



NEW METHODS OF HIGHWAY STABILIZATION

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1. INTRODUCTION

Most highway engineers are by now well acquainted with the use of prestressed rock and soil anchorages in slope stabilization programs. Such anchorage systems may be applied:

- directly to the slope face above and/or below the highway (Reference 1)
- to stabilize primary retaining structures such as piled or slurry trench walls (Reference 2)
- to stabilize existing structures affected by changed geotechnical or structural parameters (Reference 3)

In addition, through the efforts of researchers, constructors and suppliers alike, and via the medium of conferences such as this, other techniques are gaining wide recognition; for example, the use of stone columns, dynamic consolidation and geotextiles is reviewed elsewhere in this Symposium.

The aim of this paper is to highlight three other groups of options for stabilizing slopes in appropriate conditions:

- insitu reinforcement (by nails, micropiles and dowels)
- large-scale drainage (by trenches and large diameter wells)
- ground treatment (by jet grouting)

Although each group is relatively new to American practice, each represents a powerful tried and tested solution with several years of application in Western Europe and the Far East.

Each group is introduced in turn, and typical applications are described. Key references are provided for the reader who wishes to pursue the details of design and construction.

2. INSITU REINFORCEMENT

From the early 1970's, engineers in Western Europe and Western Canada have exploited the special advantages of slope stabilization by insitu reinforcement (Reference 4). Insitu reinforcement

comprises inclusions placed in the soil to maintain equilibrium under the soil self weight loading, and surcharge loading on the soil. Today, one of the methods - soil nailing - is one of the fastest growing geotechnical processes in the States. This extraordinary national upsurge may be dated from the execution of the application of soil nailing to stabilize the excavation for the foundations of the PPG Building, here in downtown Pittsburgh, in 1982 (Reference 5).

2.1 Insitu Reinforcement Techniques - There are three main categories of insitu reinforcement techniques used to stabilize soil slopes and excavations. These are *nailing*, *reticulated micropiling*, and *dowelling*.

In *soil nailing*, the reinforcement is installed horizontally or subhorizontally so that it improves the shearing resistance of the soil by acting in tension (*Figure 1a*).

Reticulated micropiles are steeply inclined in the soil at various angles both perpendicular and parallel to the face (*Figure 1b*). The overall aim is similar to soil nailing, namely to provide a stable block of reinforced soil which supports the unreinforced soil by acting like a gravity retaining structure. In this technique the soil is held together by the multiplicity of reinforcement members acting to resist bending and shearing forces.

Soil dowelling is applied to reduce or halt downslope movements on well defined shear surfaces (*Figure 1c*). The slopes treated by dowelling are typically much flatter than those in soil nailing or reticulated micropile applications. Gudehus (Reference 6) has shown that the most efficient way to improve mechanically the shearing resistance on a weakened shear surface through the soil is to use relatively large diameter piles which combine a large surface area with high bending stiffness. The diameter of a soil dowel is thus generally far greater than that of a soil nail or micropile (typically up to 6").

2.2 Selecting Insitu Reinforcement - Although there are fundamental differences in the mechanical action of these three insitu reinforcement techniques, there are circumstances where more

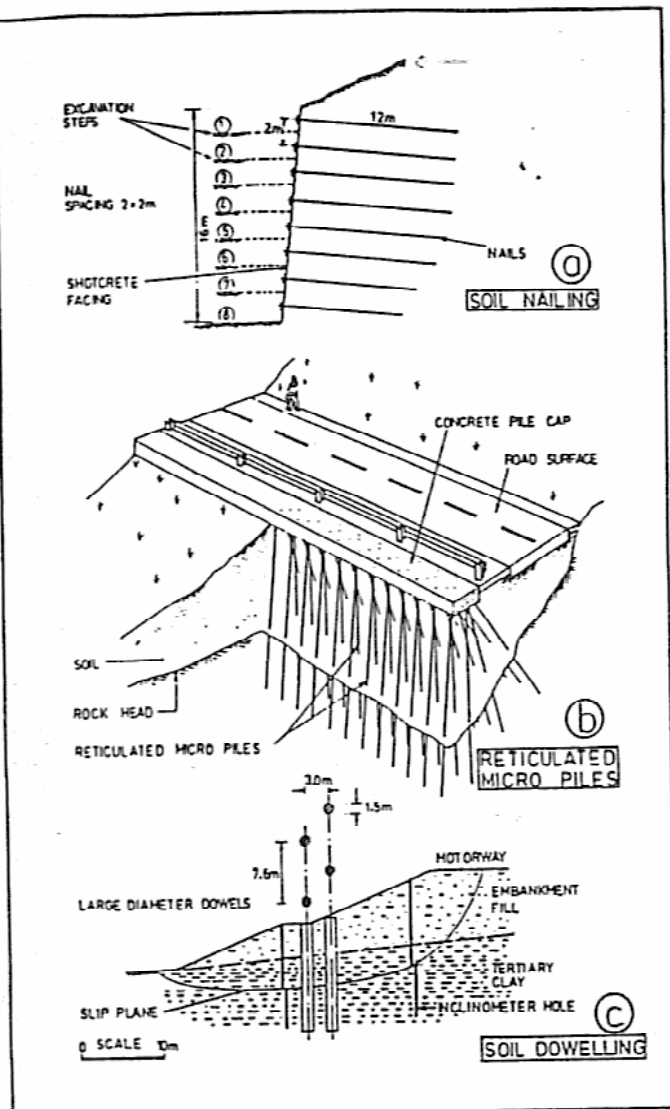


Figure 1 The family of insitu soil reinforcement techniques. (Bruce and Jewell, 1986)

than one may be applied to slope stabilization as illustrated in Figure 2. The following points should be considered when choosing the appropriate insitu reinforcement technique.

Laboratory experiments have shown the influence of the inclination and properties of reinforcing members on the shearing resistance of reinforced soil (Reference 7). These indicate that the reinforcement gives the best increase in strength when it is angled across the potential rupture surface in soil so that the reinforcement is loaded in tension. At other orientations in the soil, the reinforcement provides less benefit, and can even reduce the shearing resistance of the soil mass if it is forced to act in compression.

The conclusion, therefore, is that in applications where a steep slope is to be excavated in a homogeneous granular soil, it is most efficient to place the reinforcement through the face in a

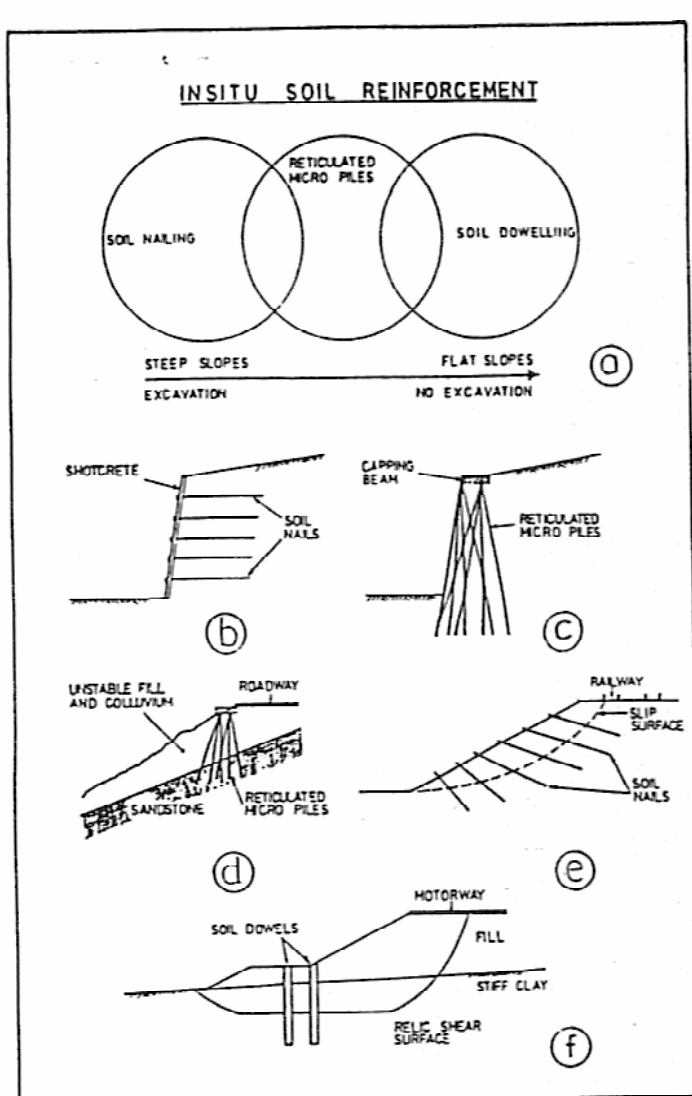


Figure 2 Overlap of insitu soil reinforcement applications (b) and (c) in excavations, (d) and (e) for general slope stabilization, and (f) to stabilize residual slip in clay. (Bruce and Jewell, 1986)

direction close to the horizontal, as in Figure 2b. To stabilize the soil with reinforcement placed in substantially vertical directions (Figure 2c) will require a much higher density of reinforcement. For this type of application soil nailing is likely to be more cost-effective than reticulated micropiling.

In marginally stable granular or scree slopes when stability must be improved, but where excavation is not foreseen, then either soil nailing or reticulated micropiling would be appropriate. Where drilling equipment cannot be placed on the slope, reticulated micropiling would be best (Figure 2d). Where access is not problematical, either technique could be applied (Figures 2d and 2e), with economic considerations being decisive.

