Geological Society of America
Reviews in Engineering Geology, Volume IX
1992

## Chapter 10

# *Recent developments in landslide mitigation techniques*

**J. David Rogers**
*Rogers/Pacific, Inc., 396 Civic Drive, Pleasant Hill, California 94523*

## ABSTRACT

**This chapter begins with a brief synopsis of landslide repair methodologies developed over the past 70 years. Early attempts at stabilizing slopes focused on emplacement of toe buttresses and inclusion of subdrainage. As earthwork equipment became larger and more capable, the removal and recompaction of entire slide masses became commonplace. Over the past decade, geotextile and geomembrane products have become available that can significantly alter the options for repair, especially under conditions of restricted access or poor weather. In the balance of the chapter, I seek to introduce the reader to some of these products and to case histories of ways in which they have been combined to effect novel solutions to slope stability problems.**

## INTRODUCTION

It has been more than 40 years since Karl Terzaghi's (1950) now-classic paper appeared in the Geological Society of America's *Applications of Geology to Engineering Practice.* In the interim, little has changed with respect to understanding the theorems of effective stress and progressive failure that promote slope instability.

Since 1950, increased development in hillside areas has underlined the importance of understanding the geologic factors promoting instability before beginning engineering analysis or repair. All too often, sites prone to landsliding have been the scene of repeated repair attempts within a few years of each other. Experience over the past half-century tends to suggest that many landslide repair attempts are made without benefit or full understanding of the geometry and hydrologic regimen of the affected sites. In addition, the blind implementation of a traditional, engineered repair scheme, for example, recompaction, may not serve to mitigate adequately all manner and form of future slope instability.

In this chapter I explore some of the more innovative techniques available for landslide repair that have come into practice in the past decade or so.

The rational design of a landslide repair cannot begin until the factors of site geology are properly evaluated. In most engineering analyses, the fundamental factors are: (1) the relative position of the ground-water table; (2) the fluctuation of groundwater levels and the flow volumes ascribable to infiltration or subaqueous flow aquifers); and (3) confirmation of the presence, character, and geometric extent of both ancient and active landslide slip surfaces.

### *Mitigation via excavation and recompaction*

The earliest engineering attempts at landslide correction likely occurred along railroad and canal embankments in England and France, beginning in the 1830s. As the industrial revolution took root in the late nineteenth century, powered excavation machinery such as track-mounted steam-powered shovels spearheaded a revolution in earthwork construction. From 1850 to 1950, most cut slopes were excavated at slopes of 1:1 (45°) or steeper, and fill was placed on embankments of about 1.5:1 (horizontal to vertical). Steeper embankments were accommodated by stacking rock or masonry blocks to create gravity retaining walls, then filling at 1.5:1 above such structures.

When disaster struck in the form of a slope failure, the style or method of repair depended on cost and the available right of way (Sharpe, 1938). In rural areas, such as cut slopes on the Panana Canal (MacDonald, 1913, 1947), failed excavations were simply laid back to a more stable inclination (from 1:1 to 10:10 in some cases). In more urbanized or mountainous areas, where there was little available right of way, concrete and masonry gravity retaining walls were most often employed (Ladd, 1935).

Self-propelled earth-moving equipment began to show up on the civil engineering scene in the 1920s as part of the ambitious road-building programs being employed throughout the

Rogers, J. D., 1992, Recent developments in landslide mitigation techniques, *in* Slosson, J. E., Keene, A. G., and Johnson, J. A., eds., Landslides/Landslide Mitigation: Boulder, Colorado, Geological Society of America Reviews in Engineering Geology, v. IX.

United States. With self-propelled equipment, landslides could be excavated and replaced with some more suitable material, such as drain rock or riprap. By the 1930s, most large landslide repairs consisted of either partial excavation of the headscarp area and/or the placement of toe buttresses, most commonly over existing creeks or gullies (Terzaghi, 1931). Such repairs were usually effected in combination with some sort of subdrainage, either withdrawal wells or trench subdrains (Larkey, 1936; Root, 1938; Greeley, 1940). A scheme typical of this early era is shown in the upper half of Figure 1.

By the mid-1940s, sheepsfoot compactors began to be employed for so-called "dry" compaction of large earth embankments (for example, the Hansen Flood Control Basin) and rock-fill dams (the San Gabriel Dam). Up to this time (1942), only smooth tire compactors with contact pressures of about 40 psi had been available. The sheepsfoot roller allowed contact pressures of around 250 psi, a six-fold increase in compactive effort (Baumann, 1936, 1937, 1941; Proctor, 1933). In the years following World War II, large earthwork projects became commonplace with the introduction of larger, self-propelled hydraulic-powered equipment and the infusion of large projects spawned by the Interstate Highways Act of 1955 and water retention, reclamation, and flood-control projects, sponsored by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers.

In constructing larger cuts and fills, some were invariably placed across ancient landslide deposits without benefit of geologic engineering input. By the late 1950s, a new style of repair came onto the scene, known by most practitioners as the "recompacted buttress fill," shown as the lower half of Figure 1.

Buttress fills remain the most commonly employed method of landslide repair in the United States. They are identical to new construction embankments in that they employ shear key benches, which are excavated beneath zones of disturbance (landslide slip surfaces) or potential distress, such as soil and organic horizons. In landslide repairs, subdrains are almost always included at the heels of the key benches in order to alleviate pore pressures that promote land slippage (Forbes, 1947).

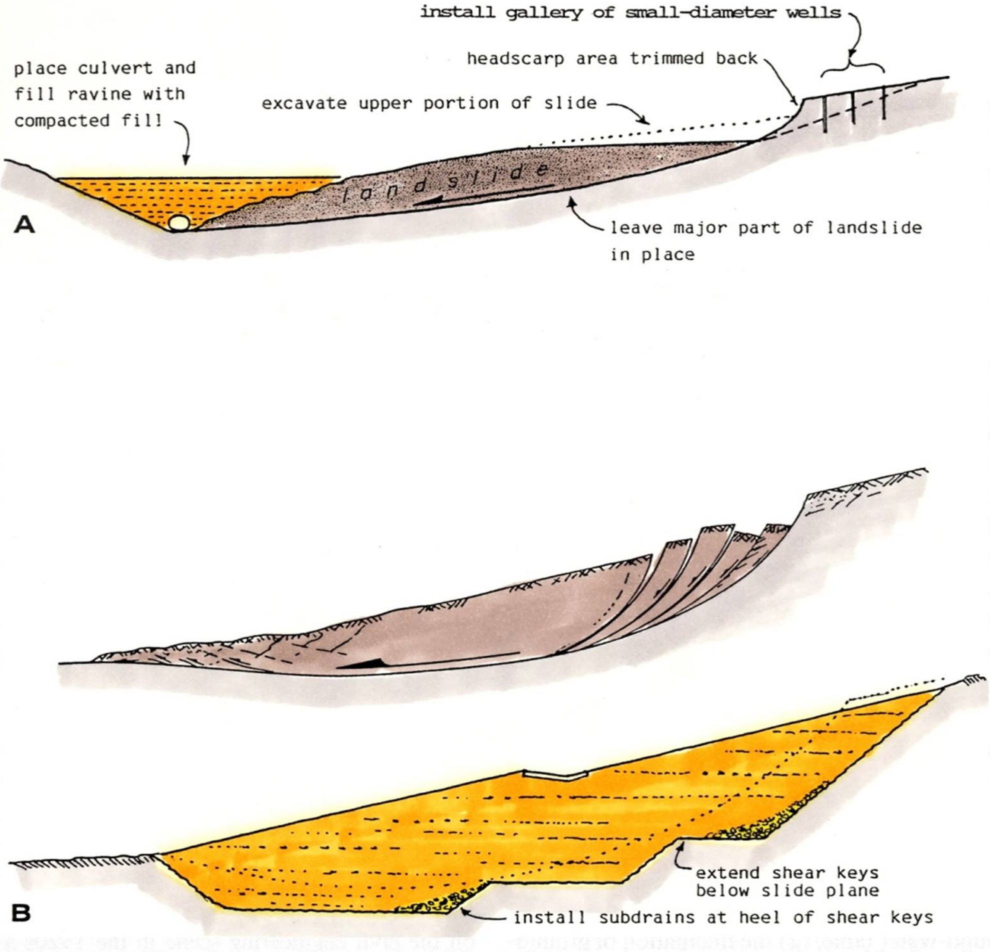


Figure 1. A, The original approach to landslide repair was to buttress toe areas in combination of limited removal of the upslope area, trimming back the headscarp and installing wells to draw down the watertable. B, As earth-moving equipment became more capable, the entire mass of a landslide could be excavated and recompacted in a buttress fill. Often such buttresses are constructed with subdrains composed of free-draining material.

Removal and replacement techniques have inherent liabilities. The landslide material must necessarily be excavated, carried, and stockpiled at an adjacent location. In large slides, it is possible to excavate in one area, simultaneously placing the excavated muck in another area already excavated and prepared for fill placement. However, in steep terrain, available stockpile area may be scant, requiring the construction of temporary fill-stockpile fences.

A second liability is the normally high moisture content of the slide material, which is usually excessive in the season following the earth movement. The slide material often requires scarification, drying, and/or mixing in order to bring moisture levels close to optimum for placement at 90% to 95% relative compaction. This requires additional handling, warm sunny days (or appreciable wind), and a larger working area.

Simple recompaction of low-strength materials can be dangerous in that compacting does not change the mineralogical makeup of the material. Soft, expansive clay will still be expansive, if not as soft. Even though compacted to a high degree of density, the fill is still able to absorb additional water through swelling or mineralogical absorption by cation exchange with percolating ground water. It is for these reasons (and subdrain clogging) that many recompacted buttress fills have failed over the past 35 years.

## *Conventional retention structures*

A variety of retention structures have been successfully employed to repair land slippage where high-value structures are inextricably involved with the repair. The types of structures are basically divisible into four main categories: (1) gravity structures; (2) cantilever structures; (3) flexible and/or bulkhead walls; (4) retained structures; in addition, combination structures can incorporate one or more of the methods.

Examples of the traditionally employed wall structures and engineered retention systems are shown in Figures 2–6.

## *Landslide mitigation using subdrainage*

***Types of subdrains.*** Where differential settlement or grievous loss of property are not immediately apparent due to distance, landslides sensitive to pore-pressure buildup can be effectively mitigated by simply providing sufficient underdrainage (Root, 1938, 1955a; Forbes, 1947; Stanton, 1948; Cedargren, 1989).

Equipment sizes now permit the construction of large continuous trench subdrain systems that can be backfilled with geotextile or rock mixtures. Figure 7 shows a typical outline of the various types of subdrain applications in landslide repairs.

***Pipes.*** The type and style of perforated pipe for conventional or trench subdrains need to be considered. The designer should consider overburden pressure, long-term maintenance, differential settlement potential, and corrosion resistance. Thick-wall ABS pipes are currently the most favored for cost, corrosion resistance, and ability to be maintained by roto-rooter or rodding. ABS-Truss pipes are particularly strong and well suited for deep subdrain applications. Clean-outs for periodic maintenance are now standard at every turn or 500 ft (~150 m) of straight section, and 90° bends are typically not allowed. PVC, polyethylene, and polypropolyene all degrade under ultraviolet radiation. Polyethylene flex lines and crush lines have very low tolerance for earth loading and should never be utilized in underdrainage applications.

***Cost considerations.*** In utilizing an approach wholly dependent upon subdrainage for landslide mitigation, several key factors will likely govern the cost: access for crawling excavator or backhoe; hauling distance of rock or gravel from the drop point; available topographic slope for drainage outlets; hauling away of spoils and waste; and availability of pervious granular backfill. Hauling of material from the drop point can be done with portable conveyers if conditions warrant. Drainage outlets (if topographic slope is insufficient) might be handled with large sump pumps or outletted via hydraugers drilled from sufficient distances downstream (Forbes, 1947; Herlinger and Stafford, 1952; Root, 1955b).

On the basis of cost only, the emplacement of subdrains alone would appear to be economic compared to other methods such as retaining structures or removal and replacement. An example of a subdrain-only repair is presented in Figure 8. A typical comparison of cost versus safety factor on a small repair is presented in Figure 9.

Most engineers are hesitant to use subdrainage alone, because there are no guarantees that the drains will continue to function for a long period of time without problems. These problems include clogging by dispersive clays, cohesionless silts, or the root systems of dense stands of vegetation. Rodents can make homes of subdrain outlet pipes, or these pipes can become overgrown near their discharge (or daylight) point, causing the system to back up. Hard ground water can deposit calcium carbonate around the percolation slits in subdrain pipes or well casings. Subdrains necessarily require maintenance or periodic replacement. As a consequence, most practitioners prefer to design with "defense-in-depth," using subdrainage in combination with other measures such as walls and recompacted buttresses.

