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# DRIVEN ANCHORS TO STABILIZE COLLUVIAL SLOPES

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16. Abstract


The experimental research with soil anchors consisted of emplacing into and removing from the ground different sizes of rebar to determine pull-out loads and thereby anchorage strength.

Driven soil nails to stabilize shallow seated slope failures appears feasible in terms of the drive process. Medium talus slopes, debris cones and slopes of colluvium are possible locations into which soil nails could be driven. A high concentration of boulders would prevent the drive process and, therefore, reduce the nail's feasibility.

The experimental maximum pull-out force was between 4,000 and 6,000 lbs. This value is less than the typical commercial minimum. Therefore, the feasibility of using only soil nails without tip anchor modfications is low. However, if deep drives are achieved or if multiple nails are used, then greater anchor forces may be obtained. In that case, the feasibility of major short term applications would be better. The feasibility of long term use is low because of the inability to protect the nail from corrosion.


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## Background

Colluvial slopes near highway construction have been a serious problem to engineers for many years. Stabilization of these slopes is an expensive process because conventional methods with vegetation are not possible. Use of drilled soil anchors is a feasible method but is expensive because of the extensive drilling required.

The construction of I-70 through Glenwood Canyon presents many geological engineering problems; among these is the stabilization of numerous colluvial slopes. Both temporary and permanent measures must be taken to allow for the construction of the roadway and protection of the highway after completion.

This report covers slope stabilization systems and the pull testing of driven anchors. There are many anchor systems available but the type tested is probably the simplest and most economical. The anchors are simply various sizes or rebar driven into the slope with jackhammers. A discussion is also included which covers the soil testing and modeling of the anchor systems.

The object of this study was to determine the feasibility of using pile-driven anchors to stabilize colluvial slopes. Sufficient pull-out resistance must be demonstrated and correlated with the geology and soil properties of the deposits.

The evaluation of driven anchors to stabilize colluvial slopes was performed as part of a masters thesis for a degree in Geological Engineering at the Colorado School of Mines. The remainder of the thesis was an in-depth study of the geology of Glenwood including the causes and location of colluvial deposits.

## Slope Stabilization Systems

Slopes are stabilized either by reducing the driving force or by increasing the resisting force. The former involves removing slope material either from its head or along its length, thus removing the driving tangential component of weight. The later involves reinforcing the toe area of the slope, but may include reinforcement methods or devices along the entire length of the slope, thus increasing the resisting normal component of weight. Slope dewatering, such as horizontal and vertical drains, results in a combination of both methods. Slope dewatering and driving force reduction methods are not included in the following discussion.

Several systems are available for increasing the resisting force, including berms, walls, anchors and a variety of slope stabilization techniques as described below. Berms consist of weighting the toe of a slope with unconsolidated fill in order to increase the normal load.

Slope stabilization techniques consist of a variety of procedures that are designed to act over a broad area of the slope, and are applied to the surfce or injected into it.

Surface stabilization includes vegetation, compaction, surface contouring organic mulching, asphalt or paving, netting and artificial mats (Jensen, 1981). Injected stabilization involves drilling an injection hole, through which concrete grout (Baker, MacPherson and Cording, 1981), chemical grout (Tallard and Caron, 1977), or any other adhesive agent such as epoxy, asphalt or lime, is transmitted into the slope mass. The bonding nature of these materials unifies the slope mass and increases stability.

Wall stabilization systems involve reinforcing the lower portion of a slope with a retaining structure. Because of the need in recent years to develop previously unacceptable areas, modern construction and design techniques have resulted in the development of many wall types, which include (Driscoll, 1979):

Gravity Type - binn walls, concrete and timber cribs, gabions and concrete structures.

Reinforced Backfill - reinforced earth, fabric reinforcement and stack sack walls.

Cantilever Pile - sheet piles and H-piles with lagging.

Anchored Walls - H-piles, sheet piles, vertical culvert pipe and horizontal sheet pile.

Standard Walls - typical wall with a foundation, usually overdesigned for strength needs.

Wall stabilization systems can also be utilized in portions of the slope other than the toe depending on needs and accessibility.

A final type of slope stabilization system is the anchor techique. This technique involves the installation of an anchoring device into the soil mass. Anchors consist of a durable material, such as concrete blocks or cylinders. They are directly emplaced and backfilled, or injected into boreholes. Tendons or rigid bars are attached to the anchors and extended to the sloe surface, to which, structural elements, such as walls are connected. The attached structural elements utilize the mobilized resistance force of the anchor within the soil via the connected tendons or bars, and apply a component of resistance to slope failure.

In addition to local geologic conditions and stability requirements, the cost and benefit of a particular stabilization system dictates its desirability. Many of the techniques described above are associated with high emplacement costs, and some require specialized installation techniques. In an effort to achieve a relatively inexpensive and effective slope stabilization program, research is on-going. One objective of this thesis has been to experimentally research the feasibility of utilizing a relatively inexpensive, effective and apparently non-typical stabilization technique. This experimental research is discussed in the following section.

## Experimental Driven Anchor Research

### Initial Phase

During the process of highway construction and utilization, the need to stabilize cut slopes, embankment fills, and retaining walls is one of the prime concerns of highway engineers. Variable geological conditions within, and the narrow nature of, Glenwood Canyon have resulted in the use of an experimental or test wall design for retaining structures. Several innovative retaining test walls have been constructed to maximize highway stabilities. The performance of these test walls, based on monitored instruments within the, indicates a preliminary successful stability achievement (Barrett, Derakhshandeh and Ruckman, 1983).

The initial phase of this anchor research involved a survey of commercially available systems. The survey indicated the availability of several anchors which include tie-backs, grouted tendon anchors, reinforced backfill, screw anchors, upset anchors, expanding plate anchors and deadman anchors (Smith, 1983). Figure 1 shows several types of anchors. Tiebacks, deadman anchors and reinforced backfill anchorage systems are installed as backfilled material is emplaced, and are not applicable to natural and cut slopes.

Grouted tendon anchors, expanding plate anchors and deadman anchors are installed either by drilling or excavating to the desired anchorage depth. The anchorage or resistance to pull-out is a function of either the socket-soil shear strength or overburden load. The above anchors are labor intensive, and require relatively high emplacement costs. Anchors that do not require backfill, drilling or excavation are not labor intensive, and involve relatively low emplacement costs.

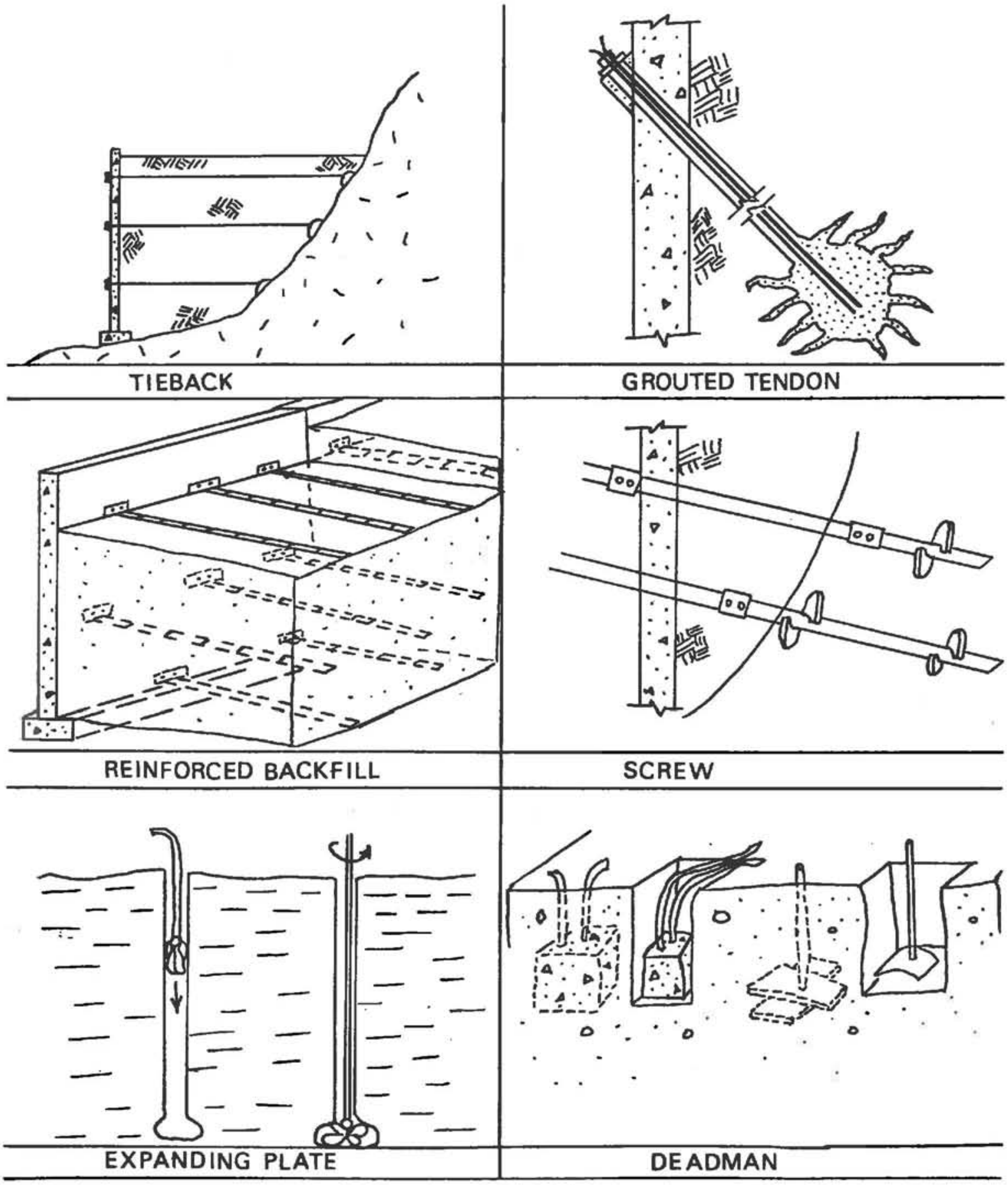



FIGURE 1 SEVERAL TYPES OF ANCHORS (SOURCE, MODIFIED FROM SMITH,1983)

The initial survey indicated that low cost anchors include screw anchors and upset anchors. Screw anchors, as their name implies, are torqued and pushed, or "screwed" into the ground. They consist of hardened steel shafts with attached plates. The shaft and plates are modeled after a screw with a very wide thread spacing. Resistance to pull-out is obtained by the combined section of soil-screw shear strength and overburden load.

Upset anchors are pushed, driven, or "pounded" into the ground. Two types of upset anchors ave available; duckbill and arrow and their names indicate their shape. Both are driven into the ground by a driving shaft. Their long axis is parallel to the direction of drive, and a tendon cable is attached to them. Upon reaching the desired depth, the driving shaft is removed and the cable is loaded with a pull-out tension force. Because of their design geometries, the tensile load causes these anchors to shift. This results in a realignment of their long axis, which rotates into a new position approximately perpendicular to the original drive position. The new position provides a larger contact area between the anchor and the overburden, and the anchor becomes locked within the soil mass. Additional pull-out resistance is encountered and becomes a function of overburden load. Figure 2 shows the operation of upset anchors.

The purpose of anchors is to provide a force which can be applied to resisting structural or slope failure. This force is a function of resistance to pull-out. Pull-out resistance varies with slope and soil geotechnical properties, the type of anchoring system used, prevailing moisture conditions and depth of anchor emplacement. The anchor that is used depends on the pull-out forces require and specific application needs, such as short or long term stability requirements. Another application need is a low emplacement cost. As stated above, driven anchors have lower emplacement costs. Apparently, the only commercially available driven anchors are upset anchors and in a sense screw anchors. All of the other systems require drilling, augering, excavating or backfilling, and have higher costs of emplacement and replacement.

