CHAPTER 2

Examples and Causes of Slope Failure

Experience is the best teacher but not the kindest. Failures demand attention and always hold lessons about what not to do again. Learning from failures—hopefully from other people's failures—provides the most reliable basis for anticipating what might go wrong in other cases. In this chapter we describe 10 cases of slope failure and recount briefly the circumstances under which they occurred, their causes, and their consequences. The examples are followed by an examination of the factors that influence the stability of slopes, and the causes of instability, as illustrated by these examples.

EXAMPLES OF SLOPE FAILURE

The London Road and Highway 24 Landslides

The London Road landslide in Oakland, California, occurred in January 1970 during a period of heavy rainfall. Front-page headlines in the January 14, 1970, *Oakland Tribune* exclaimed ''Storm Hammers State—Slide Menaces 14 Homes—The Helpless Feeling of Watching Ruin Approach.'' Figure 2.1 shows houses in the slide area that were destroyed by the slide and had to be abandoned by their owners. The slide covered an area of about 15 acres, and the sliding surface was estimated to be as deep as 60 ft (20 m) beneath the surface of the ground. As evident from the height of the headscarp in comparison with the houses on the right in Figure 2.1, the slide movements were very large. Some 14 houses were destroyed, and a jet fuel pipeline at the bottom of the hill was never used again because of the danger that it would be ruptured by slide movements. Because the cost of stabilizing the massive slide was greater than the economic benefit, it was not repaired, and an entire neighborhood was lost permanently.

Not only was there heavy rainfall during January 1970, the entire preceding year had been unusually wet, with about 140% of the average rainfall recorded at the nearest rain gage station. As discussed later, these prolonged wet conditions played a significant role in the occurrence of the massive landslide.

The Highway 24 landslide shown in Figure 2.2 occurred in January 1982, when a storm blew in from the Pacific Ocean and stalled over the San Francisco Bay area. In a 24-hour period in early January, the storm dumped nearly 10 in. of rain on the area, where the normal yearly rainfall is about 25 in. The sudden enormous deluge resulted in literally thousands of landslides in the San Francisco Bay area. Typically, these slides were shallow. The intense rainfall saturated the upper few feet of the ground on the hillsides, which came sliding and flowing down in many places, knocking down trees, destroying houses, and blocking roads. Figure 2.2 shows Highway 24 near Orinda, California, partially blocked by a flow of sloppy, saturated soil that flowed onto the roadway, just one of the thousands of slides that occurred during the storm.

The London Road landslide and the flow slide on Highway 24, less than 10 miles apart, illustrate two very different types of slope failures that occur in the same area. The London Road landslide was very deep-seated and followed two successive years of above-normal rainfall. Although detailed soil exploration was not carried out at the London Road site, some interesting facts may be surmised based on what could be seen at the ground surface. The slide movement exposed serpentine rock in one area. Serpentine is a metamorphic rock that can be hard and strong but is subject to rapid deterioration to a weak powdery mass when exposed to air and water. Although the exposed serpentine retained its rocklike appearance, it could be penetrated several inches with a bare hand and had

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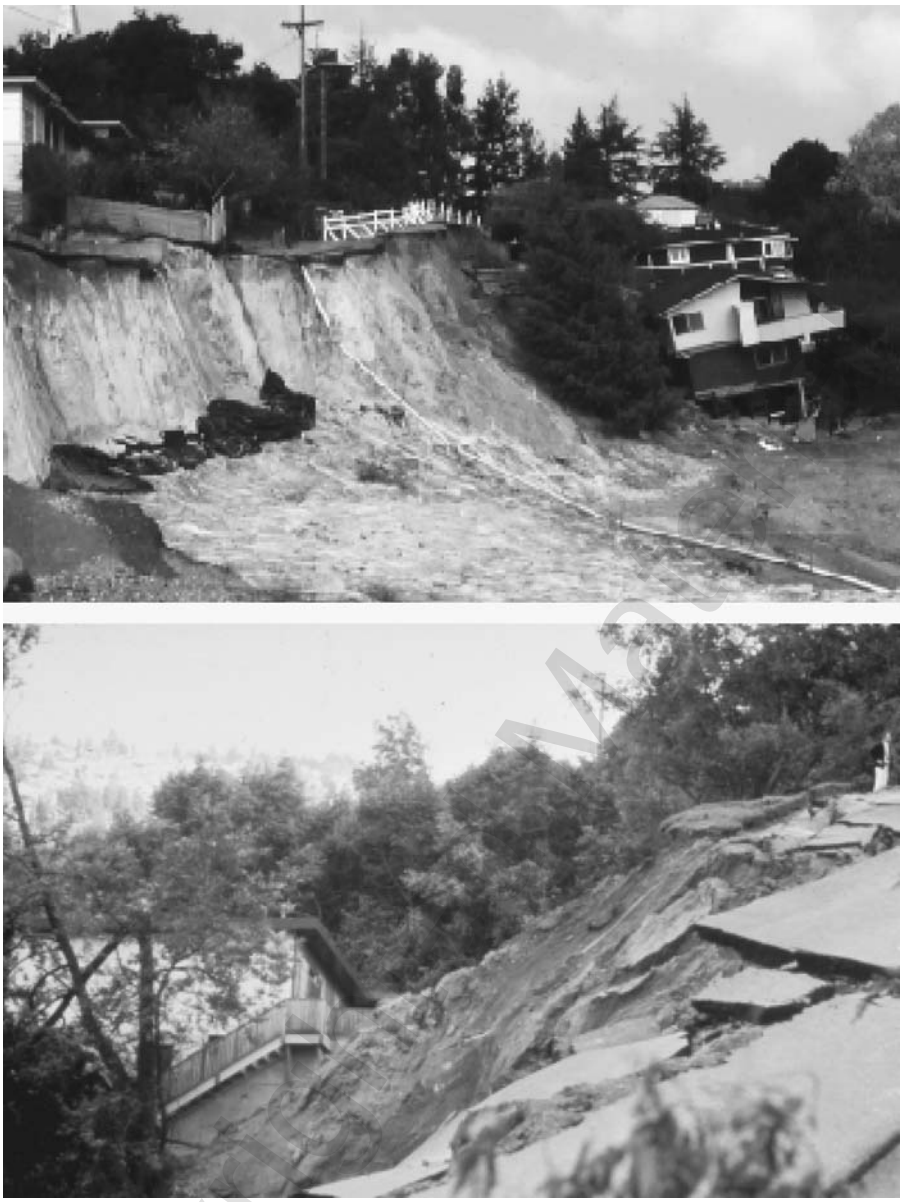