## *Soil reinforcement using geomembranes and geosynthetics*

***Types of geosynthetics and geomembranes.*** Over the past decade a plethora of geosynthetic materials has become available for use as seepage membranes, pavement crack stoppers, tank liners, and soil reinforcements. These products are marketed in the following categories.

1. *Pavement cloths* are usually marketed as crack-stopper membranes such as Petromat; they are tack-coated as an overlay to existing pavement. Then a new surface of asphalt-concrete is placed upon the membrane. Crack-stopper membranes can be relatively impervious (woven fabrics) or pervious (spun fabric).
2. *Filter cloth* membranes such as Mirafi, Supac, and Tri-

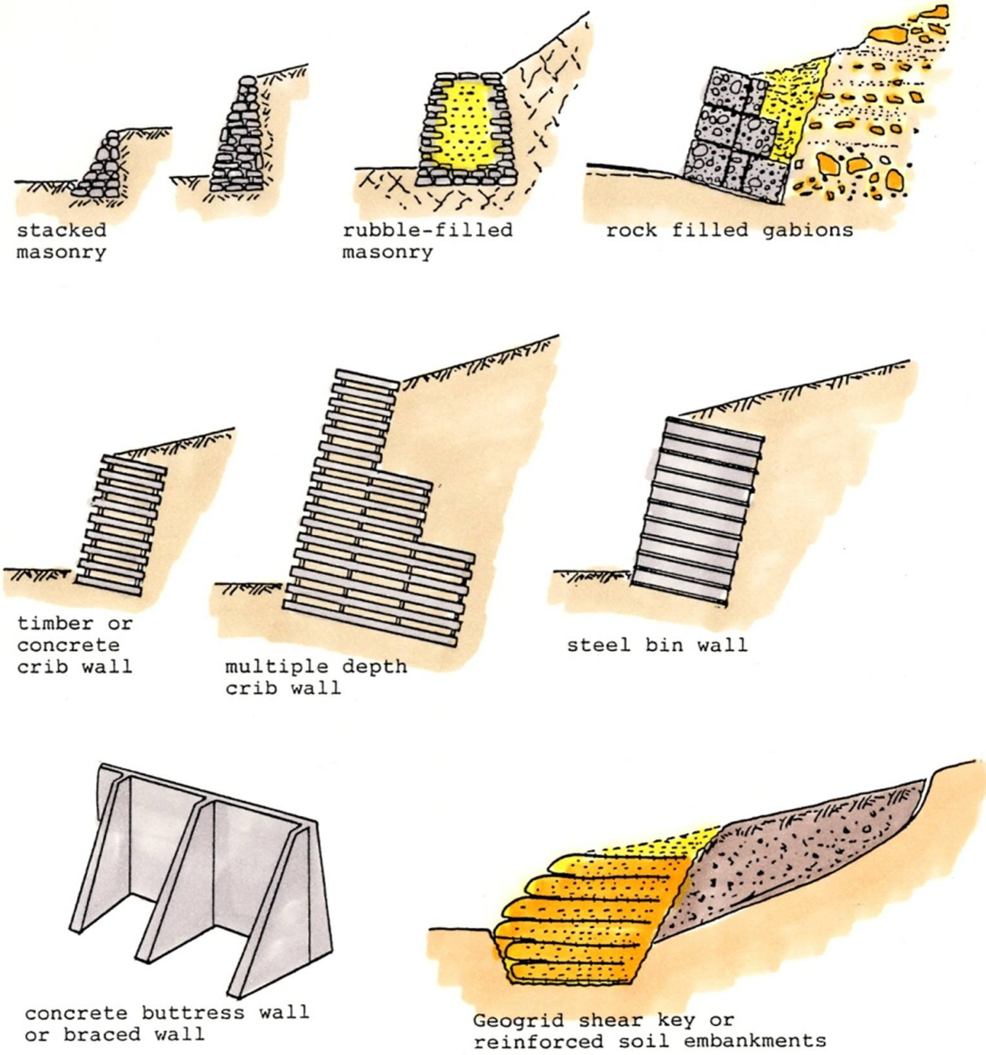


Figure 2. Various types of gravity retention structures. Such structures depend upon their sheer mass as a resisting force to the load imposed by a hillside. This is the earliest type of retention structure, having been used by Assyrians and Egyptians beginning around 2900 B.C.

vera are marketed mainly as seepage-filtration barriers. these are most commonly used beneath railroad ballast and highway aggregate base courses to prevent infiltration and settlement of the gravel into the underlying soils. Filter cloths can also be used to line subdrains constructed in drainage swales, hillsides, fill embankments, and areas prone to landslides or debris flows. Filter cloths are pervious, usually being composed of a spun, needle-punched cloth. These cloths can be lapped or sewn together in the field or at the factory.

3. *Liner membranes,* such as Hypalon, are designed as impervious membranes to effect cut off of contaminated ground water, "clean" ground water (for example, from swimming pools), or leachates from dump areas or embankments. Liner members are impervious, and are usually composed of rubbery compounds that can be sealed with the use of hand-applied solvents, so two sections of membrane can be joined together.

4. *Drainage membranes* are composites of the above materials; they necessarily combine some sort of seepage membrane with an attached filter cloth. Drainage membranes are beginning to be widely employed in the construction of retaining walls.

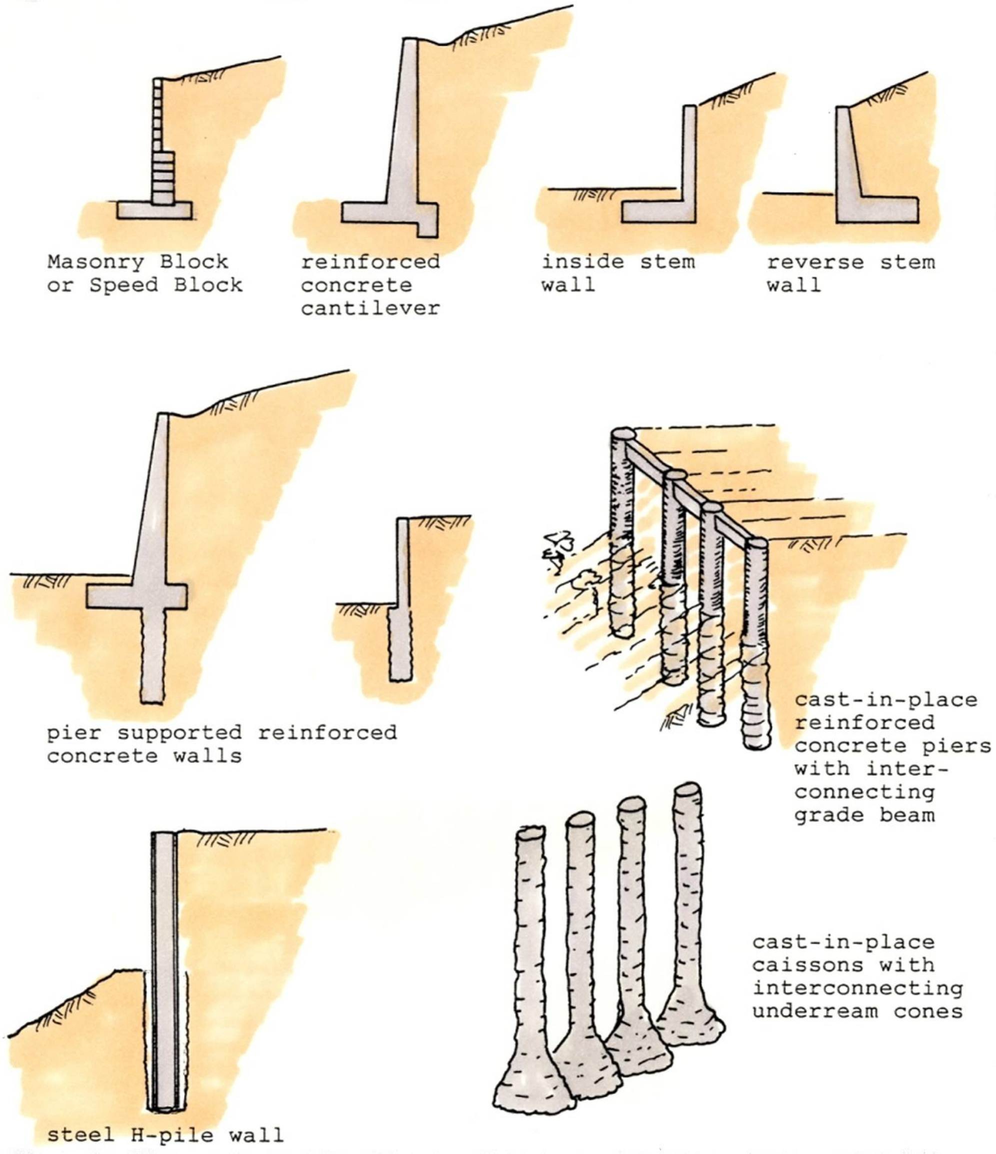


Figure 3. Various types of cantilever retention structures. Such structures came into use with the advent of pile driving, which dates back to Roman times. The use of large-diameter augers allows such structures to be constructed in stiff soils and soft rock.

These products include Enkamat and Enkadrain, Miradrain, Tensar DC-1200, and TENAX TN, TENAX MNT, and TENAX TNT.

5. *Soil reinforcement grids* include Geogrid products from Tensar, Nicolon, and Tenax. They are beginning to be widely employed in the construction of soil and rock embankments. The grids are of open-mesh construction, usually composed of polypropylene or polyethylene with carbon-black ultraviolet radiation inhibitors. The grids are constructed by mechanical pulling after roller extrusion. As a consequence, they typically possess anisotropic strength properties, one direction being stronger than the other. This stronger direction is typically aligned parallel to the fall line of the adjacent slope, to reinforce the soil most efficiently and resist downslope movement.

***Corrosion resistance and expected longevity.*** The "laboratory" of engineering theory is experience. Because geotextiles are a recently introduced technology, the data relating to their actual use are only for about 10 years. However, experience to