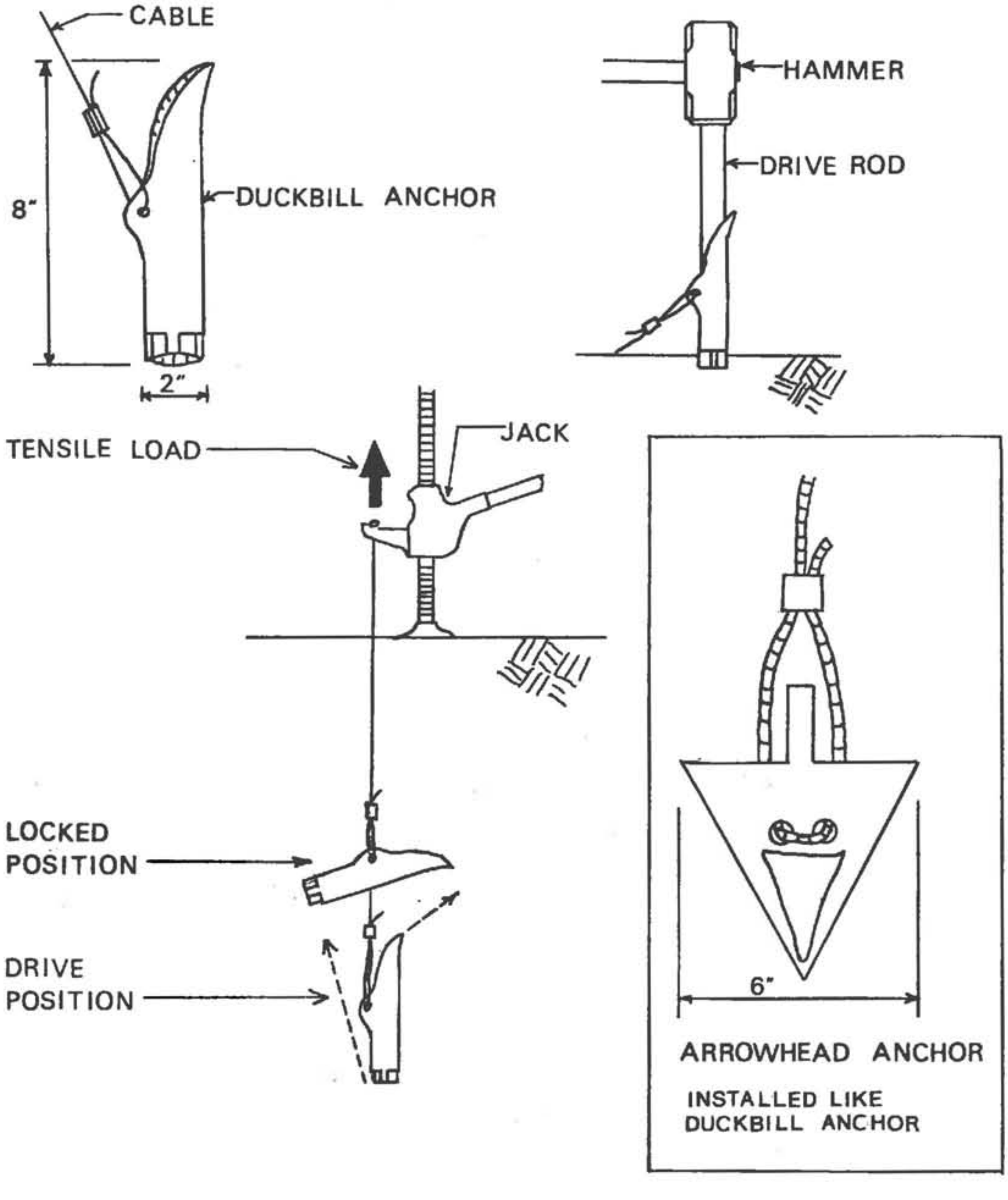
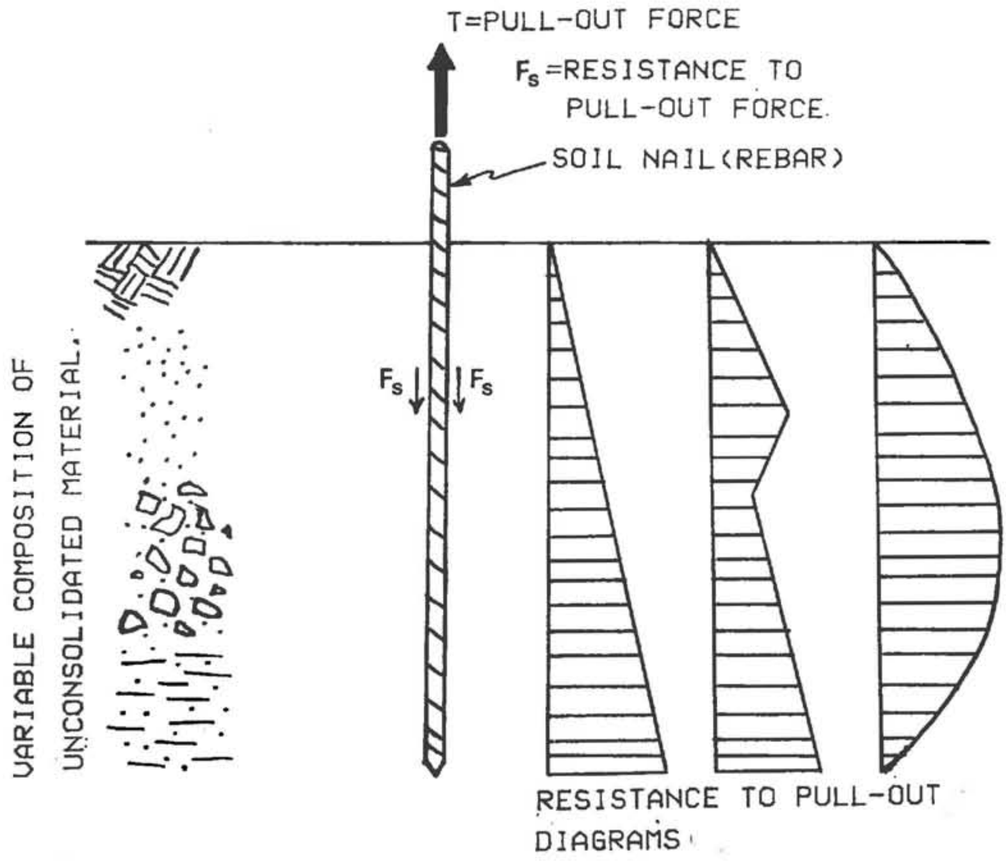



**FIG. 2 TWO KINDS OF UPSET ANCHORS.** (SOURCE, MODIFIED FROM SMITH, 1983)

This study pursued the anchoring potential of a driven system. Upset or screw anchors were not used because of their commercial availability and thus proven capacity. Instead, a system which employs slender rods was investigated. This technique, called soil nailing by some, involves the insertion of small diameter (20-30mm) rods into the ground by driving them with percussion equipment. This system does not appear to be commercially available as indicated in Appendix A. However, other research projects with them have been done, and Mitchell and Katti (1981) state that work on soil nailing has been summarized by others and that research on the subject is continuing. A research proposal draft copy, entitled, "Soil Nailing, A New Ground Reinforcement Technique" (Holtz and Juran, circa 1982) has been submitted to the U.S. Department of Transportation, Federal Highway Administration.

For this study, soil nails were driven into the ground, and pull-out tests were performed to determine resistance to pull. Subsequent to pull-out test, test pits and trenches were dug and soil was sampled at varying depths and prepared for laboratory analysis. One of the main points of concern in this research was to determine a resistance to pull diagram. Figure 3 depicts some anticipated results. The pull-out force T is applied to the rod. It was anticipated that resistance to pull is due to the skin friction (fs) between the nail surface and surrounding material. This skin friction represents the shear strength between rod and soil. However, this shear strength cannot be equated to the shear strength of the natural soil because the coefficient of friction between the two does not equate to tan o of the soil. Furthermore, the driving process undoubtedly disturbs the soil, probably compacting it as the nail passes through by displacing the soil particles. Natural soil properties within the vicinity of the nail are presumed to be replaced by disturbed soil properties.




**LOAD DIAGRAMS ARE BASED ON PERSONAL ASSUMPTIONS.**

FIG. 3 : SOME POSSIBLE RESISTANCE TO PULL-OUT DIAGRAMS.

The complete research program for this study involved several phases. Because of the non-commercial use of this method, predesigned materials and equipment were not readily available. Suitable and compatible material and equipment for the research were selected based on anticipated needs. In order to complete the research inquiries of available supplies were made to local vendors. In-house design, fabrication and lab tests were employed and advice from technical service companies were sought.

Planning Phase and Test Procedures

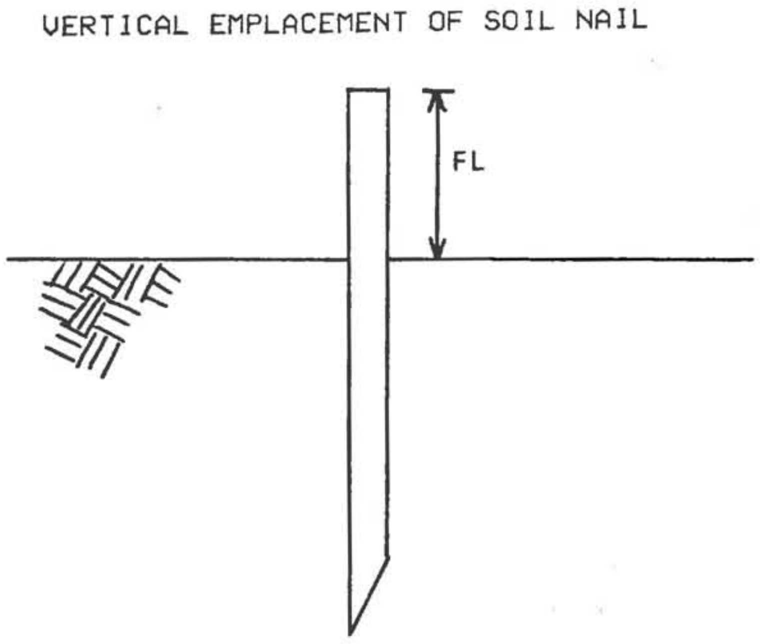
Obtaining a suitable nail was central to the entire program. It was decided that rebar rods best matched the description of soil nails. Several different sizes of rebar in terms of length and diameter were planned for the test. In addition, two tip geometries, OCP (off-center point) and CP (center point), were used. Rebar lengths included 6, 8, 13, and 18 foot. Diameters were one-half, three-quarters and one inch. Different size rebar and point geometries were used in order to determine if pull-out load was affected by rebar variability. Rebar is not a smooth steel rod, but has a surface of corrugated helixical torus-like ribs located along its entire length. Because of this arrangement resistance to pull is higher than for a smooth surface nail. Rebar specifications, point geometries and shape are indicated in Appendix B.

Two different emplacement angles were desired; a vertical emplacement into a flat surface and an off-vertical emplacement into a sloping hillside. Although the ultimate application of soil nails deals with off-vertical emplacement into hillsides to achieve slope stability, it was decided to also test vertically emplaced nails. Hillside emplacement, because of the sloping surface, would involve an awkward working environment. This fact, coupled with the fact that test procedures would be performed by inexperienced personnel, suggested the need for a controlled test site.

It was decided that a level surface into which vertical nails were emplaced, would provide a satisfactory working environment within which test procedure skills and experience could be obtained. The experience gained from the vertical test site would be applied to a better handling of the test within the more difficult condition of the hillside. Furthermore, data from boreholes within the vertical test site provided sub-surface information relating to what soil types would be encountered. Also the vertical test site was used as a control or reference to which the off-vertical test site could be compared. Figure 4 is a diagrammatic representation of the two methods of emplacement.

