Landslides, Bell (ed.) © 1995 Balkema, Rotterdam, ISBN 90 5410 032 X

Keynote paper: Recent advances in slope stabilization

Robert L. Schuster
US Geological Survey, Denver, Colo., USA

ABSTRACT: This paper reviews physical means of slope stabilization, particularly subsurface drainage and rock/soil control and retention systems, which have been improved and technically advanced in recent years because of innovative changes in design, analysis, and construction methods. The subjects dealt with in greatest detail are drainage, control of rockfall hazards, and the stabilization of soil slopes by means of geosynthetic retention systems. Several cases are presented in which new slope-stabilization techniques have been applied.

1 INTRODUCTION

The term "landslide" traditionally has been defined as the downward and outward movement of slope-forming materials: rock, soils, artificial fills, or combinations of these materials (Varnes 1958). In theory, landslides comprise the group of slope movements wherein shear failure occurs along a specific surface or combination of surfaces; thus, strictly speaking, it does not apply to all types of slope movements (Varnes 1978). However, because of the common usage of "landslides," particularly in civil engineering literature, the author has decided to follow common practice and to use landslides as the collective term for all types of slope movements in soil or rock.

Effective landslide hazard management has done much to reduce economic and social losses due to slope failure by avoiding the hazards or by reducing the damage potential. This has been achieved primarily by four mitigative approaches: (a) restriction of development in landslide-prone areas, (b) evolvement and application of excavating, grading, landscaping, and construction codes, (c) use of physical measures to prevent or control slope failures, and (d) landslide warning systems. The third of these methods, physical measures to prevent or control the gravitational and/or dynamic failure of slopes, is known as <u>slope stabilization</u>.

The use of crude slope-stabilization techniques, such as surface drainage and simple retaining walls, predates written history. However, some techniques for implementing these methods are relatively new or have undergone considerable upgrading in the past few years in terms of technology, lower costs, or increased harmony with the environment. This paper will discuss these relatively recent advances. The use of product, trade, proprietary, and company names is for clarity of expression; it does not imply endorsement or superiority of these specific procedures or of the equipment used.

2 CATEGORIES OF SLOPE STABILIZATION

The most commonly used physical approaches for control of unstable slopes are:

(a) <u>Drainage</u> -- Because of its high stabilization efficiency in relation to cost, drainage of surface water and groundwater is the most widely used, and generally the most successful, slope-stabilization method. Underground drainage systems and pumping wells collect and remove groundwater; surface water is diverted from unstable slopes by ditches.

(b) <u>Slope modification</u> -- Increased slope stability can be obtained by removing all or part of the landslide mass.

(c) <u>Earth buttresses</u> -- Earth buttress counterforts placed at the toes of unstable slopes often are successful in preventing failure. In many nations, this is the most common mechanical (as contrasted to hydrologic) method of landslide control.

(d) <u>Earth retention systems</u> -- Where methods (a) to (c) will not ensure slope stability by themselves, structural controls, such as retaining walls, piles, caissons, fences, anchors, or internal reinforcement of the earth materials, are commonly used to prevent or control slope movements. Properly designed retention systems are useful in stabilizing most types of slope failures where these failures do not involve large volumes and where lack of space precludes slope modification.

Earth retention systems often are used in conjunction with drainage, slope modification, and/or construction of earth counterfort berms. Outstanding recent examples of large-scale use of combined remedial measures are the stabilization of slopes associated with Tablachaca Dam in Peru and the Clyde Power Project in New Zealand. In the early 1980's, Tablachaca Dam was endangered by a 3-million-m^3 creeping mass of rock and colluvium on the right abutment (Fig. 1). Deere and Perez (1985) noted that approximately US$40 million was spent by the Peruvian government in landslide stabilization measures consisting of (1) a 460,000 m^3 toe buttress founded on compacted river sediments, (2) 405 prestressed rock anchors, (3) 1,300 m of drainage tunnels, 190 radial drains, and 3,300 m of surface ditches, (4) 68,500 m^3 of rock excavation, (5) improvement of the river-channel flow pattern, and (6) numerous inclinometers, piezometers,

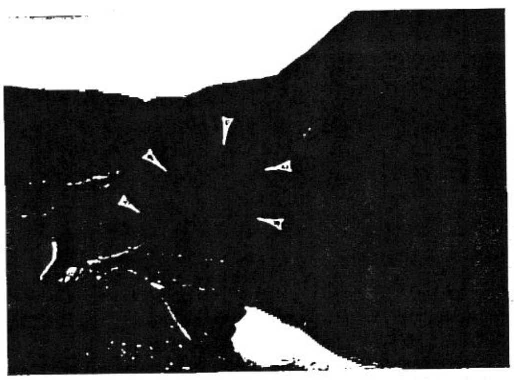


Fig. 1. Creeping rock slide (outlined by arrows) endangering Tablachaca Dam and Reservoir, Mantaro River, Peru. Complex stabilization measures, consisting of a reservoir-level earth buttress, surface and subsurface drainage, and rock anchors, were used to reduce the threat of a catastrophic slope failure to Peru's largest hydropower dam. Photograph was taken in February 1982 before most mitigative measures had been installed.

extensometers, and other instrumentation (Morales Arnao et al. 1984).

The toes of several major actively creeping landslides in the Cromwell Gorge of the Clutha River on the South Island of New Zealand will be inundated by the planned 1992 filling of Lake Dunstan behind Clyde Dam (Gillon & Hancox in press). This 102-m-high concrete gravity dam is now complete, but impoundment of Lake Dunstan has been delayed pending stabilization of several large landslides that threaten the dam and reservoir. The following extensive multiple remedial measures currently are being implemented to offset the effects of reservoir filling on the landslides: (1) gravity drainage, using drainage tunnels, drill holes, and surface drainage systems, (2) free-draining rock- and gravel-fill buttresses at the landslide toes, (3) grout curtains, and (4) pumped drainage.

These types of physical control methods have been discussed at length in the landslide literature (e.g., Baker & Marshall 1958, Veder 1981, Zaruba & Mencl 1982, Hausmann 1990). All are in common use worldwide, and all are continually being improved by modern methods of analysis, design, and construction. However, in the author's opinion the greatest innovations in slope stabilization in recent years, in terms of technology, economy, and environmental improvements, have been in drainage and in rock/soil retention systems; thus, this paper will accentuate recent advances in these stabilization methods.

3 RECENT APPROACHES TO SUBSURFACE DRAINAGE AS A METHOD OF SLOPE STABILIZATION

Subsurface drainage as a method of lowering the groundwater table in an unstable slope has traditionally consisted of one or more of the following technologies: (1) drainage trenches, (2) drainage wells, (3) drainage galleries, tunnels, or adits, (4) subhorizontal (commonly called "horizontal") drains drilled either from the slope surface or from drainage wells or galleries, and (5) subvertical drains drilled upward from drainage galleries. Most often, these systems drain by means of gravity flow; however, pumps are occasionally used to lift water from low-level collector galleries or wells. This section will discuss recent advances in the above types of drainage systems; in addition, it will briefly mention less commonly used, but innovative, means of drainage, such as electroosmotic dewatering, vacuum and siphon drains, geosynthetic drains and filters, and blasting.

3.1 Drainage tren

Trench drains,
materials, have
shallow subsurfa
decades. If th
bedrock, they are
drains. Backhoe-
depths of 5-6 m
depths. For c
diaphragm metho
excavation and
panels (Collotta e
(1987) reviewed
drains in thei
groundwater prc
dams, and natur
study of the effici
carried out by Hu
finite-element me
dimensional stead
followed, using f
three-dimensional
sloping surfaces.
(1987) have consid
flow case based or
An increasingl
vertical drains to
bottom of draina

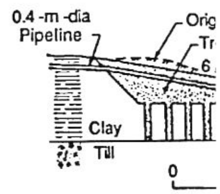


Fig. 2. Trench an
stabilization syste
of the Assiniboine
& Graham 1988).

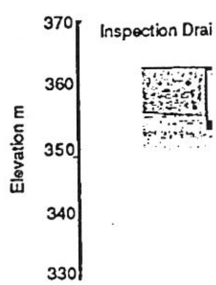


Fig. 3. Vertical d
Motorway, Italy (

methods (a) to (c) will not ensure slope stability by themselves, structural controls, such as retaining walls, piles, caissons, fences, anchors, or internal reinforcement of the earth materials, are commonly used to prevent or control slope movements. Properly designed retention systems are useful in stabilizing most types of slope failures where these failures do not involve large volumes and where lack of space precludes slope modification.

Earth retention systems often are used in conjunction with drainage, slope modification, and/or construction of earth counterfort berms. Outstanding recent examples of large-scale use of combined remedial measures are the stabilization of slopes associated with Tablachaca Dam in Peru and the Clyde Power Project in New Zealand. In the early 1980's, Tablachaca Dam was endangered by a 3-million-m^3 creeping mass of rock and colluvium on the right abutment (Fig. 1). Deere and Perez (1985) noted that approximately US$40 million was spent by the Peruvian government in landslide stabilization measures consisting of (1) a 460,000 m^3 toe buttress founded on compacted river sediments, (2) 405 prestressed rock anchors, (3) 1,300 m of drainage tunnels, 190 radial drains, and 3,300 m of surface ditches, (4) 68,500 m^3 of rock excavation, (5) improvement of the river-channel flow pattern, and (6) numerous inclinometers, piezometers,

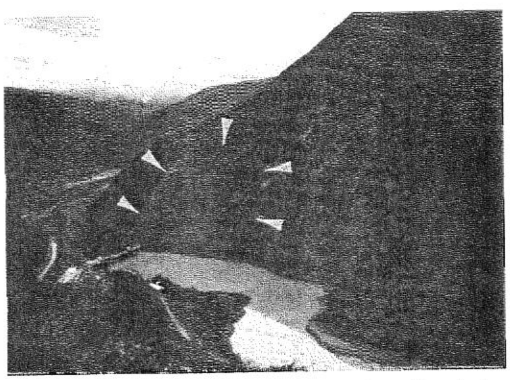


Fig. 1. Creeping rock slide (outlined by arrows) endangering Tablachaca Dam and Reservoir, Mantaro River, Peru. Complex stabilization measures, consisting of a reservoir-level earth buttress, surface and subsurface drainage, and rock anchors, were used to reduce the threat of a catastrophic slope failure to Peru's largest hydropower dam. Photograph was taken in February 1982 before most mitigative measures had been installed.

extensometers, and other instrumentation (Morales Arnao et al. 1984).

The toes of several major actively creeping landslides in the Cromwell Gorge of the Clutha River on the South Island of New Zealand will be inundated by the planned 1992 filling of Lake Dunstan behind Clyde Dam (Gillon & Hancox in press). This 102-m-high concrete gravity dam is now complete, but impoundment of Lake Dunstan has been delayed pending stabilization of several large landslides that threaten the dam and reservoir. The following extensive multiple remedial measures currently are being implemented to offset the effects of reservoir filling on the landslides: (1) gravity drainage, using drainage tunnels, drill holes, and surface drainage systems, (2) free-draining rock- and gravel-fill buttresses at the landslide toes, (3) grout curtains, and (4) pumped drainage.

These types of physical control methods have been discussed at length in the landslide literature (e.g., Baker & Marshall 1958, Veder 1981, Zaruba & Mencl 1982, Hausmann 1990). All are in common use worldwide, and all are continually being improved by modern methods of analysis, design, and construction. However, in the author's opinion the greatest innovations in slope stabilization in recent years, in terms of technology, economy, and environmental improvements, have been in drainage and in rock/soil retention systems; thus, this paper will accentuate recent advances in these stabilization methods.

3 RECENT APPROACHES TO SUBSURFACE DRAINAGE AS A METHOD OF SLOPE STABILIZATION

Subsurface drainage as a method of lowering the groundwater table in an unstable slope has traditionally consisted of one or more of the following technologies: (1) drainage trenches, (2) drainage wells, (3) drainage galleries, tunnels, or adits, (4) subhorizontal (commonly called "horizontal") drains drilled either from the slope surface or from drainage wells or galleries, and (5) subvertical drains drilled upward from drainage galleries. Most often, these systems drain by means of gravity flow; however, pumps are occasionally used to lift water from low-level collector galleries or wells. This section will discuss recent advances in the above types of drainage systems; in addition, it will briefly mention less commonly used, but innovative, means of drainage, such as electroosmotic dewatering, vacuum and siphon drains, geosynthetic drains and filters, and blasting.

3.1 Drainage tren

Trench drains, materials, have shallow subsurfa decades. If th bedrock, they are drains. Backhoe- depths of 5-6 m depths. For c diaphragm metho excavation and panels (Collotta et (1987) reviewed drains in thei groundwater pro dams, and natur study of the effici carried out by Hu finite-element me dimensional stead followed, using f three-dimensional sloping surfaces. (1987) have consid flow case based or

An increasingl vertical drains to bottom of draina

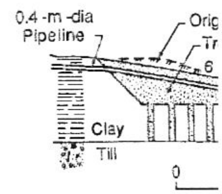


Fig. 2. Trench an stabilization syste of the Assiniboine & Graham 1988).

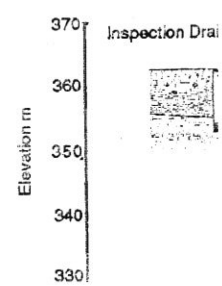


Fig. 3. Vertical d Motorway, Italy (

;trumentation

vely creeping ;orge of the ;land of New ; planned 1992 d Clyde Dam is 102-m-high :omplete, but ın has been several large ıe dam and ısive multiple are being s of reservoir (1) gravity ınnels, drill systems, (2) gravel-fill es, (3) grout nage. ıtrol methods ngth in the :er & Marshall Mencl 1982, common use inually being of analysis, wever, in the nnovations in ars, in terms environmental ainage and in us, this paper ces in these

SUBSURFACE) OF SLOPE

od of lowering ınstable slope ne or more of (1) drainage (3) drainage adits, (4) "horizontal") ope surface or ries, and (5) upward from often, these gravity flow; lly used to lift r galleries or iscuss recent s of drainage riefly mention ıtive, means of ic dewatering, geosynthetic g.

3.1 Drainage trenches

Trench drains, filled with free-draining materials, have been used for effective shallow subsurface drainage for several decades. If they fully penetrate firm bedrock, they are often called "counterfort" drains. Backhoe-type excavators are used to depths of 5-6 m and clamshells for greater depths. For clamshell excavation, the diaphragm method is used, undertaking excavation and concreting in alternating panels (Collotta et al. 1988). Cancelli et al. (1987) reviewed the theory behind trench drains in their excellent paper on groundwater problems in embankments, dams, and natural slopes. A systematic study of the efficiency of trench drains was carried out by Hutchinson (1977) using the finite-element method and assuming two-dimensional steady-state flow. Stanic (1984) followed, using finite-element analysis for three-dimensional flow in trench drains on sloping surfaces. Di Maio and Viggiani (1987) have considered the non-steady-state-flow case based on intermittent rainfall.

An increasingly used technique utilizes vertical drains to remove water from the bottom of drainage trenches. Lew and Graham (1988) described a project along the Assiniboine River in Canada in which compacted sand drains were connected to the base of a drainage trench to stabilize a slope in which a high-pressure gas pipeline was buried (Fig. 2). The design used 0.75-m-diameter augered holes backfilled with sand connecting to trench drains placed on either side of the pipeline.

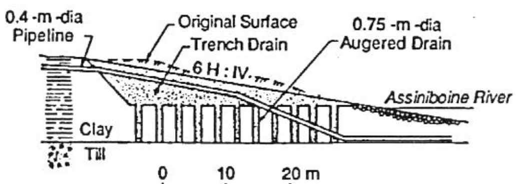


Fig. 2. Trench and augered sand-drain slope stabilization system for gas-pipeline crossing of the Assiniboine River, Canada (after Lew & Graham 1988).

3.2 Drainage wells

Deep wells increasingly are being used to drain unstable slopes, particularly where the depths needed are too deep for economical construction of drainage trenches. Collotta et al. (1988), Bianco & Bruce (1991), Beer et al. (in press), Bianco and Bruce (in press), and Peila et al. (in press) have described the development and use in Italy of large-diameter (up to 2-m) vertical wells (trade name: RODREN) spaced in rows at 5-20 m, center-to-center, and reaching depths of as much as 50 m. Each shaft is connected to its neighbor by a horizontal drill hole, placed just above the base of the well. These holes are lined with 76-to-100-mm PVC pipe and serve as gravity collector drains. Two thirds of the wells are filled with 3-to-20-mm-diameter free-draining material and serve as vertical drains; every third well is kept open for (1) physical inspection and cleaning, and (2) monitoring and adjustment of flow rates. The innovation of this system is that it allows gravity discharge of water from the wells by means of the small-diameter PVC pipes that are installed using mini-drilling rigs placed on the bottoms of the wells. This method has been successfully used to stabilize a slope along the Florence-Bologna Motorway (Fig. 3) (Collotta et al. 1988) and to control the edge of a large landslide in the city of Ancona on the Adriatic Coast (Beer et al. in press).

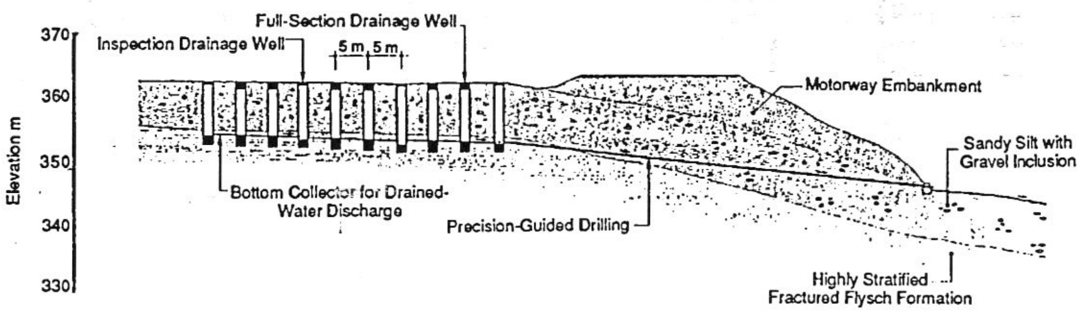


Fig. 3. Vertical drainage shafts connected to a horizontal PVC outlet drains, Florence-Bologna Motorway, Italy (after Collotta et al. 1988).

A similar drainage scheme was used for a slide area encountered in relocation of German Federal Highway B-10 where it crosses the Rhine valley near the village of Albersweiler (Wichter et al. 1988). As the main part of the stabilization scheme, 172 wells (1.5-m-diameter) and 13 maintenance shafts (1.8-m-diameter) were installed across the landslide to depths of 16-25 m. The shafts were filled with filter sand. Unlike the Italian examples noted above, these wells were spaced very closely (3 m, center-to-center); however, the outlet drain from the system required a 170-m-long subhorizontal directional borehole from the foot of the hill to one of the maintenance shafts. The short connecting holes between the wells were drilled using augers driven by compact drill rigs that fit within the casing of the vertical wells.

Gabus èt al. (1988) have described an extensive stabilization effort on the Arveyes landslide in the Swiss Alps in which 16 vertical pumped drains were installed in a 1-km^2 unstable area. The deepest well reached a depth of 95 m, and the piezometric level locally was lowered more than 50 m.

In 1991, "belled" (i.e., bell-shaped) drainage caissons backfilled with fine gravel were used to stabilize a slow-moving slide in a construction cut on Colorado Highway 93 near the city of Golden (Chou, N.N.S. 1991, personal communication, Colorado Department of Transportation, Denver). As shown in Fig. 4, the caissons were closely spaced so that the bells overlapped, allowing gravity drainage along the line of caissons to the ground surface outside the perimeter of the landslide. The use of bells for horizontal drainage precluded the need for horizontal drilling and insertion of drain pipe between the bottoms of the wells.

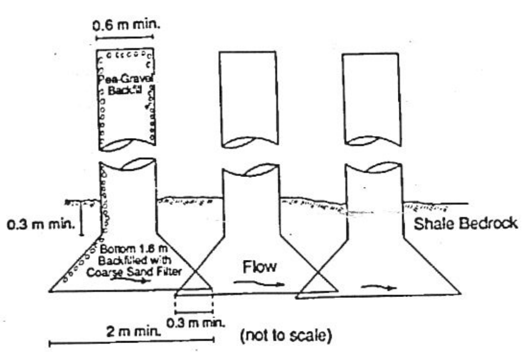


Fig. 4. Overlapping belled drainage caissons for deep drainage of slopes (courtesy of Colorado Department of Transportation).

In most cases, collected water flows from the bottoms of drainage wells or caissons by gravity. However, if the base of the well is low enough, pumping may be needed. Olcese et al. (1991) have reported on a slope-stabilization project near Genoa, Italy, in which submersible motor-driven pumps were used to remove water from the bottoms of small-diameter (200-mm) drainage wells. The pumps were automatically activated when the water surface in the wells reached predesignated levels.

3.3 Drainage tunnels, adits, or galleries

On large slope-stabilization projects, such as those sometimes needed for hydropower projects, deep drainage galleries are being used increasingly. Groundwater is intercepted directly by the galleries and by collection from fan drains drilled from the galleries. The galleries commonly are placed by conventional tunneling techniques. In addition to being drainage collectors, galleries and tunnels provide access for at-depth study of the landslides; they also enable at-depth installation of monitoring equipment, such as piezometers and extensometers.

As is the case for deep drainage wells, most drainage galleries are designed to drain by gravity. In some cases, however, to be effective as collector drains the wells must be placed at sufficiently low elevations that the collected water has to be pumped to the surface.