In flatter clay slopes where stability is governed by a well defined shear zone, larger diameter soil dowels would be most appropriate (Figure 2f). (Also References 8 to 10).

2.3 Local Examples of In Situ Reinforcement -

2.3.1 Soil Nailing: Cumberland Gap: Bearing in mind that the classic PPG soil nailing contract in Pittsburgh was to secure a deep excavation, the work on US 25E at Cumberland Gap (Reference 11) represents a milestone in slope stabilization by this method. This FHWA project was conducted to stabilize a slope near the Kentucky portal of a pilot tunnel under the Cumberland Gap.

The parent rocks in the area are a series of shales, sandstones, and siltstones that have been differentially weathered to form a soil mantle ranging in thickness from 5 to 20 feet. Beneath the soil overburden is weathered rock of varying degrees of competence and thickness. In general, competent bedrock is encountered at a depth of from 10 to 40 feet.

The design of the 40-foot-high section of the wall, based on the studies of University of California at Davis (Reference 12), resulted in a design as shown on Figure 3. The spacing of the nails was a grid 5-foot square with an assumed nail diameter of 0.58 inch. The upper 3 or 4 rows of reinforcing members were to be #8 bars, and the lower rows, #10 bars - all grade 60 steel. (During construction, #11 bars were substituted for the #10 bars because of availability.) Up to 8 rows of nails were installed, and a total of 335 nails were installed in the 9,000 square feet cut.

Prior to construction, two vertical test bars were installed and load tested to verify design assumptions on interfacial bonds. During construction, randomly selected nails were tensioned to 90% yield, as quality assurance.

The wall has been instrumented with three different types of load and movement measuring devices:

1. slope inclinometers on and behind the slope
2. strain gauges on 16 bars
3. electronic distance measuring points on bar heads

Instrument groups 1 and 3 are in agreement, showing maximum total head movements of 1/4 to 3/8 inch. This is less than 0.1% of slope height--a performance within the "typical" range of 0 to 0.3% identified by Bruce and Jewell (Reference 4). Strain gauge data have proved more difficult to decipher, but frost-related fluctuations in head load have been detected. The FHWA continues to monitor the installation and to analyze its performance.

This was the first use by the FHWA of nailing to solve a combination of design and environmental problems, and was recognized by an award in the "Excellence in Highway Design," 1986 Biennial Awards of the Federal Highway Administration (Category VIII - Highway Improvements in Federally-owned Lands).

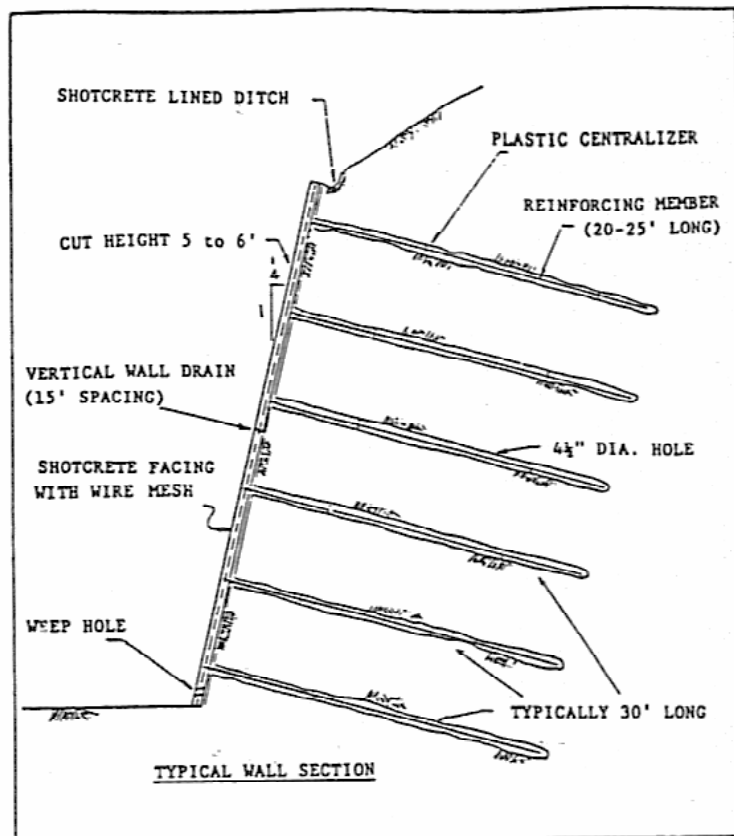


Figure 3 Details of Cumberland Gap soil nailing (Nicholson, 1986)

2.3.2 Reticulated Micropiling: LR69: Legislative Route 69 between Vandergrift and Leechburg, PA runs along the Kiskiminetas River. Much of it is cut into the steep hillside along the bank of the river, with the cut material used as downhill side fill to provide the highway bench. In a number of places this embankment material proved unstable and slides occurred.

One such slide repair was designed by The Pennsylvania Department of Transportation (PennDOT) as a tie-back caisson wall. This project was bid in the Summer of 1984 with bids accepted on the "as designed" wall or contractor designed alternatives. The authors' company was successful based on an alternate design (Figure 1b). This type of wall is referred to as a Type "A" INSERT WallSM (Reference 13).

The 310 feet long slide had previously been repaired by filling and repatching, as the closeness of the river prohibited placing a fill buttress below. Given its role as a major traffic route, detours would have proved lengthy and inconvenient and so the stabilization method had to allow passage of one lane of traffic throughout.

The design height for both the as-designed wall and the insert wall, was 34 feet. The spacing on the inserts (Figure 4) was chosen to achieve the

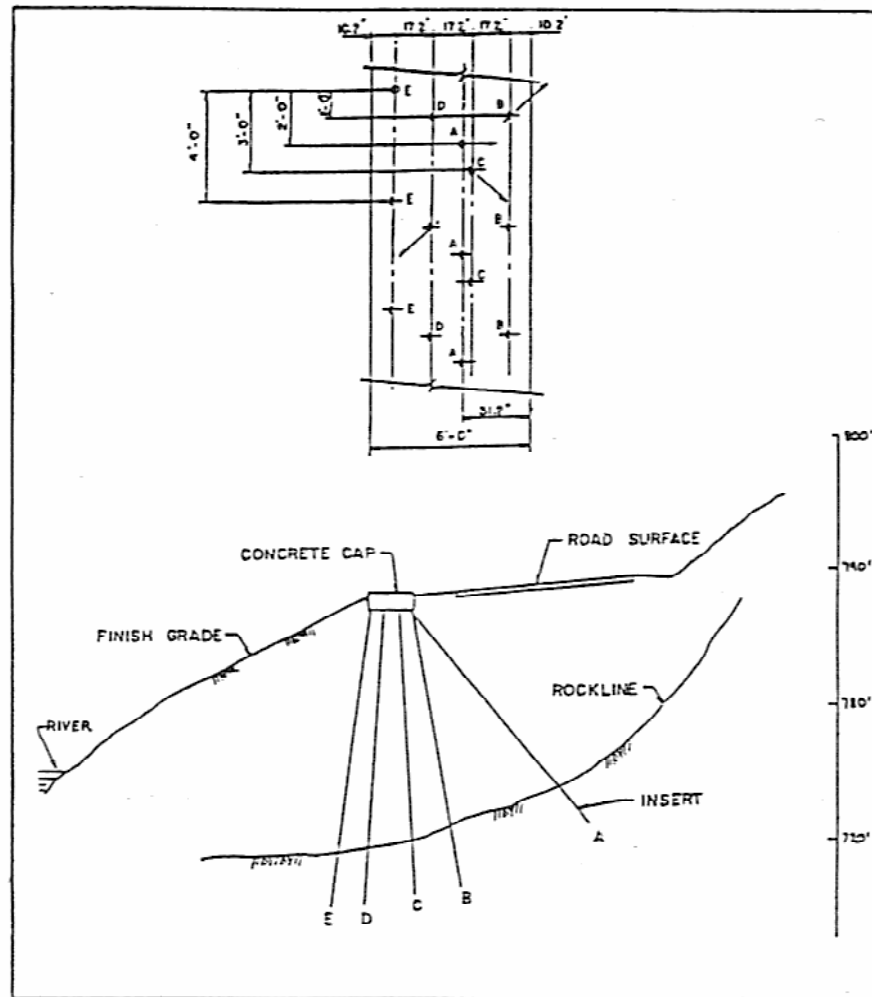


Figure 4 Details of LR69 insitu reinforcing - Type A wall
(Boley and Crayne, 1985)

maximum width at the top of rock for stability purposes. It was determined that the minimum density would be one reinforcing unit per 20 square feet at the top of rock. The resultant width of the gravity mass at the top of rock was approximately 20 feet. The stabilized mass thus achieved was, however, still potentially unstable with respect to overturning during maximum rapid drawdown conditions following a flood. For this reason, an additional row of tension members was added (Row A, Figure 4), of full design load 80 kips. These additional tension members gave a minimum factor of safety of 1.24 during rapid drawdown conditions and a factor of safety against overturning during steady-state seepage conditions of 1.95. A density of 1.33 nr 6" dia. holes per lineal foot of wall satisfied all the stability criteria, when equipped with #11 or #14 rebar, grouted with a neat cement mix to 40 psi excess pressure.