Having decided on the type of soil nails and their emplacement geometries, the next part of the planning phase was to determine an appropriate test site. The primary limitation of driven soil nails is that they cannot be utilized within areas of high boulder concentrations because of the inability to drive them into rock. It was assumed that high clay content soils would be inappropriate for soil nails because the apparent creep properties of these soil would tend, in time, to reduce the holding capacity of the anchor. Granular soils appeared to provide the best anchoring medium.

Based on these limiting conditions, in conjunction with borehold log data (Figure 5), the existing surface expression of the side slopes, suggestions from R.K. Bartett, and the ease of accessibility, an area approximately 700 feet east of Grizzly Creek appeared to offer the most appropriate test site within the study area. The site was divided into two locations. The vertical soil nails were planned for an alluvial terrace environment ($at_1$) located between boreholes TH 502 and TH 503. The off-vertical nails were planned for a colluvial slope environment located in an area slightly uphill from borehole TH 501. Designated boreholes are referenced to the numbering system on file with the Colorado Division of Highways. Figure 6 delineates the two test site areas.



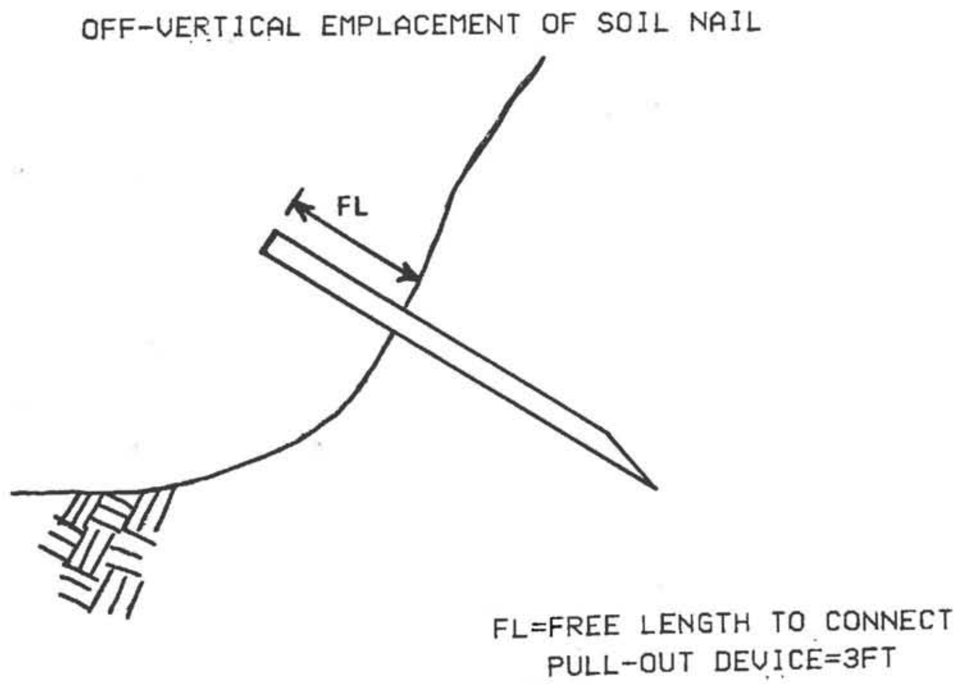




FIG. 4 TWO EMPLACEMENT GEOMETRIES.

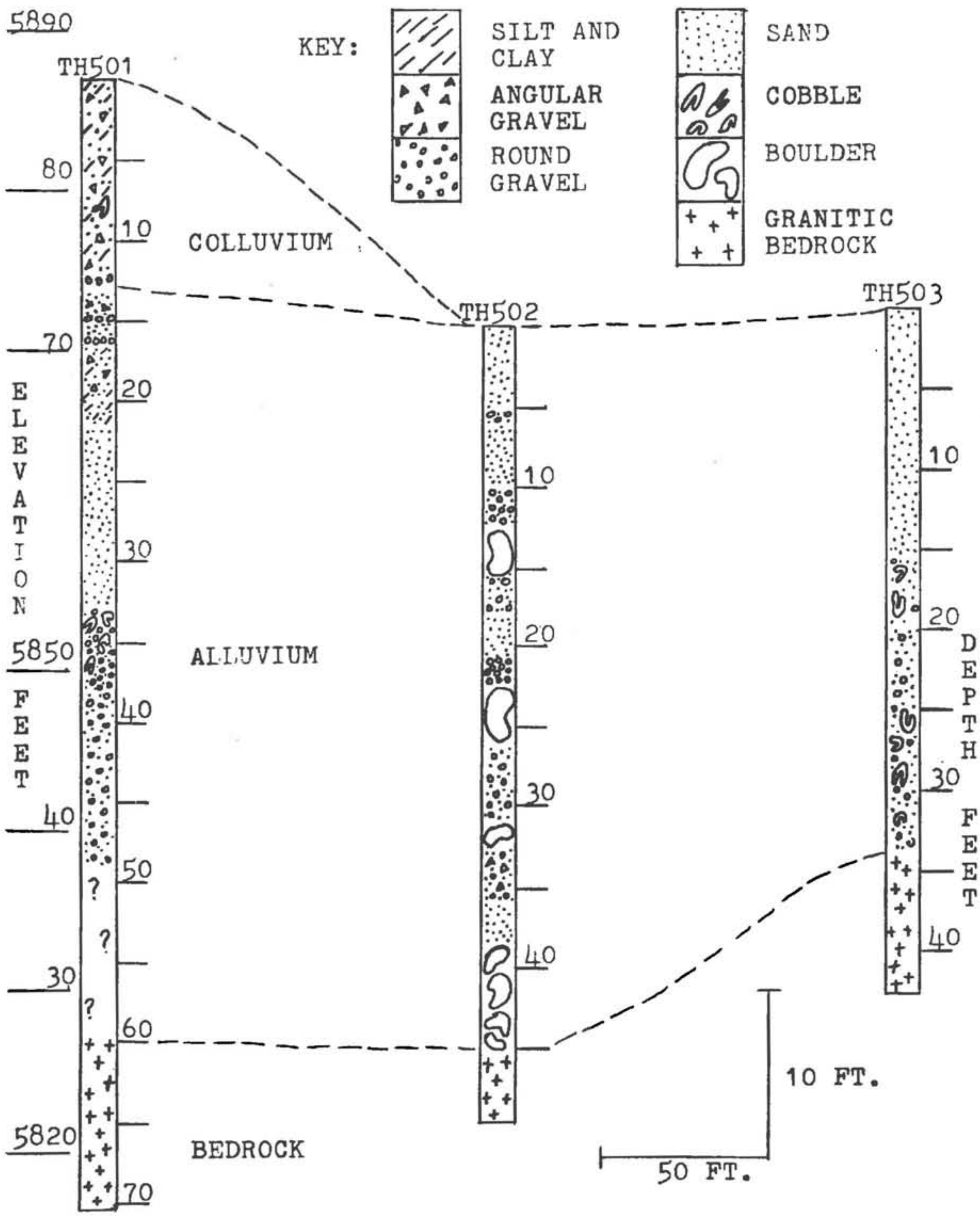



FIG. 5 BOREHOLE LOGS FROM THE TEST SITE.

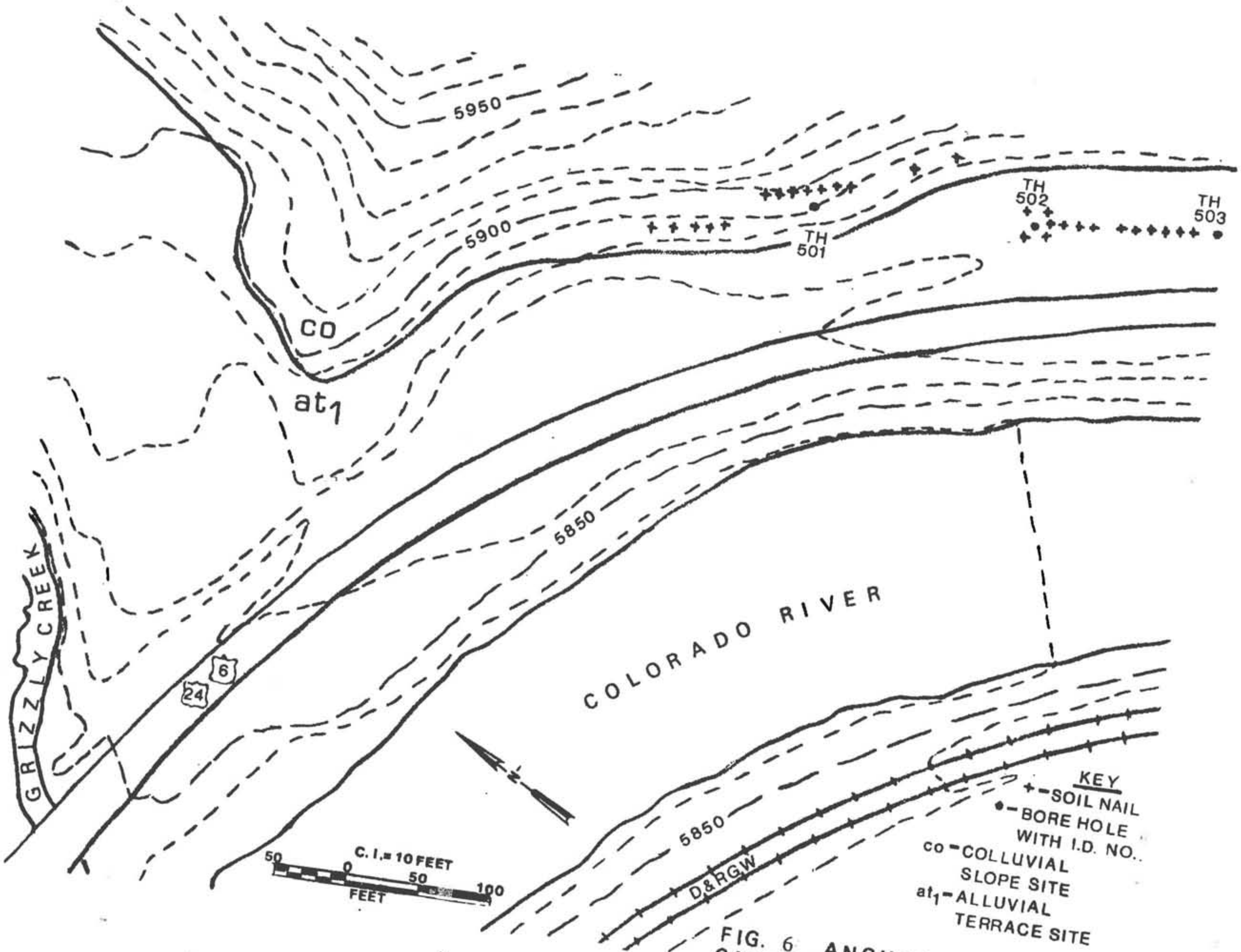



FIG. 6 ANCHOR TEST SITE LOCATION WITH TOPOGRAPHY.

A total of 28 rebar soil nails were planned for emplacement. In order to compare test results, rebar size and point geometry variabilities were duplicated, in both areas of the test site. It was proposed that 14 different nails be driven into the alluvial terrace site and a duplicate set of 14 different nails be driven into the colluvial test site. A chart of rebar size and point geometry is given, in Appendix C, where one each of the different types were used for each test site.

The alluvial site consists of river quartz sands. The granular nature of this site more or less ensured that emplacement would be successful. The colluvial site was located in the lower portion of a hillside. The soil of this site consists of "P" size angular gravels of feldspar and quartz intermixed with a finer fraction of clay and sand. Sporadic cobbles and small boulders are scattered within the soil matrix. The colluvial site is underlain by alluvial deposits at a depth range of 10-15 feet. The low concentration of cobbles and boulders indicated that the drive process would probably not be hampered.