Classic recent examples of the use of drainage tunnels and galleries are the slope-stabilization projects at the Downie slide on the Revelstoke (power) Project, British Columbia, Canada, and at the above-mentioned Tablachaca Dam in Peru and the Clyde Power Project in New Zealand. To prevent reactivation of the 1.5-billion-m^3 Downie rockslide by filling of the reservoir behind 160-m-high concrete-gravity Revelstoke Dam, deep drainage adits and connecting vertical drains were installed in the lower part of the slide before the reservoir was filled in 1984 (Imrie et al. in press). The drainage works were successful in lowering piezometric levels considerably below where they were before river level was raised 70 m, and thus stabilized the slide. At Tablachaca Dam, 1,300 m of gravity-drained collector tunnels were driven in the valley wall. On the Clyde Power Project, 49 tunnels (total length: 16,392 m) were driven into nine landslides around the reservoir (Gillon & Hancox, in press). Most of these tunnels were situated above reservoir level, and thus

could be drained
stabilize the Bre
which is at low
reservoir level o
necessary to inst
river-level) coll
which water mus
Interesting featu
landslide stabili
earthfill blanke
curtain; both wil
the reservoir th
drainage works (
zoned earthfill b
lining that is m
shoulder, transit
The low-level d
drains will inter
through the blan

3.4 Subhorizonta

Conventional
(henceforth ref
drains," a more
worldwide. For
California more t
drains were us
period 1940-1980
draining unstabl

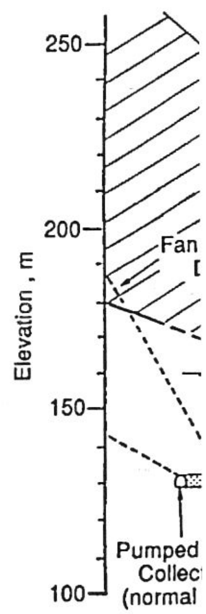


Fig. 5. The Bre
(after Gillon et a

could be drained by gravity. However, to stabilize the Brewery Creek slide, the toe of which is at lower elevation than the pre-reservoir level of the Clutha River, it was necessary to install a low-level (i.e., below-river-level) collector gallery (Fig. 5) from which water must be pumped to the surface. Interesting features of the Brewery Creek landslide stabilization scheme are a zoned earthfill blanket and connected grout curtain; both will act as barriers to flow from the reservoir through the landslide to the drainage works (Gillon et al. in press). The zoned earthfill blanket is in effect a canal lining that is made up of wave-armoring, shoulder, transition, core, and filter zones. The low-level drainage tunnel and its fan drains will intercept any water that passes through the blanket and the grout curtain.

3.4 Subhorizontal drains

Conventional subhorizontal drains (henceforth referred to as "horizontal drains," a more common usage) are used worldwide. For example, in the State of California more than 300,000 m of horizontal drains were used effectively during the period 1940-1980 as an economical method of draining unstable slopes (Smith 1980). Horizontal drains can be inserted from the ground surface or by drilling from drainage galleries, large-diameter wells, or caissons. The typical horizontal drain hole is 120-150 mm in diameter and is lined with slotted plastic casing (commonly PVC) 60-100 mm in diameter. Conventional rotary techniques are adequate for drilling most horizontal drain holes; however, it often is necessary to use precision drilling to connect a horizontal drain to a drainage shaft. In recent years, precision drilling techniques have progressed to where horizontal directional boreholes as long as 200 m have been installed (Sembenelli 1988).

In 1980, the California Department of Transportation conducted a study of the long-term effectiveness of 20 of its horizontal-drain installations (Smith 1980). The main conclusions reached were: (1) a 30-40-year life span is about the maximum that can be expected from perforated metal pipe casing, (2) slotted PVC will provide longer service life than metal pipe casing, (3) slotted PVC casing allows considerably less sediment to enter the drain than the standard U.S. 3/8-inch (9.5-mm) perforated metal pipe, (4) high-pressure water cleaning systems used for cleaning sewers and unplugging culverts can be adapted for horizontal-drain cleaning, and (5) most

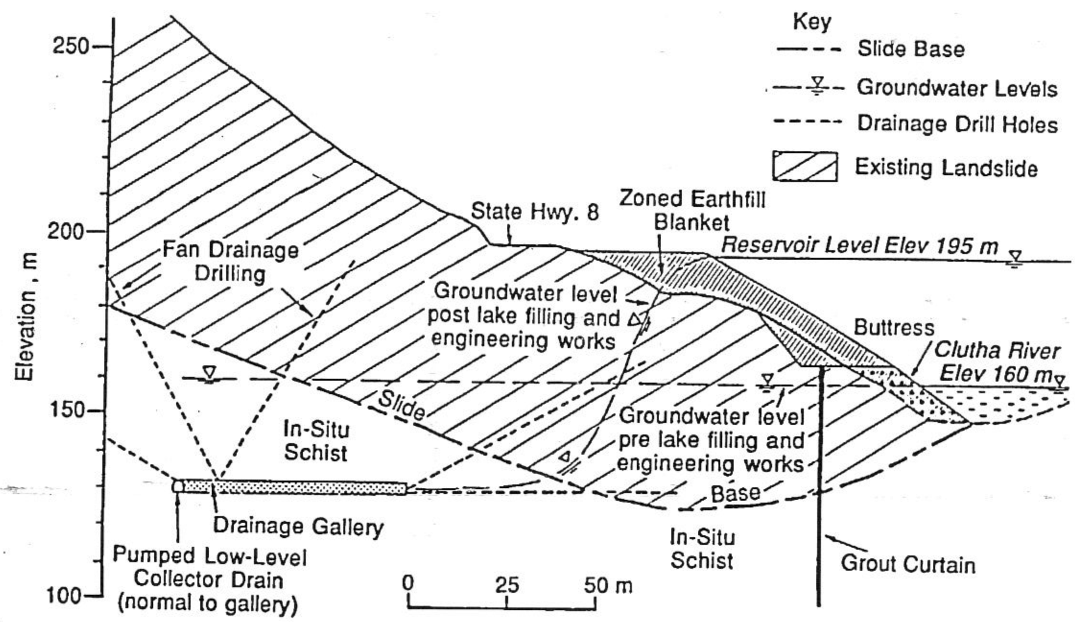


Fig. 5. The Brewery Creek landslide-stabilization scheme, Clyde Power Project, New Zealand (after Gillon et al. in press).

drains need to be cleaned once every 5-8 years (heavy root growth or exceptionally fine-grained sediments can change this requirement to every 2 years).

When using horizontal drains to dewater an unstable slope, the length, spacing, and position of the drains must be chosen. These drain-placement parameters are usually selected on the basis of engineering judgment or design charts. However, Nakamura (1988) has developed a numerical-analysis technique to evaluate time-related drop in groundwater level in a slope as a result of use of horizontal drains, and Resnick and Znidarcic (1990) have tested model slopes in a geotechnical centrifuge to determine the influence of horizontal drains on distribution of piezometric levels within the slopes.

As noted above, there is an annular space between the horizontal casing and the soil or rock of the drillhole. Filling this space with uniformly graded filter sand is difficult, and checking the actual quality of the work in the hole is impractical. Sembenelli (1988) has noted that many attempts have been made to eliminate the need for this awkward sand filter. In the past, pipes with very fine slots have been used; however, these are being supplanted by pipes with coarse slotting protected on the outside by one- or two-ply synthetic geotextile stockings.

3.5 Subvertical drains drilled upward from tunnels

Fans of subvertical drains are now commonly driven upward from drainage galleries or tunnels. Groundwater flows by gravity from these drains into the larger collectors, from which it either drains by gravity or is pumped to the surface. The use of a system of subvertical fan drains to remove groundwater from the Brewery Creek landslide in New Zealand is discussed above and illustrated in Fig. 5.

3.6 Less commonly used drainage methods

3.6.1 Electroosmotic dewatering

In the mid 1930's, Professor Leo Casagrande introduced the concept of electroosmotic drainage into geotechnical engineering practice, and many papers (e.g., Casagrande, 1948) have since been published on this subject. Despite some success in the past in slope stabilization, this process has not received wide application because of the costs of operation and remaining technical uncertainties. However, Lo et al. (1991a, 1991b) have improved the process by using specially designed copper electrodes to prevent gas accumulation around the electrode and to allow free water to flow from the cathode without pumping. In field tests conducted on the soft, sensitive Leda clay of eastern Canada, undrained shear strength increased by about 50 percent in a period of 32 days throughout the depth of the electrodes. Because no pumping was needed, both installation and electricity costs were reduced considerably compared with earlier electroosmotic drainage installations.

3.6.2 Vacuum dewatering

Arutjunyan (1988) has reported on the use of vacuum in drill holes to dewater soils of low permeability in slopes in the Soviet Union. The technique has been applied to landslides as a quickly installed temporary measure until long-term stabilization could be effected. The vacuum treatment increases soil suction and accelerates the process of soil consolidation. It has proved to be successful to depths of 30-35 m by applying the vacuum for a period of 2-4 weeks.

3.6.3 Drainage by siphoning

Siphon drains for slope stabilization have been installed at 40 sites in France in the past 5 years (Gress in press). These drains have the advantage of being able to raise water to the surface without pumping. Siphoning of water from unstable strata is accomplished by sealed PVC pipe systems. A recent example of the successful use of siphoning to lower the groundwater table under a highway embankment on an unstable slope occurred at Venarey-Les-Laumes near Dijon, France, where five vertical siphon drains, spaced at intervals of 10 m, lowered the piezometric level from an original depth of 2 m beneath the highway to a depth of 8 m (Fig. 6).

3.6.4 Use of geosynthetics for drainage

ASTM Committee D-35 (1991) defined a "geosynthetic" as "a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system." Geosynthetics include geotextiles, geogrids, geomembranes, geonets, and geocomposites (Koerner 1990). Correctly designed and installed geotextiles ("thin, flexible, permeable sheets of synthetic

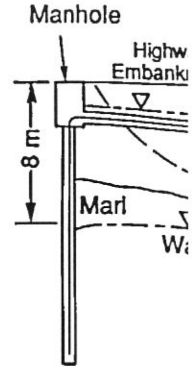


Fig. 6. Cross
unstable slope

material used
performance o
engineering w
filter, drain,
geologic materi
They can be
drainage of fine
clays) if the re
When used for
direction, rath
plane of the
geotextiles are
punched fabric
mm thick and
provide the
particularly wh
common usage
embankments,
and as pore-
retaining walls
In-place sul
can also be acc
the most recent
geosynthetics
defined a geo
material form
integrally joine
drainage with f
or any other ge
an integral pa
structure, or
geonets are us
extrusion pro
netlike config
which is free-d
geonet mats a
Because they
geonets are alw
geomembrane,
their outer su
materials are k

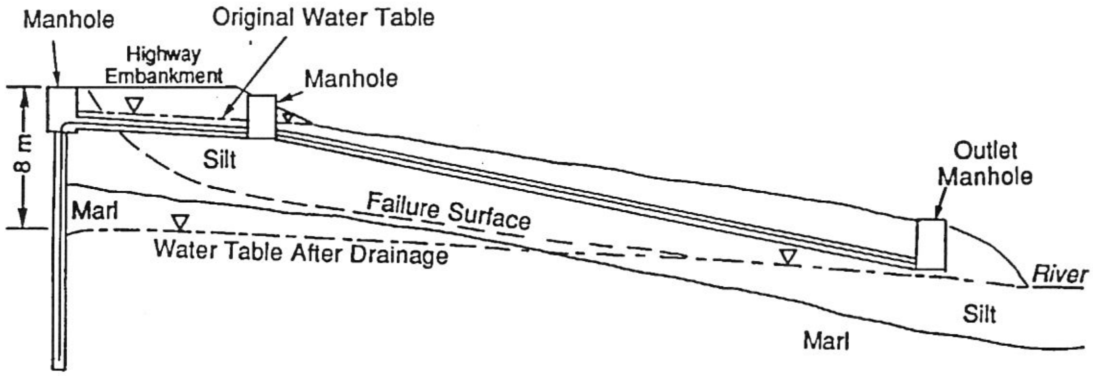


Fig. 6. Cross section of siphon-drain stabilization of highway embankment and underlying unstable slope near Dijon, France (after Gress in press).

material used to stabilise and improve the performance of soil associated with civil engineering works") have the ability to filter, drain, reinforce, and separate geologic materials (Ingold & Miller 1988)). They can be used for lateral or vertical drainage of fine-grained soil masses (silts or clays) if the required flow rates are modest. When used for drainage, flow is in the planar direction, rather than moving across the plane of the fabric as is the case when geotextiles are used as filters. Needle-punched fabrics, that often are as much as 6-mm thick and can be made much thicker, provide the most effective drainage, particularly when used in layers. Their most common usage on slopes is for drainage of embankments, within reinforced earth walls, and as pore-pressure dissipators behind retaining walls (Koerner 1990).

In-place subsurface drainage capability can also be accomplished by use of geonets, the most recently introduced members of the geosynthetics family. Koerner (1990) has defined a geonet as "a netlike polymeric material formed with intersecting ribs integrally joined at the junctions used for drainage with foundation, soil, rock, earth, or any other geotechnical-related material as an integral part of a human-made project, structure, or system." In manufacture, geonets are usually formed by a continuous extrusion process of polyethylene into a netlike configuration of intersecting ribs which is free-draining. In use for drainage, geonet mats are commonly 5-13 mm thick. Because they do not function as filters, geonets are always confined by a geotextile, geomembrane, or other material placed on their outer surfaces; such combinations of materials are known as geocomposites.

A recent example of the use of geofabric drains to help stabilize a steep slope in residual soil and weathered rock in Singapore has been presented by Broms & Wong (1986). Woven geofabric was wrapped around sand-and-gravel drains that cut across failure surfaces. The fabric functioned as both a filter around the drains and as reinforcement in the slope.

3.6.5 Blasting for drainage of rock slopes

List (1988) has described the reduction of pore pressures in unstable bedrock open-pit mine slopes by means of blasting. The procedure was used in an open-pit oil-sand mine in northeastern Alberta, Canada. During oil-sand excavation, large block slides occurred in the 60-m-high sedimentary-rock wall of the pit. Local stability was obtained by setting off explosive charges of as much as 1500 kg of Heavy AN/FO per hole in boreholes along the face. The explosive forces reduced pore pressures by forming microfractures and fractures in the rock.

4 RECENT ADVANCES IN USE OF EARTH-RETENTION SYSTEMS IN SLOPE STABILIZATION

In this paper, I will treat earth-retention systems as a broad category of remedial or control measures that include conventional retaining walls, rock fences and nets, soil and rock anchors, soil nailing, and reinforced-earth-type walls, as well as changes in physical/chemical character of the earth mass by processes other than drainage.

For clarity, the discussion will be divided into uses of retention systems on (1) rock slopes and (2) soil slopes, although there are obvious overlapping uses in these two categories of geologic materials.

4.1 Retention systems for rock faces and slopes

Although several different categories of failures occur on rock slopes, only rockfalls, rock topples, rock slumps, and rock slides can be prevented or controlled at reasonable expense. Rock avalanches, another major type of rock-slope failure, in most cases are too large to be controlled; however, rock avalanches often start as relatively small falls, topples, slides, or slumps that are controllable at the source. This section will deal mainly with stabilization of slopes that are subject to rockfall, because it is in this area of rock-slope engineering that the most recent advances have been made. Several of the new techniques considered here also are applicable to rock topples, slumps, and slides.

Increasing traffic volumes in mountainous areas have heightened public awareness of the danger of potential rockfalls, resulting in significant ongoing research and development of innovative measures for rockfall prediction and control. In much of the United States, rockfall accidents are no longer considered to be "acts of God;" the traveling public increasingly demands protection from rockfalls on mountain highways (Barrett & White 1991). An important factor in stabilization of slopes subject to rockfall has been the prioritization of slopes that are most subject to catastrophic failure; this prioritization enables unstable slopes to be stabilized before accidents occur. An example of a system of prioritization is the Rockfall Hazard Rating System (RHRS) developed by the Oregon Department of Transportation (Pierson 1991). Rockfall risks have been rated on the basis of: (1) slope height, (2) ditch effectiveness, (3) average vehicle risk (percentage of time a vehicle will be in the rockfall hazard zone), (4) sight distance for drivers, (5) roadway width, (6) geologic character of the slope, (7) block size or quantity of rockfall per event, (8) presence of water on the slope, and (9) rockfall history. The Oregon RHRS system is intended to be a proactive tool that will allow transportation agencies to rationally address rockfall hazards instead of simply reacting to rockfall accidents.

The measures most commonly used to prevent rockfall from encroaching upon a highway, railway, or other structure or development are (1) rock nets, fences, walls, attenuators, benches, and ditches, and (2) rock bolting and buttresses (Fig. 7). In addition, even though it is expensive, under extreme conditions tunneling can be used to avoid rockfall.

4.1.1 Rock nets, fences, walls, attenuators, benches, and ditches

The most important rockfall input factors in design of systems to prevent rockfall from reaching a highway, railway, or other critical structure include (1) trajectory (height of bounce), (2) velocity, (3) impact energy, and (4) total volume of accumulation. Nearly 30 years ago, Ritchie (1963) documented that by varying slope steepness and length, height of rock bounce, velocity, and distance of travel are greatly affected. In recent years, computer programs have been developed to provide statistical analysis of probable rockfall behavior for a given slope

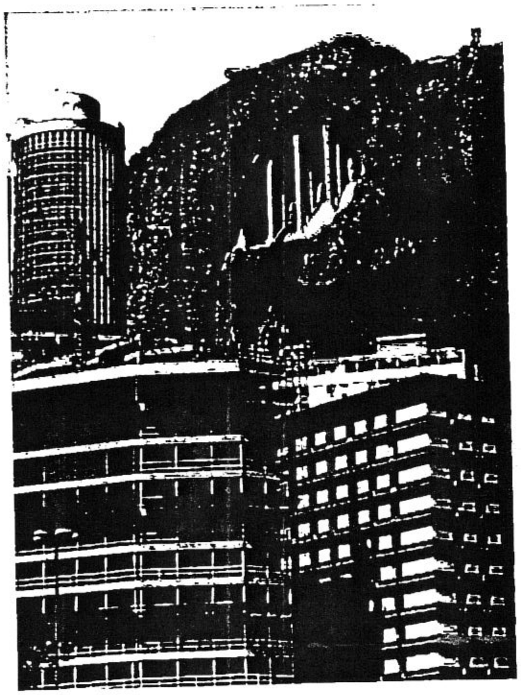


Fig. 7. Anchored-concrete-pillar support of huge block of gneiss above high-rise buildings, Cantaglo Hill, Rio de Janeiro. The 25-m maximum-height pillars were installed by the Rio de Janeiro Geotechnical Control Office in 1967. (1990 photograph courtesy of Rio de Janeiro Geotechnical Control Office).

by modeling
by convertir
cartesian-coo
change in slc
different line
most recent oi
Rockfall Simu
incorporates
to slope and r
et al. 1990).
relating velc
properties ai
interaction oi
model utilize
acceleration
describe the
rock. The
rockfalls is m
angle at whicl
limits set t
differences,
geometry. T
of probable
various locat
10).
Programs
being refinec
application t
walls, fences
Colorado De
using CRSP
walls in const
(I-70) throug
Canyon in we
1991). An e
design and
faced, geosy
concrete roc
70 near the t
m design hei
based on roc
CRSP (Figs.
Highways
North Ameri
from rockfal
fencing sup
thus, in the
fence" ofter
style, comm
fence (Bar
fence, as a
inexpensive
rockfalls. H
rockfall catc
for many y
Europe to p
mountain co
these syst
protection s
protect Alpi
The Euro
used extens
the United S

For clarity, the discussion will be divided into uses of retention systems on (1) rock slopes and (2) soil slopes, although there are obvious overlapping uses in these two categories of geologic materials.

4.1 Retention systems for rock faces and slopes

Although several different categories of failures occur on rock slopes, only rockfalls, rock topples, rock slumps, and rock slides can be prevented or controlled at reasonable expense. Rock avalanches, another major type of rock-slope failure, in most cases are too large to be controlled; however, rock avalanches often start as relatively small falls, topples, slides, or slumps that are controllable at the source. This section will deal mainly with stabilization of slopes that are subject to rockfall, because it is in this area of rock-slope engineering that the most recent advances have been made. Several of the new techniques considered here also are applicable to rock topples, slumps, and slides.

Increasing traffic volumes in mountainous areas have heightened public awareness of the danger of potential rockfalls, resulting in significant ongoing research and development of innovative measures for rockfall prediction and control. In much of the United States, rockfall accidents are no longer considered to be "acts of God;" the traveling public increasingly demands protection from rockfalls on mountain highways (Barrett & White 1991). An important factor in stabilization of slopes subject to rockfall has been the prioritization of slopes that are most subject to catastrophic failure; this prioritization enables unstable slopes to be stabilized before accidents occur. An example of a system of prioritization is the Rockfall Hazard Rating System (RHRS) developed by the Oregon Department of Transportation (Pierson 1991). Rockfall risks have been rated on the basis of: (1) slope height, (2) ditch effectiveness, (3) average vehicle risk (percentage of time a vehicle will be in the rockfall hazard zone), (4) sight distance for drivers, (5) roadway width, (6) geologic character of the slope, (7) block size or quantity of rockfall per event, (8) presence of water on the slope, and (9) rockfall history. The Oregon RHRS system is intended to be a proactive tool that will allow transportation agencies to rationally address rockfall hazards instead of simply reacting to rockfall accidents.

The measures most commonly used to prevent rockfall from encroaching upon a highway, railway, or other structure or development are (1) rock nets, fences, walls, attenuators, benches, and ditches, and (2) rock bolting and buttresses (Fig. 7). In addition, even though it is expensive, under extreme conditions tunneling can be used to avoid rockfall.