All the inserts were extended into rock a minimum of 15 feet to ensure that they would function as shear keys to thus prohibit sliding of the gravity mass and also to allow the transfer of any axial load, whether in tension or compression, into bedrock. Special diesel hydraulic drilling rigs were

used to ensure fast penetration through both soil and rock in one pass.

Monitoring of the slide by slope indicators during 1984 showed movement of the slide at a depth of 30 feet of about 0.5" during a seven-month period. The slope indicator readings taken between August and November at 30 feet depth showed additional movement of about 3.75". On about November 10, 1984, after the concrete cap had been placed and the drilling of the inserts just started, some additional movement occurred. A section of roadway along almost the entire length of the cap moved vertically downward about 4" and a crack about 1" wide opened up in the middle of the road surface. No movement or cracking of the cap was detected so installation of the inserts in the slide area was accelerated to provide support as soon as possible. About a week later, after installation of about 40 of the inserts, movement appeared to have stopped. No further movements of the roadway have been detected and other slope indicator readings taken to date have shown little or no further movement.

Savings to the State of about 15% of the total contract price as foreseen using the conventional design were generated. The cost per foot of roadway for the repairs, not including other roadway items such as drainage, paving and traffic control, is about \$1,000 to \$1,500 per foot of roadway. The exact cost depends on many factors including the size of the project, depth to stable material or rock, and composition of the sliding soil.

3. DRAINAGE

An unfavorable groundwater regime is a common cause of slope instability. Pore pressure decreases the effective stress level, and may reduce the shearing resistance of the material along a potential slip surface to a degree where unacceptable movement and failure may occur. In such conditions it is logical and effective to reduce the groundwater potential, thereby reducing pore pressures and increasing the effective stress and the factor of safety against sliding. Drainage is most effective (Reference 14) when placed as close as possible to the potential failure zone, when influencing the largest possible portion of the zone, and if placed in the direction of maximum slope inclination. Stabilization by drainage is appropriate to materials of both high and low permeabilities, as change in flow net patterns is also very important (Reference 15).

Small diameter drains inclined just above horizontal are commonly used to drain steep slopes or faces, while in other, usually temporary, applications, vertical dewatering wells are used to provide the same function within their pump out influence zones.

However, such solutions may not be practical or economic where slopes are shallow and cover large areas, or where urban structures or underground services otherwise preclude their use. Two excellent systems which are being used routinely in Western Europe, and which have major potential for application here are:

- Drainage Trenches (for shallower slides in open areas), and
- Drainage Wells (for slides of all depths in urban and rural areas.

3.1 Drainage Trenches - In conditions where surface access is good and continuous and the depth to the potential failure plane is less than 30 feet, then drainage trenches are economic and practical. Shallow conventional diaphragm wall equipment and techniques are used to form vertical continuous panels in the soil, but without the use of bentonite slurry. These panels are backfilled with an appropriately graded granular fill material, in a sequence which ensures perfect continuity between adjacent panels. The trenches are arranged in an interconnecting "herringbone" pattern (e.g., Figure 5) generally at right angles to the contours (they have no shear strength to resist lateral thrust) and are later covered with the natural soil to conceal

their presence. At the downslope end of the system, the water collected along its length flows, by gravity, into a collector drain or simple discharge well.

The rationale of design has been addressed by Stanic (Reference 14), while the application in conjunction with the companion technique of drainage wells, is described by Bianco and Beligni (Reference 16) and Bianco (Reference 17).

A good recent example was seen by the authors at a site near Ancona in Eastern Italy. In a rural farming area of rolling hills, the development of a very large areal slip was threatened in a thickness of 15-33 feet of new cohesive colluvials and alluvials overlying similar but stronger and stabler materials of Middle Pliocene age. The piezometric level (about 10 feet below the surface) was largely responsible for the marginal stability of an area of about 250,000 ft.² (600' down slope and up to 450 feet wide). It was calculated that a reduction of the piezometric level by 20 feet would increase the factor of safety against slippage to an acceptable minimum of 1.30. The slope height difference down the 600' length was about 110 feet.

Panels 30 feet deep, about 30" wide and 7 feet long were excavated in sequence with a hydraulic rope suspended grab. The leading edge of each successive panel was temporarily protected by a full height protective steel plate, later removed upon backfilling of the adjacent panel. Gravel was placed in each trench to about 5 feet from the ground surface and topped with a geotextile sheet. Soil backfill was then graded over. In this way about

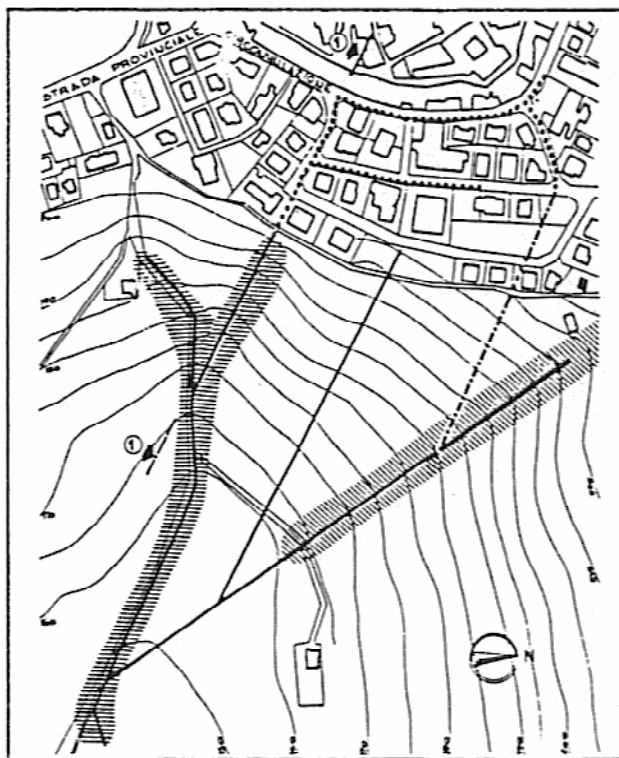


Figure 5 Stabilization of landslide at Monte S. Giusto, Italy by drainage trenches (and deep wells) (Bianco, 1986)

80,000 ft.² of drainage trenches were formed in a number of interconnecting "arms" aggregating about 2,500 lineal feet of trenches. At the bottom of the slope, the system discharged into a horizontal outlet drain.

Daily productions per machine can reach over 2,000 ft.² of trench per day, but can be as low as 300 ft.² per day in collapsing trench conditions where biodegradable slurry must be used as support. In this particular project, the approximate cost of slope stabilization was about \$2.50 per square foot of slope.

3.2 Drainage Wells - A series of major landslides which caused extensive structural damage and tragic loss of life in the Ancona-Tresenda area of Italy in the early '80's spurred the specialist geotechnical contractor, Rodio, to devise new systems of large slope stabilization for urban areas. Large diameter vertical wells are excavated by grab or auger to full depth which may be well over 60 feet. These can either be backfilled completely with filter material, or can be constructed with a combination of materials to provide both drainage and insitu reinforcement of the soil dowell type (Figure 1c). The key element of the system, however, is the horizontal interconnecting collector drain: this is a small diameter plastic pipe linking adjacent wells at depth and installed immediately after well drilling but before backfilling (Figure 6).

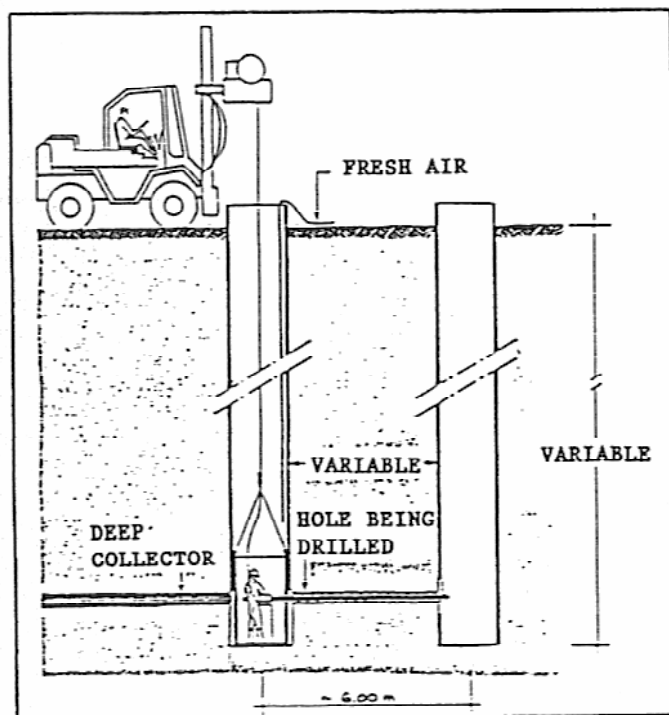


Figure 6 Drilling horizontal collector pipe between adjacent deep drainage wells (Bianco, 1986)

Wells are typically placed at distances of around 15-20 feet, but applications with up to 60 feet spacing have been recorded. Spacing is dictated primarily by the hydrogeological requirements (Reference 16) but practical considerations such as the site surface access restraints (especially in urban areas) and the accuracy with which the horizontal collector can be drilled are also limiting factors.

There are three types of wells: (1) normal full section drainage, (2) inspectable/controllable, and (3) reinforced.

