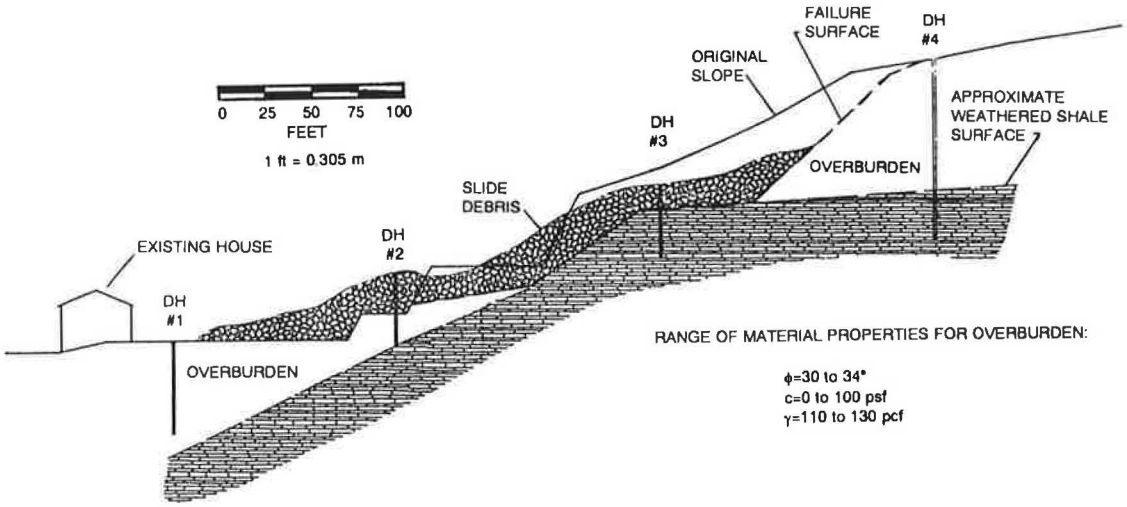

FIGURE 11 Profile through Oak Hills slide.

relatively small increases in the factor of safety against sliding even though the design forces were greater than in the first two cases. Legal issues concerning responsibility for the slide have delayed any actions regarding repair of the slide for more than 8 years, during which time the roadway has been blocked to traffic.

CONCLUSIONS

1. Drilled-shaft walls when used in combination with appropriate drainage provisions can be a practical, effective, and reasonably economical means of stabilizing small to medium-size landslides such as those discussed in the first two case histories—slope heights less than 30 ft and slope lengths less than 60 ft.
2. Significant increases in the factor of safety (from 1.0 to 1.5) are possible for small slides, but only moderate increases can be expected for larger slides such as those discussed in the next two case histories—slope heights of 80 ft and slope lengths of 200 ft.
3. The design of the shaft walls for landslide control requires a relatively good understanding of the mechanism controlling the slope failure, the strength of the soil, and the force distribution on the shaft.

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