4.1.1 Rock nets, fences, walls, attenuators, benches, and ditches

The most important rockfall input factors in design of systems to prevent rockfall from reaching a highway, railway, or other critical structure include (1) trajectory (height of bounce), (2) velocity, (3) impact energy, and (4) total volume of accumulation. Nearly 30 years ago, Ritchie (1963) documented that by varying slope steepness and length, height of rock bounce, velocity, and distance of travel are greatly affected. In recent years, computer programs have been developed to provide statistical analysis of probable rockfall behavior for a given slope

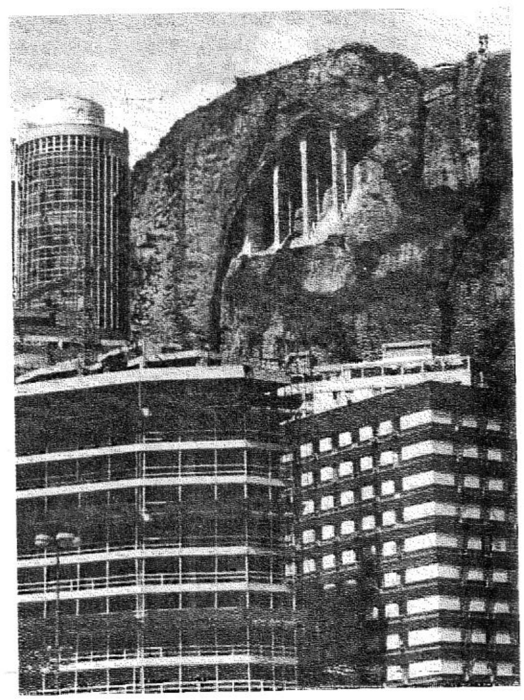


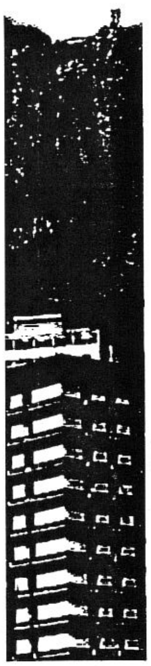
Fig. 7. Anchored-concrete-pillar support of huge block of gneiss above high-rise buildings, Cantaglo Hill, Rio de Janeiro. The 25-m maximum-height pillars were installed by the Rio de Janeiro Geotechnical Control Office in 1967. (1990 photograph courtesy of Rio de Janeiro Geotechnical Control Office).

by modeling
by convertir
cartesian-coo
change in sl
different line
most recent o
Rockfall Simu
incorporates
to slope and r
et al. 1990).
relating vel
properties a
interaction o
model utilize
acceleration
describe the
rock. The
rockfalls is m
angle at whic
limits set b
differences,
geometry. T
of probable
various locat
10).
Programs
being refined
application t
walls, fences
Colorado De
using CRSP
walls in const
(I-70) throug
Canyon in we
1991). An e
design and
faced, geosy
concrete roc
70 near the t
m design hei
based on roc
CRSP (Figs.
Highways
North Ameri
from rockfall
fencing sup
thus, in the
fence" ofter
style, comm
fence (Bar
fence, as a
inexpensive
rockfalls. H
rockfall catc
for many y
Europe to p
mountain co
these syst
protection s
protect Alpi
The Eur
used extens
the United

ənces, walls,
ıes, and (2)
ʼig. 7). In
ısive, under
n be used to

attenuators,

ut factors in
ʼockfall from
other critical
y (height of
ıact energy,
tion. Nearly
ımented that
and length,
and distance
In recent
have been
l analysis of
given slope



ır support of
e high-rise
Janeiro. The
ere installed
nical Control
h courtesy of
ıtrol Office).

by modeling the slope; this is accomplished by converting slope survey data into a cartesian-coordinate system in which each change in slope angle is represented by a different line segment or cell. One of the most recent of these programs is the Colorado Rockfall Simulation Program (CRSP), which incorporates numerical input values assigned to slope and rockfall characteristics (Pfeiffer et al. 1990). Empirically derived functions relating velocity, friction, and material properties are used to model the dynamic interaction of the rock and the slope. The model utilizes equations of gravitational acceleration and conservation of energy to describe the motion of falling and bouncing rock. The statistical variation among rockfalls is modeled by randomly varying the angle at which a rock impacts the slope within limits set by rock size, slope material differences, and irregularities of slope geometry. The program provides estimates of probable bounce height and velocity at various locations on a slope (Figs. 8, 9, & 10).

Programs such as CRSP are constantly being refined as experience is gained in their application to design of rockfall retention walls, fences, benches, and ditches. The Colorado Department of Transportation is using CRSP for design of rockfall retention walls in construction of Interstate Highway 70 (I-70) through the rugged gorge of Glenwood Canyon in western Colorado (Barrett & White 1991). An example of this use was the 1990 design and construction of a rubber-tire-faced, geosynthetically reinforced soil and concrete rockfall retention wall to protect I-70 near the town of Gypsum, Colorado; the 5-m design height of the constructed wall was based on rock rebound height as estimated by CRSP (Figs. 8 & 9) (Andrew in press).

Highways and railways in the mountains of North America commonly have been protected from rockfall by traditional single-twist mesh fencing supported by fixed, rigid posts; thus, in the United States the term "rock fence" often is associated with this basic style, commonly referred to as "chain-link" fence (Barrett & White 1991). This basic fence, as a single-mesh layer, is relatively inexpensive and will effectively contain small rockfalls. However, larger and heavier duty rockfall catch fences or nets have been in use for many years in the Alpine countries of Europe to protect highways, railways, and mountain communities from rockfall events; these systems evolved from avalanche protection systems developed in the 1950's to protect Alpine villages (Yarnell 1991).

The European technology is now being used extensively in the mountainous parts of the United States in spite of its higher cost

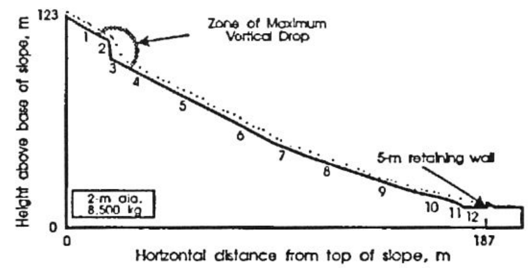


Fig. 8. Schematic rockfall trajectory produced by Colorado Rockfall Simulation Program (Pfeiffer et al. 1990) for rockfall on Interstate Highway 70 at Gypsum, Colorado. The slope was divided into 12 segments with surface characteristics determined by field study. For this analysis, individual rock spheres were chosen with a diameter of 2 m and weight of 8,500 kg each. Design of 5-m-high retaining wall shown at a horizontal distance of 187 m from the top of the slope was based on this rockfall simulation.

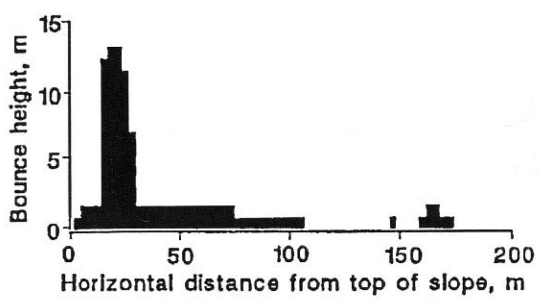


Fig. 9. Computer-produced Colorado Rockfall Simulation Program histogram of bounce heights for 25 2-m-diameter rocks on slope at Gypsum, Colorado (Fig. 8).

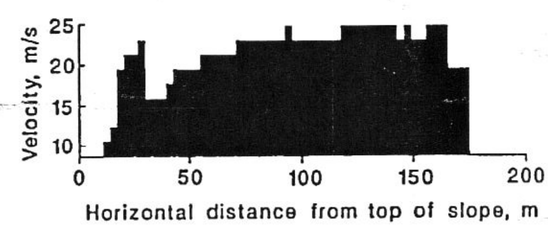
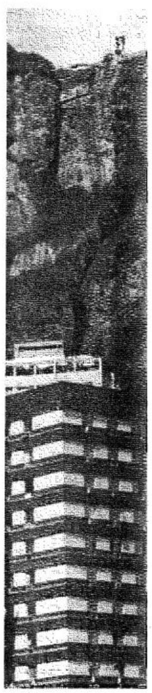


Fig. 10. Computer-produced Colorado Rockfall Simulation Program histogram for velocities of 25 2-m-diameter rocks on slope at Gypsum, Colorado (Fig. 8).

ənces, walls,
hes, and (2)
Fig. 7). In
asive, under
n be used to

attenuators,

ut factors in
rockfall from
other critical
y (height of
pact energy,
tion. Nearly
umented that
and length,
and distance
In recent
have been
l analysis of
given slope



ar support of
e high-rise
Janeiro. The
ere installed
nical Control
h courtesy of
trol Office).

by modeling the slope; this is accomplished by converting slope survey data into a cartesian-coordinate system in which each change in slope angle is represented by a different line segment or cell. One of the most recent of these programs is the Colorado Rockfall Simulation Program (CRSP), which incorporates numerical input values assigned to slope and rockfall characteristics (Pfeiffer et al. 1990). Empirically derived functions relating velocity, friction, and material properties are used to model the dynamic interaction of the rock and the slope. The model utilizes equations of gravitational acceleration and conservation of energy to describe the motion of falling and bouncing rock. The statistical variation among rockfalls is modeled by randomly varying the angle at which a rock impacts the slope within limits set by rock size, slope material differences, and irregularities of slope geometry. The program provides estimates of probable bounce height and velocity at various locations on a slope (Figs. 8, 9, & 10).

Programs such as CRSP are constantly being refined as experience is gained in their application to design of rockfall retention walls, fences, benches, and ditches. The Colorado Department of Transportation is using CRSP for design of rockfall retention walls in construction of Interstate Highway 70 (I-70) through the rugged gorge of Glenwood Canyon in western Colorado (Barrett & White 1991). An example of this use was the 1990 design and construction of a rubber-tire-faced, geosynthetically reinforced soil and concrete rockfall retention wall to protect I-70 near the town of Gypsum, Colorado; the 5-m design height of the constructed wall was based on rock rebound height as estimated by CRSP (Figs. 8 & 9) (Andrew in press).

Highways and railways in the mountains of North America commonly have been protected from rockfall by traditional single-twist mesh fencing supported by fixed, rigid posts; thus, in the United States the term "rock fence" often is associated with this basic style, commonly referred to as "chain-link" fence (Barrett & White 1991). This basic fence, as a single-mesh layer, is relatively inexpensive and will effectively contain small rockfalls. However, larger and heavier duty rockfall catch fences or nets have been in use for many years in the Alpine countries of Europe to protect highways, railways, and mountain communities from rockfall events; these systems evolved from avalanche protection systems developed in the 1950's to protect Alpine villages (Yarnell 1991).

The European technology is now being used extensively in the mountainous parts of the United States in spite of its higher cost

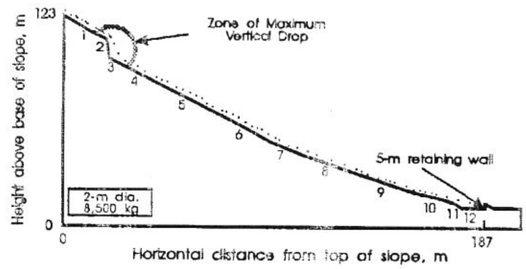


Fig. 8. Schematic rockfall trajectory produced by Colorado Rockfall Simulation Program (Pfeiffer et al. 1990) for rockfall on Interstate Highway 70 at Gypsum, Colorado. The slope was divided into 12 segments with surface characteristics determined by field study. For this analysis, individual rock spheres were chosen with a diameter of 2 m and weight of 8,500 kg each. Design of 5-m-high retaining wall shown at a horizontal distance of 187 m from the top of the slope was based on this rockfall simulation.

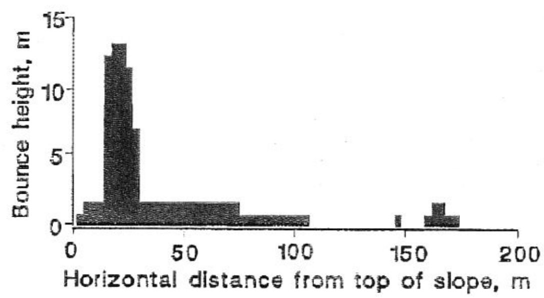


Fig. 9. Computer-produced Colorado Rockfall Simulation Program histogram of bounce heights for 25 2-m-diameter rocks on slope at Gypsum, Colorado (Fig. 8).

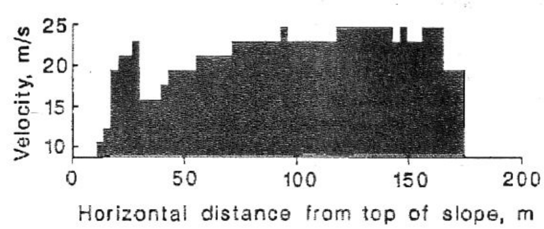


Fig. 10. Computer-produced Colorado Rockfall Simulation Program histogram for velocities of 25 2-m-diameter rocks on slope at Gypsum, Colorado (Fig. 8).

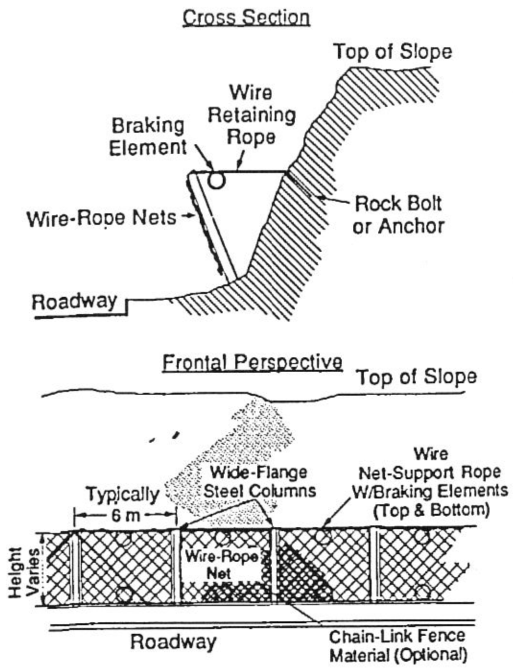


Fig. 11. Schematic cross section and frontal perspective of a typical wire-rope rock-fall barrier net (after Yarnell 1991.)

as compared to chain-link fence. Most United States applications of wire-rope safety nets are found in the eastern States (Yarnell 1991). This may be the result of limited rights-of-way along eastern transportation corridors, which preclude the use of other mitigative measures. However, rockfall nets/fences also can be used to control rock slopes of great heights, such as those found in the western United States.

Rockfall problem areas have been identified along 5,000 km of California highways (McCauley et al. 1985). The 1985 study concluded that rolling rocks up to 0.6 m in diameter can be restrained by chain-link fence; however, this type of restraining device frequently suffers severe damage when hit by rocks of this size and is inadequate to stop larger rocks. Thus, a rigorous field testing program of "European-style" rock fences (Fig. 11) was conducted by the California Department of Transportation (Caltrans) (Smith & Duffy 1990). Many of these fences are proprietary. The two types tested by Caltrans were high-impact wire-rope net systems developed by Brugg Cable Products, Inc., (Switzerland) and The Industrial Enterprise Corporation (France). Both types of fences rely on

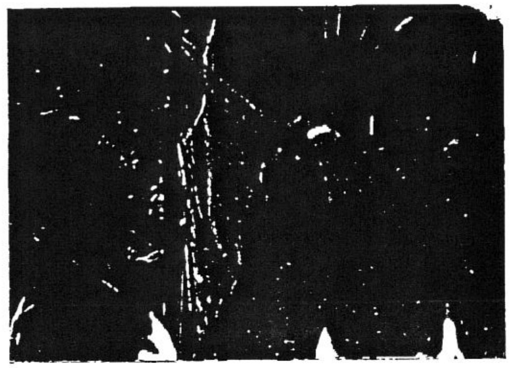


Fig. 12. Three-ton boulder impacting panel of rock net in field tests by California Department of Transportation. (Photograph by J. L. Walkinshaw, Federal Highway Administration, U.S. Department of Transportation.)

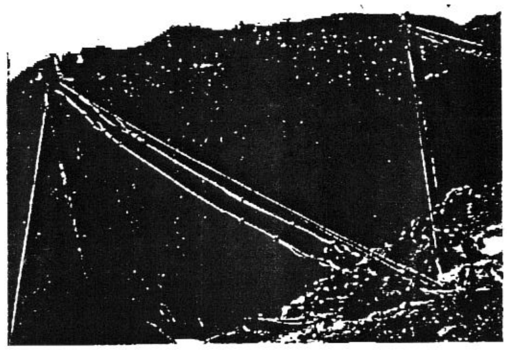


Fig. 13. Upslope side of Isomat Italia s.p.a. wire-rope rockfall-retaining-net fence. Friction brakes are enclosed in modules at lower right. (Photograph courtesy of Isomat Italia s.p.a., Milan, Italy.)

friction braking devices. When the bouncing rocks collide with the fences, producing deformation of the nets, the nets engage the energy-absorbing friction brakes, thus extending the time of collision, and thereby significantly increasing the capacity of the nets to restrain the rocks. This approach allows the use of lighter, less costly elements, thus reducing the cost of the fences. The Caltrans field tests were conducted on an 80-m-long, 34° slope below State Highway 1 between Big Sur and San Simeon, California. Large boulders (135 to 6000 kg) were rolled down the slope into the net (Fig. 12), imparting rotational energy on individual components of the fence. Design-load rockfalls were effectively stopped by

both types of ro
(Fig. 13) with fr
produced by Isc
control in northe
been used succe
the relocated r
highway at the
Val Pola landslic
One disadva
friction energy-
that the frictio
after each sign
long-term maint
For this reason,
Transportation
Colorado Flex
(Barrett & Wh
grouting bundle
casings, posts
ability to be flex
support the mes
the system is
redirects bour
dissipating co
immediately afte
posts rebound,
the next en
maintenance (B
insure that this
to the public, i
Colorado Depa
thus, its design
An untested v
post fence utili
that causes bou
go underneath)
slope surface
dissipated (Bar
If, as a result
rock becomes e
large forces ma

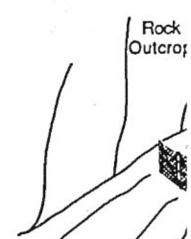


Fig. 14. Sch
Department of
Rockfall Fence
The mesh is dou

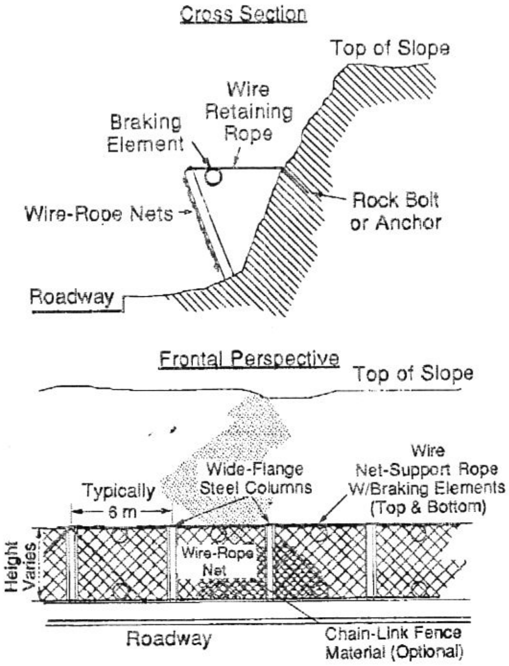


Fig. 11. Schematic cross section and frontal perspective of a typical wire-rope rock-fall barrier net (after Yarnell 1991.)

as compared to chain-link fence. Most United States applications of wire-rope safety nets are found in the eastern States (Yarnell 1991). This may be the result of limited rights-of-way along eastern transportation corridors, which preclude the use of other mitigative measures. However, rockfall nets/fences also can be used to control rock slopes of great heights, such as those found in the western United States.

Rockfall problem areas have been identified along 5,000 km of California highways (McCauley et al. 1985). The 1985 study concluded that rolling rocks up to 0.6 m in diameter can be restrained by chain-link fence; however, this type of restraining device frequently suffers severe damage when hit by rocks of this size and is inadequate to stop larger rocks. Thus, a rigorous field testing program of "European-style" rock fences (Fig. 11) was conducted by the California Department of Transportation (Caltrans) (Smith & Duffy 1990). Many of these fences are proprietary. The two types tested by Caltrans were high-impact wire-rope net systems developed by Brugg Cable Products, Inc., (Switzerland) and The Industrial Enterprise Corporation (France). Both types of fences rely on

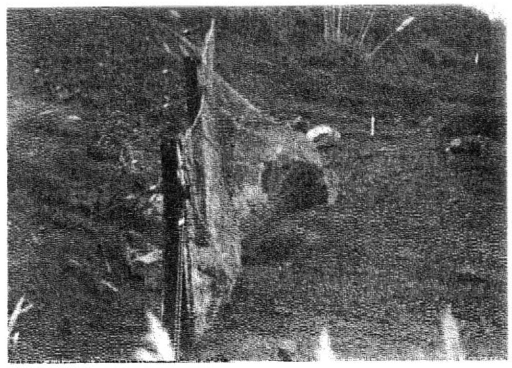


Fig. 12. Three-ton boulder impacting panel of rock net in field tests by California Department of Transportation. (Photograph by J. L. Walkinshaw, Federal Highway Administration, U.S. Department of Transportation.)

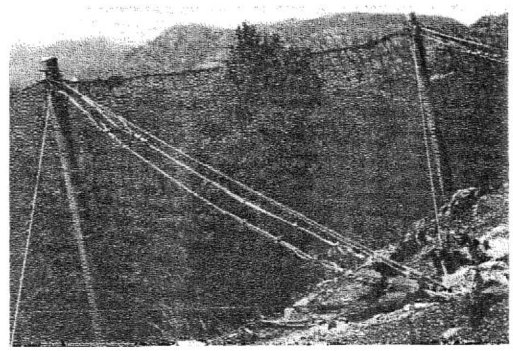


Fig. 13. Upslope side of Isomat Italia s.p.a. wire-rope rockfall-retaining-net fence. Friction brakes are enclosed in modules at lower right. (Photograph courtesy of Isomat Italia s.p.a., Milan, Italy.)

friction braking devices. When the bouncing rocks collide with the fences, producing deformation of the nets, the nets engage the energy-absorbing friction brakes, thus extending the time of collision, and thereby significantly increasing the capacity of the nets to restrain the rocks. This approach allows the use of lighter, less costly elements, thus reducing the cost of the fences. The Caltrans field tests were conducted on an 80-m-long, 34° slope below State Highway 1 between Big Sur and San Simeon, California. Large boulders (135 to 6000 kg) were rolled down the slope into the net (Fig. 12), imparting rotational energy on individual components of the fence. Design-load rockfalls were effectively stopped by

both types of ro
(Fig. 13) with fr
produced by Iso
control in northe
been used succe
the relocated r
highway at the
Val Pola landslic

One disadva
friction energy-
that the frictio
after each sign
long-term maint
For this reason,
Transportation
Colorado Flex
(Barrett & Wh
grouting bundle
casings, posts
ability to be flex
support the mes
the system is
redirects bou
dissipating co
immediately afte
posts rebound,
the next en
maintenance (B
insure that this
to the public, i
Colorado Depa
thus, its design

An untested v
post fence utili
that causes bou
go underneath)
slope surface
dissipated (Bar
If, as a result
rock becomes e
large forces ma

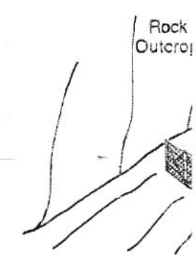
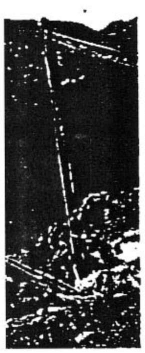


Fig. 14. Sch
Department of
Rockfall Fence
The mesh is do

acting panel
y California
(Photograph
·al Highway
ırtment of



Italia s.p.a.
net fence.
ı modules at
esy of Isomat

the bouncing
, producing
s engage the
·akes, thus
and thereby
pacity of the
his approach
less costly
cost of the
tests were
slope below
Sur and San
lders (135 to
lope into the
al energy on
ıce. Design-
stopped by

both types of rock nets. A similar fence/net (Fig. 13) with friction braking has been produced by Isomat Italia s.p.a. for rockfall control in northern Italy. Isomat fences have been used successfully to control rockfall on the relocated route of the Bormio-Sondrio highway at the toe of the 1987 35-million-m^3 Val Pola landslide in the Italian Alps.