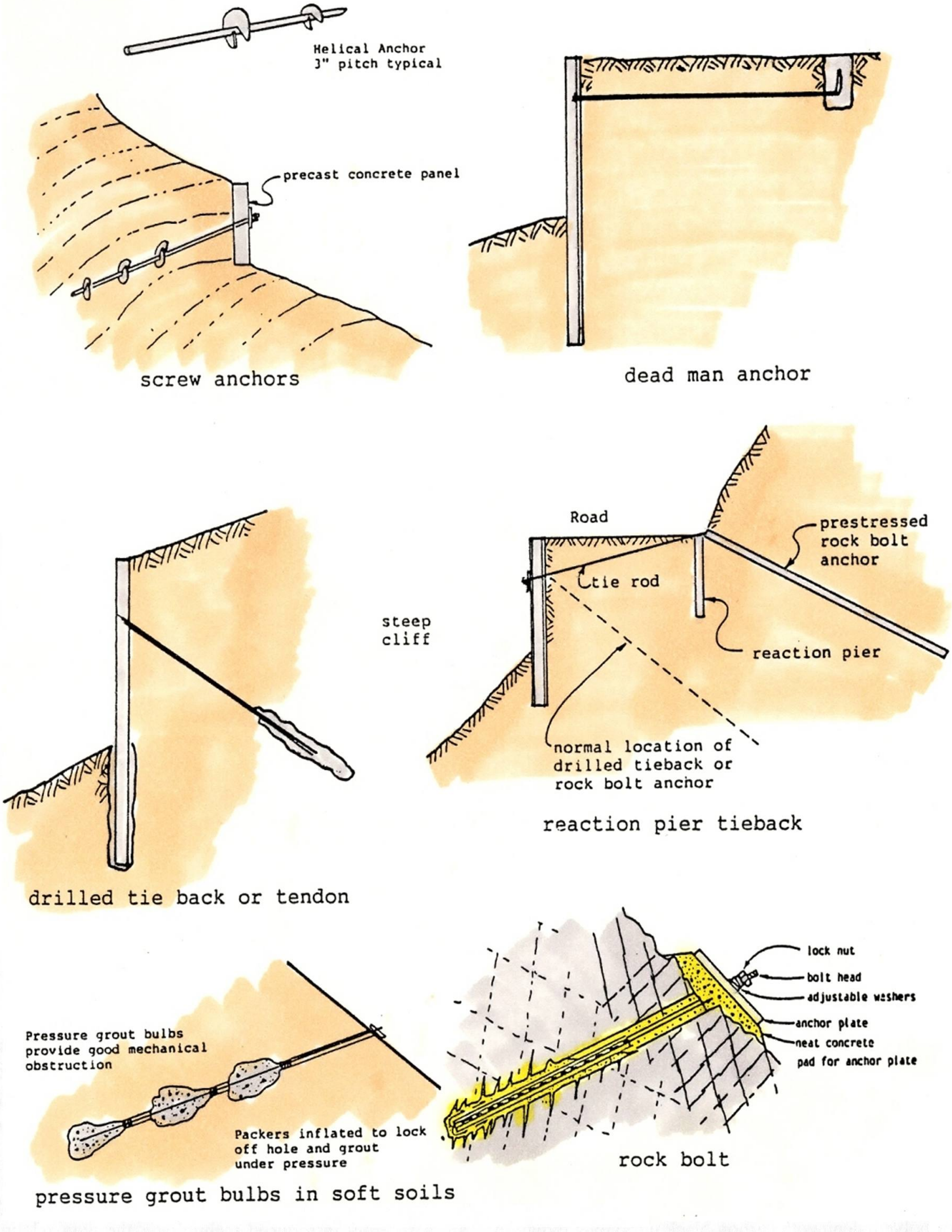
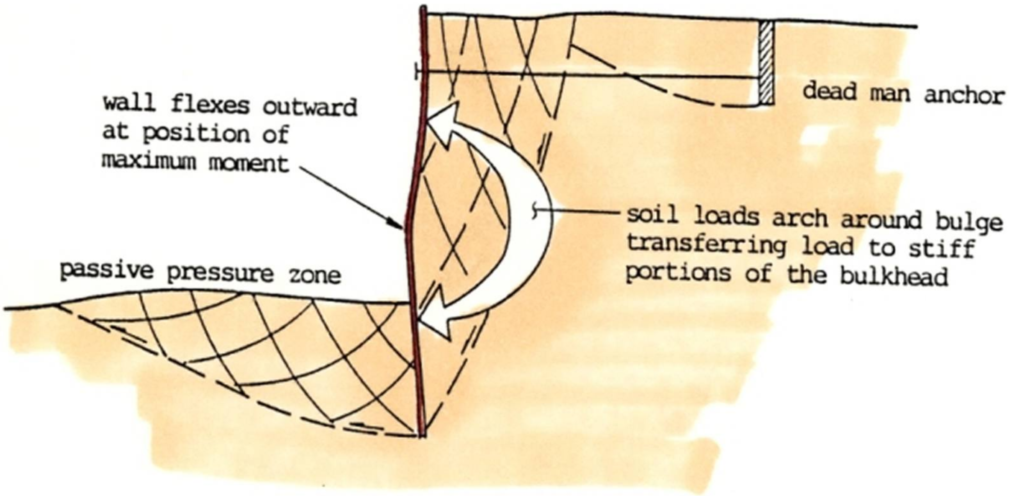
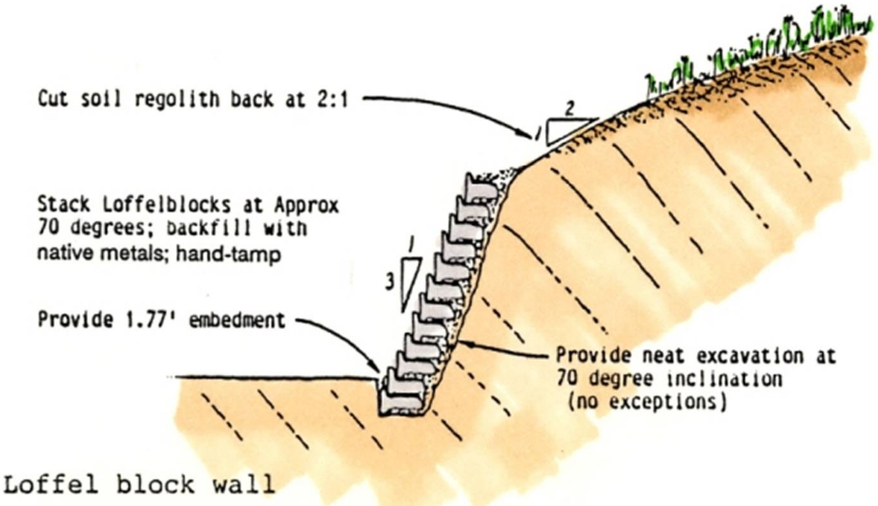


Figure 4. Various types of retained structures—those employing tension elements. The cost and feasibility of such structures is almost wholly dependent on drill rig access and drillability of the ground.





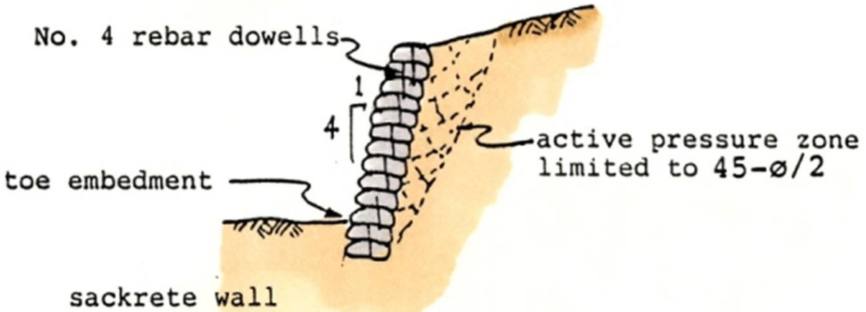


Figure 5. Various types of flexible retention structures, or those that deflect in order to shed their imposed loads. Such deflection lessens wall loads by allowing the ground mass to mobilize its shear strength (Rankine active pressure theory).

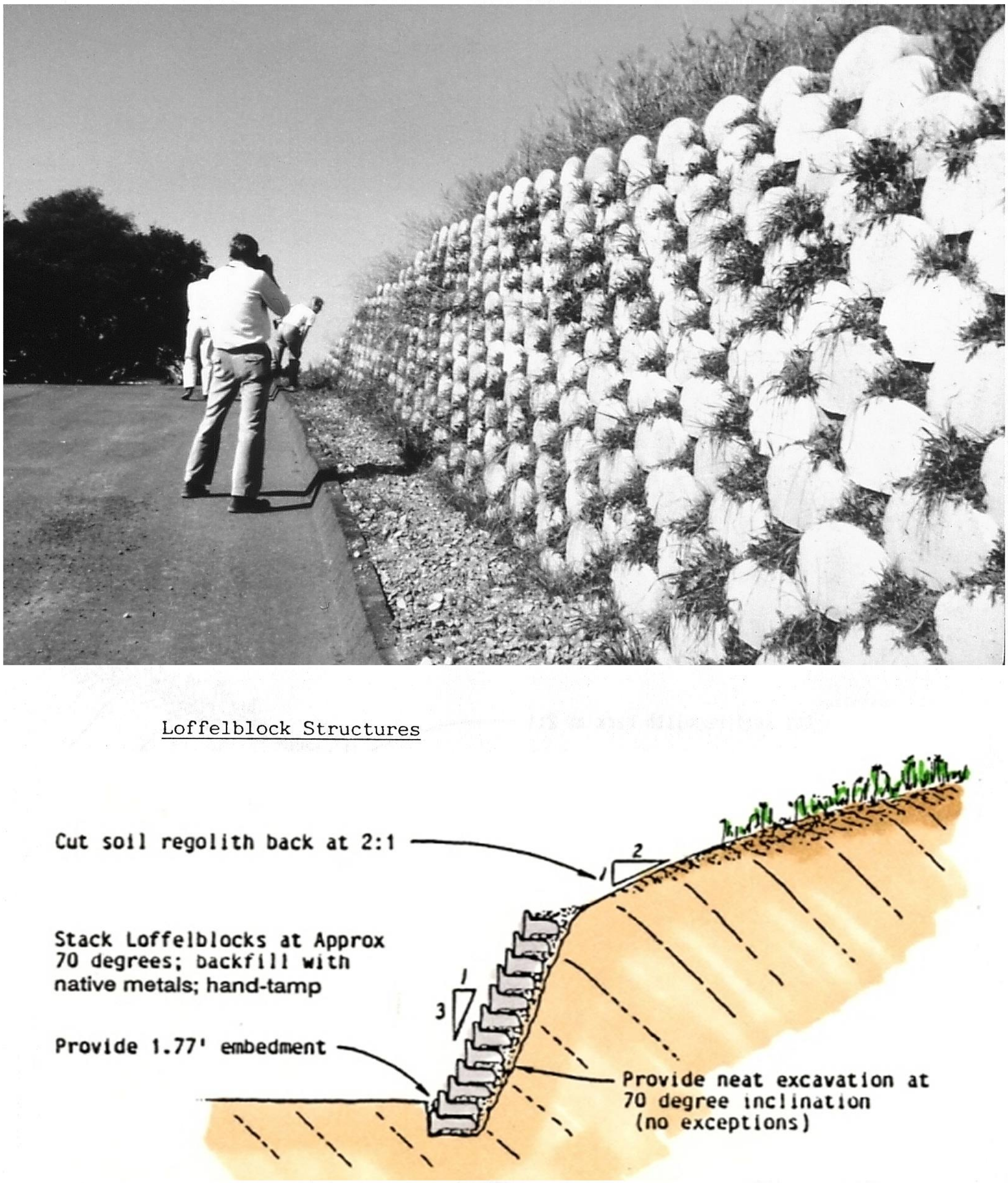


Figure 6. The Loffelstein, or Loffelblock retaining wall is a design concept emanating from Austria, and is now produced in the United States. Extremely economic, its primary application is for slopes under 22 ft (~6.6 m) high with an angle of internal friction, $\phi$, greater than 30°. In the case shown, the wall was constructed on a 20% longitudinal gradient to support a highway cutslope. Such walls can be built for \$12 to \$15 per square foot (in 1988 U.S. dollars).

date with such materials has been very promising. Geogrids can melt when exposed to flames, but are not combustible.

*Metalliferous Products.* Engineers began using geotextiles to extend the expected lifetime of buried structures. In the 1950s and 1960s, metalliferous subdrain pipes, collectors, conduits, and culverts were buried in embankments as part of rationally designed civil engineering works. These included such products as perforated metal pipes (PMP), corrugated metal pipes (CMP), steel and aluminum culverts, and steel binwalls, iron pipe, threadbar reinforcing rods (tiebacks), steel H-piles, and galvanized metal reinforcing strips.

The presence of chlorides, in any dose, was found to be extremely detrimental to the longevity of buried metal elements. Salt, in any concentration, has an appreciably hard effect on buried metal objects. Steel or iron structures subject to high tensile stresses near water have been found to be susceptible to

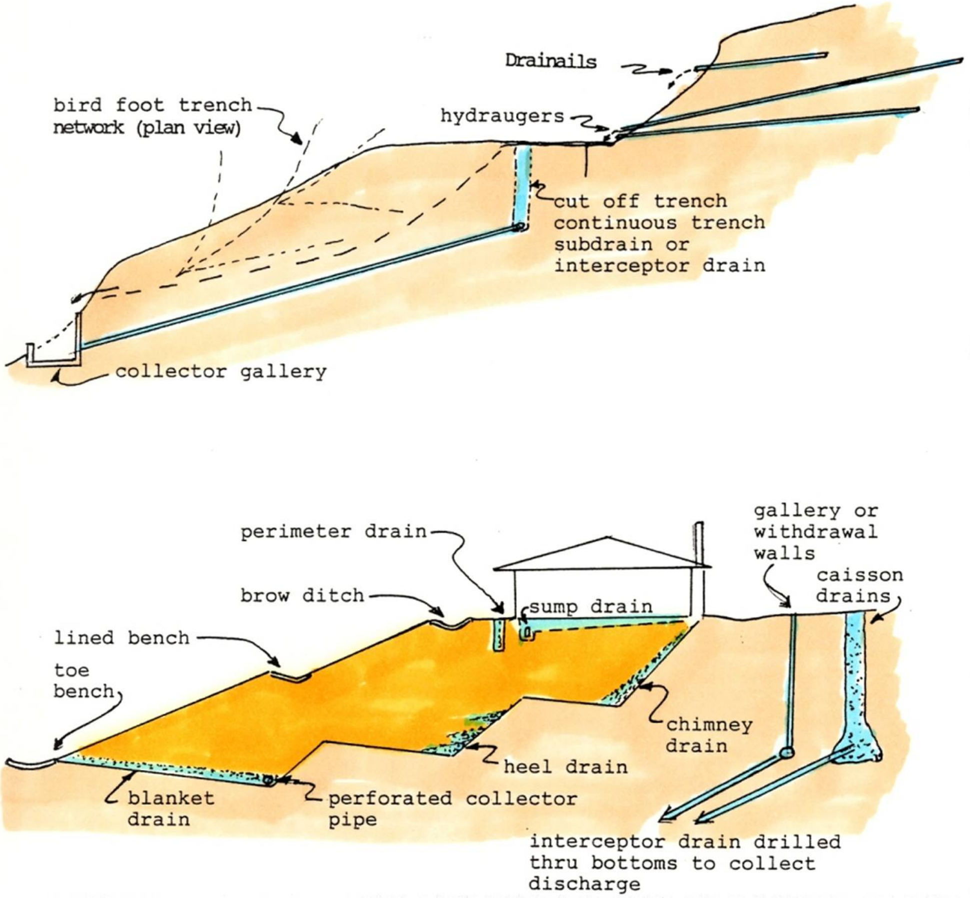


Figure 7. Traditionally employed nomenclature of the various types of subdrainage measures used by most geotechnical practitioners.

hydrogen embrittlement, a physiochemical process by which the steel structure is attacked and snaps under load, causing catastrophic failure. Mercury, in even the smallest concentrations, attacks aluminum with noticeable severity.