Figure 2.1 London Road landslide, Oakland, California. (J. M. Duncan photos.)

essentially no strength or stiffness. It can be surmised that the strength of the serpentine, and other soils and rocks underlying the London Road area, had been deteriorating slowly over a period of many tens, hundreds, even thousands of years since the hillside was formed. Such deterioration results from chemical and physical processes that can gradually change the properties of earth materials. Eventually, this reduction in strength, combined with two years of heavy rainfall and resulting high groundwater levels, led to the very deep-seated landslide.

In contrast, the slide that blocked Highway 24 was very shallow, probably no more than 3 ft (1 m) deep. This type of slide develops very quickly as a result of relatively brief, extremely intense rainfall. Infiltration within a brief period affects only the upper few feet of soil. Within this depth, however, the soil may become saturated and lose much of its strength. In the area east of San Francisco Bay, the hillsides are blanketed by silty and sandy clays of low plasticity over the top of less weathered and stronger rock. The thickness of the soil cover ranges from zero to 15 ft (5 m). The soil

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Figure 2.2 Flow slide on Highway 24 near Orinda, California. (J. M. Duncan photo.)

has formed from the underlying rocks and has reached its present condition through processes of weathering, erosion, shallow sliding, and deposition farther downhill. When dry, the soils that blanket the hillsides are stiff and strong, and the slopes they form are stable. During intense rains, however, water infiltrates the ground rapidly because the ground contains many cracks that provide secondary permeabilty. Although the processes leading to this type of slide are still the subject of research study, it is clear that conditions can change very quickly, and that the transition from stable ground to a fluid mass in rapid motion can take place within minutes. The high velocities with which these slides move makes them very dangerous, and many lives have been lost when they flowed down and crushed houses without warning.

The London Road and the Highway 24 slides illustrate a relationship between rainfall and landslides that has been observed in many places: Long periods of higher-than-average rainfall cause deep-seated, slow-moving slides, with shear surfaces that can extend tens of feet below the ground surface. One or two days of very intense rainfall, in contrast, tend to cause shallow slides involving only a few feet of soil, which move with high velocity once they are in motion.

The Landslide at Tuve, Sweden

In December 1977 a large landslide occurred on a gentle hillside in the town of Tuve, Sweden, a suburb north of Göteborg. A photograph and three cross sections through the slide are shown in Figure 2.3. The soil at the site was a layer of quick clay overlying a thin layer of permeable granular material on top of rock. The slide covered an area of 15 ha (40 acres), destroyed about 50 houses, and took 11 lives. Quick clays of the type involved in this slide are noted for their great sensitivity and extremely brittle behavior. When they fail, they lose practically all shear strength and flow like a viscous liquid.

The slide is believed to have started as a small slope failure in the side of a road embankment. The small slide left unstable the slope it slid away from, and a slightly larger failure of that slope took place. The process was repeated as the slide grew in the uphill direction, covering a larger and larger area. Houses in the area were undermined by the retrogressing slides and cruised downhill on the weakened slippery clay, crashing into other houses. As soil and houses from the failed area moved downhill, the soil in the area into which it moved was loaded and disturbed, and it began to fail also. The slide thus grew uphill and downhill from the original small slope failure in the middle.

Slides that grow uphill by increments are called *retrogressive;* slides that grow downhill by increments are called *progressive.* The Tuve slide was both. The small embankment failure that started the Tuve slide is believed to have been caused by erosion steepening the slope of the roadway embankment at the location of a small drainage culvert. This small slope failure was the trigger for a slide of immense proportions because of the metastable structure of the Swedish quick clay. It is likely that the strength of the bottom part of the quick clay layer was unusually low as a result of ar-