The soil nails were to be driven by manually operated 80 and 90 pound pavement breakers (jack hammers), using compressed air as the driving force. Because of the long rebar length (maximum = 18 feet), the nails could not be driven by simply mounting the hammer to their tail portion. Therefore, a drive tool, which could be secured to the nail at a conveniently reachable height, was required. Appendix H shows the design of the drive tool. It was to be attached to the nail at about 3 feet from the penetrating tip, and then driven with hammers. Upon reaching the level of the drive tool, the tool would be detached, lifted to a new drive position, resecured and driving resumed. This process was repeated until the nail was driven to the desired depth. In all cases, a 3 foot free length of nail was left protruding above the ground surface in over to facilitate the pull-out process. The in-ground length of each nail would be 3 feet shorter than the total rebar length indicated in Appendix C.

The pull-out process required several pieces of component equipment, in total called the pull-out jack. The jack, a hydraulically driven system, consisted of a hollow-center ram, a hydraulic pump with gauge to indicate psi pressures and a clamping device which was to be attached to the rebar, into which the ram would push as it was loaded and thus pull the soil nails from the ground. A schematic of the pull-out jack is shown in Appendix E.

Upon removal of the soil nails, test pits and trenches would be dug by a backhoe tractor in the vicinity of the previously emplaced nails. Soils samples at specific depths were to be taken, catelogued and prepared for shipment to the Colorado Division of Highways soil laboratory for testing of soil geotechnical properties.

Material and Equipment Procurement Phase

Soil nails were obtained from a rebar supplier (Colorado West) in Grand Junction. Desired lengths and point geometries were cut and fabricated by the supplier.

A drive tool was commercially unavailable. Therefore, one had to be specifically designed as indicated in Appendix D. Based on this design, the drive tool was fabricated and modified by Colorado Division of Highways machine shop personnel from Grand Junction and Glenwood Springs. Extra effort to fabricate and modify the drive tool was provided by Steve Foster (driller, Colorado Division of Highways).

The pull-out jack design and fabrication were beyond the available resources of the Colorado Division of Highways. Therefore, it had to be purchased. Jack suppliers within the Denver metro area were contacted and a list of appropriate suppliers was generated. The critical jack specification necessary to perform the pull-out test was that the diameter of the ram's hollow center had to be large enough to accommodate the largest nail diameter. the minimum diameter of the hollow ram was to be 1-1/8 inch. The pull-out force capacity of this ram is 30 tons, and was considered to be sufficient for pulling the nails. Because of additional future applications and a small extra cost, a ram with twice the pull-out capacity (60 tons) and three times the minimum hollow ram diameter (3 inch), was ultimately decided upon.

<u>Drive Phase</u>

Soil nail rebars of different lengths, diameters and point geometries were driven into each of two test sites. A total of 28 nails were emplaced. Table 1 indicates the number code, length, diameter and point geometry of each nail.

To drive a nail, the drive tool was attached, drive hammers were positioned on the tool, activated and the nail was driven into the ground. The drive tool was designed to employ two drive hammers, but for the shorter length nails, one hammer was sufficient. Longer nails required to hammers.

The vertical nails within the alluvial test site were driven first. The driving time was directly related to the driving length and the number of driver hammers used. The shorter length and smaller diameter bars were driven to their desired depth with 20 seconds of drive time. The larger bars took from 150 seconds to 600 seconds of drive time. The actual emplacement time, because of setup time, reset time, rest time and drive time, took from a few minutes for the smaller bars to well over two hours for the larger ones.

TABLE 1 Identification code for driven soil nails.

| Vertical Drive (at$_1$site) | | | | Inclined Drive (co. slope site) | | | |
|---|---|---|---|---|---|---|---|
| Code No. | Length[1] (ft.) | Diam.[2] (in.) | Point[3] | Code No. | Length[1] (ft.) | Diam.[2] (in.) | Point[3] |
| 1 | 6 | .5 | OCP | 15 | 6 | .5 | OCP |
| 2 | 6 | .5 | CP | 16 | 6 | .5 | CP |
| 3 | 8 | .5 | OCP | 17 | 8 | .5 | OCP |
| 4 | 8 | .5 | CP | 18 | 8 | .5 | CP |
| 5 | 8 | .75 | OCP | 19 | 8 | .75 | OCP |
| 6 | 8 | .75 | CP | 20 | 8 | .75 | CP |
| 7 | 13 | .75 | OCP | 21 | 13 | .75 | OCP |
| 8 | 13 | .75 | CP | 22 | 13 | .75 | CP |
| 9 | 8 | 1. | OCP | 23 | 8 | 1. | OCP |
| 10 | 8 | 1. | CP | 24 | 8 | 1. | CP |
| 11 | 13 | 1. | OCP | 25 | 13 | 1. | OCP |
| 12 | 13 | 1. | CP | 26 | 13 | 1. | CP |
| 13 | 18 | 1. | OCP | 27 | 18 | 1. | OCP |
| 14 | 18 | 1. | CP | 28 | 18 | 1. | CP |

[1] In-ground length is 3 ft. less than indicated total length.

[2] Off-rib diameter.

[3] OCP - off-center point.
CP - center point.

In all cases, when the drive process was complete, it was observed that the surface within the vicinity of the nail was distorted. The surface distortion consisted of a very shallow hole around the nail. These holes closed or caved at a depth of 1 to 2 inches, in a tapering fashion. The surface expression of these holes was elliptical with dimensions ranging from 0.05 feet to 0.35 feet for the two axes without any preferred orientation. It is believed that vibrations from the drive process had caused the underlying granular material to compact, and experience a volume reduction at depth causing a shallow surface collapse within a few inch radius of the nail. The maximum sideward looseness of the rebar nails at the ground surface ranged from negligible ( .01 feet) to 0.06 feet. It appeared that the nails were solidly locked in place within a few inches of the surface and the surface collapse holes did not extend beyond their apparent visible depth.

The off-vertical soil nails were driven into a colluvial slope shortly after the vertical drives, in late April, 1983.
However, the earlier success of the vertical drives was not duplicated in the later off-vertical drives. Equipment malfunction, redesign, and failure, in conjunction with the need to use alternative drive techniques, and the very difficult use of unsupported jackhammers on the hillside caused installation delays and non-standardized drive processes. Ultimately, all soil nails were driven and set at their required depth as planned. The average slope of the colluvial test site is 37$^{o}$, and the average angle between nail and slope was 84$^{o}$. Surface distortion holes were elliptical and ranged from negligible to 0.75 feet. The sideward looseness of the nails ranged from no deflection to 0.05 feet.
Comments and conclusion on the drive process are as follows:

1. A continuous resetting of the drive tool is time consuming.

2. It is extremely difficult to use unsupported hammers on a hillside. Balancing and supporting the jackhammer by the operator rather than driving the nails became the primary task. The hammer's vertical weight component caused the rebar nail to drift downhill, excessively distorting the drive hole and possibly affecting pull-out resistance.

3. Excessive vibrations from the hammer caused welds on the drive tool to deteriorate with hairline cracks and ultimately fail. This vibrational component of the drive energy was wasted in breaking up the drive tool rather than driving rebar. If a drive tool is used in future applications, unibody rather than weld fabrication is recommended. Also, the drive tool should be solidly locked onto the rebar nail in order to reduce excessive drive vibrations. Trapani (Glenwood Canyon project coordinator) indicated (pers. comm. April, 1983) that the Swiss have a pneumatic drive tool, which might have potential applications in soil nail driving processes.

4. In some cases during the hillside drive process, the hammer operators managed to attached the hammers directly onto the back end of longer nails by standing on top of the compressor truck. Driving the nails in this manner resulted in a substantial reduction of drive time. The rapidity and ease of this direct drive process substantiated the fact that some amount of drive energy was lost when the project designed drive tool was employed. a discussion with the hammer operators dealing with the probable productivity of the direct approach method, indicated that three or four, 15-foot long nails could be driven per hour. The drive tool method took 2 hours to drive one nail. In terms of production, the direct approach is most appealing.

5. In order to directly drive very long nails without the use of scaffolding, either a layered cut slope or segmented nail is required. The segmented nail consists of driving the nail in 5 foot sectional lengths of threaded (dywidag) of other spliceable rabar. Rock bolts used in tunneling are the dywidag type. The dywidag is very convenient because it is threaded, and the 5 foot sections could be jointed by threaded couplers similar in diameter to the nail. This would reduce drive hole distortions. Splices and couplers need to withstand both compressive and tensile loads.

<u>Pull-Out Phase</u>

Because procurement of the pull-out jack was delayed by several month, the soil nails were not removed until the later part of the summer of 1983. The nails were embedded for three months before tensile loads were applied.

The pull-out process consisted on the following:

1. Place the hollow cylinder ram over the rebar nail, and lower into pull-out position.

2. Place protective steel plate with center hole over the top surface of the ram.

3. Lower rebar clamping device (upper drive tool collar) down the length of the rebar and come into flush contact with the protective steel plate.

4. Lock clamping device onto rebar.

5. Pump the ram up with hydraulics, and as the ram moves upward immediate contact with the clamping device causes the rebar to be pulled from the ground.

6. Record pump gauge pressure reading at the first incremental upward movement of the rebar nail. Measure incremental displacement of the nail and correlate to pump pressure readings. Continue pumping and record pressure changes with corresponding ram displacement values until the full-length of ram travel is reached.

7. Reverse hydraulic flow, pump ram into collapsed position, lower protective plate, unlock and lower clamping device into original position, relock clamping device, and resume pull-out process as in 5 and 6 above.

8. Repeat 5, 6, and 7 resetting the pull-out arrangement until the nail has broken free, and no more pull-out resistance is observed.

Due to the fact that the maximum extended pull-out length of the ram is 12 inch, steps 5, 6, and 7 had to be repeated several times for each nail. In order to avoid settlement of the ram into the ground resulting in erroneous pull-out displacement readings, the ram was positioned onto two 3"x3"x3.5' timbers, which displaced the pull-out reaction load over a larger area. The recorded data consisted of pump gauge pressure (psi) readings, ram displacement in 0.05 feet increments, time of pull-out in seconds, and any other pertinent observational data associated with the pull-out process.

The minimum number of personnel required for the pull-out test was three; a pumper and gauge reader, a ram displacement measurer, and a data recorder.

Two types of hydraulic pumping procedures were used: manual and motorized. The first 2 feet of rebar pull-out was performed by manually pumping up the ram, and detailed pull-out readings were obtained. The number of pump strokes (up and down) for a complete ram displacement (1 foot) was 600. This multiplied by several set ups per nail and by 28 nails constituted a major human work effort.

Therefore, a procedural modification, in terms of using a motorized hydraulic pump (log splitter), was suggested by Roy Garner, a Colorado Division of Highways drill forman. The modification was successful and proved to save much time and effort. Following the initial manually pumped 2 feet pull-out for all the nails, the motorized pump was used to pull the remaining lengths. Pressure readings for the two methods were consistent, and only the rate of displacement and data detail differed.

The vertical nails were removed first in order to gain procedural experience in the less awkward environment of the level surface. Modifications, as indicated above, and test procedure adjustments, which improved the operation, were made. The manual pumping phase of the test, as stated above, consisted of individual discrete strokes of the pump, which required about 2 seconds per cycle (one up and one down stroke) to complete. With 300 cycles (600 strokes) per 1 foot of pull-out, the manual process required 10 to 15 minutes per foot of pull-out. The manual pump pull-out reflected a series of individual discrete 2 second load conditions. In contrast, the motorized process, whose time requirement per foot of pull-out ranged from 40 to 80 seconds (depending on the in-ground length of rebar), reflected a continuous load condition. Due to the different pull-out methods and a non-sophisticated timing of the events, a rate of pull-out analysis was not pursued in this study.