1. **Normal Full Section Drainage (Figure 7)**: A shaft about 4 feet diameter is drilled, and a temporary steel liner placed. A small purpose built drilling rig is then lowered to the base and a 4-1/2" hole drilled horizontally to penetrate the adjacent well, at a level about 4 feet above the base. A 3" pvc pipe is then placed in the hole and the drill removed. A 3'-4" thick concrete base plug is then cast. After setting, the well is backfilled with filter material as the temporary liner is extracted. The filling is stopped 4'-5' below the surface and covered by geotextile. The rest of the hole is then filled with soil or concrete, depending on the environment of the well.

2. **Inspectable/Controllable (Figure 8)**: A slightly larger shaft (say 5 feet diameter) is formed and a 4 feet diameter permanent liner (of corrugated galvanized steel) is placed to full depth. The outer annulus is filled with filter aggregate. The horizontal drilling and placing of the pvc tube is then executed as before, except that the tube has a valve system attached inside the well. A concrete base plug, top hatch arrangement and ladder are placed following removal of the drilling equipment.

Such wells are located between groups of 3-10 normal drainage wells, at changes of direction or intersections of "runs," and at final discharge locations. They permit the total system through-flow to be monitored, and controlled if necessary, either manually (by operating the valves) or automatically (by a preset float arrangement).

3. **Reinforced (Figure 9)**: Where structural strength is also required, then wells of this type can be integrated into the system, or substituted for the standard inspectable well. A 7 feet diameter hole is formed to the total depth required for structural stability, a 6 feet diameter galvanized lining tube placed, with filter material placed. The standard 4 feet diameter liner is then homed to the required depth for drainage, carrying an outer reinforcement cage. The resulting inner annulus of 1-foot width is then concreted, as is the basal plug. The horizontal collector drain is then drilled, as before. In addition, further "fans" of normal, unconnected, drainage holes can also be drilled from the shaft at any elevation or direction dictated, for example, by the need to penetrate under certain buildings or into particular strata.

In particular circumstances, the concrete upper section of well can be thickened to accommodate the

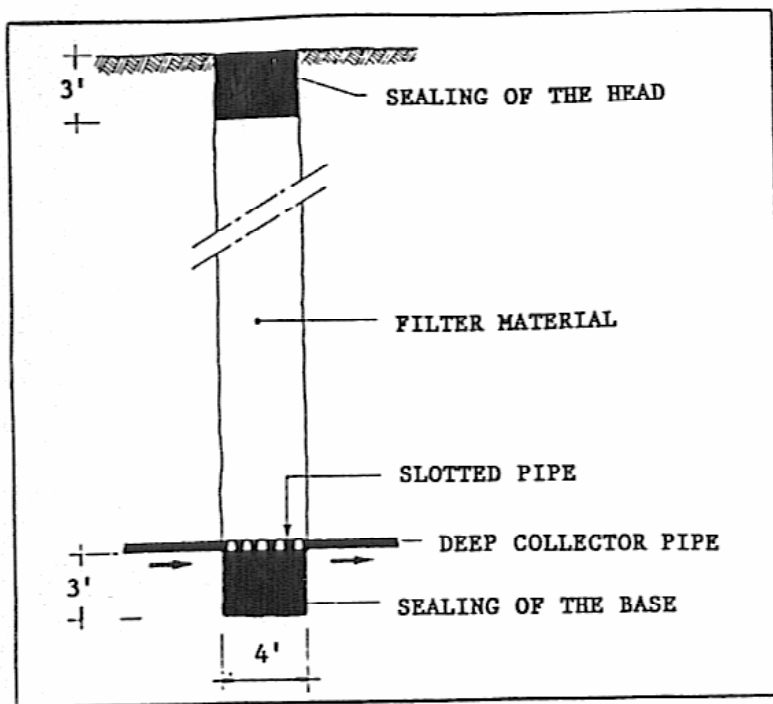


Figure 7 Normal full section drainage well (Bianco, 1986)

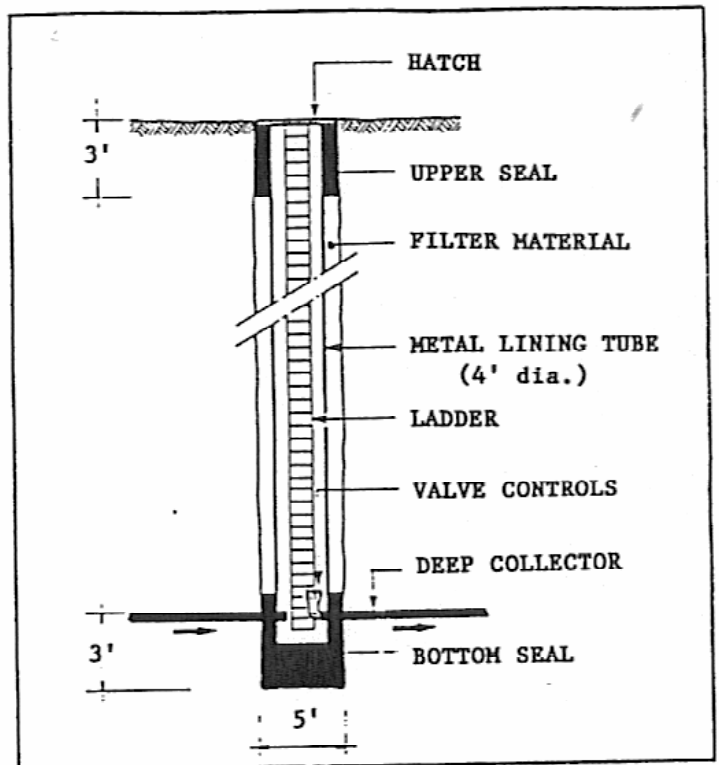


Figure 8 Inspectable/controllable well (Bianco, 1986)

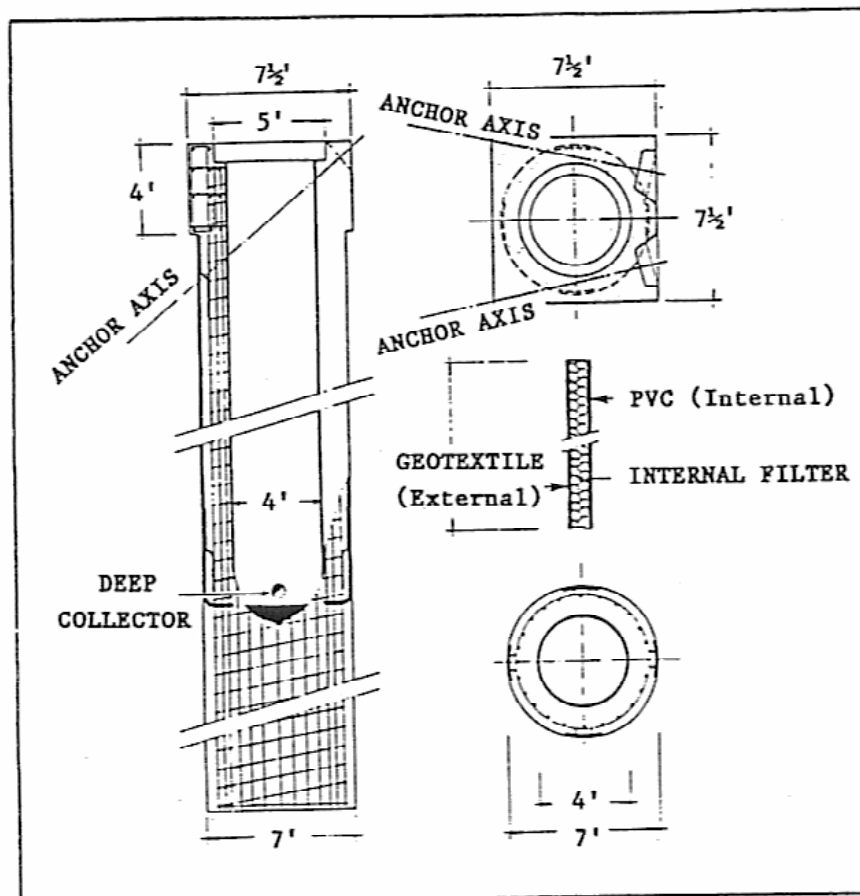


Figure 9 Reinforced well, with anchorages (Bianco, 1986)

stresses imposed by prestressed ground anchors installed to resist overturning forces. The edge distance between such structural wells is typically 0.5-3.0 times diameter, to ensure an efficient "arching" effect in the intermediate ground, as well as reflecting hydrological factors.

Design criteria - based on the classic formulae - and excellent data on six recent large and successful case histories in Italy are provided in References 16 and 17. Most of the examples were executed in the middle of urban developments with minimal disturbance to the environment.

All these case histories confirm the following major advantages of the method:

(1) Its facility for continual monitoring allows it to be limited or expanded during construction or in service in direct response to performance.

(2) It is ideal for even densely populated areas, providing minimal disruption to services, adjacent structures, and the environment. It can, therefore, be used to safeguard all kinds of structures including bridges, roads, and buildings.

(3) After installation, operational and maintenance costs are minimal: the system requires no external power source.

(4) The method of drainage is natural and self-regulating, responding by gravity feed to daily and seasonal changes in the groundwater characteristics. As there is no "forced pumping," there is no danger of instabilities due to rapid drawdowns. Manual regulation of a system's inspectable wells can further ensure optimal groundwater/stability conditions.

(5) The inspectable and structural wells permit the drilling of further families of drains, allowing the controlled dewatering of particular areas and horizons, as desired.

(6) No fines are extracted from the ground, and so no surface settlements, or clogging of the drainage system is possible.

(7) Observation and monitoring of the concrete of the reinforced wells will indicate significant ground strains.