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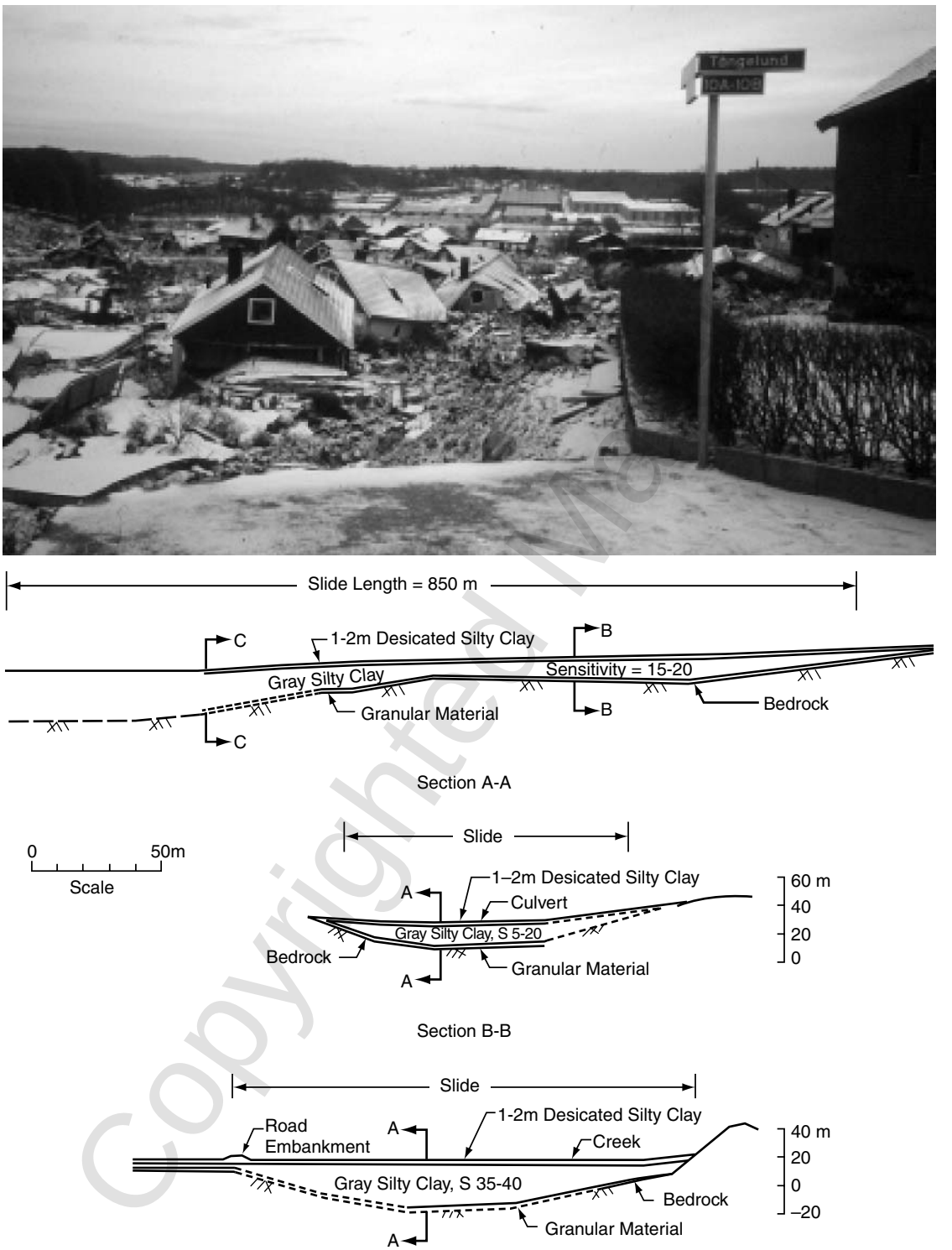


Figure 2.3 Landslide in quick clay near Tuve, Sweden. (J. M. Duncan photo and drawing.)

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tesian pressures in the permeable granular material beneath the clay. Similar conditions in Norway have been found to be especially treacherous.

Slope Failures in Highway and Dam Embankments

Pinole, California, slide. Figure 2.4 shows a slope failure that occurred on a section of Interstate 80 near Pinole, California, where the road was supported on an embankment of well-compacted clayey soil. It can be seen that the back scarp of the failure is very steep. This is an indication that the embankment material was very strong, or it would not have remained stable in this nearly vertical slope, which was about 30 ft (10 m) high. The weak link was the foundation, which contained organic soil that had not been removed when the highway was constructed.

The natural ground sloped upward away from the south side of the embankment (the left side in Figure 2.4) and downward away from the right side. During rains, water tended to pond against the embankment because there was no underdrainage, and water seeped from south to north through the foundation of the embankment. The slide occurred after a period of heavy rain in the winter of 1969.

Houston, Texas, slide. A slide in a highway embankment near Houston, Texas, is shown in Figure 2.5. The embankment was constructed of compacted highly plastic clay and was built with 2 (horizontal):1 (vertical) side slopes. The fill was well compacted. The embankment was stable when it was built and remained stable for many years afterward. However, as time went by and the fill was wetted and dried repeatedly during alternating rainy and dry periods, it gradually swelled and grew softer and weaker. Finally, about 20 years after the embankment was built, the failure shown in Figure 2.5 occurred.

San Luis Dam, California, slide. On September 4, 1981, a massive slide occurred in the upstream slope of San Luis Dam, about 100 miles southeast of San Francisco, California. A photograph of the slide is shown in Figure 2.6. At the left end of the slide the movements were about 35 ft (about 10 m). The amount of displacement decreased to the right, diminishing to zero in a length of about 1100 ft (350 m). In the area where the slide occurred, the embankment was 200 ft (60 m) high and was constructed on a layer of highly plastic clay slope wash overlying the rock that formed the hillside. This material was formed from the underlying rocks by the same processes of weathering, erosion, shallow sliding, and deposition farther downhill that formed the soils on the hills east of San Francisco Bay.