After each foot of ram pull-out, it was reset to continue the process until low psi readings indicated that the rebar had broken free. After pulling out about half of the vertical nails, it was noted that inconsistent psi values were being registered and creaking noises were issuing from the ram housing. Equipment malfunction was suspected, and therefore, ram calibration tests with known loads and no load were performed. The calibration tests and resulting data curves indicated a ram malfunction, and further pull-out tests were discontinued. The ram was returned to the manufacturer for malfunction determinations and repairs. Apparently, either improper sealing, packing or machined tolerances had caused a gradual deterioration of the ram's ability to perform. Pre-pull-out calibration tests indicated a 50 psi internal ram frictional load. Later calibration tests, when malfunction was suspected, indicated a maximum internal ram frictional load of 900 psi. Following repair, the ram frictional load was again 50 psi; a load which is apparently typical for a 60-ton capacity ram (pers. comm., Hy-land Hydraulic, ram manufacturer in Glenwood Springs).

Pull-out of the remaining vertical and all the off-vertical nails was resumed upon repair of the ram. Because of its heavy weight (130 lbs.), the use of the ram on the off-vertical nails proved to be difficult. Field adjustments such as sandbag supports, pre-locking and extra human effort were required to stabilize the ram and allow for proper pull-out testing procedures. The photographs of Figure 7 depict the off-vertical pull-out process. Photo (a) is of the manual pump method and photo (b) is of the motorized method. One assistant in (a) is measuring ram pull-out displacement and calls out readings as the other assistant at the pump indicates a pressure change and also calls the new value. The pull-out jack system consists of hand pump, psi gauge (not shown), two hydraulic hoses (one for lift and the other for ram return), two port cylinder with partially extended ram, protective steel plate at the top of the ram and bolted rebar clamping device for transferring ram push-out load to rebar pull-out load. Note flagged rebar in the background awaiting pull-out, and sandbags for support and proper

(a) MANUAL PUMP METHOD.

(b) MOTORIZED PUMP METHOD.

FIG. 7 PHOTOGRAPHS OF PULL-OUT PROCESS.

ram alignment to ensure that pull-out load is normal to the slope. In photo (b) note the different view of the rebar clamping device, the ram in its collapsed position, supporting sandbags, timers and motorized hydraulic system with hoses attached.

Pull-Out Test Results and Interpretations

Approximately 1550 pump pull-out pressure versus displacement readings were recorded for 26 soil nails. Data for soil nails code number 18 and 22 were discarded because of the inability to properly attach the jack. Pull-out pressure data was converted to pull-out force values (lbs), by multiplying the ram's piston effective area (11.95 in.$^2$) times the recorded pressure reading. The ram's internal friction was subtracted from the recorded pressure readings. The pull-out displacement for individual nails were cumulated to reflect rebar pull-out lengths. These lengths were correlated to their respective pull-out forces and the correlation was reduced to graphical form: Figure 8 and Plate 2.

Figure 8 shows resistance to pull-out curves. The vertical axis is pull-out length in feet and the horizontal axis is pull-out force in 500-pound increments. The dimension of each vertical axis is the total in-ground length of each respective nail. Center point (CP) nails are indicated by dotted curves and off-center point (OCP) nails by solid curves. The curves are arranged for convenient comparisons. The upper portion of the figure represents the vertical alluvial terrace site and the lower portion represents and off-vertical colluvial hillside site. In-ground nail lengths and diameters generally increase from left to right on the figure.

An obvious observation from Figure 8 is that resistance to pull increases with nail diameter and in-ground length, and decreases as the nail is pulled out. The shortest length and smallest diameter nails (code number 1, 2, 15, and 16) had an initial force of approximately 500 pounds. The longest length and largest diameter

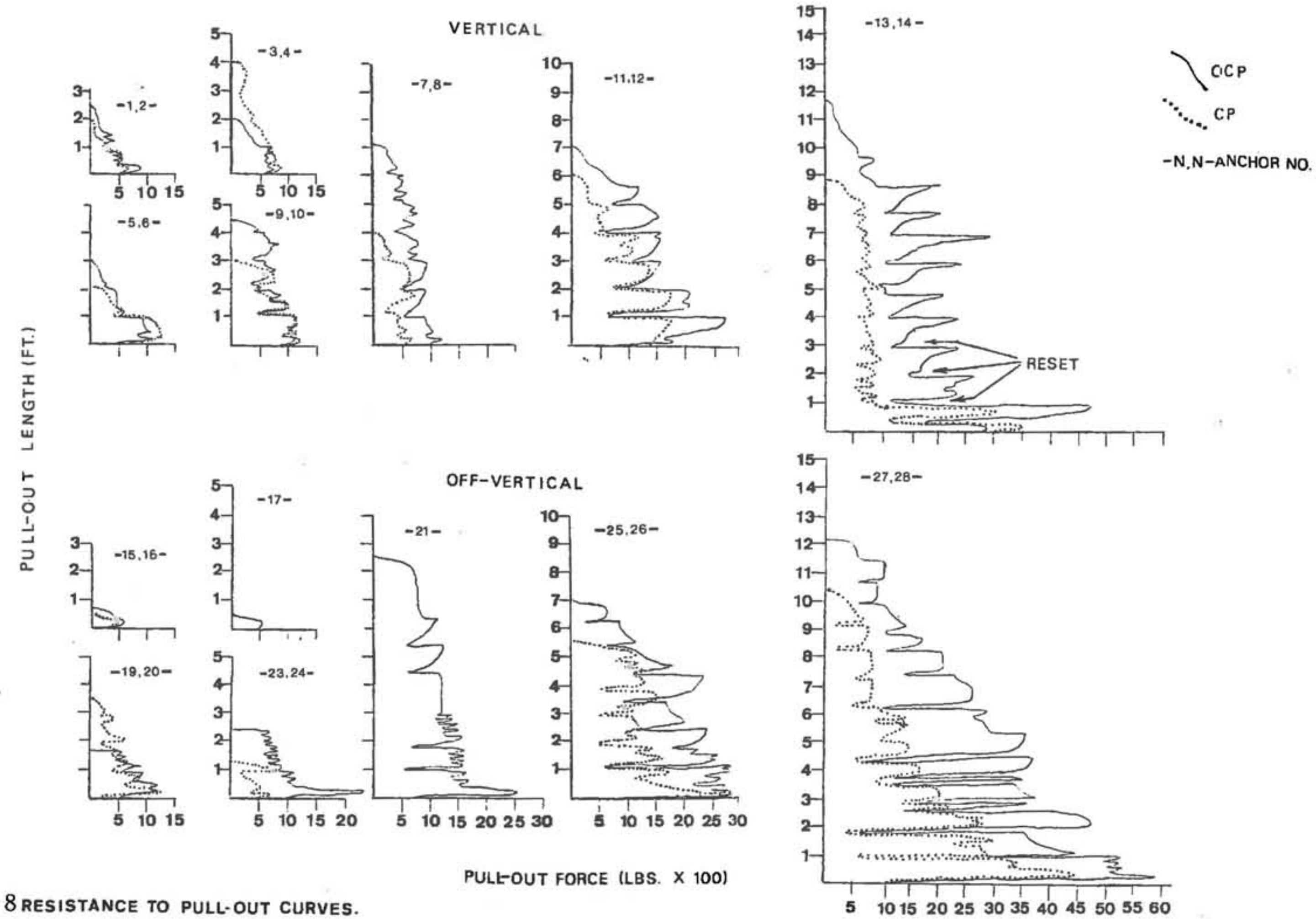



Figure 8 RESISTANCE TO PULL-OUT CURVES.

nails (code number 13, 14, 27, and 28) had an initial pull-out force that exceeded 2,500 pounds.

Other observations from Figure 8 are as follows:

1. Except for nails 11 and 13, the initial maximum pull-out force occurred within the first few hundredths of a foot.

2. Oscillating pull-out forces occurred for most of the longer nails. These oscillating values reflect the fact that as the nail was pulled out shear stresses within the soil were mobilized to resist pull. At some critical shear strength value, a local maximum resistance to pull was achieved and with continued force this critical value was exceeded, local failure occurred and the resistance to pull-out dropped. As pull-out continued, the soil shear strength was remobilized and another critical maximum strength was achieved and eventually exceeded as another local rupture occurred. The very deep drops in the pull-out force as indicated in several places on the figure represent periods of ram reset, where the pull-out force was reduced to zero as the ram was prepared for another cycle of pull-out. It can be seen from the figure that following reset, as the ram was pumped up, the pull-out force rose to a near pre-reset value as shear strength was again mobilized. Reset points typically occured at 1 foot increments.

3. With some of the longer nails, it appears that the OCP nails had a higher pull-out force than the CP nails. Perhaps the asymmetric geometry of the point caused them to bend and be driven into the ground in an arcuate manner. Upon pull-out, force components that were not parallel to the nail would develop, resulting in a larger pull-out load.

4. For the larger nails, it appears that resistance to pull is greater in the colluvial test site than the alluvial site. This difference is due to different shear strength values within the two areas. In addition, compressive forces, radial to the axis of the nail, due to displaced soil particles, probably contributed to the total pull resistance. Finally, the overburden weight, N, did not directly load the vertical nails. However, the inclined nails did experience a component of overburden weight, which resulted in a higher shear strengthen. This, in turn, required a greater pull-out force. A detailed investigation of how c, o and soil lithology differences between the two areas affect, the ultimate shear strength is beyond the scope of this study. However, soil properties of the two areas, for comparison are discussed in the next section.

Figure 8 shows the pull-out curves for the entire nail pull-out length, and only overall trends were plotted. For design and application purposes, a more detailed plot is preferred. Plate 2 is a detailed bar-graph plot of the correlated test results. The horizontal axis is pull-out force in 100 pounds increments. The vertical axis is pull-out length in 0.2 feet increments, and extends to the 1.5 foot pull-out length. This vertical limit is used because it is believed that the fist 1.5 feet of pull-out is critical in defining the potential stability of the nail.

### Soil Sampling and Laboratory Test Results

Five test pits in the vertical site and two test trenches in the off-vertical site were dug following nail pull-out tests. Figure 9 shows the relative location of pits and trenches. The pits were dug to depths of 5, 10, or 15 feet and the trenches were dug to inclined depths of 13 feet. Soil samples were removed at 3, 5, 8, 10, and 15 feet for the pits, and 3, 5, 8, 10, and 13 feet for the trenches. Bag samples of 30 pounds for classification and jar samples for