One disadvantage of systems that use friction energy-absorbing brake systems is that the friction brakes require resetting after each significant rockfall, a factor in long-term maintenance costs of the systems. For this reason, the Colorado Department of Transportation has recently developed the Colorado Flexible-Post Rockfall Fence (Barrett & White 1991) (Fig. 14). By grouting bundles of wire tendons into steel casings, posts are produced that have the ability to be flexible, yet are rigid enough to support the mesh netting. The principle for the system is: the fence catches and redirects bouncing rocks to energy-dissipating collisions with the slope; immediately after each collision, the flexible posts rebound, leaving the fence ready for the next encounter without needing maintenance (Barrett & White 1991). To insure that this innovative system is available to the public, it has been patented by the Colorado Department of Transportation; thus, its design and use are public property.

An untested variation of the Colorado flex-post fence utilizes a long tail of wire mesh that causes bouncing rocks to "mole" (i.e., go underneath) between the mesh and the slope surface until all energy has been dissipated (Barrett & White 1991) (Fig. 15). If, as a result of its rotation, a bouncing rock becomes entangled in the wire mesh, large forces may suddenly be applied to the mesh, cable, and posts, resulting in flexing of the posts.

Another type of energy-absorbing fence has recently been used to contain rockfall on a slope above a housing development in Springdale, New Foundland, Canada (Boyd 1991). The 240-m-long fence, which was designed for a rock impact energy of 100 KNm, incorporates a series of high-strength cables that run horizontally along the slope and parallel to each other, and are supported by 3-m-high galvanized-steel posts set in concrete foundations. The fence absorbs energy from rock impacts by stretching of the cables. The cable system is covered by galvanized-steel mesh to aid in absorbing rotational energies and to intercept flying rock fragments.

Polymer grids (discussed as a means of soil reinforcement in section 4.2.1.1.3) have been used in Hong Kong as fences to protect a housing development from rockfall (Threadgold & McNichol 1984). In Norway, similar geogrid fences are being used as protection against avalanches during the spring thaw (Bush 1988).

Another approach to controlling rockfall is to partially absorb or attenuate the energy of bouncing or rolling rocks without actually stopping them. The Colorado Department of Transportation has developed an attenuation system that utilizes columns of used tires and rims mounted on vertical 75-mm-diameter steel pipes suspended from a large-diameter wire rope mounted across the rockfall chute (Barrett & White 1991; Andrew in press) (Fig. 16). Rock anchors are used to secure the ends of the wire rope to the bedrock walls of the gully. To address aesthetic concerns, a facade of wooden timbers is usually suspended from a wire rope immediately

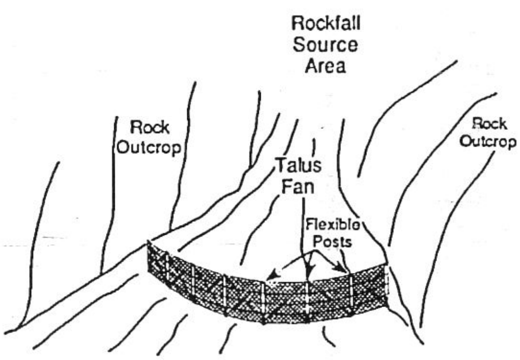


Fig. 14. Schematic diagram of Colorado Department of Transportation Flexible-Post Rockfall Fence (after Barrett & White 1991). The mesh is double-twist hexagonal wire.

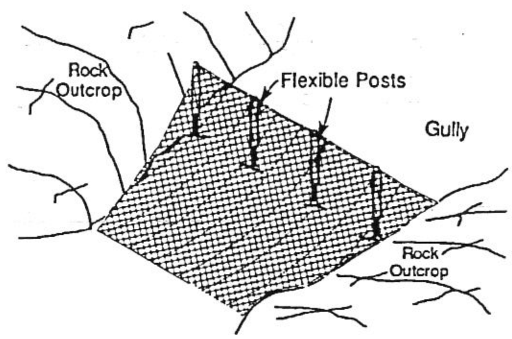


Fig. 15. Colorado "mole" fence (after Barrett & White 1991). Posts are flexible; mesh is double-twist hexagonal wire.

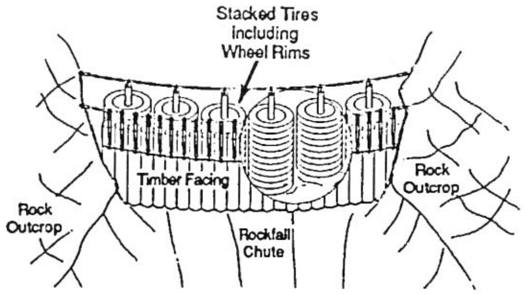


Fig. 16. Schematic diagram of Colorado Department of Transportation rockfall attenuator (after Andrew in press).

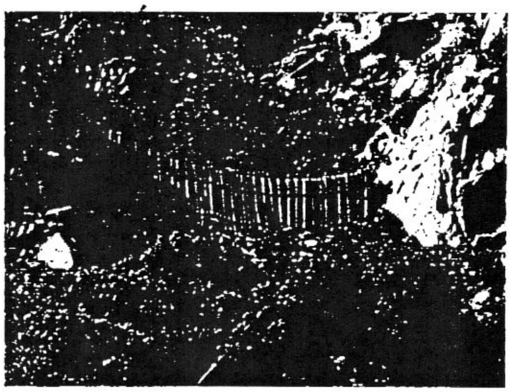


Fig. 17. Uphill view of Colorado Department of Transportation rockfall attenuator located in rockfall chute above Interstate Highway 70, Glenwood Canyon, Colorado. Note timber facing on downchute side of the attenuator.

downslope of the hanging tires (Fig. 17). The Colorado "rockfall attenuator" is designed to absorb most of the kinetic energy and to reduce rebounding heights from incoming rockfall; after a rock passes through the attenuator, the system returns to its original position without maintenance.

4.1.2 Rock bolting

Rock bolting has been used for several decades to stabilize rock slopes. Tensioned rock bolts are positioned across potential rock failure surfaces and anchored in stable rock beyond the surfaces. The application of tensile stress in the bolts results in increased normal stress in the direction of the bolts and decreased shear stress on the failure surface. Methods of securing the distal end of the bolt in the drill hole include cement anchors). For permanent anchorage, grout, resin, and mechanical anchors (e.g., slot-and-wedge and expansion-shellgrouted bolts are preferred because they provide better bonding between the rock and the bolt, and are resistant to corrosion (Wyllie 1991). If the face plate for the bolt fails or corrodes away, the bolt force continues to be transferred to the rock by means of the grout bond. The grout can be a nonshrinking Portland-cement mix or a plastic (high-density polymer). Plastics with variable setting rates often are chosen so that the grout in the anchorage zone hardens first, allowing tensioning before the rest of the grout sets. Current world practice in design and installation of rock anchors has recently been summarized by Littlejohn (1990).

Barley (1991) has described five recent cases in the United Kingdom in which rock-bolting systems were used to stabilize rock and soil slopes. Unique methods of rotary percussive drilling, water jetting to clean the holes, and grouting were used in these installations. An interesting concept of multiple "unit anchors" within a single borehole is provided by the Single Bore Multiple Anchor (SBMA) system. Each anchor has its own tendon, and each anchor is encapsulated in a corrosion-protection system; the anchor "capsules" are located at staggered depths within the borehole, so that each anchor, loaded with its own stressing jack, transfers its load to a discreet length of the anchor bore. This system almost eliminates the effect of the progressive failure mechanism that occurs in normal anchorage systems and allows the simultaneous mobilization of almost the entire ground strength throughout the full length of the borehole (Fig. 18). The SBMA system has been used successfully to stabilize a coastal slope at Alexander Quay, Southhampton, England (Barley 1991).

Rock anchors often are combined with other methods of stabilization. One of the best-known examples was the use of 405 rock anchors to control the slide on the right abutment of Tablachaca Dam, Peru, mentioned earlier in this report. These anchors accompanied a large toe buttress, surface and subsurface drainage, rock excavation, and improvement of the river-channel flow pattern (Morales Arnao et al., 1984).

4.1.3 Tunneling as a means of avoiding rock-slope failures

Because of its expense, tunneling seldom is used to avoid rock-slope problems. However, after a section of the main line of

BC Rail in Br
been blocked f
years, and
continuing roc
the line, the ra
new 1.2-km-lo
unstable slopes
was completed
million. Thi
permanent, s
problem of roc

4.2 Retention s

Figure 19 summ
earth retentio
presented in
whether they
stabilization.
internally stab
are presented
stabilized syst
walls against
mobilized. Pri
walls, mainly
were the pre
structures.
external walls
not be discuss
earth retention
that is installe

Figure 19. Cl

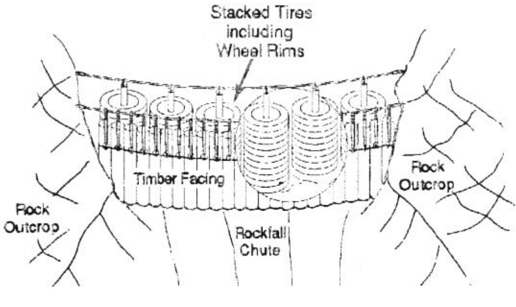


Fig. 16. Schematic diagram of Colorado Department of Transportation rockfall attenuator (after Andrew in press).

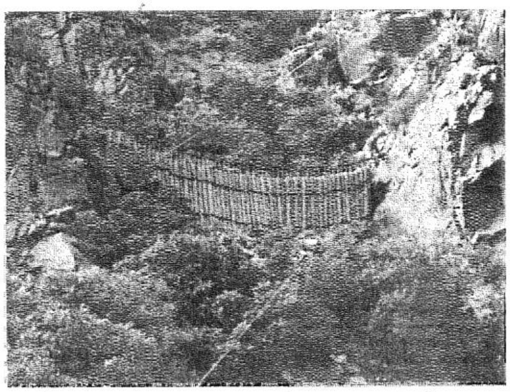


Fig. 17. Uphill view of Colorado Department of Transportation rockfall attenuator located in rockfall chute above Interstate Highway 70, Glenwood Canyon, Colorado. Note timber facing on downchute side of the attenuator.

downslope of the hanging tires (Fig. 17). The Colorado "rockfall attenuator" is designed to absorb most of the kinetic energy and to reduce rebounding heights from incoming rockfall; after a rock passes through the attenuator, the system returns to its original position without maintenance.

4.1.2 Rock bolting

Rock bolting has been used for several decades to stabilize rock slopes. Tensioned rock bolts are positioned across potential rock failure surfaces and anchored in stable rock beyond the surfaces. The application of tensile stress in the bolts results in increased normal stress in the direction of the bolts and decreased shear stress on the failure surface. Methods of securing the distal end of the bolt in the drill hole include cement anchors). For permanent anchorage, grout, resin, and mechanical anchors (e.g., slot-and-wedge and expansion-shellgrouted bolts are preferred because they provide better bonding between the rock and the bolt, and are resistant to corrosion (Wyllie 1991). If the face plate for the bolt fails or corrodes away, the bolt force continues to be transferred to the rock by means of the grout bond. The grout can be a nonshrinking Portland-cement mix or a plastic (high-density polymer). Plastics with variable setting rates often are chosen so that the grout in the anchorage zone hardens first, allowing tensioning before the rest of the grout sets. Current world practice in design and installation of rock anchors has recently been summarized by Littlejohn (1990).

Barley (1991) has described five recent cases in the United Kingdom in which rock-bolting systems were used to stabilize rock and soil slopes. Unique methods of rotary percussive drilling, water jetting to clean the holes, and grouting were used in these installations. An interesting concept of multiple "unit anchors" within a single borehole is provided by the Single Bore Multiple Anchor (SBMA) system. Each anchor has its own tendon, and each anchor is encapsulated in a corrosion-protection system; the anchor "capsules" are located at staggered depths within the borehole, so that each anchor, loaded with its own stressing jack, transfers its load to a discreet length of the anchor bore. This system almost eliminates the effect of the progressive failure mechanism that occurs in normal anchorage systems and allows the simultaneous mobilization of almost the entire ground strength throughout the full length of the borehole (Fig. 18). The SBMA system has been used successfully to stabilize a coastal slope at Alexander Quay, Southhampton, England (Barley 1991).

Rock anchors often are combined with other methods of stabilization. One of the best-known examples was the use of 405 rock anchors to control the slide on the right abutment of Tablachaca Dam, Peru, mentioned earlier in this report. These anchors accompanied a large toe buttress, surface and subsurface drainage, rock excavation, and improvement of the river-channel flow pattern (Morales Arnao et al., 1984).

4.1.3 Tunneling as a means of avoiding rock-slope failures

Because of its expense, tunneling seldom is used to avoid rock-slope problems. However, after a section of the main line of

BC Rail in Br
been blocked f
years, and
continuing roc
the line, the re
new 1.2-km-lo
unstable slopes
was completed
million. Thi
permanent, s
problem of roc

4.2 Retention s

Figure 19 summ
earth retentio
presented in
whether they
stabilization.
internally stab
are presented
stabilized syste
walls against
mobilized. Pri
walls, mainly
were the pre
structures.
external walls
not be discuss
earth retention
that is installe

Figure 19. Cl

orage, grout,
; (e.g., slot-
lgrouted bolts
rovide better
the bolt, and
llie 1991). If
s or corrodes
inues to be
ıs of the grout
nonshrinking
lastic (high-
with variable
ı so that the
ardens first,
e rest of the
tice in design
; has recently
(1990).
d five recent
ı which rock-
stabilize rock
ods of rotary
ıg to clean the
sed in these
ɛ concept of
ıin a single
Single Bore
stem. Each
l each anchor
on-protection
are located at
ehole, so that
wn stressing
reet length of
ystem almost
progressive
's in normal
allows the
ost the entire
ie full length
SBMA system
o stabilize a
der Quay,
y 1991).
ombined with
One of the
se of 405 rock
on the right
Dam, Peru,
ort. These
oe buttress,
inage, rock
of the river-
Arnao et al.,

ıvoiding

ing seldom is
problems.
: main line of

BC Rail in British Columbia, Canada, had been blocked for 22 days within a period of 2 years, and geologists had predicted continuing rockslide/rockfall problems for the line, the railroad company constructed a new 1.2-km-long tunnel that bypassed the unstable slopes (Leighton 1990). The tunnel was completed in 1989 at a cost of US$6.6 million. This was an expensive, but permanent, solution to a very difficult problem of rock slope instability.

4.2 Retention systems for soil slopes

Figure 19 summarizes the current methods of earth retention; these methods have been presented in two groups depending on whether they provide external or internal stabilization. Examples of externally and internally stabilized earth retention systems are presented in Figure 20. Externally stabilized systems rely on external structural walls against which stabilizing forces are mobilized. Prior to the late 1960's, external walls, mainly gravity and cantilever walls, were the predominant types of retaining structures. The principles and use of external walls are well understood, and will not be discussed here. Internally stabilized earth retention systems rely on reinforcement that is installed within the slope and extends

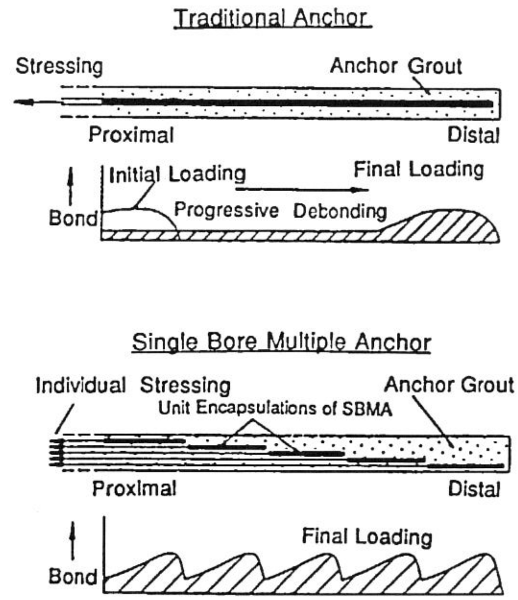


Fig. 18. Schematic illustration of load distributions for a traditional rock anchor and the Single Bore Multiple Anchor (SBMA) system (after Barley 1991).

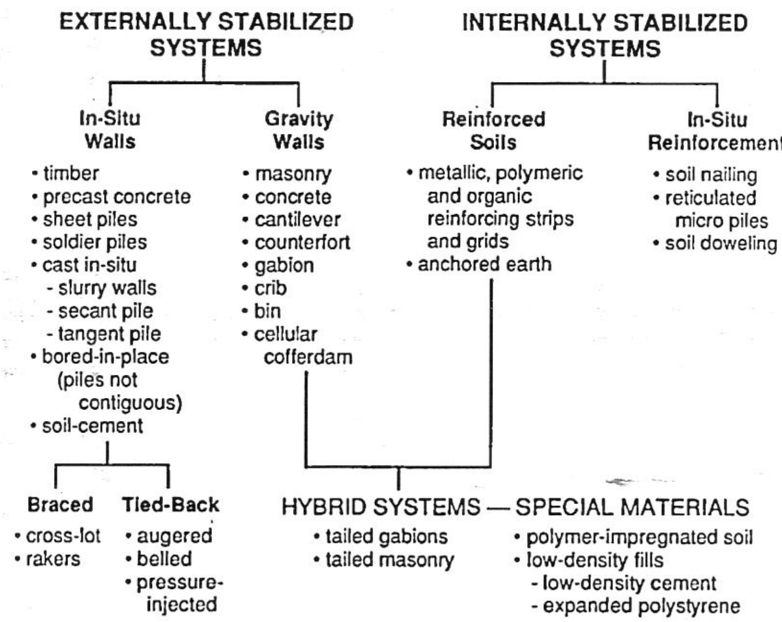


Figure 19. Classification scheme for earth retention systems (after O'Rourke & Jones 1990).

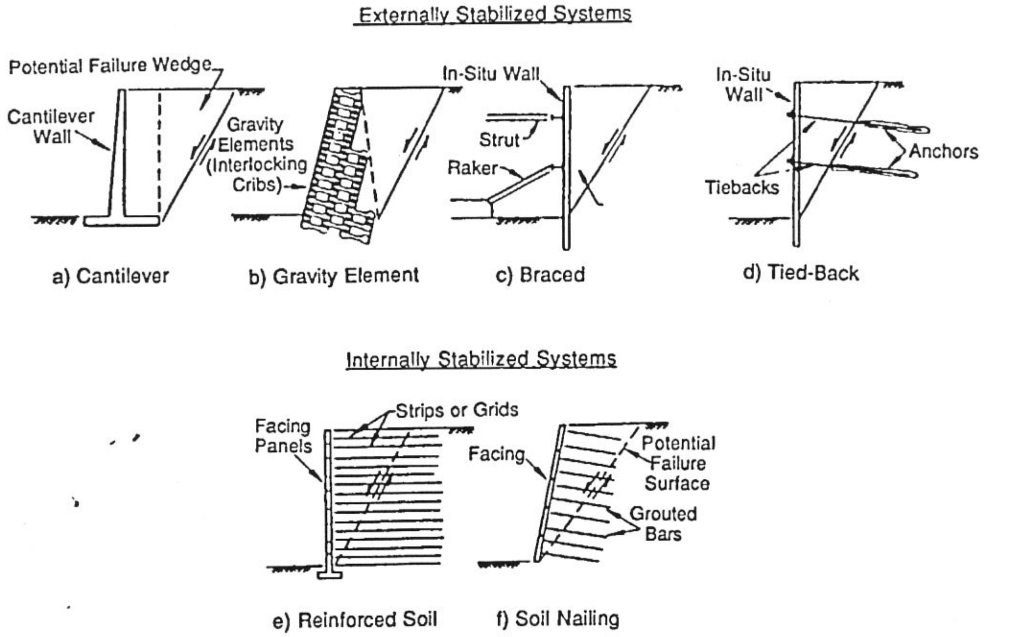


Fig. 20. Examples of externally and internally stabilized earth retention systems (after O'Rourke & Jones 1990).

beyond the potential failure surfaces into stable ground. General references on this topic are Jones (1985), Mitchell & Villet (1987), Christopher et al. (1989 & 1990), Mitchell & Christopher (1990), and O'Rourke & Jones (1990). This section will deal with advances in the use of these internally stabilized earth retention systems, which are known generically as "reinforced soil."

4.2.1 Reinforcement of soil slopes and embankments by internal stabilization

Earth reinforcement (reinforced soil), which can be defined as the inclusion of resistant elements in a soil mass to improve its overall strength, has emerged over the past 25 years as a technically attractive and cost-effective technique for extending the use of soil as a stable construction and slope-forming material. Internal reinforcement can be used to stabilize natural slopes or the slopes of embankments, or to retain excavations. Reinforced soil structures have the following advantages over traditional retaining walls: (1) they are coherent and flexible, and thus are tolerant of large deformations, (2) a wide range of backfill materials can be used, (3) they are easy to construct, (4) they are resistant to seismic loadings, (5) the variety of available facing types makes possible aesthetically pleasing structures, and (6) they are often less costly than conventional retaining structures or piles (Mitchell & Villet 1987). Steep slopes of reinforced soil reduce the required width of new transportation rights of way and are especially suitable for the widening of existing constricted rights of way.