Over the past 30 years, the empirical relation between corrosivity and soil resistivity has been recognized and researched by the American Society of Testing and Materials, the Electric Power Research Institute, and other governmental agencies, such as state highway departments. These organizations have concluded that soil pH is a basic determinator of longevity and a cause of problems. Soil resistivity of less than 2000 ohm-cm usually indicates that some sort of corrosion protection is required, and resistivities of less than 500 ohm-cm indicate extremely high corrosivity, thereby negating the use of metals for long-term applications.

*Plastics and Composites.* In the late 1960s, products like polyvinyl chloride (PVC) pipe began to be utilized in some buried pipe–conduit–subdrain applications due to its light weight, high strength, and small cost. However, time has shown that plastics possess their own problems, including (1) long-term embrittlement due to absorption and/or exposure to hydrocarbons and acid rain; (2) embrittlement due to ultraviolet radiation exposure; and (3) strength loss ascribable to long term creep under sustained loading.

As plastics were used more, new elements were marketed. These included the following.

Styrene plastic is lighter, more brittle, and subject to the same detractions as PVC, but cheaper.

Polyethylene pipes are impregnated with carbon-black to better resist ultraviolet deterioration, but are of insufficient strength to withstand any sort of sustained loading.

Acrylonitrile Butadiene Styrene (ABS) was introduced in the late 1970s to provide a high-strength plastic with built-in defenses for those environmental factors that caused problems for PVC. ABS possesses few reaction problems, even with corrosive fluids. It is not sensitive to ultraviolet radiation and is constructed with sufficient sidewall thickness to be sewer-standard and capable of withstanding rooter cleaning. Since 1984, ABS has also been available in truss construction, permitting the highest degree of bending resistance within a minimum weight section.

Polypropylene products such as geotextiles and some geogrid materials are the least suitable for long-term exposed applica-

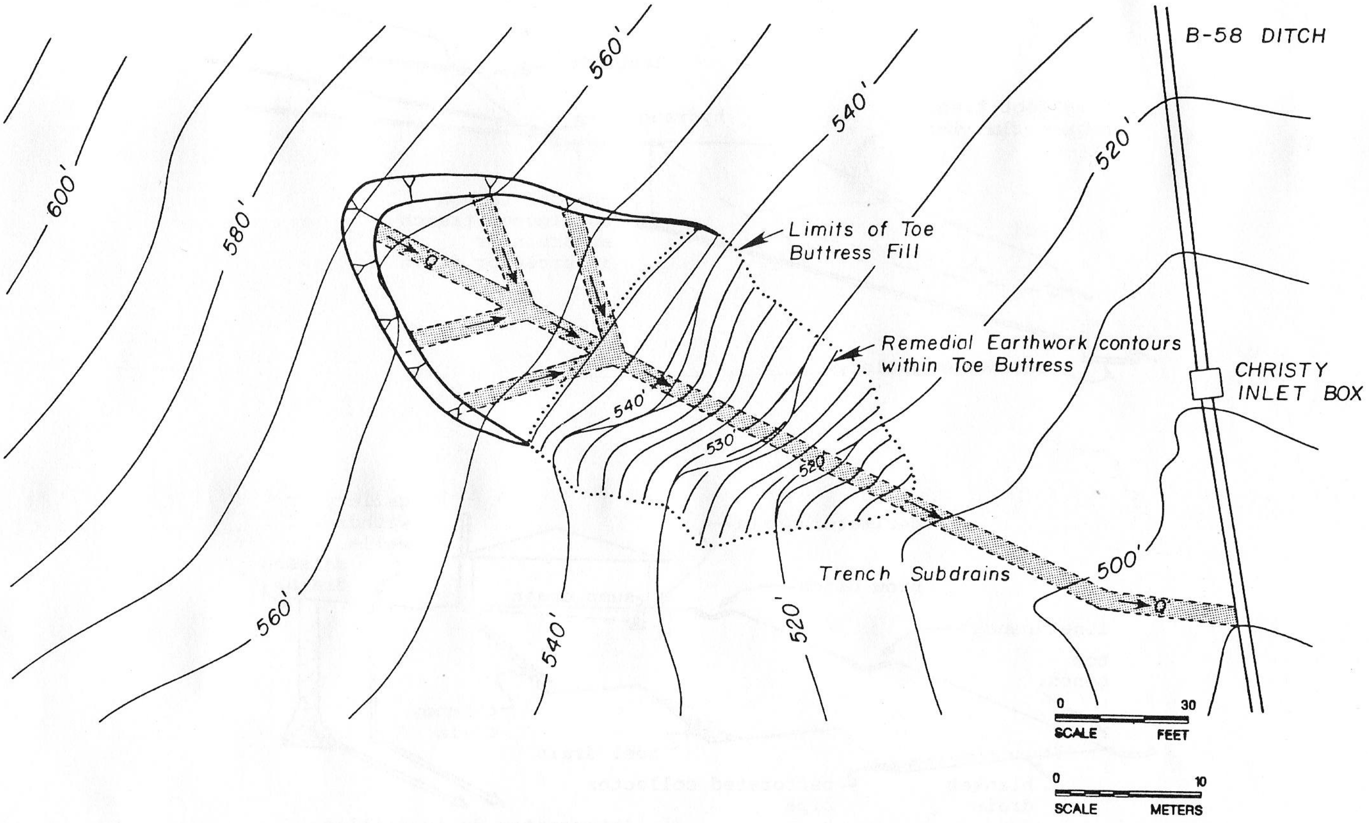


Figure 8. Birdfoot-style trench subdrain network, as seen in plan view. Simple trench subdrains offer a low-cost alternative for slide repairs in rural areas where subsequent ground movement due to consolidation and/or creep is of little consequence or economic concern.

tions because of their susceptibility to breakdown under ultraviolet radiation. Ultraviolet inhibitors such as carbon-black and protective coatings can be used, but they only serve to retard the breakdown process, not to prevent it. Ultraviolet breakdown is most acute in higher elevations (>5000 ft [~150 m]) where there is less filtration of the sun's ultraviolet rays.

Polyester products such as Trivera Spunbond or Bidim filter cloths are the most stable (and most expensive) product with regard to corrosion resistance and inherent resistance to ultraviolet radiation breakdown.

High-density polyethylene (HDPE) is utilized in a wide array of products, such as flex-wall pipes, Geogrids, Geoweb cells, impermeable membranes, and erosion-control mats. All of those products are impregnated with carbon-black to help retard ultraviolet breakdown.

Fiberglass–nylon–rayon roving fibers are beginning to be used in soil reinforcement. These can be of continuous strands (roving) or discontinuous strips. Fibers can be crimped or smooth. They are generally mixed with cohesionless granular fill materials such as sand and decomposed granite.

***Applications.*** In the following pages, section views of various geosynthetics and slope facing elements are presented. The remaining figures present case histories of applications that I and others have used to mitigate slope instability.

### *Mechanically stabilized embankments*

Beginning in the early 1960s, the French architect Henri Vidal proposed reinforcing beach sand with pine needles to increase its bearing capacity, and patented the idea of Reinforced Earth. The concept is similar in precept to what occurs naturally with tree roots (see Fig. 10). By providing some form of tensile reinforcement, the sand was engendered with some degree of cohesion, thereby enabling it to support greater loads., The reinforced Earth concept spread to the United States by 1969, when a 50-ft-high (~15 m) vertical Reinforced Earth® wall was constructed on California Route 39 near Islip Saddle in the Angeles National Forest.

Other Reinforced Earth, and VSL Corporation's competing Retained Earth, systems were utilized mostly on highways through the 1970s. The California and Georgia Departments of Transportation developed their own similar retention systems. In the early 1980s, Netlon Corporation of Great Britain introduced Tensar Geogrid, a high-density polyethylene (HDPE) grid im-

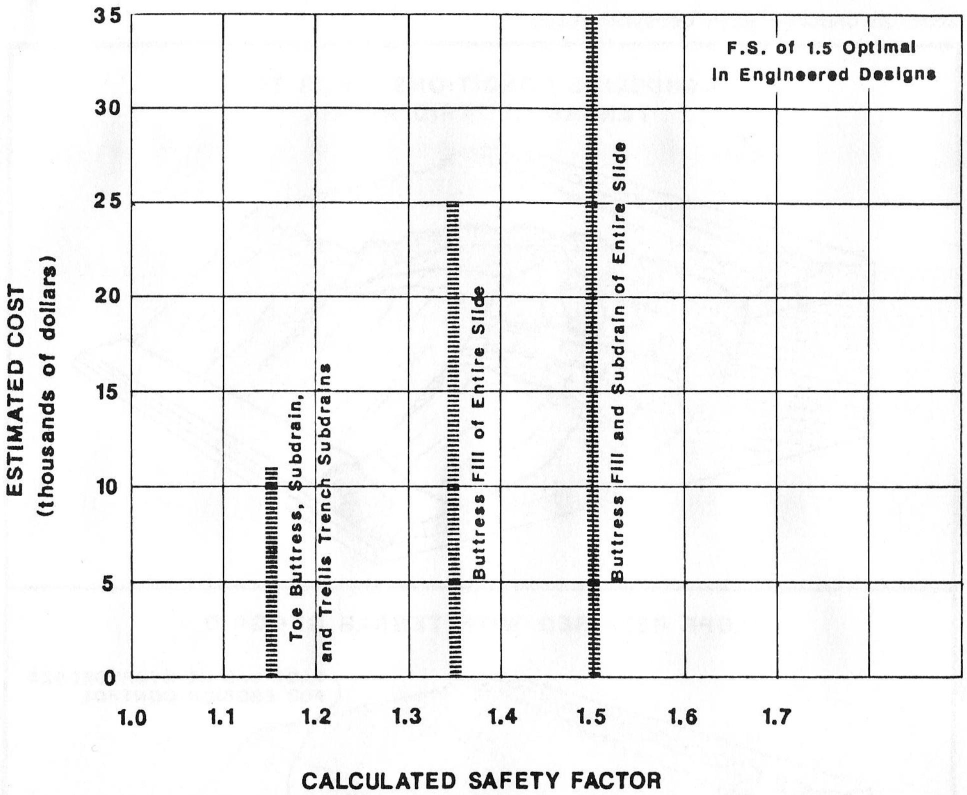


Figure 9. Comparison of estimated cost versus safety factor for three styles of landslide repair (in 1984 U.S. dollars). In this case, the partial toe buttress and trench subdrain alternative cost the least, but also offered the lowest safety factor. If the consequences of future failure are not deemed to be excessive, trench subdrains can be very cost effective.

Figure 10. Nature's concept of soil reinforcement is shown to good effect in the root network of a banyan tree, here stabilizing a nearly vertical cut in colluvium on a National Park Service trail at Diamond Head, Oahu, Hawaii. Water percolating through the colluvium serves to propagate the expanding root system. The new technologies of micropiles, soil nailing, fiber-reinforced soil, and bioengineering emanate from this natural example. In terms of frictional contact area, such tropical trees are easily capable of engendering 10,000 psf increased shear strength to the soil.

pregnated with carbon-black. The grid is manufactured by heating and stretching thick perforated stock. Geogrids come in a variety of sizes, depending on the level of intended loading once buried in the ground. These soil-reinforcement grids work on the same principle as the Reinforced Earth and Retained Earth concepts; that of providing tensile reinforcement through frictional contact with the surrounding soil. The basic concept of entraining soil-reinforcement grids in landslide stabilization is presented in Figure 11.