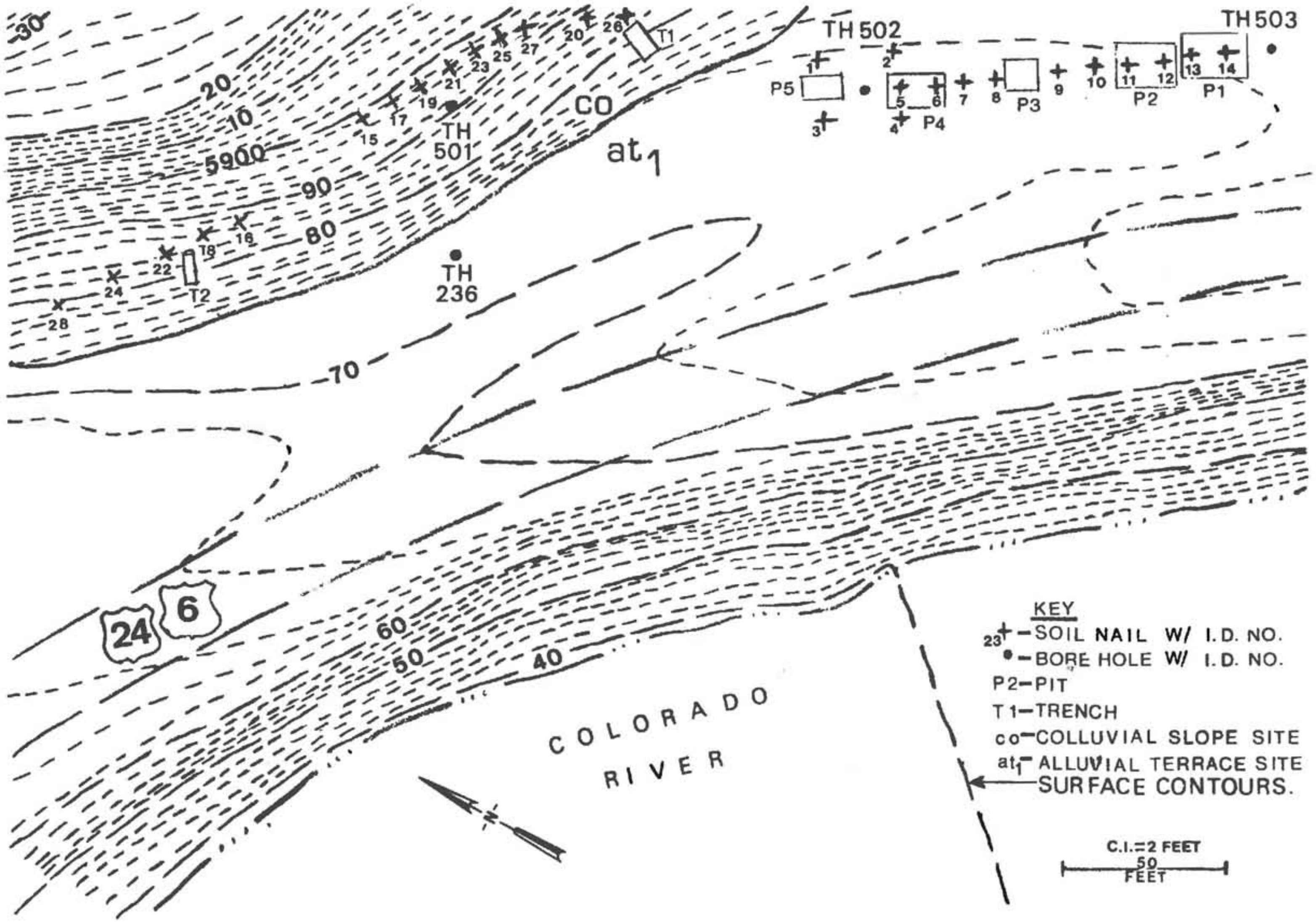



FIG. 9 TEST PITS & TRENCHES RELATIVE TO EMBEDDED SOIL NAILS.

moisture determinations were obtained. Appendix F lists sieve analysis results, liquid limit and plasticity index values, AASHTO classifications and moisture content by percent for 27 soil samples.

For the vertical site, the moisture content within 15 feet of the surface was very low, averaging 7.4%. Beneath the first 3 feet of the surface, the sieve analysis indicates generally well sorted sands with no liquid limit and no plasticity. Minor gravel lenses occur at depths in excess of 8 feet.

Sieve analysis for the first 3 feet of the vertical site indicates an appreciable amount of fines ( 0.002mm), with minor amounts of gravel and sand. The liquid limit for this upper zone averaged 23%, with a low plasticity index of 4%. In general, the vertical site, below 3 feet, consists of non-cohesive granular material of an alluvial environment as indicated by borehole logs for the area. The top 3 feet is a colluvium-like silty soil.

The laboratory tests for the hillside test site indicate an equally low moisture content averaging 7.7%, a high concentration of fines ( .002mm), with an overall liquid limit to the 10 feet depth of 24%, and a low plasticity index of 4%. In general, the upper 10 feet of inclined depth consists of a diverse mixture of poorly sorted colluvium, with a higher than anticipated fraction of fines. Below the 10 feet inclined depth, the material consists of alluvial sands and gravels. Figure 10 depicts a detailed sub-surface cross section of the two test sites, and demonstrates the relative relationship of the pits and trenches.

Four samples from pit number 1 from the 3, 5, 8, and 10 feet depths were tested for effective c and o stresses. The 3 feet depth sample which closely approximated the colluvium from the hillside was triaxially tested as a remolded, consolidated undrained sample; $c$ = 2.1 psi and $o = 31.0^{o}$. The other samples were tested by direct

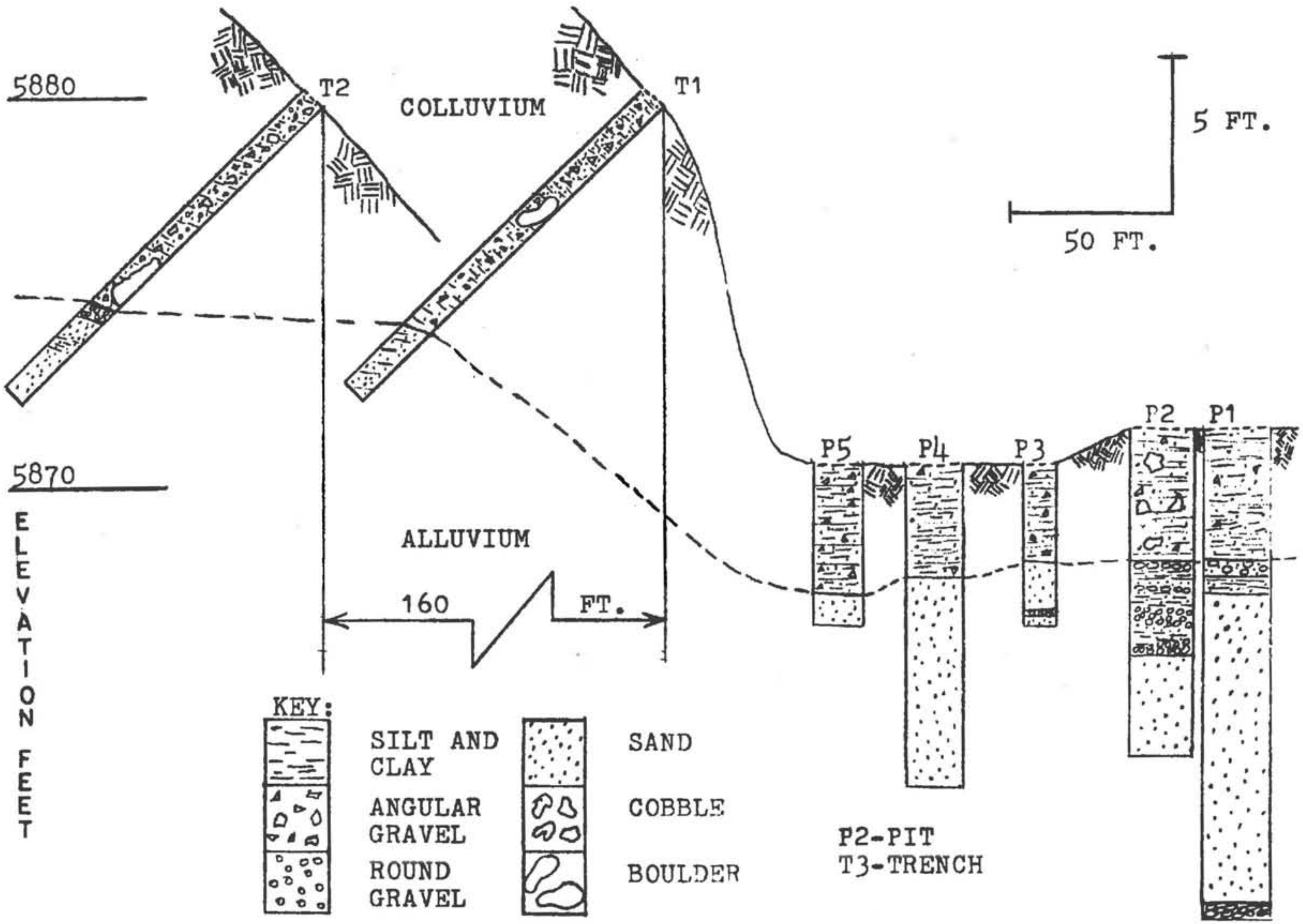



FIG. 10 DETAILS OF TEST SITE SUB-SURFACE.

shear for which c = o in all cases, and o = $46.5^{o}$, $41.0^{o}$, and $37.0^{o}$ for the 5, 8, and 10 feet depths respectively.

Conclusions and Recommendations

State of the art soil anchors incorporate the use of injected grout which spreads into a bulbous mass within the soil and to which tie rods are attached. The grouted bulb is intimately bonded to the soil mass by the concrete grout flowing into voids. Essentially, the length of the bulb is the bond length. Based on the knowledge of this length and soil shear strength properties, anchor pull-out loads can be determined. Alternatively, if an anchor design load is specified, calculations can be backtracked to determine bond lengths, and thus bulb dimensions.

In the case of driven soil anchors, or soil nails, a grout bulb is non-existent. Therefore, the nail, itself is both anchor and tie rod. Determining the bond length of the nail enables a theoretical estimation of pull-out resistance.

The length of the nail that is bonded to the adjacent soil mass develops a resistance to pull-out. In a reverse sense, this can be compared to the shear strength that develops in a purely skin friction supported pile. The pull-out resistance that develops in a nail appears to be a combination of particle adhesion to the nail and soil shear strength within some small disturbed radius of the nail. Because of the rib-like nature of the rebar nail used in this study, it was observed that soil particles (granular and cohesive) had adhered to the nail in the space between the ribs. Furthermore, because of the drive process, it is believed that soil within some small radius of the nail is disturbed and thus normal in-situ radius of the nail is disturbed and thus normal in-situ geotechnical properties are changed. Therefore, the pull-out shear stress, based on the natural c and o soil parameters would probably be erroneous. Additional parameters such as the coefficient of friction between the nail and soil, and the properties of the disturbed soil around the nail need to be considered.

As an example, reference is made to Figure 11, which represents soil nail number 27, in its embedded state. From Figure 8 it is observed that the maximum pull-out force for this nail was 5800 pounds. As a theoretical estimation, the pull-out force can be approximated from:

P = Lb x  x d x  (PTI, 1980)

where:
P = pull-out force
Lb = bond length
d = nail diameter (1 inch)
 = assumed shear stress between nail and soil

The value of  , as indicated in previous sections, is dependent on soil parameters c,  , and o. Repeating the shear stress equation:

 = c +  tan o

where:  = (soil unit weight) x H(depth of soil)

From field penetrometer tests,  = 140 pounds/cubic feet; from slope geometry and rebar average midpoint, H = 10 feet; from lab tests of sample pit #1 @ 3 feet, which is similar to the colluvial slope, c = 302 pounds/square feet, o = $31^{o}$; based on nail break-free length, bond length is assumed to be 10 feet. Solving these equations yields a theoretical pull-out force of 3000 pounds. If the full 15 foot of nail length is assumed to be bonded, the pull-out force is 4,100 pounds. Both calculated values are less than the test pull-out force of 5,800 pounds. Factors other than the soil geotechnical properties appear to influence pull-out force, such as point geometry, near-nail disturbed soils and soil-nail adhesion.