The modern concept of earth reinforcement was originated by Professor Arthur Casagrande, who proposed reinforcing weak soils by laying high-strength membranes between layers of soil (Westergaard 1938). Internally stabilized earth retention systems rely on transfer of shear forces to mobilize the tensile capacity of closely spaced reinforcing elements. Advances in this concept have led to increased use of internal reinforcing elements, either by incremental burial to create reinforced soils (Fig. 21) or by systematic in-situ installation of reinforcing elements, such as soil nails. The common types of inclusions are steel strips, steel or polymeric grids, geotextile sheets, and steel nails, that are capable of withstanding tensile loads, and, in some cases, shear and bending stresses as well (Mitchell & Christopher 1990). The two main mechanisms of stress transfer between the reinforcement and the soil are: (1) friction

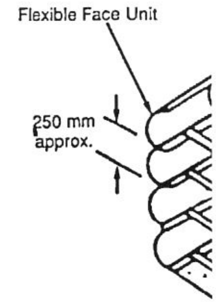


Fig. 21. Examp
strip reinforcing
Jones 1990).

between the soil
reinforcement, a
resistance on reir
oriented normal
movement betwee
soil.
Finite-element
(1990) and centr
Jaber et al. (
reinforced soil w
mats, geogrids,
have indicated
predicted and
reinforcment elen
As coherent, b
reinforced soil
well suited for us
(Mitchell & Villet
degree of struct
the dynamic ener

Table 1. Sumr
constructed in N

Wall height
Low < 3 m
Medium 3-7 m
High > 7 m
Total Number (Total Percent)

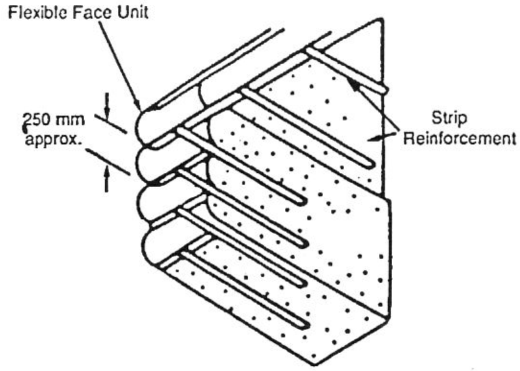


Fig. 21. Example of incremental burial of strip reinforcing elements (after O'Rourke & Jones 1990).

between the soil and the surface areas of the reinforcement, and (2) passive soil bearing resistance on reinforcement surfaces that are oriented normal to the direction of relative movement between the reinforcement and the soil.

Finite-element studies by Adib et al. (1990) and centrifuge model experiments by Jaber et al. (1990) of four types of reinforced soil walls (using steel strips, bar mats, geogrids, and non-woven geotextiles) have indicated good agreement between predicted and measured stresses in reinforcment elements.

As coherent, but flexible, gravity masses, reinforced soil structures are particularly well suited for use in seismically active areas (Mitchell & Villet 1987). They provide a high degree of structural damping that absorbs the dynamic energy associated with earthquakes. Seismic design of reinforced soil systems can be based on numerical calculations and shaking-table model tests (Bonaparte et al. 1986; Segrestin and Bastick 1988).

An important recent element in earth reinforcement is the use of geosynthetics as the reinforcing components. As noted earlier, geosynthetics include geotextiles, geogrids, geonets, geomembranes, and geocomposites. Each of these classes of geosynthetics is currently being used for soil reinforcement or slope drainage. A limitation of geosynthetics as reinforcement is that they possess low stiffness (relative to steel); consequently the amount of deformation required to attain maximum shear strength can exceed the allowable deformation of the soil structure.

All materials used as elements in soil reinforcement are subject to deterioration with time when exposed to soil, groundwater, and the elements. Elias (1990) has discussed the durability/corrosion of soil-reinforced structures in detail.

Table 1 summarizes the use of geosynthetically reinforced permanent retaining walls in North America through 1987. Most of the recent walls have used geogrids as the primary means of reinforcement (Koerner 1990).

As suggested by Mitchell & Villett (1987) and Christopher et al. (1990), this section is divided into placed soil reinforcement systems: (1) strip reinforcement, (2) sheet reinforcement (3) grid, bar, and mesh reinforcement, (4) placed soil-anchor reinforcement, and (5) fiber reinforcement, and in-situ soil reinforcement systems: (1) soil nailing, (2) inserted soil anchors, and (3) root piles.

Table 1. Summary of types of geosynthetically reinforced permanent retaining walls constructed in North America through 1987 (after Yako & Christopher 1988).

Wall height	Woven geotextile	Nonwoven geotextile	Geogrid	Other
Low < 3 m	4	3	5	1
Medium 3-7 m	5	7	14	1
High > 7 m	1	1	4	0
Total Number (Total Percent)	10 (22%)	11 (24%)	23 (50%)	2 (4%)

Fig. 22. Tiered Reinforced Earth wall on Vail Pass, Interstate Highway 70, Colorado. (Photograph courtesy of Reinforced Earth Company, MacLean, Virginia.)

4.2.1.1 Placed soil reinforcement systems

4.2.1.1.1 Strip reinforcement

In strip reinforcement systems, a coherent strengthened material is formed by placing metal or geosynthetic strips horizontally between successive backfill layers. The modern concept of soil reinforcement by means of galvanized steel strips was introduced by the French architect and engineer Henri Vidal in the early 1960's. Vidal named his development "Terre Armee," or "Reinforced Earth," terms that have become generic in many countries, being used to describe all forms of soil reinforcement. However, in some countries, including the United States and Canada, Reinforced Earth is trademarked (Jones 1985). As of 1991, 16,000 Reinforced Earth walls with a total face area of 9,600,000 m^2 have been constructed worldwide (McKittrick, D., 1991, personal communication, Reinforced Earth Company, McLean, Virginia). Schlosser (1990) has noted that 33 percent of the Reinforced Earth wall area in the world has been built in Europe and 34 percent in the United States (e.g., Fig. 22) and Canada.

The introduction of Reinforced Earth by Vidal led to rapid development of the concept of soil reinforcement. Much fundamental research was sponsored by government agencies, notably the Laboratoire des Ponts et Chaussees in France, the United States Department of Transportation, and the United Kingdom Department of Transport. Recently, Yoo and Ko (1991) have conducted centrifuge tests on Reinforced Earth models to investigate the behavior and failure mechanisms of walls subjected to self loading and to surcharge. Experiments were conducted by changing strip lengths, materials, and geometries. Test results were analyzed and compared to various current design methods to verify design feasibility for Reinforced Earth.

Vidal originally proposed the use of fiber-glass-reinforced polymers as the strips in Reinforced Earth (Schlosser 1990). However, in 1966 an experimental wall using fiber-glass-reinforced plastic strips failed after 10 months, apparently as the result of bacterial attack. This resulted in the use of stainless steel and aluminum strips for Reinforced Earth walls built in France. However, as of 1990 (10-15 years after construction), a large number of the stainless steel strips and some of the aluminum strips were corroded, indicating that these materials were not sufficiently resistant to corrosion to be used in soil. As a result, all Reinforced Earth walls currently are constructed using galvanized steel strips. However, even galvanized steel is subject to corrosion, and is thus restricted to use as reinforcement in cohesionless, granular, free-draining backfills to reduce the potential for chemical and water attack (Carroll & Richardson 1986). As one result, epoxy-coated steel reinforcements have been developed that offer potential for high resistance to corrosion.

In recent years non-metallic reinforcing materials, such as geotextiles, fiberglass, plastics, and composites, have been used extensively for soil reinforcement. These materials do not corrode, but may undergo other chemical and/or biological forms of deterioration. Many of these materials are new, and the effects of long-term burial and exposure to the elements are not well known (Elias 1990). For this reason, research is currently being undertaken on their weathering characteristics.

In 1973, the first polymeric strips were introduced in construction of a highway retaining wall in Yorkshire, United Kingdom (Holtz 1978). The reinforcing strips were made of continuous glass fibers embedded lengthwise in a protective coating of resin. An example of a recently marketed geosynthetic strip is the Paraweb strip (Fig. 23), in which the fibers are made of high-tenacity polyester or polyamarid.

4.2.1.1.2 Shee

Sheet reinfo
geotextiles plac
of embankmen
reinforced soi
transfer and
predominantly
Christopher et
Ltd. & Geoserv
geotextiles wit
properties and
be used, includ
or heat-bonded
and woven p
(Christopher e
the geotextil
reinforcement
polypropylene
from silty sand

Fig. 23. Sche
strip reinforc
Villet 1987).

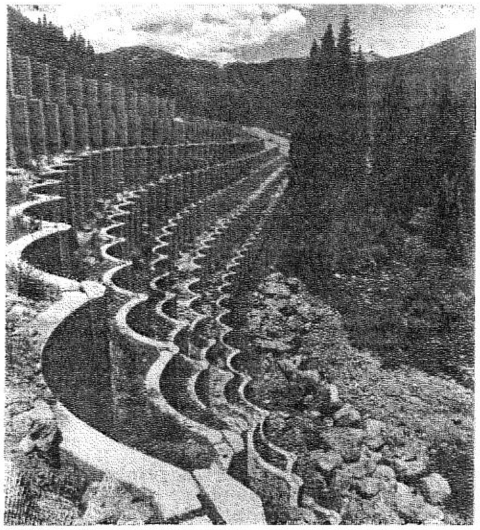


Fig. 22. Tiered Reinforced Earth wall on Vail Pass, Interstate Highway 70, Colorado. (Photograph courtesy of Reinforced Earth Company, MacLean, Virginia.)

4.2.1.1 Placed soil reinforcement systems

4.2.1.1.1 Strip reinforcement

In strip reinforcement systems, a coherent strengthened material is formed by placing metal or geosynthetic strips horizontally between successive backfill layers. The modern concept of soil reinforcement by means of galvanized steel strips was introduced by the French architect and engineer Henri Vidal in the early 1960's. Vidal named his development "Terre Armee," or "Reinforced Earth," terms that have become generic in many countries, being used to describe all forms of soil reinforcement. However, in some countries, including the United States and Canada, Reinforced Earth is trademarked (Jones 1985). As of 1991, 16,000 Reinforced Earth walls with a total face area of 9,600,000 m^2 have been constructed worldwide (McKittrick, D., 1991, personal communication, Reinforced Earth Company, McLean, Virginia). Schlosser (1990) has noted that 33 percent of the Reinforced Earth wall area in the world has been built in Europe and 34 percent in the United States (e.g., Fig. 22) and Canada.

The introduction of Reinforced Earth by Vidal led to rapid development of the concept of soil reinforcement. Much fundamental research was sponsored by government agencies, notably the Laboratoire des Ponts et Chaussees in France, the United States Department of Transportation, and the United Kingdom Department of Transport. Recently, Yoo and Ko (1991) have conducted centrifuge tests on Reinforced Earth models to investigate the behavior and failure mechanisms of walls subjected to self loading and to surcharge. Experiments were conducted by changing strip lengths, materials, and geometries. Test results were analyzed and compared to various current design methods to verify design feasibility for Reinforced Earth.

Vidal originally proposed the use of fiber-glass-reinforced polymers as the strips in Reinforced Earth (Schlosser 1990). However, in 1966 an experimental wall using fiber-glass-reinforced plastic strips failed after 10 months, apparently as the result of bacterial attack. This resulted in the use of stainless steel and aluminum strips for Reinforced Earth walls built in France. However, as of 1990 (10-15 years after construction), a large number of the stainless steel strips and some of the aluminum strips were corroded, indicating that these materials were not sufficiently resistant to corrosion to be used in soil. As a result, all Reinforced Earth walls currently are constructed using galvanized steel strips. However, even galvanized steel is subject to corrosion, and is thus restricted to use as reinforcement in cohesionless, granular, free-draining backfills to reduce the potential for chemical and water attack (Carroll & Richardson 1986). As one result, epoxy-coated steel reinforcements have been developed that offer potential for high resistance to corrosion.

In recent years non-metallic reinforcing materials, such as geotextiles, fiberglass, plastics, and composites, have been used extensively for soil reinforcement. These materials do not corrode, but may undergo other chemical and/or biological forms of deterioration. Many of these materials are new, and the effects of long-term burial and exposure to the elements are not well known (Elias 1990). For this reason, research is currently being undertaken on their weathering characteristics.

In 1973, the first polymeric strips were introduced in construction of a highway retaining wall in Yorkshire, United Kingdom (Holtz 1978). The reinforcing strips were made of continuous glass fibers embedded lengthwise in a protective coating of resin. An example of a recently marketed geosynthetic strip is the Paraweb strip (Fig. 23), in which the fibers are made of high-tenacity polyester or polyamarid.

4.2.1.1.2 Sheet

Sheet reinfor
geotextiles place
of embankment
reinforced soil;
transfer and
predominantly fr
Christopher et
Ltd. & Geoservi
geotextiles with
properties and e
be used, includi
or heat-bonded p
and woven po
(Christopher et
the geotextile
reinforcement ar
polypropylene fa
from silty sand t

Fig. 23. Schem
strip reinforce
Villet 1987).

ire des Ponts
Jnited States
n, and the
f Transport.
ve conducted
Earth models
and failure
o self loading
iments were
:ip lengths,
t results were
'ious current
gn feasibility

use of fiber-
the strips in
ser 1990).
tal wall using
strips failed
the result of
in the use of
ı strips for
in France.
years after
ber of the
some of the
d, indicating
sufficiently
d in soil. As
alls currently
anized steel
ized steel is
restricted to
cohesionless,
lls to reduce
water attack
s one result,
its have been
al for high

reinforcing
, fiberglass,
e been used
ient. These
may undergo
cal forms of
materials are
rm burial and
ot well known
, research is
ı on their

strips were
f a highway
ited Kingdom
strips were
rs embedded
ing of resin.
y marketed
b strip (Fig.
ade of high-
d.

4.2.1.1.2 Sheet reinforcement

Sheet reinforcement commonly consists of geotextiles placed horizontally between layers of embankment soils to form a composite reinforced soil; the mechanism of stress transfer and sheet reinforcement is predominantly friction (Mitchell & Villet 1987; Christopher et al. 1989; STS Consultants, Ltd. & Geoservices Inc, 1990). A variety of geotextiles with a wide range of mechanical properties and environmental resistance can be used, including nonwoven needle-punched or heat-bonded polyester and polypropylene, and woven polypropylene and polyester (Christopher et al. 1989). However, most of the geotextile fabrics used in earth reinforcement are made of either polyester or polypropylene fabrics. Granular soil ranging from silty sand to gravel commonly is used as backfill. Facing elements are formed by wrapping the geotextile around the soil at the face (Fig. 24) and covering the exposed fabric with gunite, asphalt emulsion, or shotcrete, or with soil and vegetation, for long-term protection from ultraviolet light and vandalism. Typical applications of geotextile-reinforced walls include slope stabilization on mountain roads and retaining walls for temporary or permanent road widening or diversion.

The use of geotextiles in reinforced soil walls resulted from the beneficial effect of geotextile reinforcement in highway embankments over weak subgrades. The first geotextile-reinforced wall was built in France in 1971. The first full-size fabric retaining wall in the United States was a 3.3-m-high wall (Fig. 25) built by the U.S. Forest Service to reconstruct a failed road fill

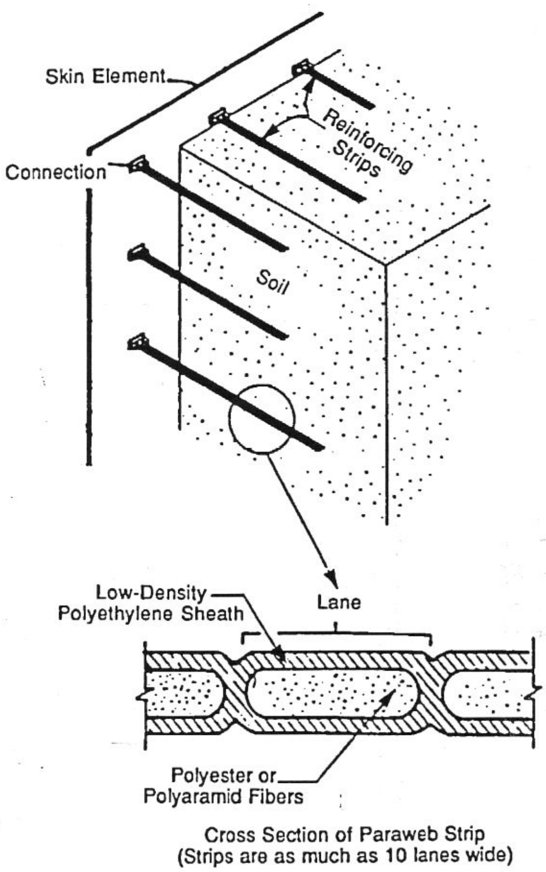


Fig. 23. Schematic diagram of a nonmetallic-strip reinforced soil wall (after Mitchell & Villet 1987).

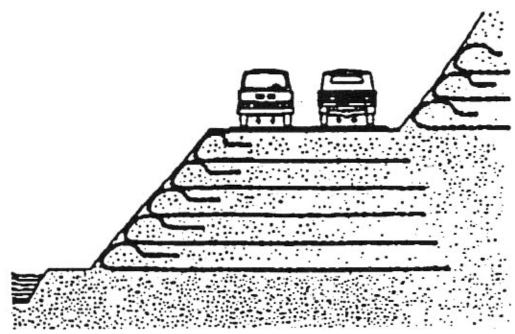


Fig. 24. Schematic cross section of a geotextile reinforced soil wall (after STS Consultants, Ltd. & Geoservices Inc 1990).

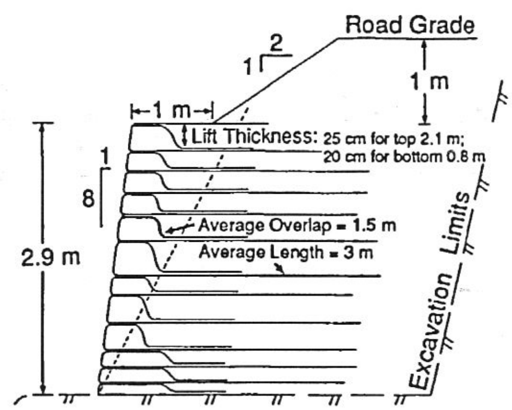


Fig. 25. Geotextile reinforced and faced wall, Siskiyou National Forest, Oregon (after Bell & Steward 1977).

in Siskiyou National Forest, Oregon; the fabric for this wall was a synthetic, nonwoven, needle-punched, spunbonded polypropylene (Bell & Steward 1977). This fabric was permeable and, when buried, was resistant to rotting. However, it was subject to deterioration when exposed to ultraviolet light; therefore, the wall was finished with a gunite facing to protect it from sunlight.

During the 1980's, studies were conducted in various countries to determine the effects of outdoor exposure on geotextiles used for slope reinforcement (Elias 1990). One of the most comprehensive of these studies was carried out by the Hong Kong Geotechnical Control Office to determine the outdoor-exposure performance of 14 geotextiles subjected to Hong Kong conditions (Brand & Pang 1991). All of the geotextiles (12 nonwoven, one woven, and one composite) lost strength and became brittle due to the combined influence of sunlight, temperature, rainfall, wind, oxygen, and atmospheric pollution. The average loss of strength in the first month was less than 16 percent. Long-term performance varied widely, with some geotextiles losing virtually all of their strength after 6 months of exposure. These data confirm that use of geotextiles for slope stabilization should be limited to short-term projects if they are not protected from the elements.

4.2.1.1.3 Grid, bar, and mesh reinforcement

Grid reinforcement systems consist of polymer or metallic elements arranged in rectangular grids, metallic bar mats, and wire mesh. The two-dimensional grid/soil interaction involves both friction along longitudinal members and passive bearing resistance against the transverse members. Because of the passive resistance developed on the cross members, grids are more resistant to pullout than strips; however, full passive resistance develops only for relatively large displacements (5-10 cm) (Schlosser 1990).

The greatest advance in development and use of rectangular grids for soil reinforcement has been in the area of polymeric "geogrids." As defined by Koerner (1990), a geogrid is "a deformed or non-deformed gridlike polymeric material formed by intersecting ribs joined at the junctions used for reinforcement with foundation, soil, rock, earth, or any other geotechnical engineering-related material as an integral part of a human-made project structure or system". Geogrids are relatively stiff, netlike materials with open spaces called "apertures," that usually measure 1-10 cm between the ribs. The first use of polymer grids was by Japanese engineers in the 1960's to reinforce subgrade soils for railway embankments (Jones 1985). Because these original grids were made of non-oriented polymers, they were relatively fragile, lacking the necessary tensile strength to serve as slope reinforcement. In the 1970's, advances in the formulation of polymers led to significant improvements in strength and stiffness of geogrids. In the late 1970's, geogrids were developed with oriented polymers, which provided increased directional strength. These new geogrids were used in 1979 in construction of a reinforced soil wall at a railroad station in Yorkshire, United Kingdom (O'Rourke & Jones 1990). In 1981, the development of soil reinforcement advanced into a new area of application when synthetic grid materials were used to repair failures in cuts on the M1 and M4 motorways in England (Jones 1985).

In 1983, the first geogrid wall in the United States was built to stabilize a landslide on the Oregon coast (Szymoniak et al., 1984). This 9-m-high geogrid wall with a face slope of 80° was selected over other alternatives because (1) it had the lowest estimated cost, and (2) the open face of the grid wall allowed establishment of vegetation, which provided a natural appearance compatible with the surroundings of an adjacent state park. The geogrid was a high-density polethylene stabilized with carbon black to provide resistance to ultra-violet light. At about the same time, Forsyth and Bieber (1984) reported on the construction of a geogrid wall to reinforce a California slope 9.5 m high with a slope angle of 48°. This wall was built along State Highway 84 near La Honda to repair a small slide. Since the construction of these early walls, more than 300 polymeric geogrid wall and slope projects have been constructed in the United States (Mitchell & Christopher 1990).

Manufacturing processes have evolved to the point where strong and durable geosynthetic soil reinforcing elements can be mass produced. The most familiar products in earth retention systems are high density polyethylene (HDPE) and polypropylene grids (O'Rourke & Jones 1990). An example is Tensar (Fig. 26), a proprietary plastic grid reinforcement developed in the United Kingdom in the early 1980's.

Several proprietary and non-proprietary systems of bar and mesh reinforcement have been developed that rely on both frictional and passive resistance to pullout. In 1974, the first "bar-mat" system of soil reinforcement was developed by the California Department of Transportation

(Caltrans) to con
Interstate High
California; these
cross-linking ste
coarse "welded-
1978). For propr
bar-mesh reinf
designated "M
Embankment" (M
Reinforced Eart
difficulties with
deterioration of t
Recent centrifug
and Elgamal (19
potential failure
localized deterio
strips in an effo
effects of this
strength.

Evolving in t
Caltrans proje
techniques: Hilf
Hilfiker Reinfor
Retained Earth,
Embankment Sys

Fig. 26. Sche
reinforced wall
geogrid reinfor
backfill.