(8) The installation does not require cuts to be made in potentially unstable slopes and will, therefore, not compromise the stability of very delicate areas during construction.

(9) It is extremely cost effective, taking over from drainage trenches (where applicable) at depths of around 30 feet.

4. GROUND TREATMENT

It is common (e.g., Reference 18) to identify four basic methods of ground treatment by grouting: hydrofracture, compaction, permeation, and replacement (Figure 10). For various reasons, the first three have little relevance or application within the framework of this conference. Furthermore, it is likely that the most contact highway engineers have had with grouting is in association with bulk infill (e.g., in areas of old coal mines), slabjacking (to inject under loosened concrete road slabs) or injection of post-tensioning ducts or anchorage holes (in concrete structures).

However, the fourth category of ground treatment, as listed above, replacement grouting -

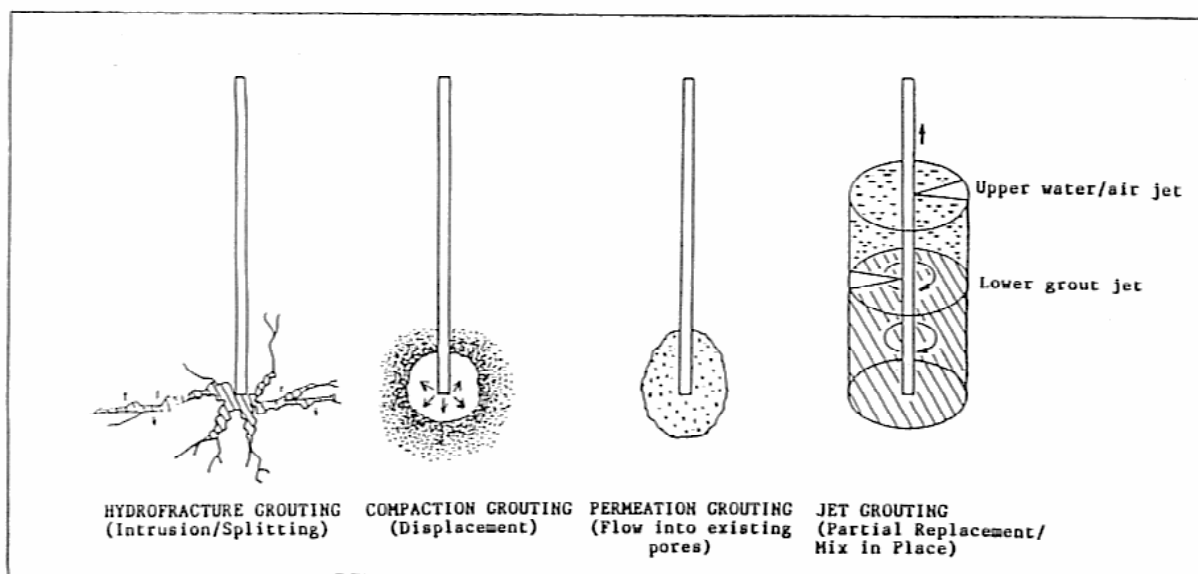


Figure 10 Basic types of ground treatment by grouting

usually referred to as jet grouting - is a tool of extreme potential and power, already proven in Europe and the Far East for over ten years.

Its development was fostered by the need to treat reliably and thoroughly soils from gravels to clays in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. In one particular form (Figure 11), the ground around the drill string is fragmented by a very high pressure (up to 8,000 psi) horizontally directed water and air jet, and largely expelled from the hole. The cavitated zone so formed is simultaneously filled from below with a cement-based grout, which does, of course, incorporate some of the native ground. A simpler variant eliminates the air/water cavitation and instead uses only the high pressure grout jet to cavitate and eject, as well as inject.

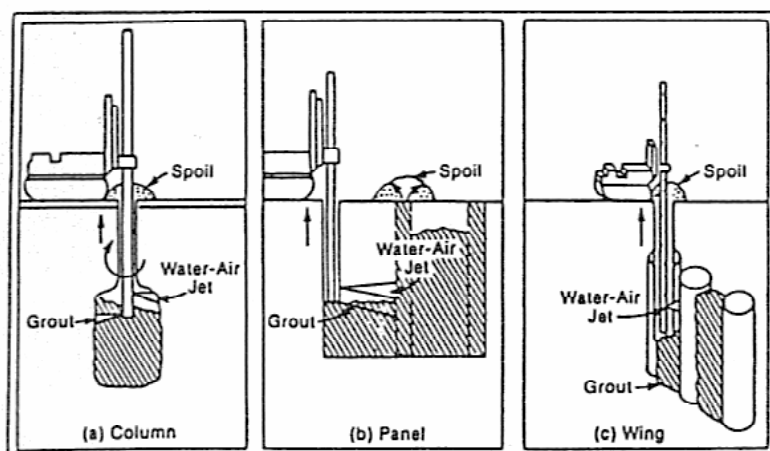


Figure 11 Jet grout construction (Bruce, 1987)

Major advantages of the technique include:

- (1) Its applicability across a wide range of soil types (Figure 12), whereas other types of grouting (e.g., by permeation) may be very expensive or simply not practical.
- (2) The relatively high strength and durability of the treated "soilcrete" mass: unconfined crushing strengths may range from 700 to 2,000 psi in sandy/gravelly soils, and 250 to 500 psi in cohesive deposits.
- (3) Treatment is usually accomplished only by using cement grouts. Other materials (e.g., flyash) can, however, be readily incorporated in response to the target ground parameters after treatment.
- (4) The system can be easily applied in areas of very restrictive access conditions or other tight environmental restraints.

Major case histories and test programs have been reported from around the world in the last five years, particularly (References 18, 19 and 20), and today jet grouting is the favored technique of specialist contractors in Western Europe and Japan. There are also indications that it is now about to seriously threaten, in the States, the dominance that compaction grouting for settlement control has previously enjoyed.

The following are the main applications of jet grouting, those of particular relevance to highway construction being illustrated in Figures 13 to 25.

1. Underpinning and Protection of Existing Structures

- (1) Enhancement of piled foundations (Figure 13)
- (2) Support of raft foundations (Figure 14)
- (3) Support of strip footings (Figure 15)
- (4) Support of isolated footings (Figure 16)

2. Waterproof Diaphragms

- (1) Under dams
- (2) Around shafts and other deep excavations (NB In these applications the bases can also be grouted to resist in-flow, and uplift forces) (Figure 17)

3. Tunnels

- (1) Ground consolidation by vertical injection of full face from ground surface.
- (2) Ground support of portals and tunnels by sub/supra horizontal "umbrella" around excavation (Figure 18)

4. Support in New Construction

- (1) As load bearing piles for concrete structures (Figures 19 to 21)
- (2) As support for embankments on soft ground (Figure 22)

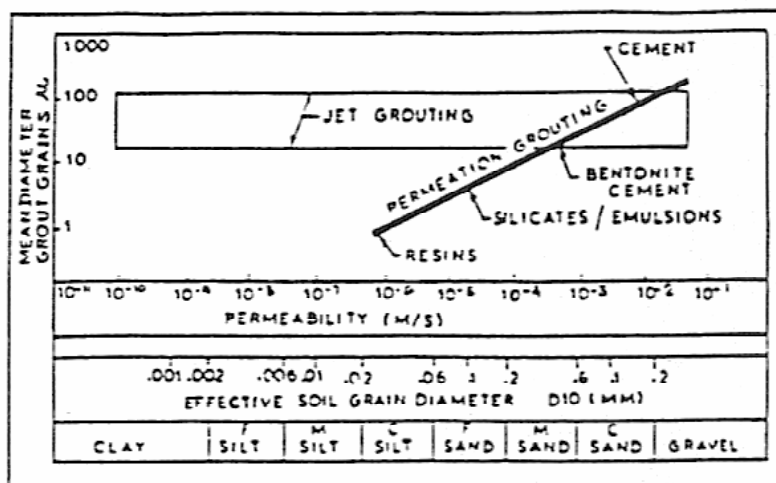


Figure 12 Groutability of soils in relation to grout and soil properties (Bruce, 1987)

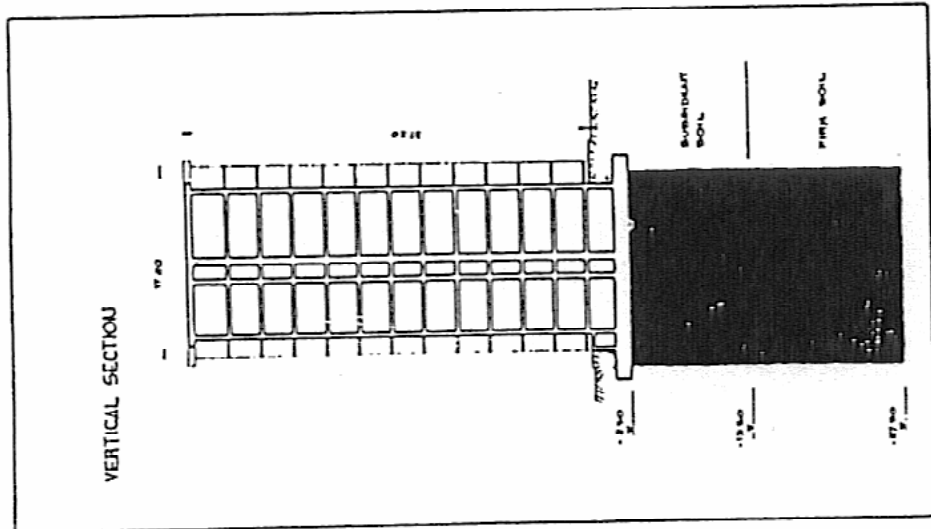


Figure 13 Enhancement of piled foundations (Courtesy GKN Keller)