The application of soil nails for slope stability and retaining wall anchorages depends on the ability to drive them. This study demonstrated that they could be driven into granular soils and colluvium. It may also be possible drived nails into medium talus debris of tabular shaped rock fragments. The flat dimensions of these fragments result in their coming to rest in a preferential semi-bedded orientation with many voids. Driving nails into this

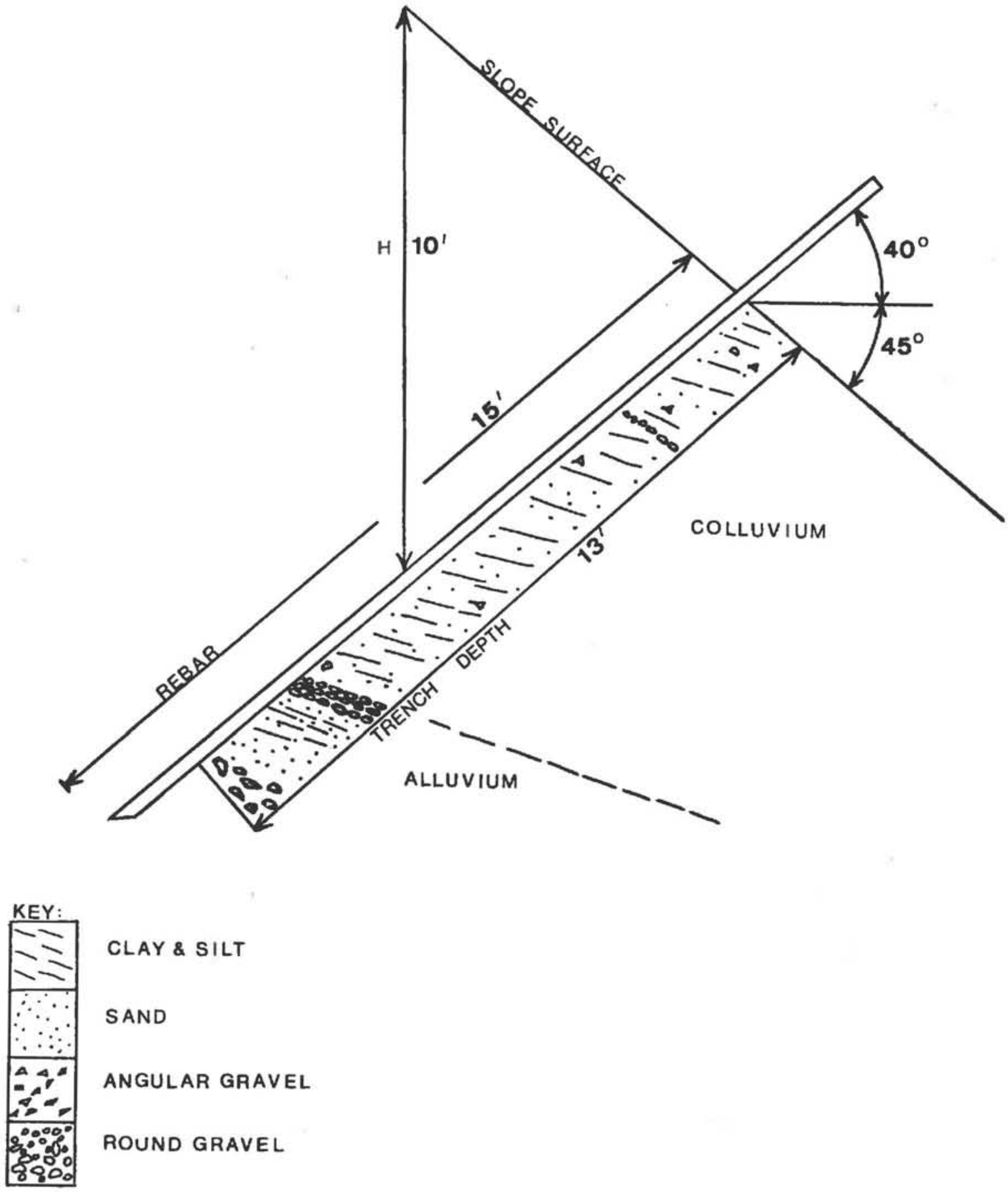



FIG. 11 SOIL NAIL NO. 27, FIELD CONDITIONS.

type of geologic environment appears to be feasible. Many medium tabular talus deposits occur adjacent to the Sawatch Formation. Medium talus chutes with ravel runs might also be stabilized in a similar manner. The effective use of soil nails within talus deposits of coarse and blocky debris is unlikely.

Because of their unprotected exposure to potential corrosive agents, soil nails have a limited useful life. Overdesigned diameter sizes would extend their usefulness. Corrosive activity would dictate this design decision In general, it appears that soil nails would best be applied to temporary construction activities.

If corrosion is not a problem, a longer term use of soil nails is possible. Stabilizing unconsolidated deposits on the upper portions of rock cuts would thus be possible.

The use of soil nails with temporary retaining walls, based on the tested pull-out values, appears feasible. The ability foresite pull-out failure from the outward directed forces of the wall would depend on driving the nail to a suitable depth. Presumably, from Figure 8 resistance to pull-out increases with the nail's in-ground length. This length is controlled by the ability to accomplish deep drives. The alternative to a very long nail is to drive many nails. The amount of multiple nail anchors would depend on design requirements, but additional soil nail testing is required to determine if cumulative resistance to pull-out is achieved. The spacing of multiple soil nails also requires additional testing. Based on field observations during the pull-out test, it appears that 1 foot spacings are feasible.

Soil nails may be applicable to shallow plane and circular failure conditions. If the nail could be driven to depths beyond the potential rupture surface, it may be possible to compress the soil mass, provided the consolidation force does not exceed the pull-out force. In effect, this consolidation would be felt as an increased normal load across the rupture surface, increasing the shear strength and improving the slope stability. Additional testing of this is required. This study did not investigate the long term effect of pull-out force and soil creep. It is recognized that this condition may prevail within a clayey colluvium deposit.

Finally, in an effort to achieve higher pull-out resistance loads, it is believed that the use of upset anchors, as described previously, would significantly increase the pull-out resistance. Commercially used soil anchors for major projects have pull resistance volumes that range from 7 kips to 60 kips, with some of the more common ones at about 20 kips (PTI, 1980). Based on the pull tests for this study, the maximum resistance values obtained just about equal the minimum commercial applications. It is believed, in order for driven soil nails to have major commercial significance a larger resistance load is required. Larger resistance loads could be obtained with tip modified upset anchors. Another apparent tip modification would include a nail with harpool-like barbs.

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## Appendices

## APPENDIX A. LETTERS INDICATING THE APPARENT UNAVAILABILITY OF COMMERCIALLY USED DRIVEN ANCHORS.

STATE OF CALIFORNIA—BUSINESS AND TRANSPORTATION AGENCY EDMUND G. BROWN JR., Governor

DEPARTMENT OF TRANSPORTATION
DIVISION OF CONSTRUCTION
OFFICE OF TRANSPORTATION LABORATORY
5900 FOLSOM BLVD., P.O. BOX 19128
SACRAMENTO, CA 95819
(91‸) 739-2353

JAN 1983
DIST. III
MATERIALS

December 29, 1982

Mr. Robert K. Barrett
District III Geologist
Colorado Department of Highways
P. O. Box 2107
Grand Junction, CO 81502

Dear Bob:

Thank you for your letter of December 3, 1982 and the information on your proposed research with soil anchors for slope stabilization. Caltrans has, on occasion, used drilled and grouted anchors. We have also collected several references on the subject. However, we are not aware of any applications where driven, ungrouted anchors were used for slope stabilization.

The utility industry uses ungrouted anchor systems for pole line guys. Information on this application is attached along with other references that may be of value.

I would like to thank you for the report you sent on the fabric wall in Glenwood Canyon. One of our transportation districts proposes similar construction.

I understand that you may be making a presentation at the '83 TRB Meeting on "Performance of Instrumentation of Earth Reinforced Retaining Wall Over Compressible Soils." I would appreciate receiving any written reports, if available, since I will not attend this year. Ray Forsyth is also unsure of his travel plans.

Thanks again. If I can be of further assistance, please let me know.

Very truly yours,

Joseph Hannon

Joseph Hannon, P.E.
Senior Materials and Research Engineer
Soil Mechanics and Pavement Branch

JBH:EH
Attachments
cc: RForsyth
GChang

APPENDIX A. LETTERS INDICATING THE APPARENT UNAVAILABILITY OF COMMERCIALLY USED DRIVEN ANCHORS.

FOSTER-MILLER ASSOCIATES, INC.
ENGINEERS
350 SECOND AVE.
WALTHAM, MA 02154
617 890-3200/TWX 710-324-1468

18 February 1983

Mr. Stan Szabelak
6886 S. Prince Circle
Littleton, Colo. 80120

Dear Stan:

I enjoyed talking about your slope stabilization reserach study for the I-70 project. After reviewing my files I realized that while we have a lot of ideas in the works, none are yet in printable and therefore, sendable form.

Our projects differ in a few critical ways. While we must anchor a cable for only three years, you must have functional systems lasting decades. While the risk of failure for our system,will in all likelyhood, cause injury and damage to equipment, these disasters occur on private land during an industrial endeavor. The consequence of failure on a public highway is much greater (at least politically).

Furthermore, we are using the earth system to stabilize an externally applied load (the cable), whereas you will be stabilizing the earth itself.

One of the largest costs associated with anchoring for logging is simply getting the installation equipment and anchors to the area of interest. Logistics are a much bigger problem and as such we are trying to minimize the volume and weight of the entire system. While logistics will be a factor for your application, it is unlikely to be the determining factor, therefore, if a larger anchor stabilizes a larger soil volume, it may payoff to do it that way, rather than sinking dozens of smaller anchors.

We should have a better idea of how our system is going to work in about 2-3 months. In the meantime I would suggest getting in touch with companies like, "Schnabel Foundation Company", 4720 Montgomery Lane, Bethesda, MD. 20014, (301) 657-3063; or the other majors such as Dames & Moore (303) 232-6262 or 213-879-9700), Golder Associates (Vancouver, Denver, etc.), Shannon & Wilson (206-632-8020), etc.

If you have any further questions, please do not hesitate to call.

Sincerely,

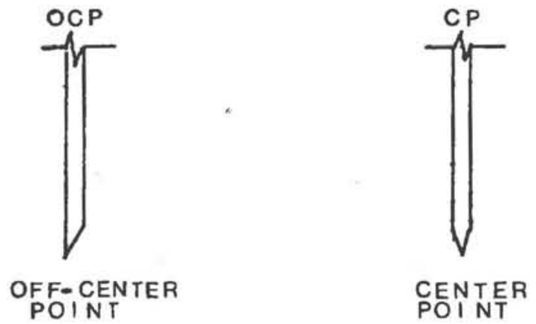
FOSTER-MILLER, INC.

Arnis Mangolds
Senior Engineer

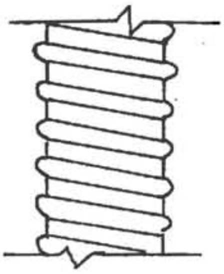
APPENDIX B. REBAR SPECIFICATIONS.

| FACTORY CODE | GRADE | DIAM. OFF-RIB (IN.) | DIAM. ON-RIB (IN.) | RIB SPACE (CM.) | YIELD STRENGTH (PSI) | TENSILE STRENGTH (PSI) | YIELD LOAD (TON) |
|---|---|---|---|---|---|---|---|
| 4 | 60 | 1/2 | 5/8 | 0.9 | 67,000 | 102,000 | 6 |
| 6 | 60 | 3/4 | 7/8 | 1.2 | 63,000 | 98,000 | 14 |
| 8 | 60 | 1 | 1 1/8 | 1.4 | 66,000 | 103,000 | 25 |

POINT GEOMETRY




RIB PATTERN



APPENDIX C. CHART OF REBAR SIZES USED IN FIELD TEST.