Soil-reinforcing grids serve to increase the unit shear strength of any soil in which they are emplaced, thereby offering much higher long-term factors of safety than are possible through simple compaction. This is because no matter how intense the original compactive effort, soil density is eventually lessened through saturation, swell, and creep, factors that occur over many years. In situ soil reinforcement also allows the designer to vary the steepness of the slope's finished face, allowing vertical faces when necessary.

Soil reinforcement can also allow compaction activities to proceed through wet weather periods, when traditional levels of compactive effort are not achievable. In wintertime slide emergencies, saturated soils involved in sliding can be mucked out and

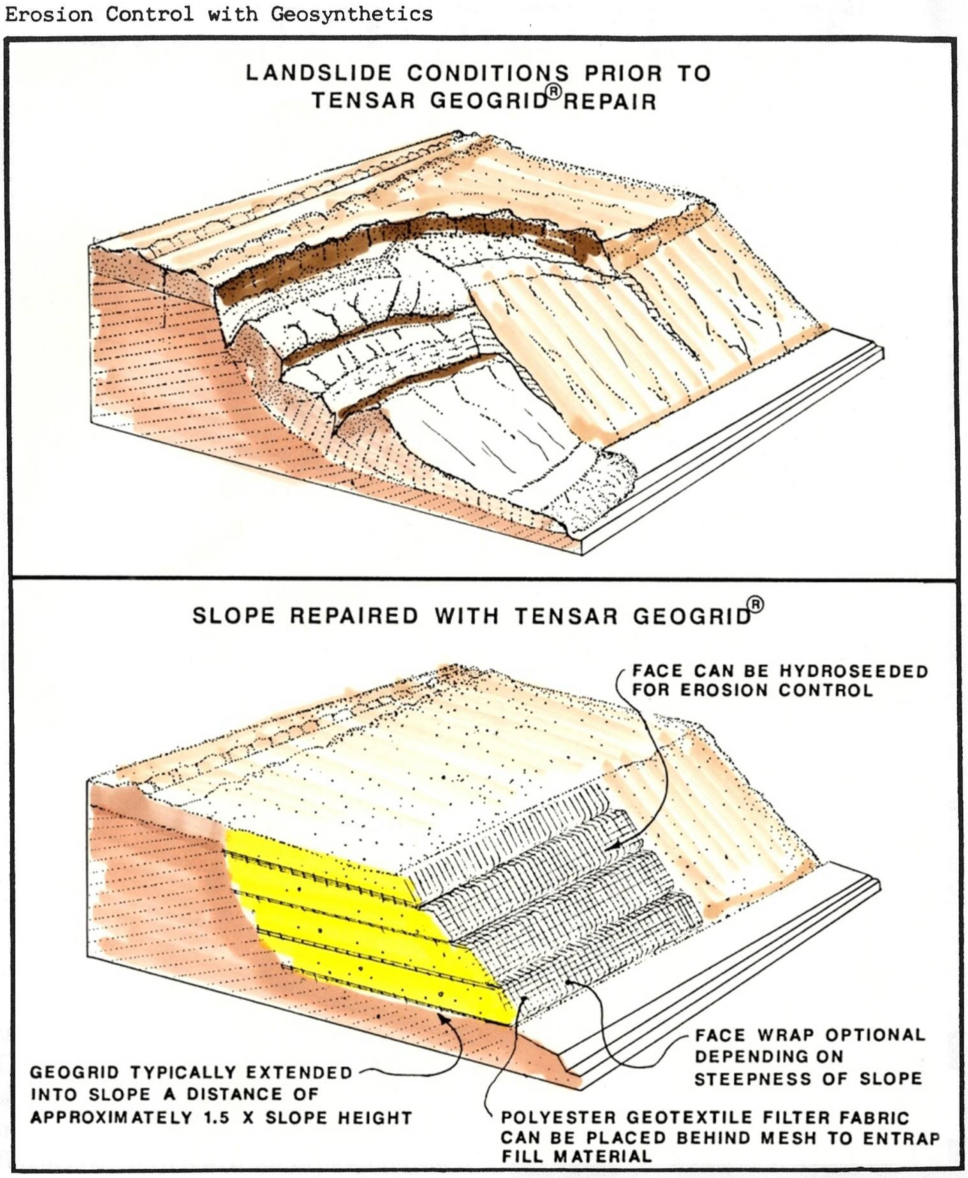


Figure 11. Schematic representation of the basic tenants of a Geogrid reinforcement repair scheme. A wide array of grid strengths is now available, as are competitive products manufactured by Tenax and Nicolon. Embedment lengths generally vary from 1 to 1.5 times the embankment height.

replaced with either drier soils or free-draining gravel as buttress fill material with benefit of the in situ soil reinforcement. With gravels, soil reinforcement allows steeper, conforming slopes to be constructed in the worst of environmental conditions.

Reinforcing grids are generally placed at lift separations of 2 to 4 ft (~0.6–1.2 m), as shown in Figure 12. Face wrapping of the grids is an option. The upper photo in Figure 12 shows a mechanically stabilized embankment under construction with a vertical face wrapping; the lower photo shows a slope under construction with a flush inclined face wrapping. The term “mechanically stabilized embankment,” or MSE, was originally coined by the California Department of Transportation to provide a generic name for all of the various proprietary systems. The Federal Highway Administration has since adopted that term to describe generically all in situ soil benefication retention systems. Some typical cross-sections of mechanically stabilized embankments are presented in Figure 13. The wide embankment portrayed in the upper half of Figure 13 is a landslide repair keyed

Figure 12. A, Placement of welded wire mesh soil reinforcement near the face of a vertical embankment structure during construction. Soil is then spread over the mesh and compacted in 6–8-in-thick (~15–20 cm) lifts. The area immediately adjacent to the free face usually requires local compaction with hand-operated vibratory equipment. B, Compaction of fill lifts in a landslide repair between successive layers of Tensar Geogrid with face warps at 4 ft (1.2 m) intervals. The slope face was repaired at a slope of 1.7:1 directly beneath a series of existing structures.

into underlying Cretaceous siltstone and shale. The lower half depicts a rock-cut repair in Miocene sandstone. Note how the length of embedment decreases with increasing slope inclination. This is because the normal force engendering friction to the grids is greater under a steeper slope due to increased overburden and the increased steepness of maximum principal stress trajectories. An additional factor governing calculation of required grip length is competency of the underlying materials. Leshchinsky and Bodeker (1989) described in greater detail how design judgment is incorporated into MSE designs.

In a weathered bedrock cut in fairly competent materials, the failure surface is generally shallow and more planar, often a wedge or slab-type failure. In such materials, weathering, relaxation and/or creep, and near-surface seepage pressures have likely precipitated the failure. Less-weathered bedrock materials generally lie a short distance beneath the failed material: there, the reinforced soil mass may be designed like a gravity retaining wall supporting very low lateral soil pressure (if adequate subdrainage is incorporated into the fill). If the structure possess insufficient capacity to resist sliding or overturning, additional tensile reinforcement of the underlying bedrock may prove both effective and economic.

Face wrapping with reinforcement grids generally provides an excellent mulch surface to resist rill erosion and promote planting. Figure 14 shows before and after photos of a face-wrapped slope taken only eight weeks apart during the winter rainy season. Face wrapping with the grid also helps retard rodent burrowing into the slope. Vegetation serves to protect the grids from ultraviolet degradation and vandalism. Because soil is exposed in grid mesh MSEs, volunteer vegetation will generally take hold on the slope regardless of initial landscaping efforts. In all cases, vegetative cover serves to provide an aesthetic surface which has the double benefit of reducing erosion.

Face wrapping the grids can also serve to create steeply inclined supporting structures such as toe buttresses and retaining walls, as shown in Figure 15. Toe buttress support capacity is greatly increased by the inclusion of reinforcement grids, because the embankment will act as a massive reinforced wall with built-in subdrainage. Face-wrapped embankments have special application to failed bedrock cut slopes, as presented in Figure 16 (the design section presented in the lower half of Fig. 13). Face wrapping is most effective in limiting subsequent erosion of the repaired slope, desired along highways and creek channels.

An alternative to face-wrapping grids is to place false layers of grid at 12 in (~30.5 cm) intervals adjacent to the embankment face (Fig. 17). These false layers usually extend only 3 to 5 ft (~0.9–1.5 m) into the embankment while conventional full-length grid layers are interspersed at 2 to 5 ft (~0.6–1.5 m) spacings. False layers effectively reduce the exposed slope height to 12 in (~30.5 cm) by providing a nonerodable, free-draining boundary that interrupts run-off velocity. Minor surface sloughage of the top few inches of the embankment is necessary to retard run-off–induced erosion. Vegetative cover then provides a sort of protective mat to retard raindrop spatter and provide a more tortuous path for overland flow. Although soil-reinforcing grids are not combustible, they can melt in brush or range fires. In such instances, exposed portions of the grid may melt, taking at the most a few inches of surface reinforcement off the slope. False layer or nonface wrap slopes can be constructed as steeply as 45° (1:1; horizontal to vertical) and are now the normal procedure for slopes of 1.5:1 or flatter inclination. Some representative exam-

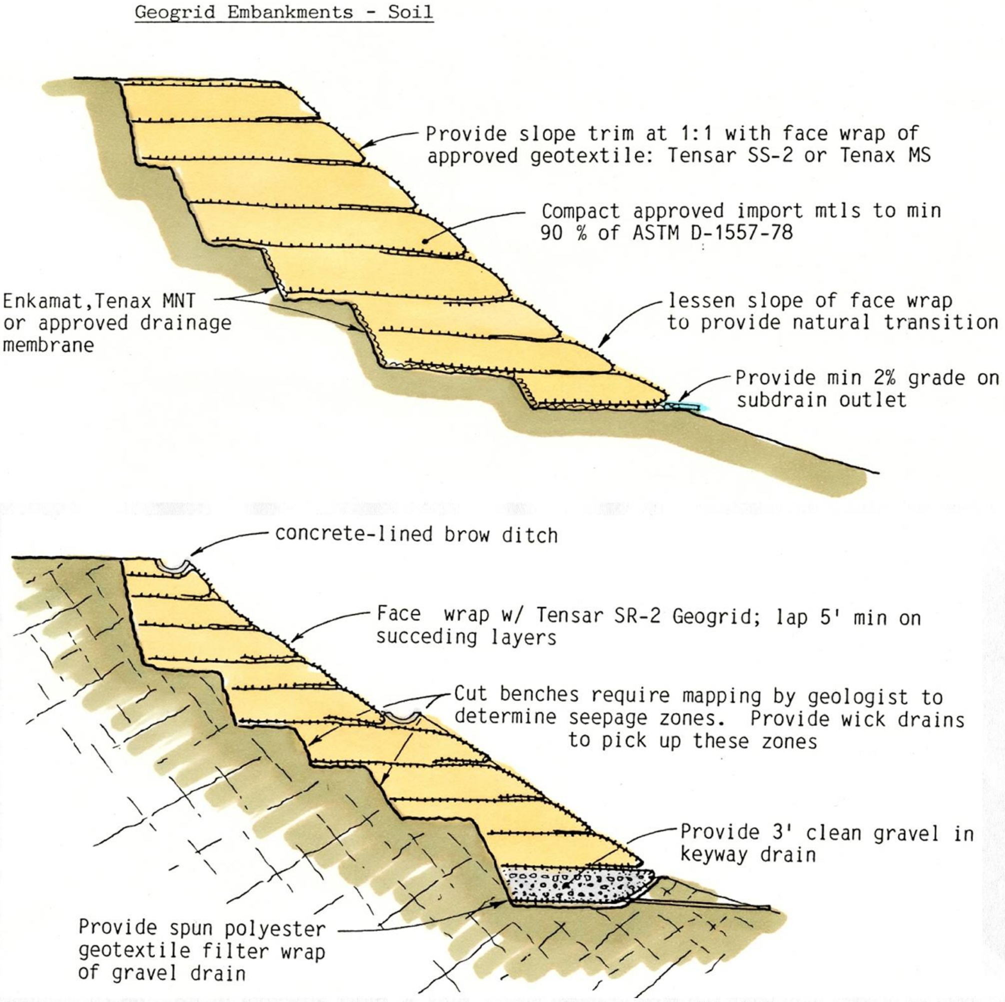


Figure 13. Typical section views of Geogrid embankment repairs accomplished on soil (top) and rock (bottom) slopes with inclinations of around 1.5:1. The employment of prefabricated drainage membranes at the heel of keyways helps to speed up jobs with steep grades and tight working areas.

ples of non-face wrap embankments are presented in Figures 18 and 19 (the slope shown in Fig. 19 suffered a brush fire four years after it was constructed).