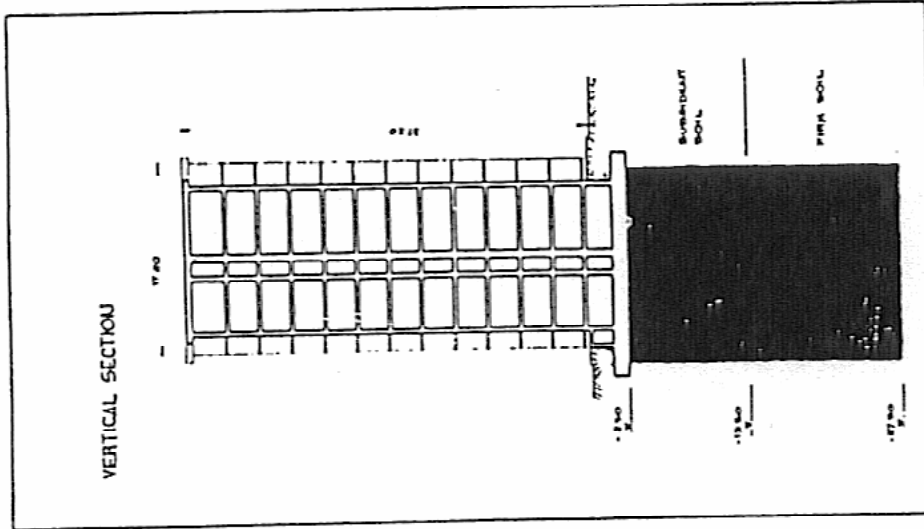


Figure 14 Support of raft foundation, Volgogradsk, USSR (Courtesy Rodio and Company)

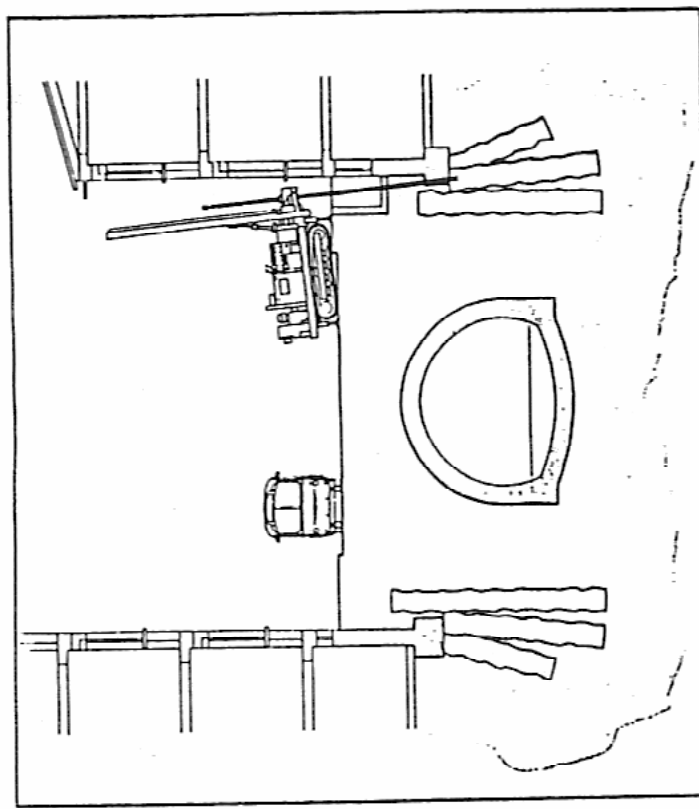


Figure 15 Support of strip foundation (Courtesy Pacchiosi Ltd.)

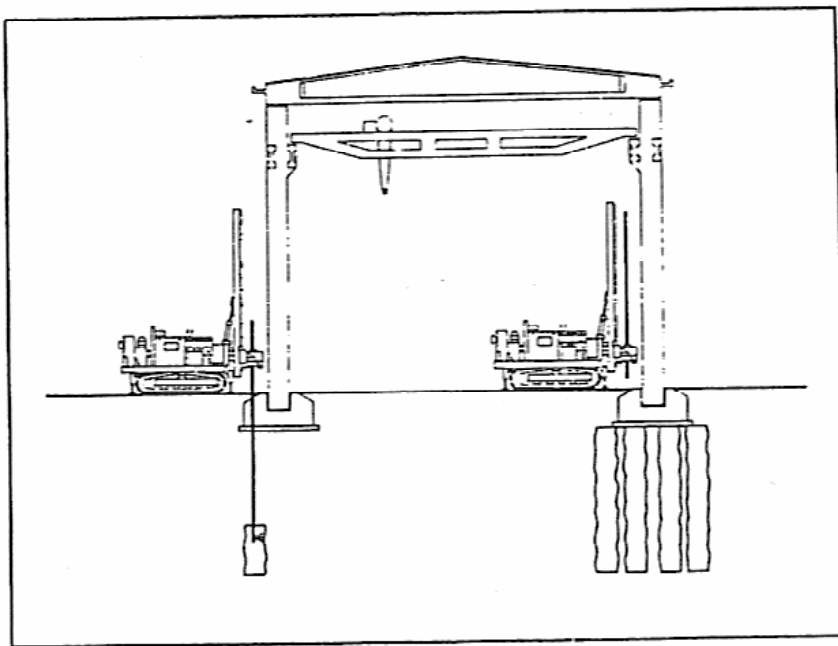


Figure 16 Support of isolated footings (Courtesy Pacchiosi Ltd.)

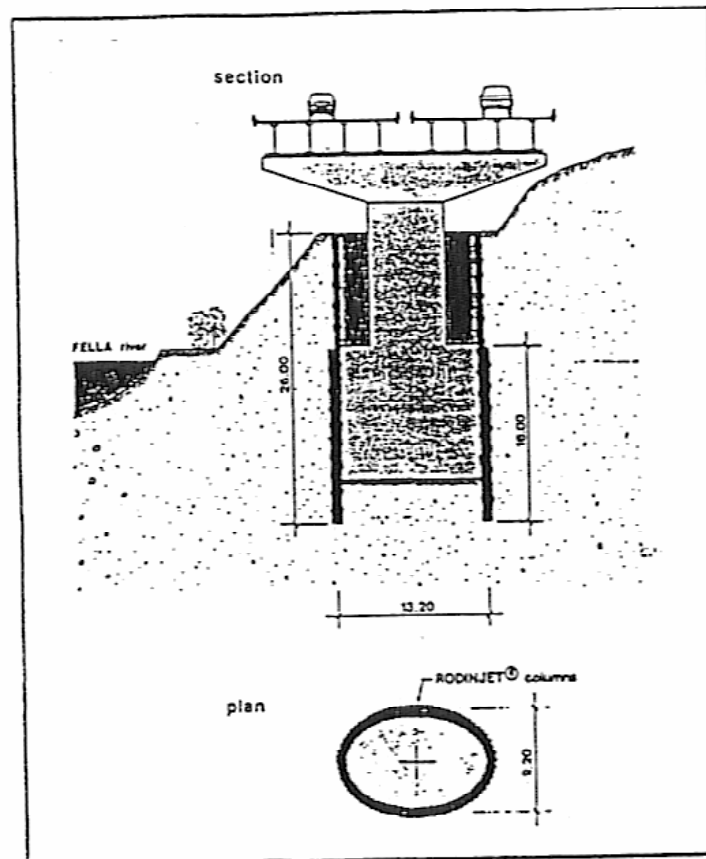


Figure 17 Shaft construction for excavation of pier foundations, Chiusaforte, Italy (Courtesy, Rodio and Company)

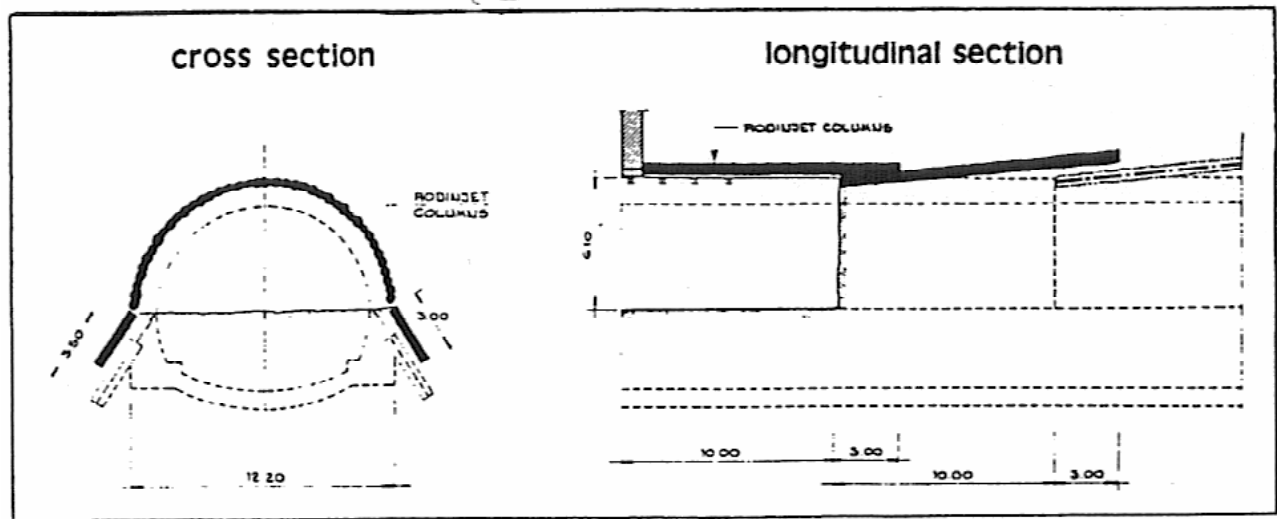


Figure 18 Ground support for tunnel excavation, Moggio Udinese Railway, Italy (Tornaghi and Cippo, 1986)