When the San Luis Dam embankment was constructed in 1969, the highly plastic clay slope-wash that covered the foundation was dry and very strong. However, when it was wetted by the water stored in the reservoir, it became much weaker. Furthermore, over the period of 12 years between construction of the dam and the slide shown in Figure 2.6, the water level in the reservoir moved up and down several times as the pumped-storage reservoir was filled in the wet season and as water was withdrawn in the dry season. Stark and Duncan (1991) performed tests on the slope-wash and analyses of the slide which indicated that the



Figure 2.4 Interstate 80 embankment slope failure.

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Figure 2.5 Slide in highway embankment near Houston, Texas. Embankment constructed of highly plastic clay.



Figure 2.6 San Luis Dam upstream slope failure.

strength of the slope-wash was gradually reduced to a low residual value due to the wetting and the cyclic variations in shear stress caused by the changes in the water level in the reservoir. Finally, in September 1981, the slide shown in Figure 2.6 occurred following the largest and fastest drawdown of the reservoir. The slide was stabilized by rebuilding the failed part of the dam, adding a 60-ft (18-m)-high buttress at the base (ENR, 1982).

The Olmsted Landslide

The Olmsted Locks and Dam project was built on the Ohio River about 50 miles (80 km) above the conflu-

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ence of the Ohio with the Mississippi. To satisfy navigational requirements, the project had to be built at a location where there was a massive active landslide on the Illinois bank of the river. The slide extends for about 3300 ft (1000 m) along the river bank. Evidence of instability on the Illinois shore at Olmsted was first discovered in 1987 during the foundation investigation for the proposed locks and dam. The extent of the unstable ground was mapped on the basis of slide scarps, cracks, leaning trees, and hummocky terrain. The difference in elevation from the toe of the slide to the scarp shown in Figure 2.7 is about 70 ft. Slope inclinometers were installed to determine the location of the shear surface, and piezometers were installed to determine water levels.

In late May and early June 1988, a drop of 7 ft (2.1 m) in the river level took place over a period of 10 days. During this period the groundwater levels within the slope dropped 1 to 3 ft (0.3 to 0.9 m). When the river level dropped, the slide moved, and nearly-vertical scarps about 3 ft (0.9 m) high developed at the head of the slide. The location of the sliding surface in the 1988 movement was determined based on data from 12 slope inclinometers, and the elevations of crimping in the riser tubes of five standpipe piezometers.

It was found that the shear surface was located within the McNairy I formation, which consists of interbedded layers of clay, silt, and sand. The thicknesses of these layers vary from fractions of an inch to as much as a foot. The layering within the McNairy I formation makes determination of shear strengths very difficult. First, the shear strength varies with the direction of the shear plane, being much higher when the shear plane crosses silt and sand layers than when it passes entirely through clay. Second, it is difficult to obtain representative undisturbed samples for laboratory testing. However, because the conditions at the time of sliding were well defined, it was possible to determine the shearing resistance of the McNairy I by *back analysis.* The strength was estimated, slope stability analyses were performed, the estimated strengths were adjusted, and the analyses were repeated until the calculated factor of safety was 1.0. The strengths back-calculated from the observed failure were used in subsequent analyses to design flatter slopes and buttresses that ensured long-term stability of the slope.

Panama Canal Landslides

The Panama Canal has been plagued by slope failures ever since the beginning of construction by the French (McCullough, 1999). To achieve even marginal stabil-

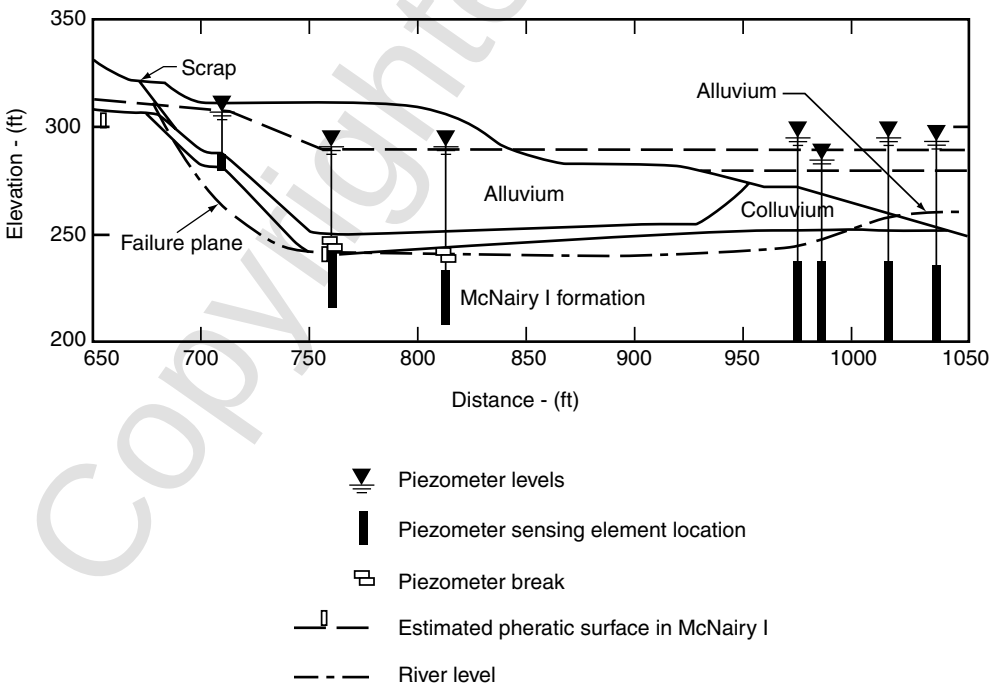


Figure 2.7 Lower bank landslide at Olmsted locks and dam site on the Ohio River.