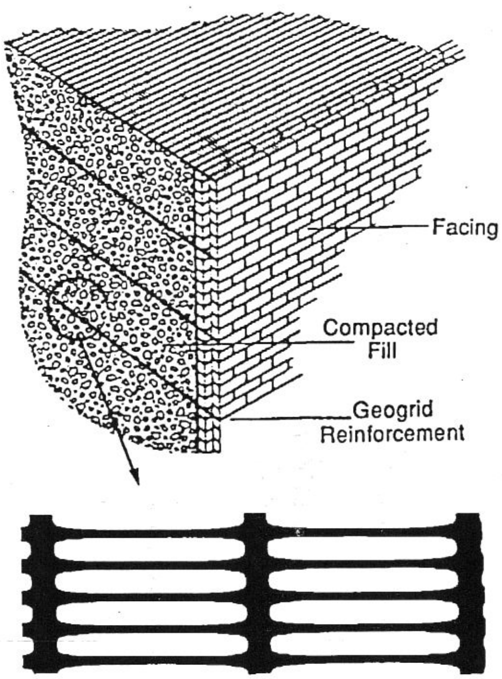
bs. The first
by Japanese
orce subgrade
(Jones 1985).
were made of
ere relatively
sary tensile
orcement. In
ormulation of
provements in
grids. In the
eveloped with
ded increased
new geogrids
ruction of a
ad station in
(O'Rourke &
lopment of soil
new area of
rid materials
cuts on the M1
Jones 1985).
wall in the
ize a landslide
niak et al.,
d wall with a
d over other
d the lowest
en face of the
of vegetation,
appearance
dings of an
eogrid was a
bilized with
ance to ultra-
time, Forsyth
ed on the
to reinforce a
a slope angle
along State
repair a small
of these early
geogrid wall
onstructed in
Christopher

ve evolved to
and durable
ements can be
iliar products
high density
polypropylene
. An example
ietary plastic
in the United

n-proprietary
rcement have
oth frictional
ut. In 1974,
em of soil
ed by the
ransportation

(Caltrans) to construct a 6-m-high wall along Interstate Highway 5 near Dunsmuir, California; these crude grids were formed by cross-linking steel reinforcing bars to form a coarse "welded-wire" bar mat (Forsyth 1978). For proprietary reasons, the Caltrans bar-mesh reinforcement technique was designated "Mechanically Stabilized Embankment" (MSE), in agreement with the Reinforced Earth Company. One of the difficulties with MSE in the field has been deterioration of the bar-mesh reinforcement. Recent centrifuge model studies by Ragheb and Elgamal (1991) have investigated the potential failure mechanisms associated with localized deterioration of MSE reinforcement strips in an effort to better understand the effects of this deterioration on long-term strength.

Evolving in the United States from the Caltrans project were other similar techniques: Hilfiker Welded Wire Wall and Hilfiker Reinforced Soil Embankment, VSL Retained Earth, and the Georgia Stabilized Embankment System.



Plan View of Tensar Geogrid Reinforcement

Fig. 26. Schematic diagram of a geogrid reinforced wall, and plan view of Tensar geogrid reinforcing element as placed in the backfill.

Hilfiker Welded Wire Wall (WWW) uses welded-wire reinforcing mesh of the type that is commonly placed in concrete slabs; the facing is a continuation of the horizontal mesh reinforcement. The material is fabricated in 2.4-m-wide mats with grid spacing of 15 x 61 cm. To the casual observer, WWW may appear to be a type of gabion wall. However, gabion walls, which are gravity walls made by encasing coarse-grained fill in wire baskets, are based on the principle of confinement and gravity retainment rather than on internal tensile reinforcement (Hausmann 1990). The first commercial WWW was built for the the Southern California Edison Power Company in 1977 for road repair along a power line in the San Gabriel Mountains of southern California. By 1980, the use of WWW expanded to larger projects, such as a 250-m-long, 5-m-high wall built by the Union Oil Company at their Parachute Creek oil-shale development in Colorado (Mitchell & Villet 1987). During the 1980's, the use of WWW for retaining structures expanded rapidly; by 1990 about 1600 WWW projects had been completed in the United States (Mitchell & Christopher 1990).

The Hilfiker Reinforced Soil Embankment system (RSE), which resembles Caltrans Mechanically Stabilized Embankment, is a continuous welded-wire reinforcement system with precast concrete facing. It was introduced commercially in 1983 on New Mexico State Highway 475 northeast of Santa Fe, where four reinforced soil stuctures were built with a total of 1600 m^2 of wall facing. By 1990, more than 50 additional RSE walls had been constructed in the United States (Mitchell & Christopher 1990).

VSL Retained Earth utilizes strips of steel-grid ("bar-mat") reinforcement that is bolted to hexagonal precast concrete panels. The first VSL Retained Earth wall in the United States was constructed in Hayward, California, in 1983. By 1990, more than 600 VSL Retained Earth walls with some 465,000 m^2 of wall facing had been built in the United States (Mitchell & Christopher 1990). The system is licensed in the United States under a Reinforced Earth patent, but it uses its own patented system for connecting the bar-mat reinforcement to the concrete facing panels.

The Georgia Stabilized Embankment, which was recently developed by the Georgia Department of Transportation, is another steel-grid, or bar-mat, reinforcing system, that has seen extensive use in the United States. It is licensed in the United States under a proprietary agreement with the Reinforced Earth Company.

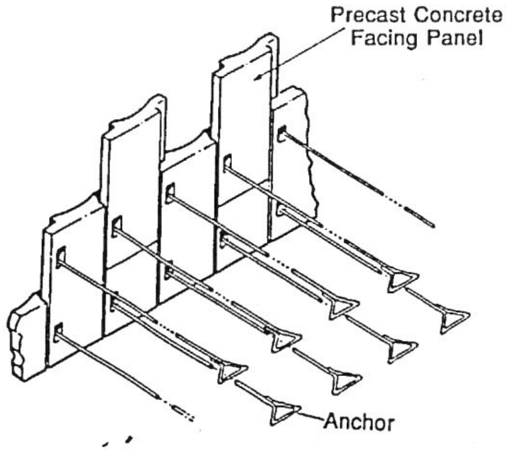


Fig. 27. Schematic diagram of an Anchored Earth retaining wall (after Murray & Irwin 1981).

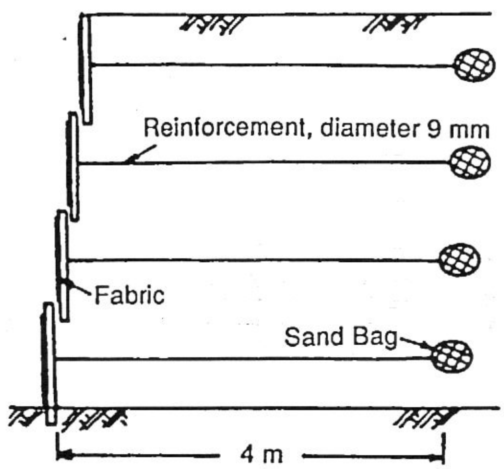


Fig. 28. Schematic cross section through proposed fabric-faced reinforced-soil wall with multiple tie bars attached to bags of sand that serve as anchors. The rods and anchors are embedded in the embankment during construction. (After Fukuoka 1986).

4.2.1.1.4 Emplaced earth-anchor reinforcement

Embankment slopes can be reinforced during construction by emplacement of slender steel rods bent at one end to form "anchors." This type of retaining wall is still in an experimental stage. Soil-to-rod stress transfer is mainly by means of passive resistance on the "anchor," which implies that the system provides stability in the same manner as tied-back retaining structures, and thus is not truly a reinforced soil system (Mitchell & Villet 1987). However, the system is discussed here because it is analogous in placement technique to other methods of soil reinforcement in embankments.

The concept of emplaced earth-anchor reinforcement was developed and patented by the Transport and Road Research Laboratory (TRRL) of the United Kingdom as "Anchored Earth" (Murray & Irwin 1981). The reinforcement consists of 16- to 20-mm-diameter mild-steel bars. The outer end of each bar is threaded to fit into concrete facing panels; the other end is formed into an anchor in the form of a "Z" or triangle (Fig. 27). Unlike other soil reinforcement methods described here, which are based on the premise that frictional stress develops along the entire length of the reinforcement, Anchored Earth is designed to rely only on passive resistance developed against the deformed ends ("anchors") of the reinforcing bars. Centrifuge model tests of Anchored Earth were conducted recently by Craig et al. (1991) to better understand the Anchored Earth concept at prototype stress levels. In the centrifuge tests, different stress paths were used to approach failure of the models, either by gravity, by increasing body forces to induce collapse, or by applying external loading. Because Anchored Earth is still in the research and developmental stages, none of its applications can be considered to be routine. However, it does appear to be a promising approach to soil reinforcement. It is likely that Anchored Earth will prove most beneficial on projects where clean granular backfill is not available.

A similar concept was used successfully in Japan in the late 1970s to construct a 5-m-high fabric-faced retaining wall with multiple anchors (Fukuoka & Imamura 1982). Each of the 20-mm-diameter steel tie bars was attached to a 40 x 40 cm concrete plate embedded in the backfill soil. Fukuoka (1986) has proposed a similar system in which the tie bars are attached to bags of sand that are embedded in fill during construction (Fig. 28).

4.2.1.1.5 Fiber reinforcement

Fiber reinforcement of backfill soil, which is analogous to fiber reinforcement of concrete, is still in the developmental stage. Materials being investigated for possible fiber reinforcement include synthetic fibers (geotextile threads), metallic fibers (metal threads), and natural fibers (reeds and other plants) (Mitchell & Villett 1987). A recent innovation is a three-dimensional

reinforcement
in France at
Chaussees (L
The techni
impregnation
mixing the
polymer filam
and a tensile
1982). Appro
composite ma
consists of
length of rei
meter of rein
wall was buil
1988). By
projects had
100,000 m
(Schlosser
demonstrated
resistance to
difficulties as
the fibers w
process mus
inclusion can
and routin
reinforcement
Another ma
for use as fi
which is one
replenishable
can also be u
other types o
has presente
durability of

4.2.1.2 In-si

4.2.1.2.1 Soi

Soil "nails"
metal tubes t
or soft rock
boreholes. T
coherent rein
stopping the
(Fig. 29)
excavations
tieback supp
passive eleme
as tiebacks
more closely
nail is used
surface area.
between the
thin layer (1
with wire me
rigid element
or by prefab
may be cove
can be used t
unstable slo
slopes, whe

; structures,
ed soil system
r, the system
analogous in
ethods of soil

earth-anchor
d patented by
h Laboratory
as "Anchored
981). The
- to 20-mm-
outer end of
into concrete
ormed into an
riangle (Fig.
ment methods
ased on the
evelops along
einforcement,
rely only on
against the
e reinforcing
of Anchored
by Craig et
the Anchored
ss levels. In
stress paths
f the models,
g body forces
ying external
th is still in
stages, none
idered to be
pear to be a
orcement. It
ll prove most
ean granular

ccessfully in
truct a 5-m-
with multiple
82). Each of
e bars was
ncrete plate
l. Fukuoka
stem in which
of sand that
construction

soil, which is
t of concrete,
ge. Materials
ssible fiber
etic fibers
bers (metal
(reeds and
tt 1987). A
e-dimensional

reinforcement technique that was developed in France at the Laboratoire des Ponts et Chaussees (LCPC) in 1980 (Schlosser 1990). The technique consists of polymer impregnation of granular backfill soil by mixing the soil with a small continuous polymer filament with a diameter of 0.1 mm and a tensile strength of 10 kN (Leflaive 1982). Approximately 0.1-0.2 percent of the composite material (known as "Texsol") consists of filament, resulting in a total length of reinforcement of 200 m per cubic meter of reinforced soil. The first Texsol wall was built in France in 1983 (Leflaive 1988). By the end of 1988, 85 Texsol projects had been completed in France, using 100,000 m^3 of Texsol-reinforced soil (Schlosser 1990). These walls have demonstrated high bearing capacity and resistance to erosion. However, there are difficulties associated with efficiently mixing the fibers with the backfill; the mixing process must be perfected before fiber inclusion can become an economically feasible and routinely used means of soil reinforcement.

Another material that has been suggested for use as fiber reinforcement is bamboo, which is one of the fastest growing and most replenishable biological materials. Bamboo can also be used as continuous elements in other types of reinforced soil. Fang (1991) has presented data on the strength and durability of bamboo as soil reinforcement.

4.2.1.2 In-situ soil-reinforcement systems

4.2.1.2.1 Soil nailing

Soil "nails" are steel bars, metal rods, or metal tubes that are driven into in-situ soil or soft rock or are grouted into predrilled boreholes. Together with the soil, they form coherent reinforced soil structures capable of stopping the movement of unstable slopes (Fig. 29) or of supporting temporary excavations (Fig. 30). Nailing differs from tieback support systems in that the nails are passive elements that are not post-tensioned as tiebacks are, and the nails are spaced more closely than tiebacks. Commonly one nail is used for each 1 to 6 m^2 of ground-surface area. Stability of the ground surface between the nails commonly is provided by a thin layer (10-15 cm) of shotcrete reinforced with wire mesh (Fig. 30), by intermittent rigid elements similar to large steel washers, or by prefabricated steel panels (which later may be covered by shotcrete). Soil nailing can be used to restrain two different types of unstable slopes: (1) potentially unstable slopes, where little or no movement is

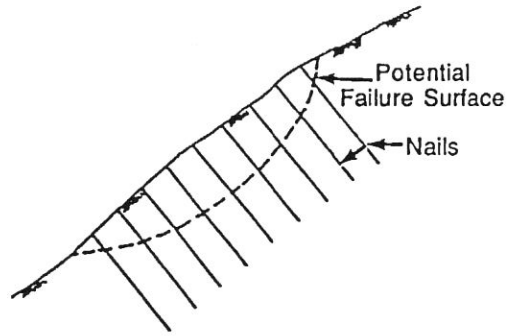


Fig. 29. Schematic cross section of soil nailing for slope stabilization.

Fig. 30. Soil nails extruding from shotcrete surface of temporary excavation for tunnel portal, Interstate Highway 70, Glenwood Canyon, Colorado.

occurring, but where safety factors are low enough to indicate a strong possibility for future movement, and (2) creeping slopes, in which movement is actually occurring.

Soil nailing has been used for slope stabilization for nearly 20 years. In North America, the system was first used in Vancouver, Canada, in the early 1970's for temporary excavation support. In Europe, the earliest reported soil-nailing projects were for retaining wall construction in Spain

; structures,
ed soil system
er, the system
analogous in
ethods of soil

earth-anchor
d patented by
h Laboratory
as "Anchored
981). The
- to 20-mm-
outer end of
into concrete
formed into an
triangle (Fig.
ment methods
based on the
evelops along
einforcement,
rely only on
against the
e reinforcing
of Anchored
by Craig et
the Anchored
ss levels. In
stress paths
f the models,
g body forces
ying external
th is still in
stages, none
idered to be
pear to be a
orcement. It
ill prove most
ean granular

ccessfully in
truct a 5-m-
with multiple
82). Each of
e bars was
ncrete plate
l. Fukuoka
stem in which
of sand that
construction

soil, which is
t of concrete,
ge. Materials
ssible fiber
netic fibers
bers (metal
(reeds and
tt 1987). A
e-dimensional

reinforcement technique that was developed in France at the Laboratoire des Ponts et Chaussees (LCPC) in 1980 (Schlosser 1990). The technique consists of polymer impregnation of granular backfill soil by mixing the soil with a small continuous polymer filament with a diameter of 0.1 mm and a tensile strength of 10 kN (Leflaive 1982). Approximately 0.1-0.2 percent of the composite material (known as "Texsol") consists of filament, resulting in a total length of reinforcement of 200 m per cubic meter of reinforced soil. The first Texsol wall was built in France in 1983 (Leflaive 1988). By the end of 1988, 85 Texsol projects had been completed in France, using 100,000 m^3 of Texsol-reinforced soil (Schlosser 1990). These walls have demonstrated high bearing capacity and resistance to erosion. However, there are difficulties associated with efficiently mixing the fibers with the backfill; the mixing process must be perfected before fiber inclusion can become an economically feasible and routinely used means of soil reinforcement.

Another material that has been suggested for use as fiber reinforcement is bamboo, which is one of the fastest growing and most replenishable biological materials. Bamboo can also be used as continuous elements in other types of reinforced soil. Fang (1991) has presented data on the strength and durability of bamboo as soil reinforcement.

4.2.1.2 In-situ soil-reinforcement systems

4.2.1.2.1 Soil nailing

Soil "nails" are steel bars, metal rods, or metal tubes that are driven into in-situ soil or soft rock or are grouted into predrilled boreholes. Together with the soil, they form coherent reinforced soil structures capable of stopping the movement of unstable slopes (Fig. 29) or of supporting temporary excavations (Fig. 30). Nailing differs from tieback support systems in that the nails are passive elements that are not post-tensioned as tiebacks are, and the nails are spaced more closely than tiebacks. Commonly one nail is used for each 1 to 6 m^2 of ground-surface area. Stability of the ground surface between the nails commonly is provided by a thin layer (10-15 cm) of shotcrete reinforced with wire mesh (Fig. 30), by intermittent rigid elements similar to large steel washers, or by prefabricated steel panels (which later may be covered by shotcrete). Soil nailing can be used to restrain two different types of unstable slopes: (1) potentially unstable slopes, where little or no movement is

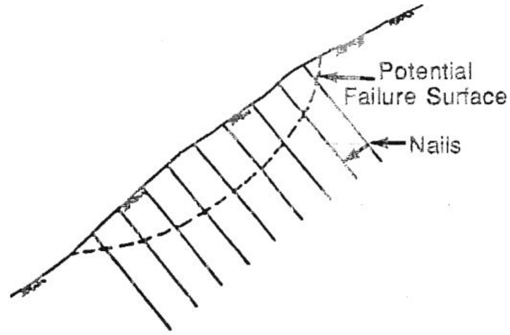


Fig. 29. Schematic cross section of soil nailing for slope stabilization.

Fig. 30. Soil nails extruding from shotcrete surface of temporary excavation for tunnel portal, Interstate Highway 70, Glenwood Canyon, Colorado.

occurring, but where safety factors are low enough to indicate a strong possibility for future movement, and (2) creeping slopes, in which movement is actually occurring.

Soil nailing has been used for slope stabilization for nearly 20 years. In North America, the system was first used in Vancouver, Canada, in the early 1970's for temporary excavation support. In Europe, the earliest reported soil-nailing projects were for retaining wall construction in Spain

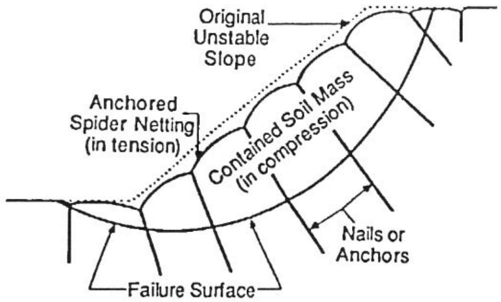


Fig. 31. Schematic cross section of anchored geosynthetic "spider netting" used with soil nails or anchors to stabilize a slope (after Koerner and & Robins 1986).

(1972), France (1973), and Germany (1976), in connection with highway or railway cut-slope construction or temporary support for building excavations (Elias & Juran 1991). Today the technique of soil nailing is widespread in France, Germany, Great Britain, Japan, and the United States.

The stability of soil-nailed reinforcement relies upon: (1) development of friction or adhesion mobilized at the soil-nail interface and (2) passive resistance developed at the face of the nail. Soil nailing is most effective in dense granular soils and low plasticity stiff silty clays. It is generally not cost-effective or practical in the following soils (Mitchell & Christopher 1990):

(1) loose granular soils with standard penetration N values lower than about 10 or relative densities of less than 30 percent,

(2) poorly graded soils with uniformity coefficients of less than 2 (nailing is not practical because of the necessity of stabilizing the cut face prior to excavation),

(3) soft cohesive soils with undrained shear strengths of less than 48 kPa, because of the inability to develop adequate pullout resistance, and

(4) highly plastic clays (PI > 20 percent), due to excessive creep deformation.

Soil nailing currently is used mainly for temporary structures because of the uncertainty of the corrosion rate of steel bars used in the process. However, new types of reinforcements and reinforcement coatings with high resistance to corrosion are being developed. For example, in 1987 fiber-glass nails were used to retain nearly vertical cuts for a freeway tunnel excavation in Reutlingen, southern Germany (Gassler in press).

Analysis of soil nailing and procedures for design have been presented by Juran et al. (1990) and Elias & Juran (1991). To increase the confidence of engineers in the potential use of this method for permanent slope stabilization, additional research is being conducted on field performance of soil-nailed structures (Plumelle et al. 1990; Stocker & Riedinger 1990). In 1986, a 4-year, US$4 million national research program, titled CLOUTERRE, was initiated by the French Minister of Transport to improve the state of knowledge and develop design and construction guidelines for soil-nailed retention systems (Schlosser & Unterreiner in press).

A new method of soil nailing uses geotextiles, geogrids, or geonets to cover the ground surface (Koerner & Robins 1986). The geosynthetic material is reinforced at distinct nodes and anchored to the slope using long rods (soil nails) at the nodes (Fig. 31). When the rods are properly fastened, they pull the surface netting into the soil, placing the net ("spider netting") in tension and the constrained soil in compression.

Soil nailing systems are both flexible and massive, and thus are resistant to seismic loading. An example of this dynamic stability was provided by the lack of damage to three California soil-nailing projects that were located within 33 km of the epicenter of the 1989 magnitude-7.1 Loma Prieta earthquake (Ferworn & Weatherby in press). However, current understanding of the dynamic behavior of soil-nailed earth structures is limited, and research is needed to develop procedures for earthquake-resistant design.

There are no proprietary restrictions on the use of soil nailing. However, some specific systems of nails and/or facing are patented. A recently patented (by Soil Nailing Limited, United Kingdom) soil-nailing technique inserts reinforcing nails into the ground by means of a compressed-air "launcher," which was originally developed in the United Kingdom for military use in shooting projectiles into the air (Bridle and Myles 1991). Under favorable conditions, the launcher can inject 38-mm-diameter nails up to 6 m long into a soil slope at a rate of one every 2-3 minutes.

4.2.1.2.2 Soil anchors

Stabilization of soil slopes by deep prestressed anchors is being used increasingly. Hutchinson (1984) has described the stabilization of a landslide in glacial deposits in southern Wales, United Kingdom (Fig. 32). Prior to treatment in 1980, movements of as much as 15 mm per year were occurring at the head of the slide and of 2-5 mm per year at the toe. Because of severe spatial constraints, anchoring into

the underlying
proved to be the
measure.