### *Combination mechanically stabilized retention structures*

As with every successful invention, a number of competing MSE systems are now available to the consumer: it is hoped that this competition will also promote some lowering of unit material prices. Soil-reinforcing grids can be mixed with any number of facing elements to provide a myriad of structure types and styles, such as gabions, wire mesh, masonry blocks, gunite over geotextile fabric, and precast concrete panels.

Figure 20 presents an example of a rock-filled, gabion-faced, mechanically stabilized embankment. This style of combination structure seeks to combine the better attributes of each support system. Gabions are free draining and extremely flexible; their as-built shape is more nondeforming with time, they provide greater roughness at high flow (thereby keeping erosive scour velocities low), and they are noneroding. Their single drawback is in the cost of imported rockfill and the labor costs associated with rock placement within the gabion baskets. By emplacing a Geogrid-reinforced embankment behind the gabion facing, on-

Figure 14. Two views of a Geogrid-reinforced landslide repair accomplished on an emergency basis between October and December 1986 in Crockett, California. The lower view shows the finished slope six weeks after completion in late January 1987. The slope had been hydroseeded and Geogrid face wrap acts as an excellent mulch mat.

Figure 15. A, As-built view of a Geogrid gravity retention structure constructed using native gravelly colluvium. Total wall height is ~10 ft (~3 m) with a grip distance of 10 ft. B, As-built view of a vertical Geogrid retention structure constructed on a 20% grade to support a roadway crossing of a colluvial-filled swale. Tensar SS-2 geogrid was utilized with crushed gravel at the outside face to act as a protection against fire, rodent activity, and to provide a reaction surface for compaction near the exposed face.

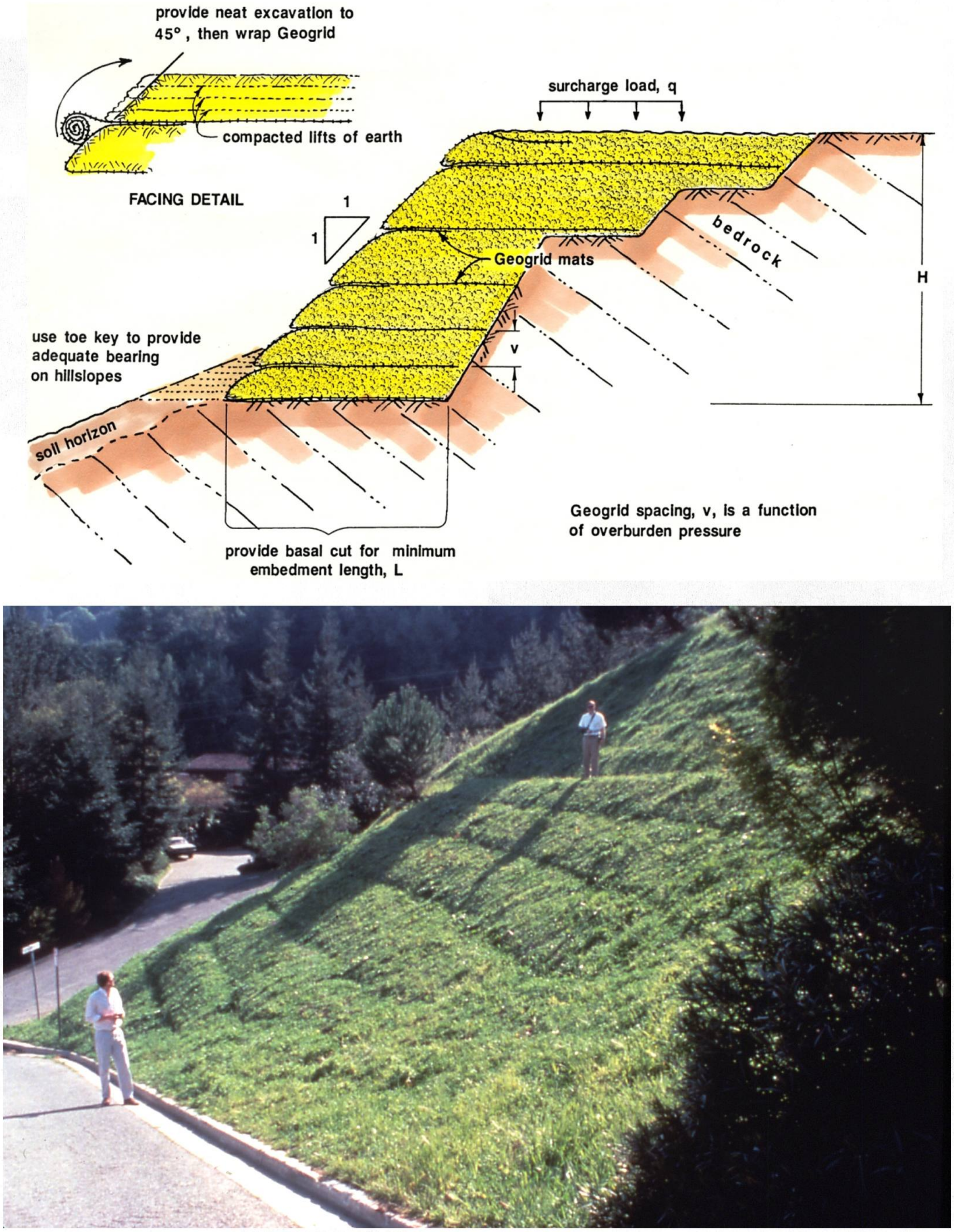
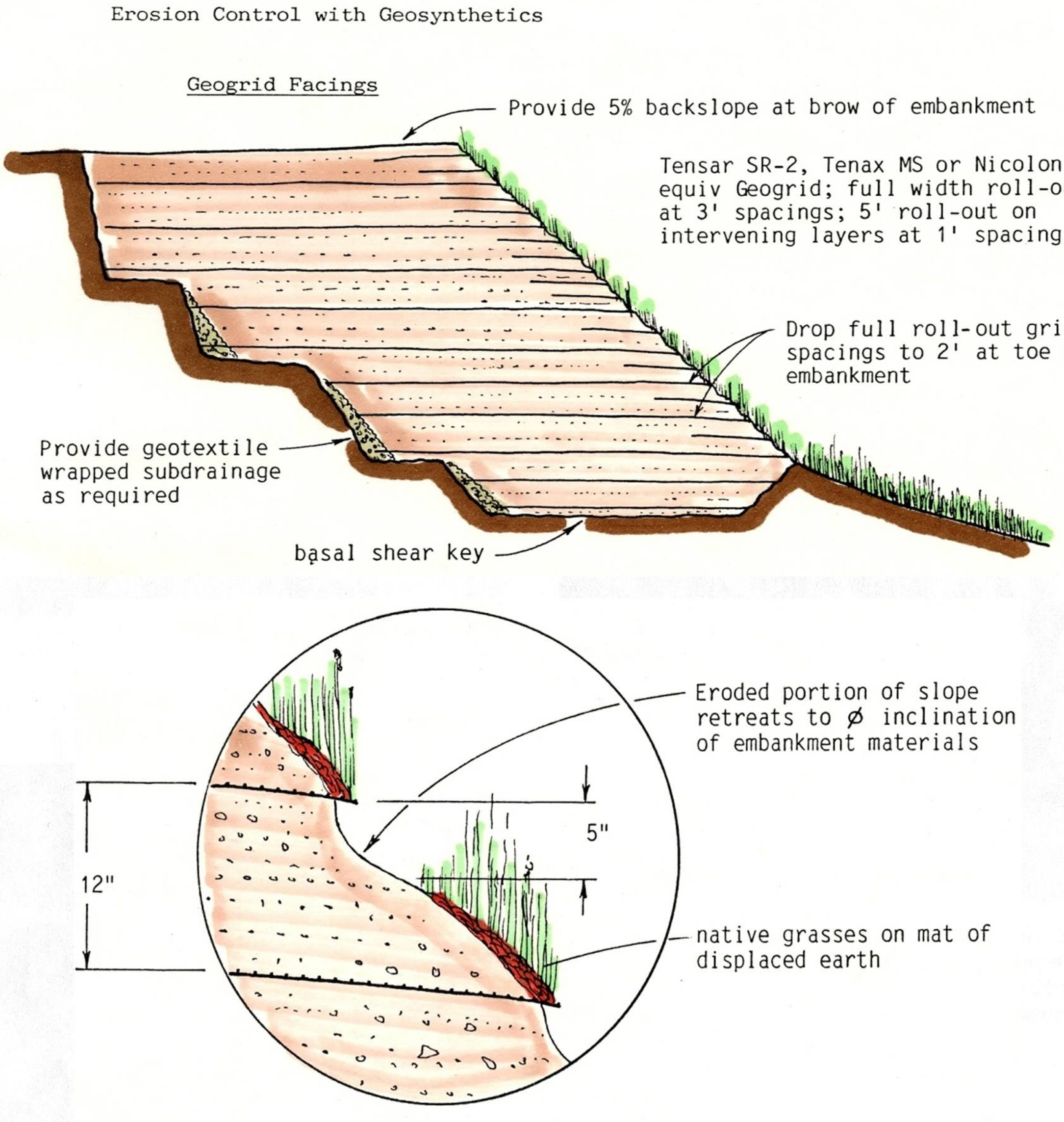


Figure 16. A, Typical face wrap detail on a Geogrid reinforced embankment steeper than 1:1. The grid provides an excellent mulch for hydroseeding, much like jute mesh. B, As-built view of a face-wrapped Geogrid repair on a 60-ft-high (~18 m) cut slope failure that involved bedrock exposed in an old 1:1 road cut. The repair of such steep slopes cannot be accomplished with traditional methods of removal and recompaction. Note excellent cover of hydroseeded grass.

site materials could then be utilized, handled, and compacted by labor-saving mechanical means.

In the late 1980s many small masonry block support systems became available in the United States. The more common of these include Keystone, Earthstone, and Loffelblock. Some representative examples of the Loffelblock type are presented in Figure 21. These interlocking blocks are basically intended to support clayey slopes of less than 5 to 6 ft (~1.5–1.8 m) high or bedrock cut slopes up to 22 ft (~6 m) high. The block systems are generally designed utilizing a 1:4 to 1:3 backward batter to reduce



Detail view of the erosion which can be expected to occur between Geogrid layers. The effective slope height is reduced to 12" by embedment of the Geogrid.

Figure 17. As an alternative to face wrapping, intervening layers of Geogrid can be placed at the slope face to create effective slope heights on the order of 1 ft (~0.3 m).

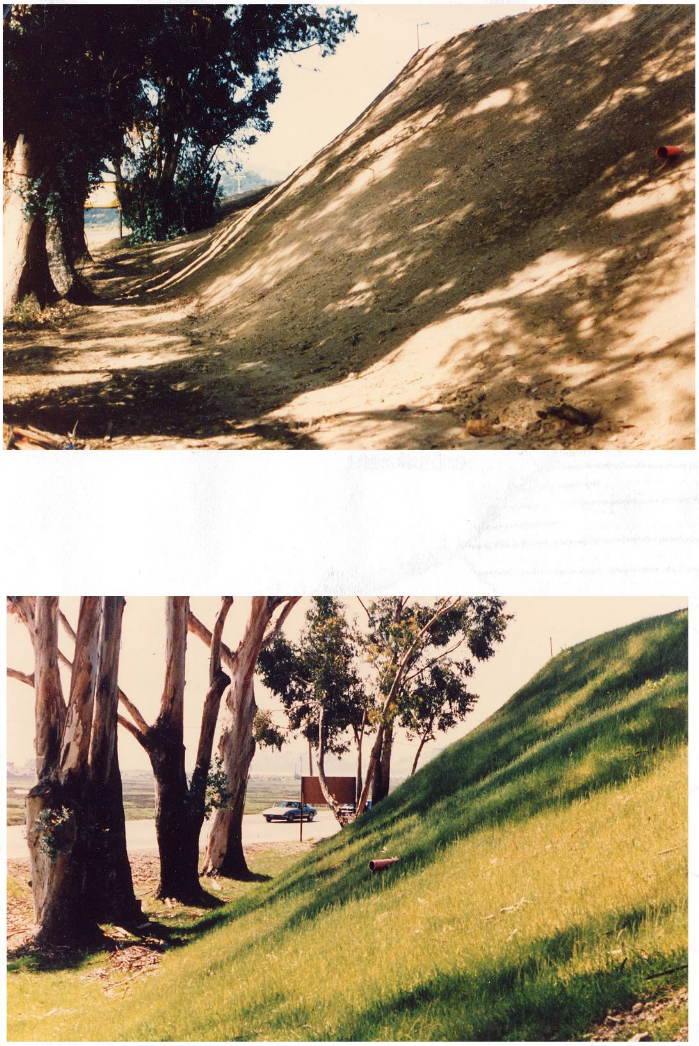


Figure 18. Two views of a 1:1 Geogrid-reinforced fill embankment constructed without benefit of face wrapping, but with short-face grid layers spaced at 12 in (~30.5 cm). The lower view shows the effects of hydroseeding several months later.