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ity, it was necessary to excavate much gentler slopes than anticipated when the first optimistic estimates of the volume of excavation were made. Unfortunately, the clay shales in which most of the slopes were cut are subject to serious deterioration over time, and many slopes that stood when first excavated failed later.

Construction of the canal was completed in 1914, but slope failures continued for many years. In October 1986 a large landslide occurred on the Cucaracha reach of the canal, where the slopes had failed many times before. The 1986 landslide, shown in Figure 2.8, closed the canal for 12 hours and impeded traffic until December 1986, when the slide mass was cleared from the navigation channel by dredging.

For some years preceding the 1986 Cucaracha slide, the budget devoted to landslide problems in the canal had gradually decreased, because no large slides had occurred. The 1986 slide engendered renewed appreciation of the important effect that landslides could have on the canal, and the Panama Canal Commission immediately devoted more resources to detection and control of landslides. The commission increased the size of the geotechnical engineering staff and instituted a landslide control program to reduce the hazard that landslides pose to the operation of the canal. The landslide control program included investigation of the causes of the Cucaracha slide and measures to stabilize it, a program of precise and essentially continuous measurements of surface movements on slopes, systematic inspections of slopes for indications of instability, improvement of surface drainage, installation of horizontal drains for subsurface drainage, and excavation to flatten and unload slopes. This approach, which treats landslides along the canal as a hazard that requires continuing attention and active management, has been highly successful.

The Rio Mantaro Landslide

On April 25, 1974, one of the largest landslides in recorded history occurred on a slope in the valley of the Rio Montaro in Peru (Lee and Duncan, 1975). A photograph of the landslide is shown in Figure 2.9, and a cross section through the slope is shown in Figure 2.10. As shown in Figure 2.10, the slope on which the landslide occurred was about 3.7 miles (6 km) long and 1.2 miles (2 km) high. It was approximately the same height and steepness as the south rim of the Grand Canyon in Arizona, also shown in Figure 2.10. The volume of earth involved in the slide was about 2 billion cubic yards (about 1.5 billion cubic meters). It was estimated that the sliding mass achieved a velocity of 120 miles/hr (190 km/h) as it moved down the slope. When it slammed into the opposite side of the valley, it splashed up to a height of 600 ft (200 m) above the bottom of the valley and then slumped back to form a landslide dam about 550 ft (170 m) high. The impact as the sliding mass hit the opposite valley wall was recorded at a seismographic station 30 miles



Figure 2.8 1986 Cucaracha landslide at the Panama Canal.

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Figure 2.9 Rio Mantaro landslide in Peru, 1974.

(50 km) away as an event comparable to a magnitude 4 earthquake.

Prior to the slide, a town of about 450 inhabitants, Mayunmarca, was situated on the slope where the landslide occurred. After the landslide, no trace was found of the town or any of its inhabitants.

Kettleman Hills Landfill Slope Failure

On March 19, 1988, a slide occurred in a 90-ft (27-m)-high slope of a hazardous-waste landfill at Kettleman Hills, California (Mitchell et al., 1990; Seed et al., 1990). The failure involved about 580,000 cubic yards of waste (Golder Associates, 1991). A plan view and cross section through the fill are shown in Figure 2.11. The failure occurred by sliding on interfaces within the composite liner beneath the waste. The liner included three geomembranes, six geotextiles, three layers of granular fill, and two layers of compacted clay. Mitchell et al. (1990) found that some of the interfaces within the liner system had interface friction angles