4.2.1.2.3 Root p

Another recer
of in-situ soils a
micropiles (comm
A root-pile syste
reinforced soil
critical failure s
difference betwe
soil-nailing sys
provided by root
by their th
geometric arran
1979).
Root piles a
concrete piles w
7.5 to 30 cm. In
these insertions
reinforcing rod
with larger diam
reinforcing bar
reinforcement ((
"Root Piles" a
were originally d
Lizzi and were
Fondedile of Na
installed the sy
underpinning);
expired (Christo
been in the past
been used for sl
root-pile slope-s
constructed with

4.2.2 Stabilizati
admixtures

One means of
engineering sta
chemical admixtu
cement; this ap
many decades fo
stabilization in
used in the mid-1
base materials
problem encount
stabilization has
insertion of the
However, in rece
obtained by mea
lime slurry into
slurry, which fo
bedding surface
weakness, is
diameter pipes fi
(Rogers 1991).
pushed into the
injected to refus

manent slope
ırch is being
: of soil-nailed
90; Stocker &
4-year, US$4
ogram, titled
y the French
ve the state of
design and
· soil-nailed
& Unterreiner

nailing uses
nets to cover
Robins 1986).
reinforced at
to the slope
at the nodes
are properly
e netting into
r netting") in
ıed soil in

ı flexible and
nt to seismic
amic stability
mage to three
s that were
center of the
a earthquake
). However,
the dynamic
structures is
d to develop
stant design.
strictions on
wever, some
or facing are
ted (by Soil
ı) soil-nailing
nails into the
mpressed-air
lly developed
litary use in
r (Bridle and
onditions, the
ıeter nails up
a rate of one

s by deep
sed
(1984) has
ı landslide in
Vales, United
treatment in
ıs 15 mm per
d of the slide
toe. Because
nchoring into

the underlying Coal Measures bedrock proved to be the most effective stabilization measure.

4.2.1.2.3 Root piles

Another recent approach to reinforcement of in-situ soils and soft rocks is the use of micropiles (commonly known as "root piles"). A root-pile system forms a monolithic block of reinforced soil that extends beneath the critical failure surface (Fig. 33). The main difference between root piles and traditional soil-nailing systems is that reinforcement provided by root piles is strongly influenced by their three-dimensional root-like geometric arrangement (Schlosser & Juran 1979).

Root piles are cast-in-place reinforced concrete piles with diameters ranging from 7.5 to 30 cm. In the smaller-diameter range, these insertions are provided with a central reinforcing rod or steel pipe, while those with larger diameters may be provided with a reinforcing bar-cage bound with spiral reinforcement (Christopher et al. 1989).

"Root Piles" and "Reticulated Root Piles" were originally developed in the 1950's by F. Lizzi and were patented by the Italian firm Fondedile of Naples, which introduced and installed the system worldwide (mainly for underpinning); the original patents have now expired (Christopher et al 1989). It has only been in the past 20 years that root piles have been used for slope stabilization, and most root-pile slope-stabilization works have been constructed within the past 10 years.

4.2.2 Stabilization by use of chemical admixtures

One means of improving a soil to meet engineering standards is to mix it with chemical admixtures, such as lime or Portland cement; this approach has been in use for many decades for highway subgrades. Lime stabilization in the United States was first used in the mid-1940's to stabilize clay-gravel base materials for highways. A major problem encountered in using lime for slope stabilization has been to obtain adequate insertion of the lime into the soil at depth. However, in recent years, insertion has been obtained by means of pressure injection of lime slurry into the soil (Fig. 34). The slurry, which follows natural fracture zones, bedding surfaces, and other surfaces of weakness, is injected through 40-mm-diameter pipes fitted with perforated nozzles (Rogers 1991). The pipes are hydraulically pushed into the ground, and the slurry is injected to refusal at depth intervals of 30-45

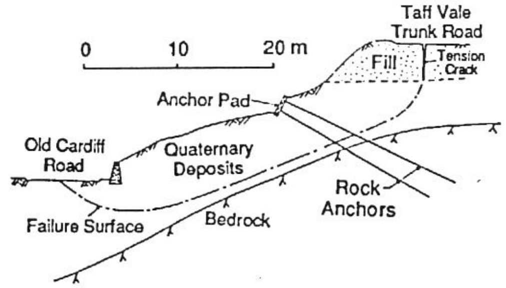


Fig. 32. Cross section of landslide in Quaternary deposits at Nangarw, south Wales, United Kingdom (after Hutchinson 1977). The slide has been stabilized by deep anchors in underlying bedrock.

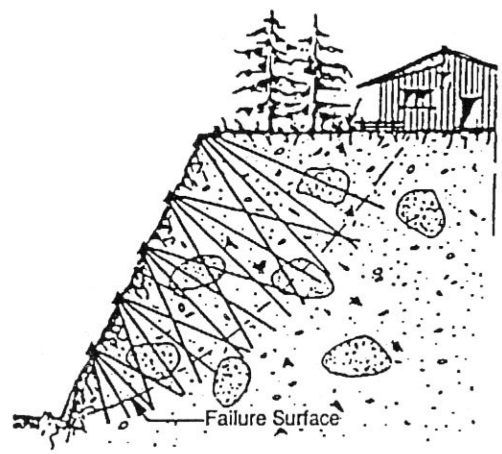


Fig. 33. Schematic cross section illustrating the use of root piles for stabilization of a slope (after Lizzi 1977).

cm. Typical injection pressures range from 350 to 1,300 kN/m^2. Depths of more than 40 m can be treated in this way. Blacklock and Wright (1986) have discussed restoration of failed soil embankments along the Interstate Highway system in Alabama, Arkansas, and Missouri using the lime- and L/FA-slurry injection method of in-situ soil stabilization. Baez et al. (1992) have evaluated the use of L/FA injection on the slope rehabilitation of a levee on the Lower Chariton River in Dalton, Missouri. Their studies showed that double injection of L/FA slurry increased the strength of the levee soil by 15-30 percent. Furthermore, since the levee was treated with L/FA injections in April 1988, there have been no slope failures in the rehabilitated stretch of the levee, whereas there have been failures in adjacent untreated areas.

A new type of retaining wall utilizing facing-panel units anchored into Portland-cement-stabilized backfill has been developed for the Texas Highway Department by the Texas Transportation Institute of Texas A&M University (Morris & Crockford 1990). Only short anchors were required, and because the strength properties of the soil were significantly improved by addition of the cement, the structure became a conventional mass-gravity structure. One of the advantages of the system is that it is non-proprietary.

5 VEGETATIVE STABILIZATION OF SLOPES

Vegetative slope stabilization can be provided: (1) directly by vegetation or (2) by biotechnical slope protection (the use of vegetation combined with structural slope-stabilization elements). The basic concepts of vegetative stabilization are not new; for example, the U.S. Forest Service made specific recommendations for the use of vegetation to stabilize slopes in the western United States as early as 1936 (Kraebel 1936). However, continuing research and new developments in design now enable more effective use of vegetation than in the past. For additional details on the effects of vegetation on slopes, refer to Gray (1970), Gray and Leiser (1982), Greenway (1987), and Wu (1991).

Vegetation contributes to stability of slopes by: (1) Restraint -- root systems physically bind or restrain soil particles, (2) Interception -- foliage and plant residues absorb rainfall energy, (3) Retardation -- above-ground residues increase surface roughness and slow the velocity of runoff, (4) Infiltration -- root and plant residues help to maintain soil porosity and permeability, and (5) Transpiration -- depletion of soil moisture by plants delays onset of saturation and runoff (Gray & Leiser 1982).

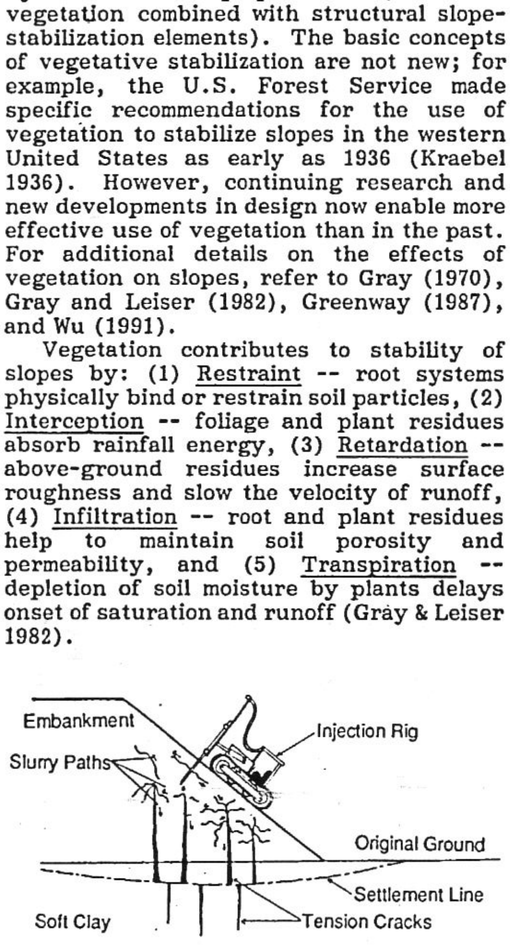


Fig. 34. Schematic cross section illustrating process of lime-slurry pressure injection for stabilization of an embankment slope (after Boynton & Blacklock 1985).

Case studies have shown that slope failures can be attributed to the loss of tree roots as slope reinforcement (e.g., Wu et al. 1979; Riestenberg & Sovonick-Dunford 1983; Riestenberg 1987). Wu (1991) has quantified this protection in terms of root reinforcement and reduction of soil moisture and pore pressures.

In recent years, trees have been planted on many slopes worldwide to increase slope stability. Examples with which the author is familiar follow:

One element of a program to correct an embankment failure on Interstate Highway I-77 near Caldwell, Ohio, was the planting of the slope with black locust seedlings at a spacing of 1.2 m. The long-term objective was to help lower the groundwater table and to develop root stabilization. As of 1987, the project appeared to be successful; however, in that short a period of time root development was not great enough to increase the factor of safety substantially (Wu 1987).

Another recent well-documented case of the planting of tree seedlings occurred as part of the stabilization program of the Cucaracha landslide in the Gaillard Cut on the Panama Canal. The historic Cucaracha slide was reactivated in 1986, almost blocking the canal (Berman 1991). As part of a comprehensive stabilization program, portions of the surfaces of the Cucaracha slide and other landslide areas in the Canal Zone were planted with 60,000 fast-growing acacia and gmelina seedlings beginning in 1987 (Rivera 1991). Although the failure surfaces in these landslides are deep-seated and generally won't be affected directly by the tree roots, the trees have significantly improved the groundwater regime, thus increasing stability.

The planting of trees to control slope failures (mostly debris torrents, flows, and avalanches) in the Vorarlberg of Austria has been documented by the Austrian Federal Service for Torrent and Avalanche Control (Olz 1990). Since 1945, this federal agency has planted 4.8 million fir and mixed-forest trees on 740 hectares of potentially unstable land. As noted by Olz, "forests grow while steel avalanche defenses grow rusty and they are and always will be an alien element in the midst of nature."

Research into the engineering role of vegetation for slope stabilization in Hong Kong may be the most comprehensive such program in the world (Barker 1991). Especially notable have been the root-reinforcement studies conducted on vegetated slopes in Hong Kong by Greenway et al. (1984), Greenway 1987), and Yin et al. (1988) under the auspices of the Geotechnical Control Office of Hong Kong. In addition,

ı that slope
ıe loss of tree
.g., Wu et al.
Dunford 1983;
ıas quantified
reinforcement
ıre and pore

been planted
ncrease slope
. the author is

to correct an
te Highway I-
ıe planting of
eedlings at a
erm objective
ater table and
.s of 1987, the
ful; however,
f time root
gh to increase
ly (Wu 1987).
ented case of
; occurred as
ogram of the
illard Cut on
ric Cucaracha
most blocking
As part of a
a program,
he Cucaracha
; in the Canal
fast-growing
beginning in
h the failure
e deep-seated
d directly by
significantly
regime, thus

control slope
ts, flows, and
of Austria has
trian Federal
anche Control
ederal agency
mixed-forest
ially unstable
ts grow while
rusty and they
element in the

ering role of
ation in Hong
ehensive such
arker 1991).
on the root-
d on vegetated
eenway et al.
d Yin et al.
e Geotechnical
In addition,

the "Geotechnical Manual for Slopes" (Geotechnical Control Office 1981) includes an excellent table noting the mechanical and hydrological effects of vegetation.

Stabilization of slopes by the combined use of vegetation and man-made structural elements working together in an integrated manner is known as "biotechnical slope stabilization." Biotechnical slope stabilization is a relatively new concept that is generally cost effective as compared to the use of structures alone; it increases environmental compatibility, and allows the use of indigenous natural materials. Vegetative treatments alone are commonly much less expensive than earth retaining structures or other constructed protection systems; however, their effectiveness in terms of arresting slope movement or preventing soil loss under extreme conditions may be much less than that of the structures (Gray & Leiser 1982).

Grasses and woody plants are used most often in biotechnical stabilization. They have a true reinforcing function and should not be considered merely a cosmetic adjunct to the structure. They may be planted on a slope above a low retaining wall, or the interstices of the structure may be planted with vegetation whose roots bind together the soil within and behind the structure. The stability of all types of retaining structures with open grid-work or tiered facings benefit from such vegetation. The following cases illustrate advances in the use of biotechnical stabilization:

Gray and Sotir (in press) have described the 1989 use of a drained rock-blanket buttress combined with an earthen brush-layer fill to stabilize a roadcut along a scenic highway in Massachusetts. The rock buttress was placed at the toe of the cut; the brush-layer fill included stems and branches of plant species, such as willow and dogwood, that rooted readily from cuttings. The branches acted as reinforcement and as horizontal drains, and rooting of the embedded stems provided secondary stabilization.

Barker (1991) has noted the recent use in the United Kingdom of a composite vegetated geotextile/geogrid reinforced structure named "Biobund." The protoype for this structure was the 1986 Schuepfen Bund along the T6 Bern-Biel Autobahn in Switzerland. This 300-m-long, 4.5-m-high visual and acoustic barrier incorporated more than 33,000 willow cuttings between pockets of geotextile-wrapped fill. Another 5,000 container-grown rooted shrubs and trees were interspersed with the willow cuttings.

Suyama (in press) has discussed the use of forests in Japan: (1) to control slope erosion, (2) to stabilize landslide scars, and (3) to absorb debris-flow and rockfall impacts. In very interesting full-scale field experiments, estimations of the resistance of trees to the impact energy of debris flows and rockfalls were determined.

6 USE OF LIGHTWEIGHT FILLS TO FACILITATE SLOPE STABILIZATION

To reduce the gravitational driving force behind slope-stabilizing retaining structures, various types of lightweight backfills have been used. Sawdust, burned coal, and fly-ash wastes have been used in areas where these waste products are readily available. In the past few years, two new types of lightweight fill have been proposed and tested for use in lightweight backfills for slope stabilization: styrofoam blocks and shredded waste car and truck tires.

The introduction of superlight expanded polystyrene (EPS; styrofoam) blocks in 1972 allowed the construction of lightweight fills for highways. In additon, it has been used successfully for road-base insulation in cold regions for several years. By 1987, the Norwegian Geotechnical Institute was involved in more than 100 projects where this superlight material had been used with great success (Flaate 1987). As superlight fill, EPS is used in the form of large blocks with a density of 0.02 t/m^3, a drastic reduction in density compared to other lightweight materials.

The most common application of EPS as superlight fill has been for highway embankments, and especially for bridge approaches. However, the Colorado Department of Transportation recently has successfully used EPS for slide correction. During the spring of 1987, an 8400-m^3 slide closed the eastbound lane of heavily traveled U.S. Highway 160 in southern Colorado. The slide area was successfully stabilized by using a counterfort berm at the toe and replacing the slide material in the highway embankment with EPS (Yeh & Gilmore in press).

Another recently applied lightweight fill for slide correction is shredded waste rubber car and truck tires. Nearly 300 million tires are discarded annually in the United States, creating a major disposal problem. Shredded tires have a compacted dry unit weight of about 0.64 t/m^3 (Humphrey & Manion in press). About 580,000 shredded rubber tires were used as lightweight fill in correction of a landslide that occurred in 1989 under a highway embankment on U.S. Highway 42 in the State of Oregon (Read et al. 1991). The force driving the slide was considerably reduced by replacing the slide material with the lightweight shredded tires.

7 FUTURE TRENDS IN SLOPE STABILIZATION

Research in analysis, design, and construction of systems for subsurface drainage, rockfall control, and soil retention will continue to provide new approaches to the development and use of these slope-stabilization systems. Particularly important is the development of new economical, strong, corrosion-resistant, and environmentally acceptable materials that can be used as elements in stabilization systems for both rock and soil slopes. For steep rock slopes, new computerized approaches will allow increased understanding of the rockfall process that will lead to better rockfall control. New experimental techniques, such as the use of the geotechnical centrifuge, will complement analytical approaches to better understand the mechanics of failure of retention systems, thus leading to improvements in design.

REFERENCES

Adib, M., J.K. Mitchell & B. Christopher. 1990. Finite element modeling of reinforced soil walls and embankments. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:409-423.

Andrew, R.D. In press. Selection of rockfall mitigation techniques. Transportation Research Record, Transportation Research Board, National Research Council, Washington, D.C.

Arutjunyan, R.N. 1988. Prevention of landslide slope process by vacuuming treatment of disconsolidated soils. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:835-837.

ASTM Committee D-35. 1991. ASTM standards on geosynthetics, second edition. American Society for Testing and Materials, Philadelphia, 104.

Baez, J.I, R.H. Borden, & J.H. Henry. 1992. Rehabilitation of Lower Chariton River Levee by lime/flyash slurry injection. Transportation Research Record, Transportation Research Board, National Research Council, Washington, D.C., 1310:117-125.

Baker, R.F., & H.E. Marshall. 1958. Control and correction. In Landslides and engineering practice, E.B. Eckel, ed., Highway Research Board Special Report, National Academy of Sciences-National Research Council, Washington, D.C., 29:150-188.

Barker, D.H. 1991. Developments in biotechnical stabilisation in Britain and the Commonwealth. In Proceedings, Workshop on Biotechnical Stabilization, University of Michigan, Ann Arbor, 21-23 August, A-83 to A-123.

Barley, A.D. 1991. Slope stabilization by new ground anchorage systems in rocks and soils. In Proceedings, International Conference on Slope Stability Engineering: Developments and Applications, Institution of Civil Engineers, Isle of Wight, U.K., 15-18 April, Thomas Telford Ltd., London, 335-340.

Barrett, R.K. & J.L. White. 1991. Rockfall prediction and control. In Proceedings, National Symposium on Highway and Railroad Slope Maintenance, Association of Engineering Geologists, Chicago, 2-3 October, 23-40.

Beer, P., U. Hegg & V. Manassero. In press. Landslide stabilization at Ancona (Italy) by deep drainage wells. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February 1992.

Bell, J.R. & J.E. Steward. 1977. Construction and observations of fabric retained soil walls. In Proceedings, International Conference on the Use of Fabrics in Geotechnics, Paris, 20-22 April, 1:123-128.

Berman, G. 1991. Landslides on the Panama Canal. Landslide News, Japan Landslide Society, 5:10-14.

Bianco, B. & D.A. Bruce. 1991. Large landslide stabilization by deep drainage wells. In Proceedings, International Conference on Slope Stability Engineering: Developments and Applications, Institution of Civil Engineers. Isle of Wight, U.K., 15-18 April, Thomas Telford Ltd., London, 341-348.

Blacklock, J.R., and Wright, P.J. 1986. Injection stabilization of failed highway embankments. Paper presented at 65th Annual Meeting of the Transportation Research Board, National Research Council, Washington, D.C., January.

Bonaparte, R., G.R. Schmertmann & N.D. Williams. 1986. Seismic design of slopes reinforced with geogrids and geotextiles. In Proceedings, Third International Conference on Geotextiles, Vienna, 7-11 April, 1:273-284.

Boyd, R.D.
measures at
Canada. In
Conference o
Development
of Civil Engi
15-18 April,
414.

Boynton, R.S.,
slurry pressu
Lime Associa

Brand, E.W.
Durability
exposure in
Geotechnical
Society of
1000.

Bridle, R. & B.
soil nailing--
presented
European Cou
des Ponts et
1991, 10.

Broms, B.B. & I
of slopes in
In Proceedi
Conference c
April 1986, 2

Bush, D.I. 1
landslide st
reinforcemen
International
Lausanne, 10

Cancelli, A.,
1987. Gener
problems in
natural slope
9th European
and Founda
3:1043-1072.

Carroll, R.G.
Geosynthetic
Proceedings
Conference c
April, 2:389-

Casagrande, L.
Proceedings,
on Soil M
Engineering,

Christopher, B
I. Juran, J.
Dunnicliff.
structures,
research and
Highway Ad
Transportati
No. FHWA-R

Christopher, B
I. Juran, J.
Dunnicliff.
structures,
construction
Administrati

Boyd, R.D. 1991. Rockfall containment measures at Springdale, Newfoundland, Canada. In Proceedings, International Conference on Slope Stability Engineering: Developments and Applications, Institution of Civil Engineers, Isle of Wight, U.K., 15-18 April, Thomas Telford Ltd., 409-414.

Boynton, R.S., & J.R. Blacklock. 1985. Lime slurry pressure injection bulletin. National Lime Association Bulletin 331:43.

Brand, E.W. and P.L.R. Pang. 1991. Durability of geotextiles to outdoor exposure in Hong Kong. Journal of Geotechnical Engineering, American Society of Civil Engineers, 117,7:979-1000.

Bridle, R. & B. Myles. 1991. A machine for soil nailing--process and design. Paper presented at the Civil Engineering European Courses Seminar, Ecole Nationale des Ponts et Chaussees, Paris, October 1991, 10.

Broms, B.B. & I.H. Wong. 1986. Stabilization of slopes in residual soils with geofabric. In Proceedings, Third International Conference on Geotextiles, Vienna, 7-11 April 1986, 2:295-300.

Bush, D.I. 1988. Recent advances in landslide stabilisation using polymer reinforcement. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:877-880.

Cancelli, A., B. D'Elia & P. Sembenelli. 1987. General Report. In Groundwater problems in embankments, dams, and natural slopes, Session 4, Proceedings, 9th European Conference on Soil Mechanics and Foundation Engineering, Dublin, 3:1043-1072.

Carroll, R.G. & G.N. Richardson. 1986. Geosynthetic reinforced retaining walls. In Proceedings, Third International Conference on Geotextiles, Vienna, 7-11 April, 2:389-394.