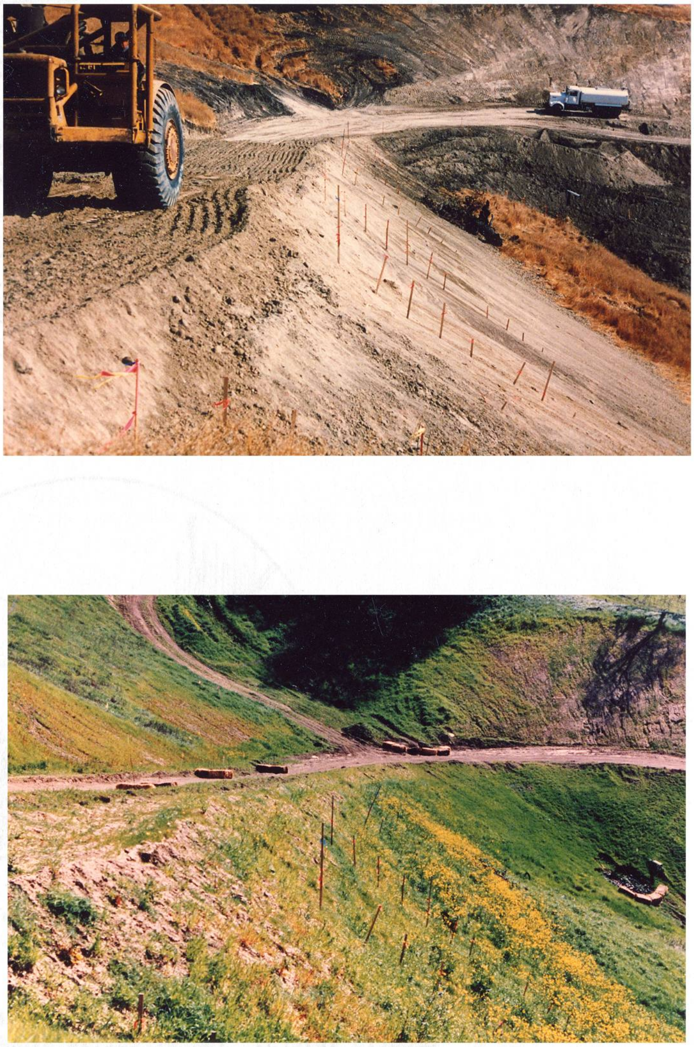


Figure 19. Two views of a 1:1 Geogrid-reinforced roadway embankment with short-face grid layers spaced at 12 in (~30.5 cm). Note the natural blending of the fill with the surrounding slopes, an aesthetic feature of soil-reinforced structures.

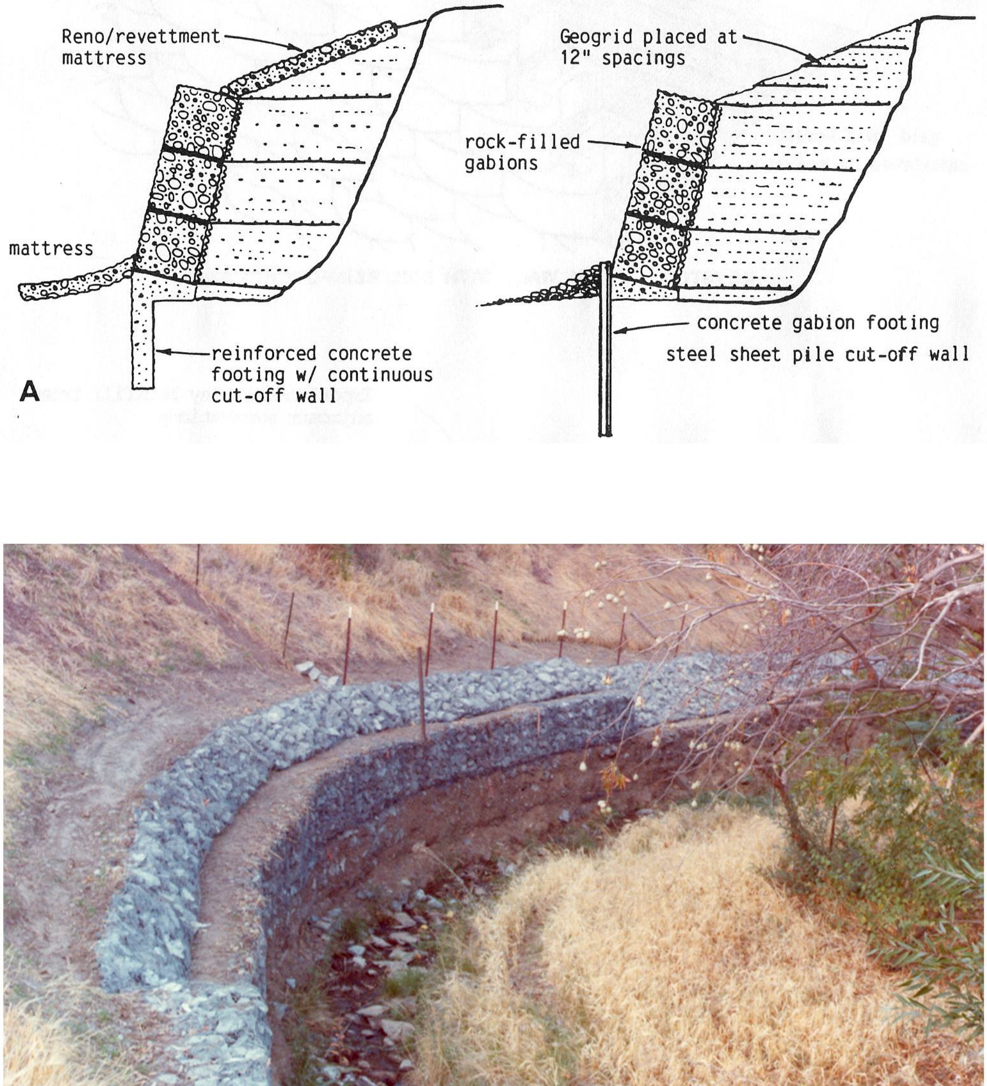
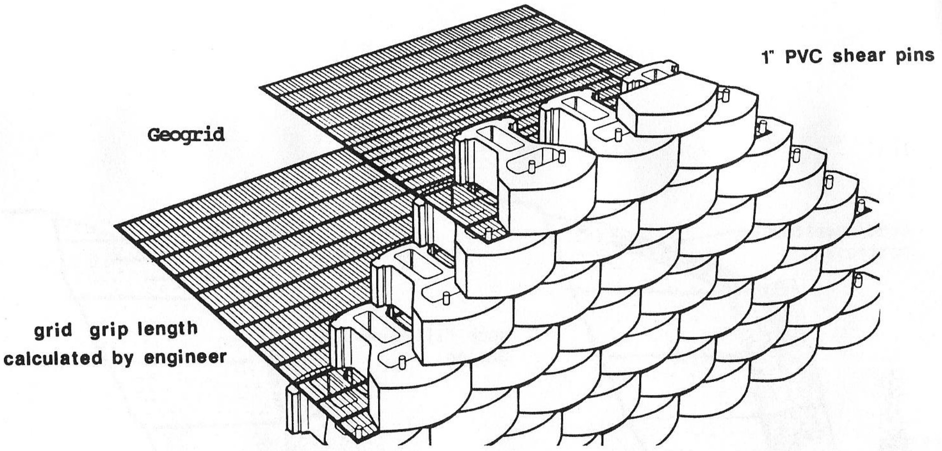
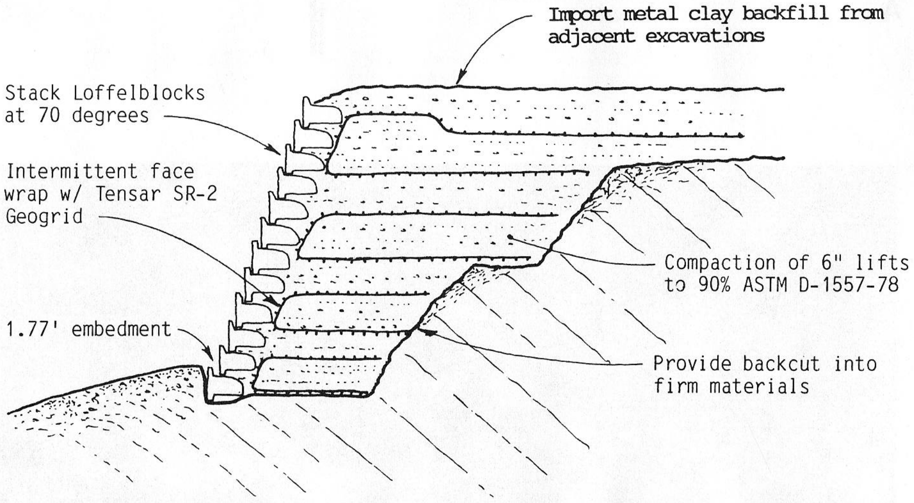


Figure 20. A, Schematic section view of gabion-faced Geogrid embankments constructed as part of a bank repair along Alhambra Creek in Martinez, California. The use of soil backfill lessened off-haul costs for excavation and negated two-thirds of the required rock fill import necessary to fill a conventional all-gabion retention structure. B, Photograph of the completed channel repair, looking downstream. Wall height is 9 to 12 ft (~2.7–3.6 m) with a 6-ft-deep (~1.8 m) footing to protect against undercutting.



KEYSTONE BLOCK WALL WITH SOIL REINFORCING GRID



LOFFELBLOCK WALL FACING ON GEOGRID-REINFORCED EMBANKMENT

Figure 21. Schematic views of Loffelblock and Keystone combination block with Geogrids.

active earth pressures acting on the blocks. Because of their thin width (generally under 24 in [~61 cm]), the resultant thrust of such walls can easily be drawn outside the middle third of the wall's footing. As a consequence, these walls are usually constructed with a reinforced concrete footing, having the basal course of blocks wet set in the concrete at the proper batter. By themselves, these blocks are most effective in facing fairly competent bedrock cut slopes (materials with an angle of internal friction greater than 35°).

Keystone makes use of 1 in (2.54 cm) diameter PVC shear dowels between the blocks, which can also be utilized to attach soil grid–reinforcement mats, similar to the example presented in Figure 21. The precast block is utilized as a facing element for the geogrid-reinforced embankment. It must be remembered that in mixing these products, they have very dissimilar stiffnesses. The reinforced soil must strain some noticeable amount to develop shear strength along the Geogrid-soil interfaces. If the abutting wall facing is not constructed with sufficient flexibility, individual blocks may crack in shear as the soil they are restraining flexes outward.

In the early 1980s, Hilfiker Corporation of Eureka, California, introduced the welded wire mesh wall, or Hilfixer mechanically stabilized embankment support system. Examples of this system are presented in Figure 22. Like gabions, this system can

Figure 22. A, Earthen lift being spread over welded wire mesh. In lifts about 2 ft (~0.6 m) high, a mechanically stabilized highway embankment with vertical face was constructed across an active landslide area. B, As-built view of a welded wire mesh embankment constructed parallel to the road's 10% grade. The face may deform outward as the fill consolidated with time, especially when wetted.

Figure 23. A, Construction view showing emplacement of welded wire mesh facing elements being used with Geogrid soil reinforcement to construct a 0.5:1 embankment slope below a proposed office complex. Welded wire mesh retention systems have been promoted for some years by Hilfiker Corporation in California and Pacific Wire in Tacoma, Washington. Such systems are employed by the same theories applicable to Geogrids and geomembranes. However, Geogrids can be less expensive to purchase and place because they do not require extensive tying. B, As-built view of the completed embankment of 11,000 yd$^3$ (~8360 m$^3$) beneath the Crest Office Park complex, Martinez, California.