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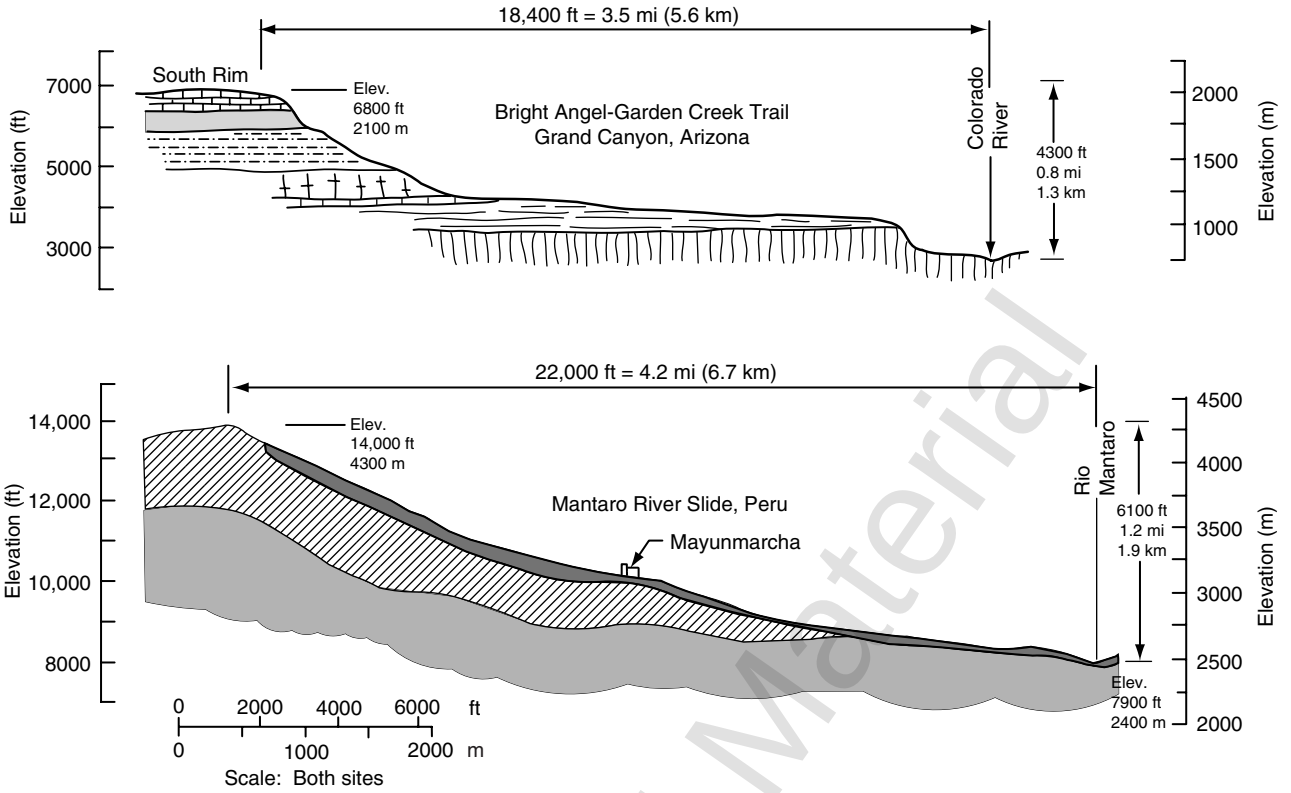


Figure 2.10 Comparison of cross sections: Grand Canyon, Arizona, and Mantaro River landslide.

as low as 8°. The lowest friction angles were found for interfaces between high-density polyethylene (HDPE) geomembranes and geotextiles, between geomembranes and geonets, and between geomembranes and compacted clay that was saturated after compaction. Seed et al. (1990) showed that factors of safety calculated for the conditions at failure were near 1.0 if the effect of wetting of the lower, nearly flat portion of the liner was taken into account and if consideration was given to three-dimensional effects. One particularly interesting aspect of the failure is that the maximum section (C1–C2 in Figure 2.11) did not have the lowest factor of safety. A shallower section near the top of Figure 2.11 had a considerably smaller factor of safety (Seed et al., 1990).

CAUSES OF SLOPE FAILURE

It is important to understand the agents of instability in slopes for two reasons. First, for purposes of designing and constructing new slopes, it is important to be able to anticipate the changes in the properties of the soil within the slope that may occur over time and the various loading and seepage conditions to which the slope will be subjected over the course of its life. Second, for purposes of repairing failed slopes, it is important to understand the essential elements of the situation that lead to its failure, so that repetition of the failure can be avoided. Experience is the best teacher—from experiences with failures of slopes come the important lessons regarding what steps are necessary to design, construct, and repair slopes so that they will remain safe and stable.

In discussing the various causes of slope failures, it is useful to begin by considering the fundamental requirement for stability of slopes: that *the shear strength of the soil must be greater than the shear stress required for equilibrium.* Given this basic requirement, it follows that the most fundamental cause of instability is that for some reason, the shear strength of the soil is *less than* the shear strength required for equilibrium. This condition can be reached in two ways:

- Through a decrease in the shear strength of the soil

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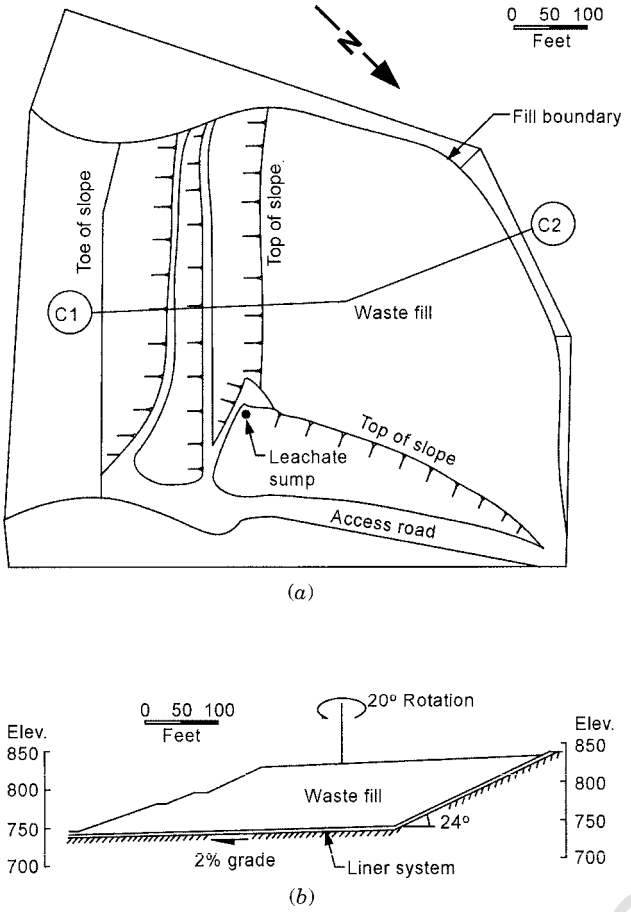


Figure 2.11 Kettleman Hills, California, landfill slope failure: (*a*) surface topography, March 15, 1988; (*b*) cross section C1–C2. (After Mitchell et al., 1990.)