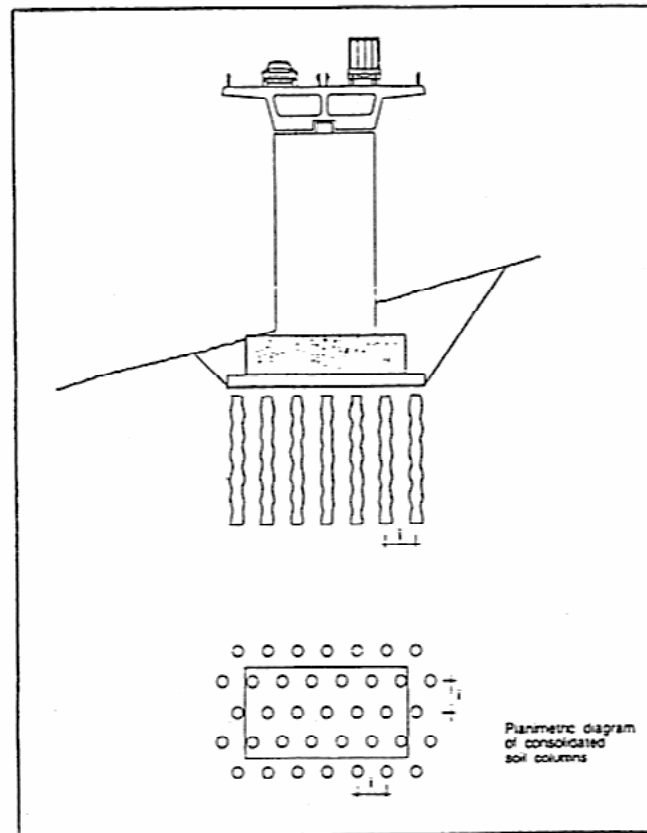


Figure 19 Columns as piles for piers (Courtesy Pacchiosi Ltd.)

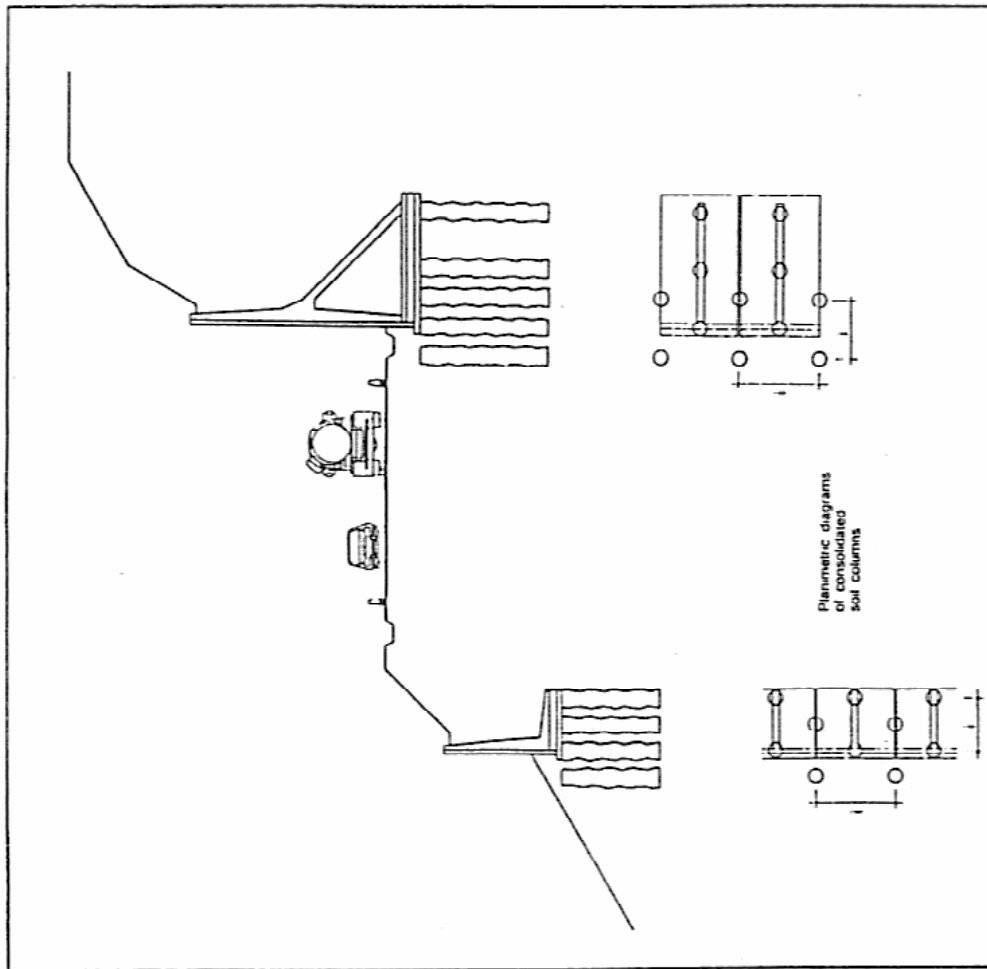


Figure 20 Columns as piles for retaining walls (Courtesy Pacchiosi Ltd.)

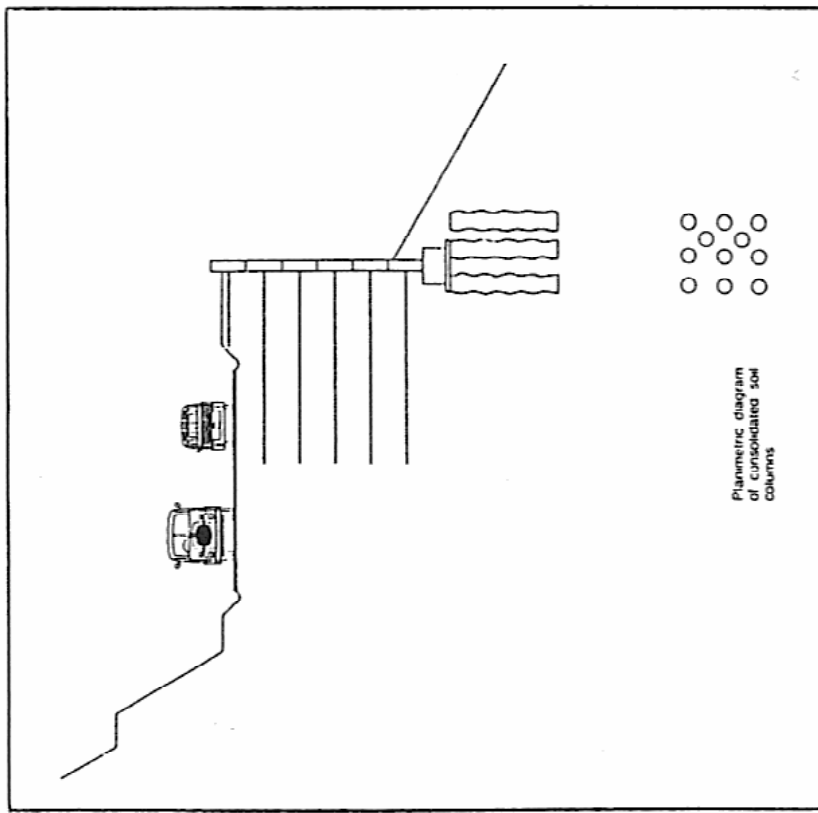


Figure 21 Columns as piles for reinforced earth wall (Courtesy Pacchiosi Ltd.)

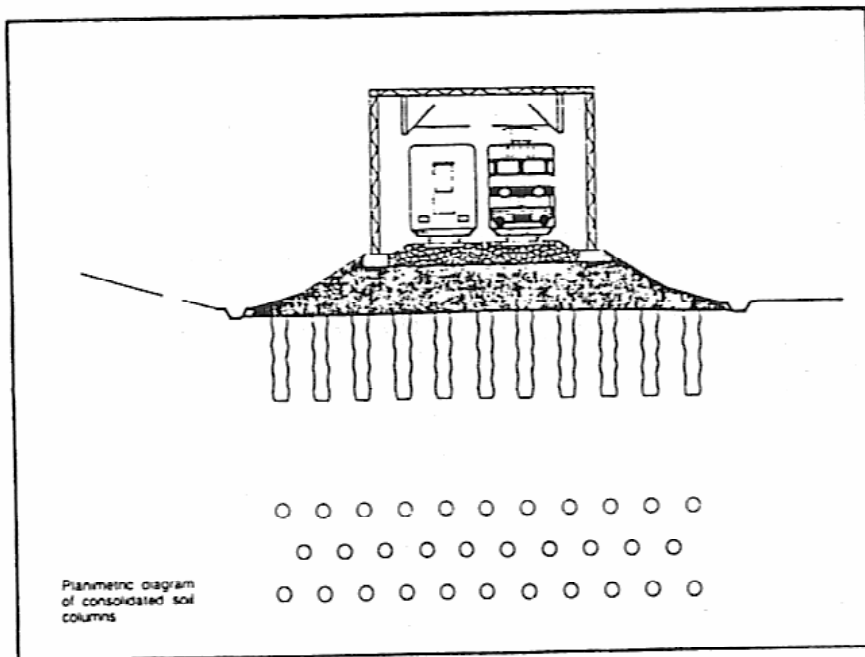


Figure 22 Columns as support for embankments (Courtesy Pacchiosi Ltd.)