Casagrande, L. 1948. Electro-osmosis. In Proceedings, 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1:218-223.

Christopher, B.R., S.A. Gill, J.P. Giroud, I. Juran, J.K. Mitchell, F. Schlosser & J. Dunnicliff. 1989. Reinforced soil structures, Volume II. Summary of research and systems information. Federal Highway Administration, Department of Transportation, McLean, Virginia, Report No. FHWA-RD-89-043, 158.

Christopher, B.R., S.A. Gill, J.P. Giroud, I. Juran, J.K. Mitchell, F. Schlosser & J. Dunnicliff. 1990. Reinforced soil structures, Volume I. Design and construction guidelines. Federal Highway Administration, U.S. Department of Transportation, McLean, Virginia, Publication No. FHWA-RD-89-043, 285.

Collotta, T., V. Manassero & P.C. Moretti. 1988. An advanced technology in deep drainage of slopes. In Proceedings, 5th International Symposium on Landslides, Lausanne, 10-15 July, 2:887-892.

Craig, W.H., R. Sapak & K.C. Brady. 1991. Model studies of an anchored earth structure. In Centrifuge 91, Proceedings of the International Conference Centrifuge 1991, H.Y. Ko & F.G. McLean eds., Boulder, Colorado, 13-14 June, 303-310.

Deere, D.U. & J.Y. Perez. 1985. Remedial measures for large slide movements. In Proceedings of P.R.C.- [People's Republic of China] U.S.-Japan Trilateral Symposium/Workshop on Engineering for Multiple Natural Hazard Mitigation, Beijing, 7-12 January: L-7-1 to L-7-15.

Di Maio, C. & C. Viggiani. 1987. Influence of intermittent rainfall on effectiveness of trench drains. In Proceedings, 9th International Conference on Soil Mechanics and Foundation Engineering, Dublin, 1:149-152.

Elias, V. 1990. Durability/corrosion of soil reinforced structures. Federal Highway Administration Publication No. FHWA-RD-89-186, U.S. Department of Transportation, McLean, Virginia, 163.

Elias, V. & I. Juran. 1991. Soil nailing for stabilization of highway slopes and excavations. Federal Highway Administration Report No. FHWA-RD-89-198, Department of Transportation, McLean, Virginia, 210.

Fang, H.Y. 1991. The use of bamboo inclusions for earth reinforcement. In Proceedings, Workshop on Biotechnical Stabilization, University of Michigan, Ann Arbor, 21-23 August, 33-62.

Ferworn, D.E. & D.E. Weatherby. In press. A contractor's experience with soil nailing. Transportation Research Record, National Research Council, Washington, D.C.

Flaate, K. 1987. Superlight material in heavy construction. Geotechnical News, Sept., 22-23.

Forsyth, R.A. 1978. Alternative earth reinforcements. In Proceedings, American Society of Civil Engineers Symposium on Earth Reinforcement, Pittsburgh, 358-370.

Forsyth, R.A. & D.A. Bieber. 1984. La Honda slope repair with grid reinforcement. In Proceedings, International Symposium on Polymer grid Reinforcement in Civil Engineering, London.

Fukuoka, M. 1986. Fabric retaining wall with multiple anchors. Proceedings, Third International Conference on Geotextiles, Vienna, 7-11 April, 2:435-440.

Fukuoka, M. & Y. Imamura. 1982. Fabric retaining walls. Proceedings, Second International Conference on Geotextiles, Las Vegas, Nevada, 2:575-580.

Gabus, J.H., C. Bonnard, F. Noverraz & A. Parriaux. 1988. Arveyes, un glissement, une tentative de correction. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:911-914.

Gassler, G. In press. German practice of soil nailing. Transportation Research Record, Transportation Research Board, National Research Council, Washington, D.C.

Geotechnical Control Office. 1981, Geotechnical manual for slopes. Public Works Department, Hong Kong, 228.

Gillon, M.D.; C.J. Graham & G.G. Grocott. In press. Low level drainage works at the Brewery Creek Slide. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February.

Gillon, M.D. & G.T. Hancox. In press. Cromwell Gorge landslides -- a general overview. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February.

Gray D.H. 1970. Effect of forest clear-cutting on the stability of natural slopes. Bulletin of the Association of Engineering Geologists, 7,1:45-66.

Gray, D.H. & A.T. Leiser. 1982. Biotechnical slope protection and erosion control. Van Nostrand Reinhold, New York, 271.

Gray, D.H. & R.B. Sotir. In press. Biotechnical stabilization of a highway cut slope. Submitted to Journal of the Geotechnical Engineering Division, American Society of Civil Engineers.

Greenway, D.R. 1987. Vegetation and slope stability. In Slope stability, M.G. Anderson & K.S. Richards, eds., John Wiley, New York, 187-230.

Greenway, D.R., M.G. Anderson, & K.C. Brian-Boys. 1984. Influence of vegetation on slope stability in Hong Kong. In Proceedings, 4th International Symposium on Landslides, Toronto, 1:399-404.

Gress, J.C. In press. Siphon drain: a technic for slope stabilization. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February 1992.

Hausmann, M.R. 1990. Engineering principles of ground modification. McGraw-Hill, New York, 632.

Holtz, R.D. 1978. Special applications. In Proceedings, Symposium on Earth Reinforcement, American Society of Civil Engineers, New York, 77-97.

Humphrey, D.N. & W.P. Manion. In press. Properties of tire chips for lightweight fill. Proceedings, Conference on Grouting, Soil Improvement, and Geosynthetics, American Society of Civil Engineers, New Orleans, 1992.

Hutchinson, J.N. 1977. Assessment of the effectiveness of corrective measures in relation to geological conditions and types of slope movement. Bulletin of the Association of Engineering Geology, 16:131-155.

Hutchinson, J.N. 1984. Landslides in Britain and their countermeasures. Journal of Japan Landslide Society, 21,1:1-24.

Imrie, A.S., D.P. Moore & E.G. Enegren. In press. Performance and maintenance of the drainage system at Downie Slide. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February 1992.

Ingold, T.S. and K.S. Miller. 1988. Geotextiles handbook. Thomas Telford, London, 152.

Jaber, M., J.K. Mitchell, B.R. Christopher & B.L. Kutter. 1990. Large centrifuge modeling of full scale reinforced soil walls. In Design and performance of earth retaining structures, P.C. Lambe and L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:379-393.

Jones, C.J.F.P. 1985. Earth reinforcement and soil structures. Butterworths, London, 183.

Juran, I., G. Baudrand, K. Farrag & V. Elias. 1990. Design of soil nailed retaining structures. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:644-659.

Koerner, R.M. 1990. Designing with geosynthetics. Prentice Hall, Englewood Cliffs, New Jersey, 2nd ed., 652.

Koerner, R.M. & J.C. Robins. 1986. In-situ stabilization of soil slopes using nailed geosynthetics. In Proceedings, Third International Conference on Geotextiles, Vienna, 7-11 April, 2:395-400.

Kraebel, C.J. 1936. Erosion control on mountain roads. United States Department of Agriculture Circular No. 380:44, Washington, D.C.

Leflaive, E. 1982. The reinforcement of granular materials with continuous fibers [in French]. In Proceedings, 2nd International Conference on Geotextiles, Las Vegas, Nevada, August, 3:721-726.

Leflaive, E. 1988. Texsol: already more than 50 successful applications. In Proceedings, International Symposium on Theory and Practice of Earth Reinforcement, Japan, 541-545.

Leighton, J.C. 1
Bluff on BC R
In Tunnelling
the Eighth An
Tunnelling
Vancouver,
November, Bit
266.

Lew, K.V. & J.
stabilization b
Proceedings o
Symposium on I
July, 2:939-94

List, B.R. 1988.
slopes through
the Fifth Int
Landslides, La
952.

Littlejohn, S.
practice. In D
earth retaining
L.A. Hansen,
Civil Enginee
Publication 25:

Lizzi, F. 1977.
structurally c
situ reinforce
International
Geotechnics
Formations, C

Lo, K.Y., I.I.
Electro-osmoti
sensitive clay
Journal 28:62-

Lo, K.Y., H.S.
Field test
strengthening
Canadian Geot

McCauley, M.L.
Naramore. 1985
of Transporta
Department
Sacramento, 1

Mitchell, J.K. &
North Americ
systems. In D
earth retaining
L.A. Hansen,
Civil Engine
Publication 25:

Mitchell, J.K.
Reinforcement
embankments.
Highway Rese
Transportatio
Research Coun

Morales Arnao, B
& J.Y. Perez.
No. 5, Peru,
Proceedings,
on Landslides,

Morris, D.V. &
Cement stabili

ion. In press.
ghtweight fill.
Grouting, Soil
osynthetics,
ngineers, New

ssment of the
measures in
ions and types
lletin of the
ing Geology,

lides in Britain
s. Journal of
,1:1-24.
. Enegren. In
itenance of the
nie Slide. In
nternational
Christchurch,
ry 1992.
Miller. 1988.
omas Telford,

. Christopher
ge centrifuge
rced soil walls.
nce of earth
. Lambe and
can Society of
nical Special

reinforcement
Butterworths,

. Farrag & V.
ailed retaining
performance of
P.C. Lambe &
can Society of
nical Special

signing with
all, Englewood
., 652.
. 1986. In-situ
using nailed
dings, Third
n Geotextiles,
100.
n control on
tes Department
No. 380:44,

nforcement of
tinuous fibers
eedings, 2nd
n Geotextiles,
t, 3:721-726.
eady more than
ations. In
Symposium on
of Earth
545.

Leighton, J.C. 1990. New tunnel at Shalalth Bluff on BC Rail's Squamish Subdivision. In Tunnelling in the 90's, Proceedings of the Eighth Annual General Meeting of the Tunnelling Association of Canada, Vancouver, B.C., 31 October-2 November, Bitech Publishers Ltd., 255-266.

Lew, K.V. & J. Graham. 1988. Riverbank stabilization by drains in plastic clay. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:939-944.

List, B.R. 1988. Stabilization of open pit slopes through blasting. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:945-952.

Littlejohn, S. 1990. Ground anchorage practice. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:692-733.

Lizzi, F. 1977. Practical engineering in structurally complex formations (the in situ reinforced earth). In Proceedings, International Symposium on the Geotechnics of Structurally Complex Formations, Capri, Italy.

Lo, K.Y., I.I. Inculet & K.S. Ho. 1991a. Electro-osmotic strengthening of soft sensitive clays. Canadian Geotechnical Journal 28:62-73.

Lo, K.Y., H.S. Ho & I.I. Inculet. 1991b. Field test of electrosoosmotic strengthening of soft sensitive clay. Canadian Geotechnical Journal 28:74-83.

McCauley, M.L., B.W. Works & S.A. Naramore. 1985. Rockfall mitigation. Office of Transportation Laboratory, California Department of Transportation, Sacramento, 147.

Mitchell, J.K. & B.R. Christopher. 1990. North American practice in reinforced soil systems. In Design and preformance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25: 322-346.

Mitchell, J.K. & W.C.B. Villet. 1987. Reinforcement of earth slopes and embankments. National Cooperative Highway Research Program Report 290, Transportation Research Board, National Research Council, Washington, D.C., 323.

Morales Arnao, B., V.K. Garga, R.S. Wright & J.Y. Perez. 1984. The Tablachaca Slide No. 5, Peru, and its stabilization. In Proceedings, IV International Symposium on Landslides, Toronto, 1:597-604.

Morris, D.V. & W.W. Crockford. 1990. Cement stabilized soil retaining walls. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:307-321.

Murray, R.T. & M.J. Irwin. 1981. A preliminary study of TRRL Anchored Earth. TRRL (Transport and Road Research Laboratory) Supplementary Report 674.

Nakamura, H. 1988. Landslide control works by horizontal drainage works. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:965:970.

Olcese, A., C. Vescovo, S. Boni & G. Giusti. 1991. Stabilisation of a landslide with submerged motor-driven pumps. Proceedings, International Conference on Slope Stability Engineering: Developments and Applications, Institution of Civil Engineers, Isle of Wight, U.K., 15-18 April, Thomas Telford Ltd., 321-326.

Olz, R. 1990. A short report about the Federal Service for Torrent and Avalanche Control in Vorarlberg. In ALPS 90, Alpine Landslide Practical Seminar, 6th International Conference and Field Workshop on Landslides, Milan, 31 August-12 September, 3 pages.

O'Rourke, T.D. & C.J.F.P. Jones. 1990. Overview of earth retention systems: 1970-1990. In Design and Performance of Earth Retaining Structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:22-51.

Peila, D., F. Lombardi & V. Manassero. In press. Stabilization of landslides using large diameter wells. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February 1992.

Pfeiffer, T.J., J.D. Higgins & A.K. Turner. 1990. Computer aided rockfall hazard analysis. In Proceedings, Sixth International Congress, International Association of Engineering Geology, Amsterdam, 6-10 August, 93-103.

Pierson, L.A. 1991. The Rockfall Hazard Rating System. In Proceedings, National Symposium on Highway and Railroad Slope Maintenance, Association of Engineering Geologists, Chicago, 2-3 October, 1-22.

Plumelle, C., F. Schlosser, P. Delage & G. Knochenmus. 1990. French national research project on soil nailing: CLOUTERRE. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:660-675.

Ragheb, A. & A.W. Elgamal. 1991. Effect of gradual reinforcement compromise on the behavior of mechanically stabilized earth walls. In Centrifuge 91, Proceedings of the International Conference Centrifuge 1991, H.Y. Ko & F.G. McLean, eds., Boulder, Colorado, 13-14 June, 333-340.

Read, J., T. Dodson & J. Thomas. 1991. Experimental project use of shredded tires for lightweight fill. Post-Construction Report, Federal Highway Administration Experimental Project No. 1, DTFH-71-90-501-OR-11, Oregon Department of Transportation, Salem, 31 plus 4 appendixes.

Resnick, G.S. & D. Znidarcic. 1990. Centrifugal modeling of drains for slope stabilization. Journal of Geotechnical Engineering, American Society of Civil Engineers, 116,11:1607-1624.

Riestenberg, M.M. 1987. Anchoring of thin colluvium on hillslopes by roots of sugar maple and white ash. Ph.D. dissertation, University of Cincinnati, Cincinnati, Ohio.

Riestenberg, M.M. & S. Sovonick-Dunford. 1983. The role of woody vegetation in stabilizing slopes in the Cincinnati area, Ohio. Geological Society of America Bulletin 94:506-518.

Ritchie, A.M. 1963. Evaluation of rockfall and its control. Highway Research Record, Highway Research Board, National Research Council, Washington, D.C., 17:13-28.

Rivera, R. 1991. Reforestation program - Gaillard Cut. Panama Canal Commission Report No. 1, January, 12 plus appendix.

Rogers, C.D.F. 1991. Slope stabilisation using lime. In Proceedings, International Conference on Slope Stability Engineering: Developments and Applications, Institution of Civil Engineers, Isle of Wight, U.K., 15-18 April, Thomas Telford Ltd., 395-402.

Schlosser, F. 1990. Mechanically stabilized earth retaining structures in Europe. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25: 347-378.

Schlosser, F. & I. Juran. 1979. Design parameters for artificially improved soils. In Proceedings, 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton, United Kingdom, 5.

Schlosser, F. & P. Unterreiner. In press. Soil nailing in France: research and practice. Transportation Research Record, Transportation Research Board, National Research Council, Washington, D.C.

Segrestin, P. & M.J. Bastick. 1988. Seismic design of Reinforced Earth retaining walls: the contribution of the finite elemnt analysis. In Proceedings, International Symposium on Theory and Practice of Earth Reinforcement, Kyushu, Japan, 577-582.

Sembenelli, P. 1988. General report: Stabilization and drainage. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:813-819.

Smith, D.D. 1980. The effectiveness of horizontal drains. California Department of Transportation, Sacramento, Report No. FHWA/CA/TL-80/16, 71.

Smith, D.D. & J.D. Duffy. 1990. Field tests and evaluation of rockfall restraining nets. California Department of Transportation Report No. CA/TL-90/05, Sacramento, 138.

Stanic, B. 1984. Influence of drainage trenches on slope stability. Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, 110,11:1624-1636.

Stocker, M.F. & G. Riedinger. 1990. The bearing behaviour of nailed retaining structures. In Design and performance of earth retaining structures, P.C. Lambe & L.A. Hansen, eds., American Society of Civil Engineers Geotechnical Special Publication 25:612-628.

STS Consultants, Ltd. & GeoServices Inc. 1990. Geotextile design & construction guidelines. Federal Highway Administration Pub. No. FHWA-HI-90-001, Federal Highway Administration/National Highway Institute, Washington, D.C., 334.

Suyama, M. In press. Assessment of biotechnical slope stability effect for urban forest in Japan. In Proceedings, Sixth International Symposium on Landslides, Christchurch, New Zealand, 10-14 February 1992.

Szymoniak, T., J.R. Bell, G.R. Thommen, & E.L. Johnsen. 1984. A geogrid-reinforced soil wall for landslide correction on the Oregon coast. Transportation Research Record, Transportation Research Board, National Research Council, Washington, D.C., 965:47-55.

Threadgold, L. & D.P. McNichol. 1984. The design and construction of polymer grid boulder barriers to protect a large public housing site for the Hong Kong Housing Authority. In Polymer grid reinforcement, Thomas Telford, London, 212-219.

Varnes, D.J. 1958. Landslide types and processes. In Landslides and engineering practice, E.B. Eckel, ed., Highway Research Board Special Report, National Academy of Sciences, Washington, D.C., 29:20-47.

Varnes, D.J. 1978. Slope movement types and processes. In Landslides--analysis

and control, R
eds., Transp
Special Repo
Sciences, Wasl

Veder, C. 198
stabilization. N
247.

Westergaard, H
elasticity sug
mechanics; sc
numerous st
Harvard Univ
Massachusetts

Wichter, L., E. K
Landslide sta
wells, concret
pile walls. In
International
Lausanne, 10-

Wu, T.H. 1987
embankment
colluvium. Fed
Report FHWA
University, C

Wu, T.H. 1991
vegetation. In
Biotechnical S
Michigan, Ann
to A-32.

Wu, T.H., W.I
Swanston. 197
landslides on
Alaska. Cana
16:19-33.

Wyllie, D.C. 19
and protection
National Sym
Railroad Slope
Engineering
October, 41-6

Yako, M.A. &
Polymerically
North America
on Polymeric
Retaining Stru
McGown, ed
Canada, Kluwe
284.

Yarnell, C.N. 1
net systems: a
to rockfall m
Proceedings,
Highway and I
Association o
Chicago, 2-3 (

Yeh, S.T. & J.
Expanded Pol
correction. In
Stability and
Embankments
American Soc
Berkeley, Cali

International
d Practice of
ıu, Japan, 577-

ıeral report:
In Proceedings
Symposium on
15 July, 2:813-

fectiveness of
ı Department of
o, Report No.

190. Field tests
straining nets.
Transportation
, Sacramento,

: of drainage
Journal of the
ıg Division,
'il Engineers,

;er. 1990. The
ıiled retaining
performance of
, P.C. Lambe &
ican Society of
ınical Special

ıoServices Inc.
& construction
ıl Highway
:WA-HI-90-001,
ration/National
ington, D.C.,

Assessment of
effect for urban
eedings, Sixth
on Landslides,
ealand, 10-14

R. Thommen, &
grid-reinforced
'rection on the
ation Research
esearch Board,
l, Washington,

:hol. 1984. The
of polymer grid
t a large public
Kong Housing
reinforcement,
212-219.
ide types and
nd engineering
ed., Highway
eport, National
hington, D.C.,

ıovement types
slides--analysis

and control, R.L. Schuster & R.J. Krizek, eds., Transportation Research Board Special Report, National Academy of Sciences, Washington, D.C., 176:11-33.

Veder, C. 1981. Landslides and their stabilization. New York, Springer-Verlag: 247.

Westergaard, H.M. 1938. A problem of elasticity suggested by a problem in soil mechanics; soft material reinforced by numerous strong horizontal sheets. Harvard University report, Cambridge, Massachusetts.

Wichter, L., E. Krauter & W. Meiniger. 1988. Landslide stabilization using drainage wells, concrete dowels and anchored bore pile walls. In Proceedings of the Fifth International Symposium on Landslides, Lausanne, 10-15 July, 2:1023-1028.

Wu, T.H. 1987. Long-term strength of embankment materials: shale and colluvium. Federal Highway Administration Report FHWA-OH-87-008, Ohio State University, Columbus.

Wu, T.H. 1991. Soil stabilization using vegetation. In Proceedings, Workshop on Biotechnical Stabilization, University of Michigan, Ann Arbor, 21-23 August, A-1 to A-32.

Wu, T.H., W.P. McKinnell III & D.N. Swanston. 1979. Strength of tree roots and landslides on Prince of Wales Island, Alaska. Canadian Geotechnical Journal, 16:19-33.

Wyllie, D.C. 1991. Rock slope stabilization and protection measures. In Proceedings, National Symposium on Highway and Railroad Slope Maintenance, Association of Engineering Geologists, Chicago, 2-3 October, 41-64.

Yako, M.A. & B.R. Christopher. 1988. Polymerically reinforced retaining walls in North America. In Proceedings, Workshop on Polymeric Reinforcement in Soil Retaining Structures, P.M. Jarrett & A, McGown, eds., Kingston, Ontario, Canada, Kluwer Academic Publishers, 239-284.

Yarnell, C.N. 1991. Steel wire rope safety net systems: a brief history of application to rockfall mitigation in the U.S. In Proceedings, National Symposium on Highway and Railroad Slope Maintenance, Association of Engineering Geologists, Chicago, 2-3 October, 135-148.

Yeh, S.T. & J. Gilmore. In press. Use of Expanded Polystyrene (EPS) for slide correction. In Proceedings, Conference on Stability and Performance of Slopes and Embankments II: a 25-year Perspective, American Society of Civil Engineers, Berkeley, California, June 1992.

Yin, K.P., L.K. Heung, and D.R. Greenway. 1988. Effect of root reinforcement on the stability of three fill slopes in Hong Kong. In Proceedings, 2nd International Conference on Geomechanics in Tropical Soils, Singapore, 293-302.

Yoo, N.J. & H.Y. Ko. 1991. Centrifuge modeling of reinforced earth retaining walls. In Centrifuge 91, Proceedings of the International Conference Centrifuge 91, Boulder, Colorado, 13-14 June, 325-332.

Zaruba, Q. & V. Mencl. 1982. Landslides and their control. New York, Elsevier, 324.