- Through an increase in the shear stress required for equilibrium

The slope failures discussed in Chapters 1 and 2 include examples of both of these causes of instability.

Decrease in Shear Strength

Several different processes can lead to reduction in the shear strengths of soils. Experience has shown that the following processes are of particular importance with regard to slope stability:

1. *Increased pore pressure* (*reduced effective stress*). Rise in groundwater levels and more adverse seepage, frequently during periods of heavy rainfall, are the most frequent reasons for increased pore pressures and associated decrease in effective stresses within slopes. All types of soils are affected. The length of time required for the pore pressures to change depends on the permeability of the soil. In soils with high permeability, changes can occur rapidly, and in soils with low permeability, changes can be slow. Although the matrix permeability of clayey soils is usually very low, clay masses can have surprisingly high secondary permeability, due to cracks, fissures, and lenses of more permeable materials. As a result, pore pressures within clay deposits can change with surprising rapidity.
2. *Cracking.* Slope failures are frequently preceded by development of cracks through the soil near the crest of the slope. These cracks develop as a result of tension in the soil at the ground surface that exceeds the tensile strength of the soil. Cracks are possible only in soils that have some tensile strength. Quite clearly, once the soil is cracked, all strength on the plane of the crack is lost.
3. *Swelling* (*increase in void ratio*). Clays, especially highly plastic and heavily overconsolidated clays, are subject to swell when in contact with water. Low confining pressures and long periods of access to water promote swell. It has generally not been possible to achieve the same amount of swell in laboratory tests as occurs in the field. Kayyal (1991) studied highway embankments near Houston, Texas, constructed of highly plastic compacted clays, which failed 10 to 20 years after construction as a result of swell and strength loss. Similar shallow slides in highly plastic clays have occurred in many areas. Skempton (1964) showed three cases of slides in the overconsolidated London clay, where zones of higher water content extended for about an inch on either side of the shear surfaces, indicating that the shear stresses within the developing rupture zone led to localized dilation of the heavily overconsolidated clay.
4. *Development of slickensides.* Slickensided surfaces develop in clays, especially highly plastic clays, as a result of shear on distinct planes of slip. As shear displacements occur on a distinct plane, platelike clay particles tend to be realigned parallel to the plane of slip. The result is a smooth surface that exhibits a dull luster, comparable in appearance to the lustrous surface of a new bar of soap. The clay separates readily across these surfaces, and they can be found by breaking hand samples in tension or by picking at the walls of trenches. Slickensided surfaces are weaker than the surrounding clay where par-

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ticles are randomly oriented. The friction angle on slickensided surfaces is called the *residual friction angle.* In highly plastic clays this may be only 5 or 6°, compared with peak friction angles of 20 or 30° in the same clay. Slickensides develop most prominently in clays that consist predominantly of clay-size particles; significant silt or sand content inhibits their formation. In some deposits randomly oriented slickensides develop as a result of tectonic movements. These have less significance for slope stability than a single set of slickensides with an adverse orientation.

5. *Decomposition of clayey rock fills.* Clay shales and claystones excavated for use as fill may break into pieces of temporarily sound rock that can be compacted into a seemingly stable rock fill. Over time, however, as the fill is wetted by infiltration or by groundwater seepage, the pieces of rock may slake and revert to chunks of disaggregated clay particles. As the clay swells into the open voids within the fill, it can lose a great deal of its strength, and the fill can become unstable.
6. *Creep under sustained loads.* Clays, especially highly plastic clays, deform continuously when subjected to sustained loading. These clays may eventually fail under these sustained loads, even at shear stresses that are significantly smaller than the short-term strength. Creep is exacerbated by cyclic variations in conditions, such as freeze–thaw and wet–dry. When the cyclically varying conditions are at their adverse extremes, movements occur in the downhill direction. These movements are permanent—they are not recovered when conditions are less adverse. The long-term result is ratcheting downslope movement that gradually increases from year to year, and this may eventually result in sliding on a continuous failure plane.
7. *Leaching.* Leaching involves changes in the chemical composition of pore water as water seeps through the voids. Leaching of salt from the pore water of marine clays contributes to the development of quick clays, which have virtually no strength when disturbed.
8. *Strain softening.* Brittle soils are subject to strain softening. After the peak of the stress–strain curve has been reached, the shearing resistances of brittle soils decrease with increasing strain. This type of stress–strain behavior makes progressive failure possible and makes it impossible to count on mobilizing the peak strength simultaneously at all points around a shear surface (see Chapter 3).
9. *Weathering.* Rocks and indurated soils are subject to strength loss as a result of weathering, which involves various physical, chemical, and biological processes (Mitchell, 1993). Physical processes break the strong soil or rock into smaller pieces, and the chemical and biological process change it into material with fundamentally different properties. Weaker soils are also subject to weathering effects, but may become stronger, rather than weaker, as a result (Mitchell, 1993).
10. *Cyclic loading.* Under the influence of cyclic loads, bonds between soil particles may be broken and pore pressures may increase. The soils most subject to loss of strength due to cyclic loads are loose soils and soils with particles that are weakly bonded into loose structures. Loose sands may liquefy under cyclic loading, lose virtually all strength, and flow like a liquid.