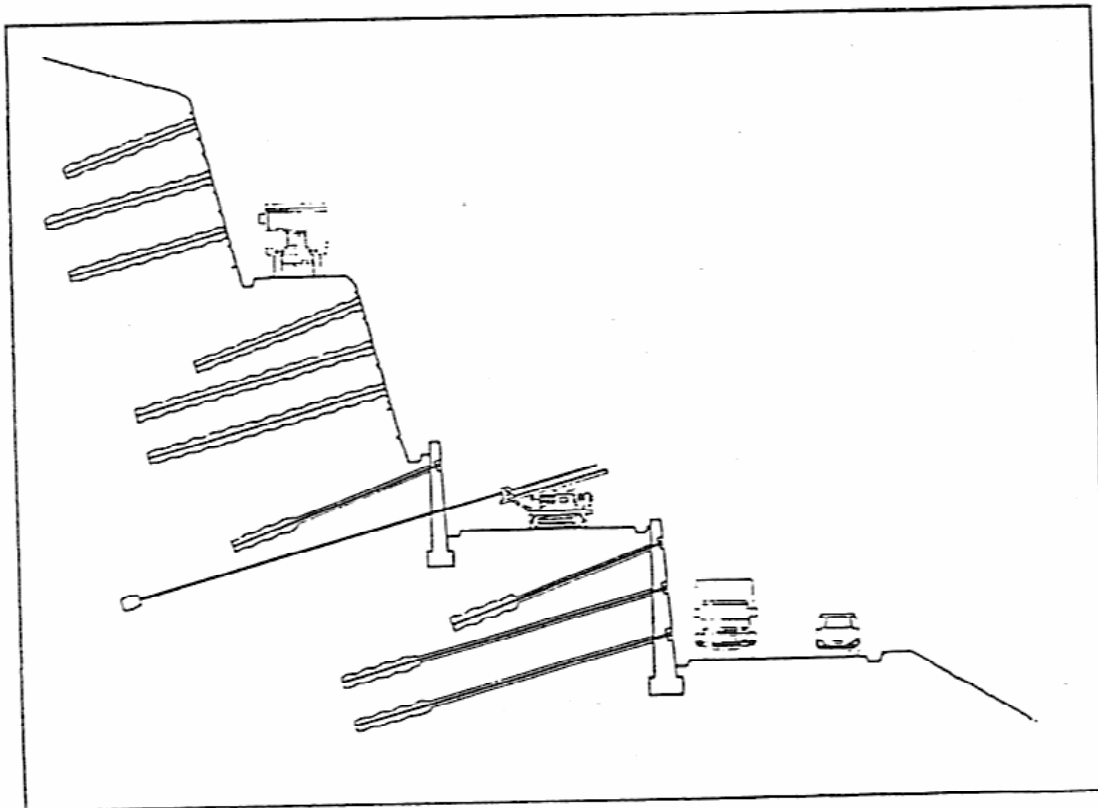


Figure 23 Slope stability, together with anchorages (Courtesy Pacchiosi Ltd.)

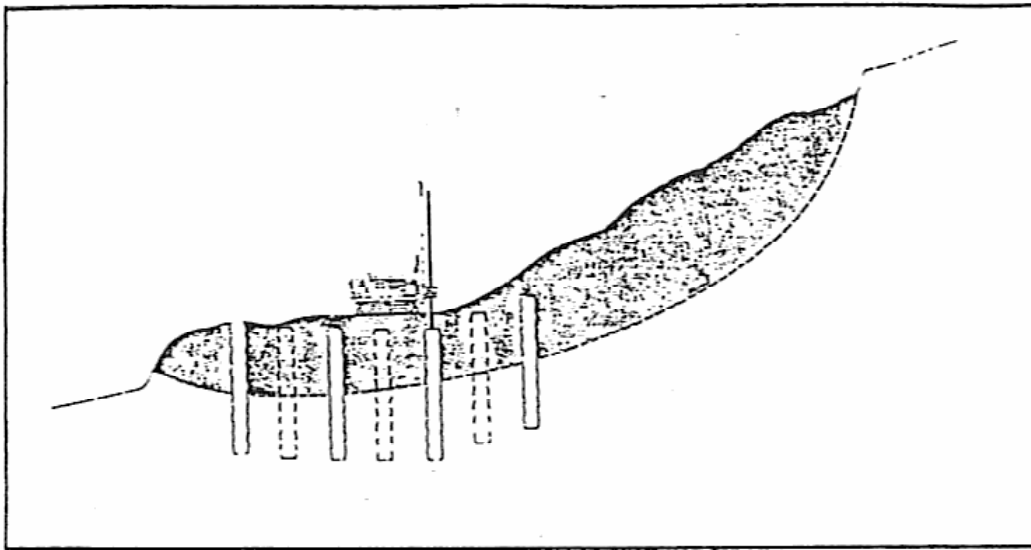


Figure 24 Shear keys in shallow slopes (Courtesy Pacchiosi Ltd.)

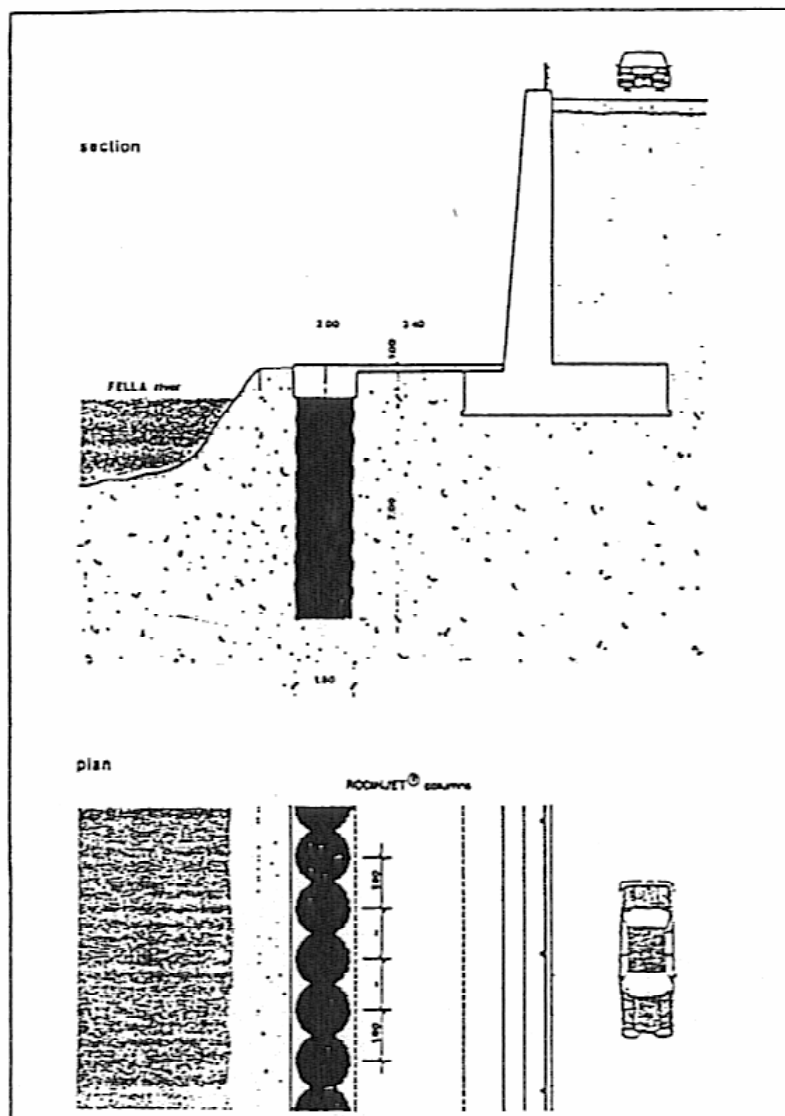


Figure 25 Protection of structure against river scour, Udine-Carnia-Tarvisio Motorway, Italy (Courtesy Rodio and Company)

5. Slope Stability

- (1) With anchorages or coil nails for steep cuts (Figure 23)
- (2) As shear keys for shallow slopes (Figure 24)
- (3) As protection against scour (Figure 25)

5. CONCLUSIONS

The techniques of insitu earth reinforcement have been demonstrated successfully in many parts of the United States, and one of these techniques - soil nailing - is one of the fastest growing geotechnical processes in the country. Although the benefits of groundwater lowering to enhance slope stability have long been practiced, the novel Italian methods of trenches and wells, interconnected at depth, afford the designer a new dimension. These techniques are of special relevance in large-scale slope stability problems which impact directly on urban developments. Ground treatment has not been a commonly applied tool of the highway engineer to date. However, the recent developments in jet grouting technology in Western Europe and the Far East could well revolutionize the way certain problems in underpinning, slope stability and deep foundations are solved in this country.

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