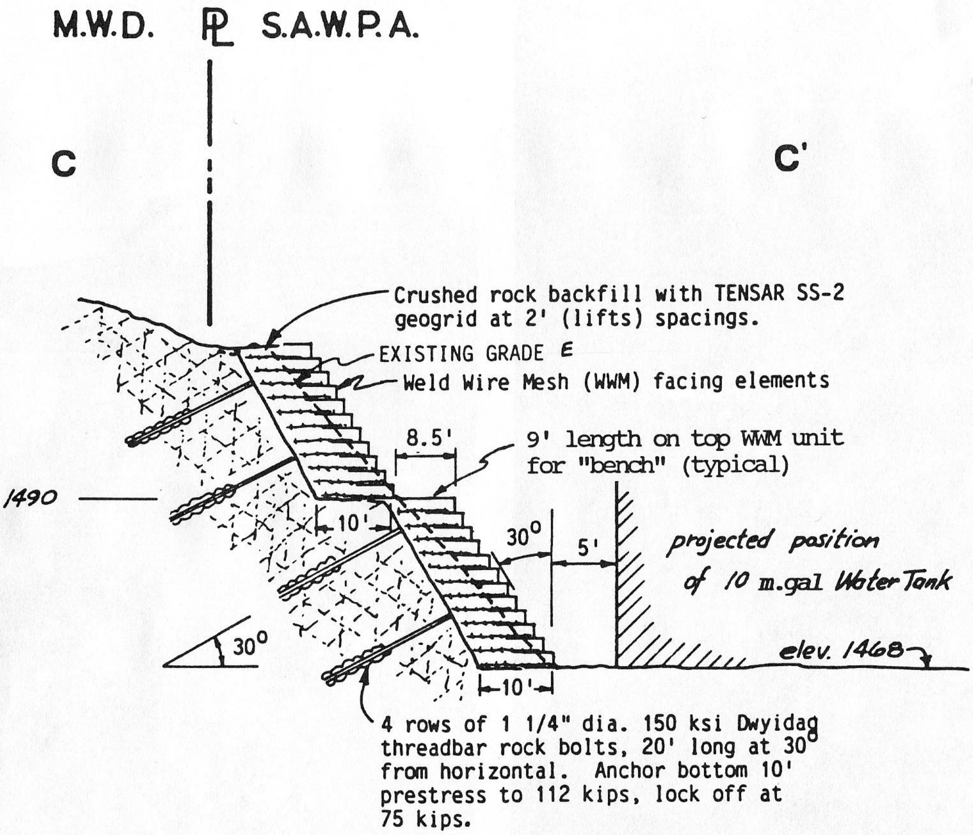


Figure 24. Schematic section view of a combination structure employed to repair a 60-ft-high (~18 m) cut slope failure in granite. The original cut slope was made at 0.5:1, greatly limiting repair measures. A neat excavation was made into the repair area by using presplit drill lines with conventional burden blasting. The burden was then mucked and crushed to a –6 in size. With 2 × 4 ft (~6 × 1.2 m) bent-L welded wire sections, a gravity structure was concurrently constructed from two starting levels, one at the slope base, the other at mid-height. Crushed rock was placed behind the wire facing and reinforced at 2 ft (~6 m) intervals with Geogrid. At four levels, prestressed rock bolts 20 ft (~6 m) long on 20 ft centers were installed to reinforce the broken rock face and reduce the required wall width from 25 ft (~7.5 m) down to 9 ft (~2.7 m) (by tying the active pressure zone into compression). The repair was effected in the winter: rainfall had no effect on rock compaction or excavation activities. The project was located at Lake Matthews, near Riverside, California.

be utilized as a facing element for soil grid–reinforcement products. Wire-mesh facing possesses a number of favorable attributes: being very light, a lot of product can be shipped to a job on a single truck and easily handled; it is basically fireproof and corrosion resistant (if the FHWA 2 oz ft$^2$ galvinizing specification is maintained), the mesh sets up easily, and, by stipulating offsets between lifts (commonly 2 ft [~0.6 m]; see upper half of Fig. 22A), any slope inclination desired can be easily constructed. Wire-mesh facing looks neat when finished and is extremely easy to landscape. An example project is presented in Figure 23.

Wire mesh–faced walls can also be utilized in steep, inaccessible terrain, as shown in Figure 24. In this case, a 70-ft-high (~21 m) rock slope cut at 0.5:1 in granodiorite had begun to undergo a toppling failure. Very little access room was available to effect a repair, and a property line existed just above the crown of the cut. A neat excavation was made into the repair area by utilizing presplit drill lines with light, outward burden blasts. The burden was then mucked and crushed on site to –6 in (~15.25 cm) size. Using 2 × 4 ft (~0.6 × 1.2 m) bent-L sections of welded wire mesh (see Fig. 25), gravity structures were concurrently constructed from two levels; one at the slope base, the other at mid-height (working down from the top). Crushed rock was placed behind the wire facing and reinforced at 2 ft (~0.6 m) intervals with geogrid. At four levels prestressed rockbolts 20 ft (~6 m) long on ~20 ft centers were installed to reinforce the exposed rock face and reduce the required wall width from 25 ft (~7.5 m) down to 9 ft (~2.7 m) (by tying the active pressure zone into compression). The repair could have been effected in winter months, because rainfall would have no effect on compaction of previous rock backfill.

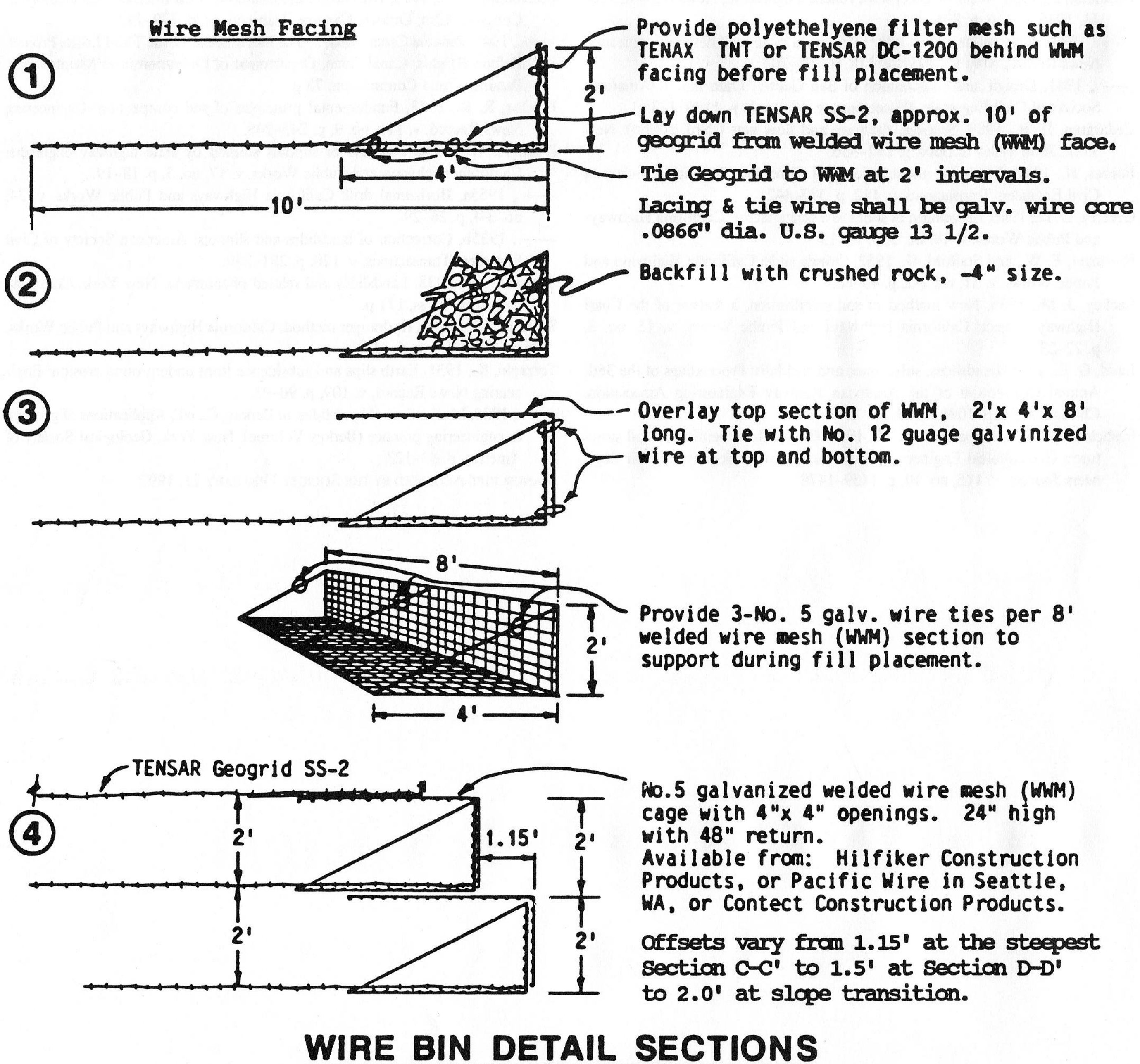


Figure 25. Placement details for utilizing welded wire mesh as facing elements for Geogrid-reinforced embankments or retention structures. In mountainous areas, the welded wire mesh has the advantage of being fireproof over the Geogrid (which won't burn, but will melt). The 2 × 4 ft (~0.6 × 1.2 m) welded wire L-sections are widely available in the western United States. A single or double overlay, as shown here, can be used. Prefabricated filter mats or polyester filter cloths can be employed behind the mesh to inhibit piping. Polypropalene filter cloth products are not employable because they degrade quickly under ultraviolet radiation.

## REFERENCES CITED

Baumann, P., 1936, Wear on sheepsfoot rollers: Engineering News Record, June 11, 1936, v. 116, p. 835–836.

——, 1937, Wear on sheepsfoot rollers decreased by revised design: Engineering News Record, May 13, 1937, v. 118, p. 707–708.

——, 1941, Design and construction of San Gabriel Dam No. 1: American Society of Civil Engineers, Proceedings, v. 67, no. 7, p. 1199–1238.

Cedargren, H. R., 1989, Seepage, drainage and flow nets (third edition): New York, John Wiley & Sons, p. 292–336.

Forbes, H., 1947, Landslide investigation and correction: American Society of Civil Engineers Transactions, v. 112, p. 377–442.

Greeley, D. H., 1940, Prevention of slides as a safety factor: California Highways and Public Works, v. 19, no. 5, p. 13–15.

Herlinger, E. W., and Stafford, G., 1952, Orinda slide: California Highways and Public Works, v. 31, no. 1-2, p. 45–60.

Lackey, J. M., 1936, New method of soil stabilization, a feature of the Coast Highway Project: California Highways and Public Works, v. 15, no. 3, p. 22–23.

Ladd, G. E., 1935, Landslides, subsidence and rockfalls: Proceedings of the 36th Annual Convention of the American Railway Engineering Association, Chicago, v. 36, p. 1091–1163.

Leshchinsky, D., and Boedeker, R. H., 1989, Geosynthetic reinforced soil structures: Geotechnical Engineering Division American Society of Civil Engineers Journal, v. 115, no. 10, p. 1459–1478.

McDonald, D. F., 1913, Excavation deformations: 12th International Geological Congress, 12th, Ottawa, Canada, Transactions, p. 779–792.

——, 1947, Panama Canal slides, *in* The Panama Canal, the Third Locks Project: Balboa Heights, Canal Zone, Department of Operations and Maintenance, Panama Canal Commission, 73 p.

Proctor, R. R., 1933, Fundamental principles of soil compaction: Engineering News Record, v. 111, no. 9, p. 245–248.

Root, A. W., 1938, Problem of slipouts studied by state highway engineers: California Highways and Public Works, v. 17, no. 3, p. 18–19.

——, 1955a, Horizontal drill: California Highways and Public Works, v. 34, no. 3-4, p. 26–29.

——, 1955b, Correction of landslides and slipouts: American Society of Civil Engineers Transactions, v. 120, p. 281–289.

Sharpe, C. F., 1938, Landslides and related phenomena: New York, Columbia University Press, 171 p.

Stanton, T. E., 1948, Hydrauger method: California Highways and Public Works, v. 27, no. 1-2, p. 6–10.

Terzaghi, K., 1931, Earth slips and subsidence from underground erosion: Engineering News Record, v. 109, p. 90–93.

——, 1950, Mechanism of landslides, *in* Berkey, C., ed., Applications of geology to engineering practice (Berkey Volume): New York, Geological Society of America, p. 83–123.

MANUSCRIPT ACCEPTED BY THE SOCIETY FEBRUARY 21, 1992

Printed in U.S.A.