Water plays a role in many of the processes that reduce strength, and as discussed in the following section, water is also involved in many types of loads on slopes that increase shear stresses. It is not surprising, therefore, that virtually every slope failure involves the destabilizing effects of water in some way, and often in more than one way.

Another factor involved in most slope failures is the presence of soils that contain clay minerals. The behavior of clayey soils is much more complicated than the behavior of sands, gravels, and nonplastic silts, which consist of chemically inert particles. The mechanical behavior of clays is affected by the physicochemical interaction between clay particles, the water that fills the voids between the particles, and the ions in the water. The larger the content of clay minerals, and the more active the clay mineral, the greater is its potential for swelling, creep, strain softening, and changes in behavior due to physicochemical effects. The percentage of clay in a soil and the activity of clay minerals are reflected qualitatively by the value of the *plasticity index* (PI). For that reason PI affords a useful first indication of the potential for problems that a clayey soil poses: The higher the PI, the greater the potential for problems. See Mitchell (1993) for a thorough discussion of the physicochemical behavior of clays.

It is safe to say that except for the effects of water and clayey soils, slope failures would be extremely rare. The truth of this statement is illustrated by the slopes on the surface of the moon, where there is neither water nor clay. In that environment, slopes remain

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stable for eons, failing only under the influence of violent meteor impacts. On Earth, however, both water and clays are common, and slope failures occur frequently, sometimes with little or no warning.

Increase in Shear Stress

Even if the strength of the soil does not change, slopes can fail if the loads on them change, resulting in increased shear stresses within the soil. Mechanisms through which shear stresses can increase include:

1. *Loads at the top of the slope.* If the ground at the top of a slope is loaded, the shear stress required for equilibrium of the slope will increase. Common occurrences that load the ground are placement of fill and construction of buildings supported on shallow foundations. To avoid significantly increasing the shear stresses in the slope, such loads should be kept away from the top of the slope. An acceptable distance can be determined by slope stability analysis.
2. *Water pressure in cracks at the top of the slope.* If cracks at the top of a slope are filled with water (or partially filled), the hydrostatic water pressure in the cracks loads the soil within the slope, increasing shear stresses and destabilizing the slope. If the cracks remain filled with water long enough for seepage toward the slope face to develop, the pore pressures in the soil increase, leading to an even worse condition.
3. *Increase in soil weight due to increased water content.* Infiltration and seepage into the soil within a slope can increase the water content of the soil, thereby increasing its weight. This increase in weight is appreciable, especially in combination with the other effects that accompany increased water content.
4. *Excavation at the bottom of the slope.* Excavation that makes a slope steeper or higher will increase the shear stresses in the soil within the slope and reduce stability. Similarly, erosion of soil by a stream at the base of a slope has the same effect.
5. *Drop in water level at the base of a slope.* External water pressure acting on the lower part of a slope provides a stabilizing effect. (This is perhaps the only good thing that water can do to a slope.) If the water level drops, the stabilizing influence is reduced and the shear stresses within the soil increase. When this occurs rapidly, and the pore pressures within the slope do not decrease in concert with the drop in outside water level, the slope is made less stable. This condition, called *rapid drawdown* or *sudden drawdown,* is an important design condition for the upstream slopes of dams and for other slopes that are partially submerged.
6. *Earthquake shaking.* Earthquakes subject slopes to horizontal and vertical accelerations that result in cyclic variations in stresses within the slope, increasing them above their static values for brief periods, typically fractions of a second. Even if the shaking causes no change in the strength of the soil, the stability of the slope is reduced for those brief instants when the dynamic forces act in adverse directions. If the cyclic loading causes reduction in soil strength, the effects are even more severe.

SUMMARY

When a slope fails, it is usually not possible to pinpoint a single cause that acted alone and resulted in instability. For example, water influences the stability of slopes in so many ways that it is frequently impossible to isolate one effect of water and identify it as the single cause of failure. Similarly, the behavior of clayey soils is complex, and it might not be possible to determine in some particular instance whether softening, progressive failure, or a combination of the two was responsible for failure of a slope. Sowers (1979) expressed the difficulties in attempting to isolate *the* cause of failure in these words: ''In most cases, several 'causes' exist simultaneously; therefore, attempting to decide which one finally produced failure is not only difficult but also technically incorrect. Often the final factor is nothing more than a trigger that sets a body of earth in motion that was already on the verge of failure. Calling the final factor *the cause* is like calling the match that lit the fuse that detonated the dynamite that destroyed the building *the* cause of the disaster.''

The fact that it is so difficult to isolate a single cause of failure highlights the importance of considering and evaluating all potential causes of failure in order to develop an effective means of repairing and stabilizing slopes that have failed. Similarly, in designing and constructing new slopes, it is important to attempt to anticipate all of the changes in properties and conditions that may affect a slope during its life and to be sure that the slope is designed and constructed so that it will remain stable despite these changes.

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