QUANTITY OF REBAR

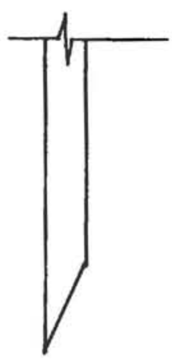
REBAR SIZE WITH POINT TYPE

| REBAR LENGTH | #4 | | #6 | | #8 | |
|---|---|---|---|---|---|---|
| | OCP | CP | OCP | CP | OCP | CP |
| 6ft. | 2 | 2 | - | - | - | - |
| 8ft. | 2 | 2 | 2 | 2 | 2 | 2 |
| 13ft. | - | - | 2 | 2 | 2 | 2 |
| 18ft. | - | - | - | - | 2 | 2 |

REBAR POINT TYPE

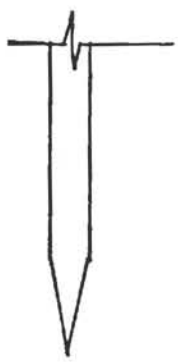
OFF-CENTER POINT

OCP

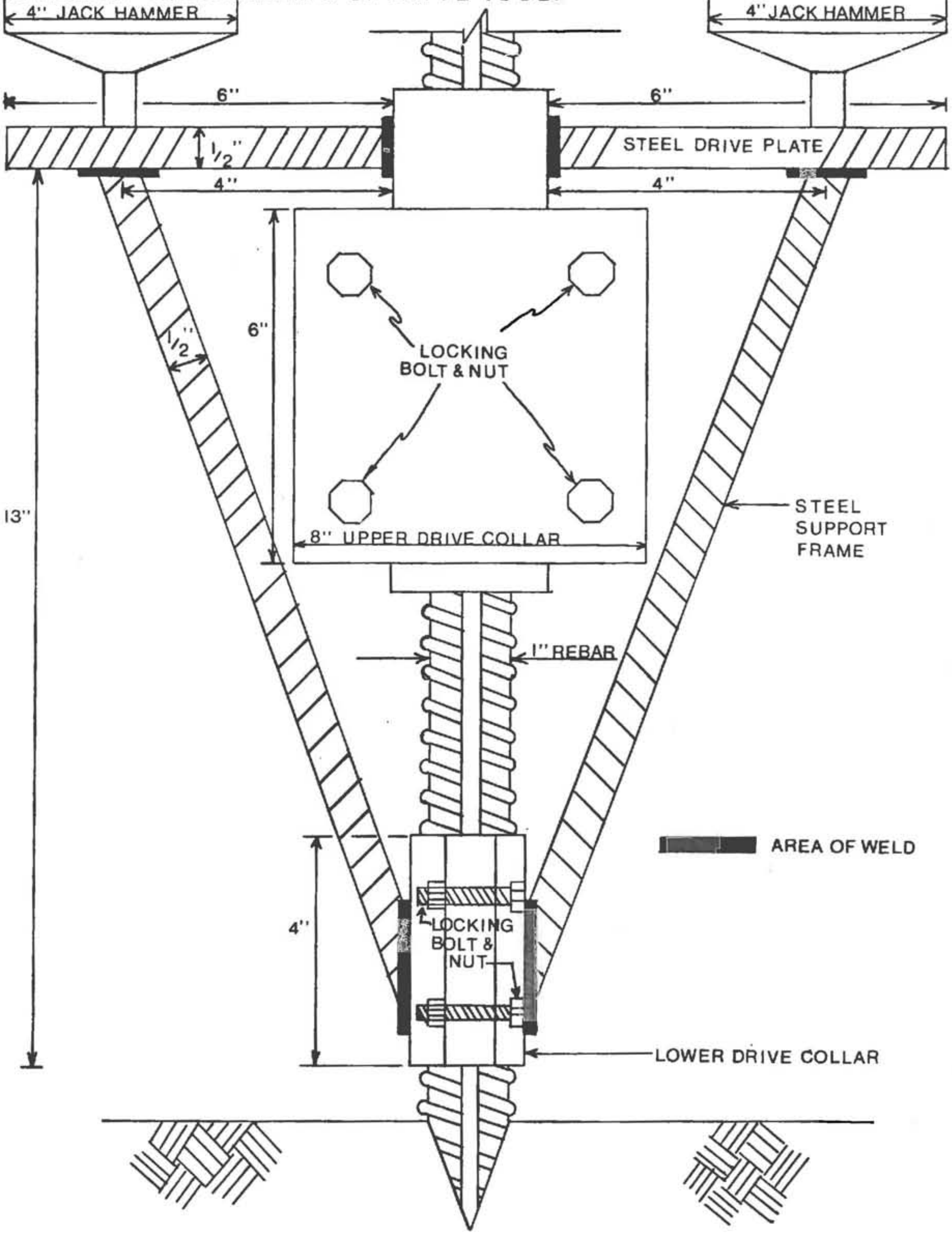


CENTER POINT

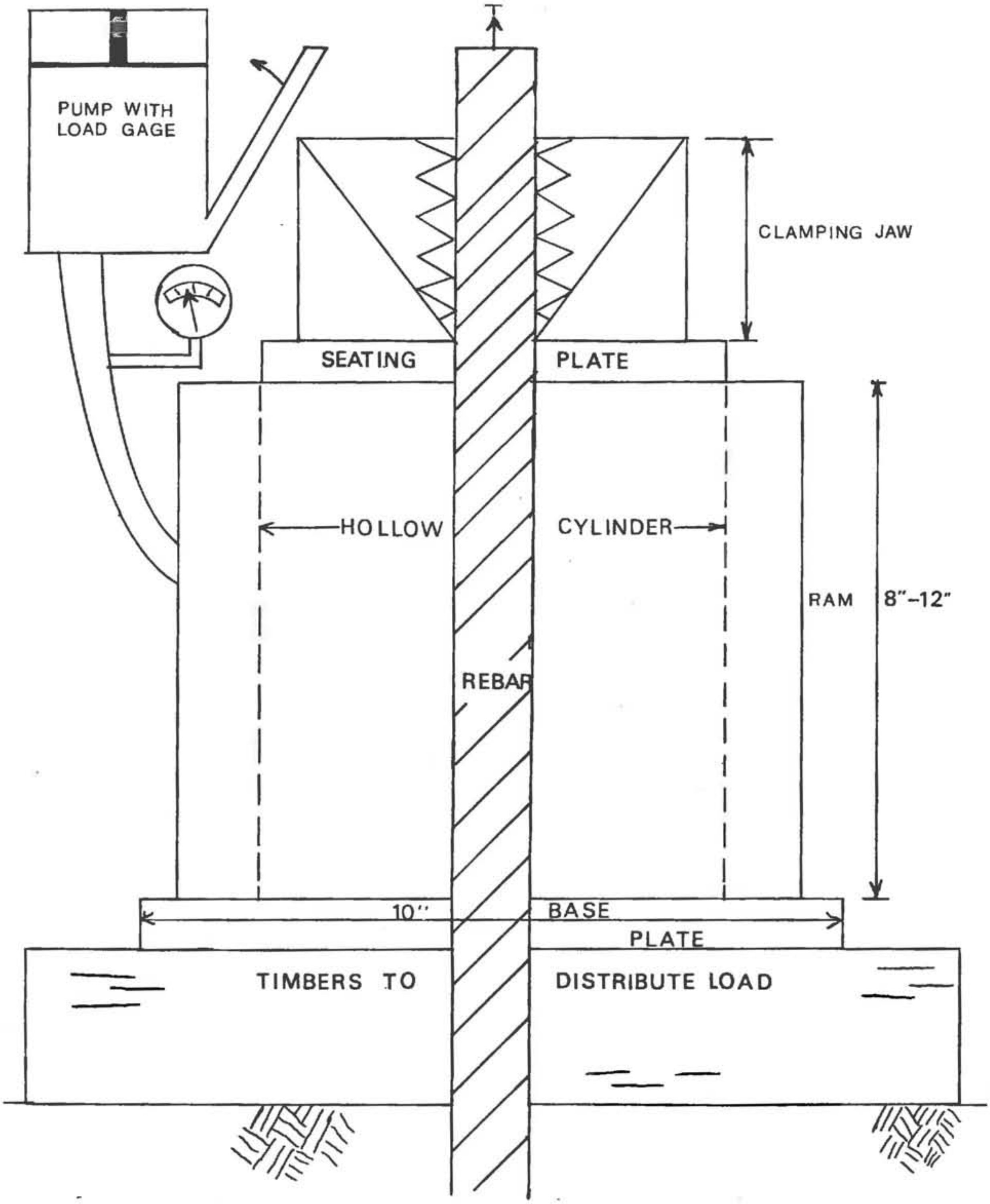
CP



APPENDIX D. SCHEMATIC OF DRIVE TOOL.



APPENDIX E DIAGRAM OF PULL-OUT JACK.



## APPENDIX F. SOIL LABORATORY TEST RESULTS.

DEPARTMENT OF HIGHWAYS
STATE OF COLORADO
DIVISION OF HIGHWAYS
DOH Form No. 323
March, 1982

**LABORATORY REPORT ON ITEM 203**
(Embankment or Borrow)
☐ PRELIMINARY
☐ CONSTRUCTION

Field Sheet No. —
Project No. Research (Soil Anchors)
Location
District 3 Date 8-15-83

| Test No. | Station and Log | Max. Size | Percent Passing 3 | 1 | 3/4 | 3/8 | #4 | #10 | #40 | #200 | L L | P I | Class, and Group Index | Moist % | R Value |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | TRENCH #1 @ 3' | | | | | | 100 | 84 | 48 | 29 | 23 | 4 | A-2-4(0) | 7.3 | |
| | " @ 5' | | 100 | 94 | 93 | 92 | 88 | 65 | 32 | 16 | 22 | 3 | A-1-b(0) | 4.6 | |
| | " @ 8' | | 100 | 88 | 86 | 83 | 80 | 61 | 25 | 13 | 23 | 3 | A-1-b(0) | 4.6 | |
| | " @ 10' | | | | | | | 100 | 99 | 63 | 24 | 3 | A-4(0) | 22.0 | |
| | " @ 13' | | | | | | | 100 | 99 | 96 | NV | NP | A-4(0) | 5.0 | |
| | | | | | | | | | | | | | | | |
| | TRENCH #2 @ 3' | | 100 | 96 | 95 | 94 | 90 | 63 | 36 | 22 | 25 | 6 | A-1-b(0) | 6.8 | |
| | " @ 5' | | | | | | 100 | 92 | 71 | 48 | 21 | 3 | A-4(0) | 4.0 | |
| | " @ 8' | | | | | | 100 | 80 | 50 | 33 | 27 | 9 | A-2-4(0) | 6.1 | |
| | " @ 10' | | 100 | 76 | 69 | 57 | 46 | 34 | 18 | 1 | NV | NP | A-1-a(0) | 1.6 | |
| | " @ 13' | | 100 | 98 | 97 | 93 | 90 | 87 | 61 | 11 | NV | NP | A-2-4(0) | 14.7 | |

☐ T-99 ☐ T-180
☐ Rigid Pavement
☐ Flexible Pavement

Notes and Samples by STAN SZABELAK

## APPENDIX E. SOIL LABORATORY TEST RESULTS.

DEPARTMENT OF HIGHWAYS
STATE OF COLORADO
DIVISION OF HIGHWAYS
DOH Form No. 323
March, 1982

**LABORATORY REPORT ON ITEM 203**
(Embankment or Borrow)
□ PRELIMINARY
□ CONSTRUCTION

Field Sheet No. ______
Project No. Research (SOIL ANCHORS
Location 1/4 mi. E. OF Grizzly CK.
District 3 Date 8-15-83

| Test No. | Station and Log | Max. Size | Percent Passing 3 | 1 | 3/4 | 3/8 | #4 | #10 | #40 | #200 | L L | P I | Class, and Group Index | Moist % | R Value |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | PIT #1 @ 3' | | | | | | 100 | 97 | 90 | 52 | 22 | 4 | A-4(0) | 12.4 | |
| | " @ 5' | | | | | | 100 | 99 | 96 | 12 | NV | NP | A2-4(0) | 4.1 | |
| | " @ 8' | | 100 | 97 | 96 | 94 | 93 | 92 | 84 | 9 | NV | NP | A-3(0) | 5.8 | |
| | " @ 10' | | 100 | 99 | 98 | 97 | 97 | 96 | 91 | 20 | NV | NP | A2-4(0) | 9.2 | |
| | " @ 15' | | 100 | 71 | 62 | 49 | 42 | 37 | 23 | 4 | NV | NP | A-1-2(0) | 4.2 | |
| | | | | | | | | | | | | | | | |
| | PIT #2 @ 3' | | | | | | 100 | 90 | 62 | 37 | 23 | 5 | A-4(0) | 8.2 | |
| | " @ 5' | | | | | | 100 | 99 | 88 | 5 | NV | NP | A-3(0) | 3.8 | |
| | " @ 8' | | 100 | 99 | 96 | 95 | 94 | 94 | 91 | 5 | NV | NP | A-3(0) | 7.3 | |
| | " @ 10' | | | | | | | 100 | 98 | 6 | NV | NP | A-3(0) | 7.5 | |
| | | | | | | | | | | | | | | | |
| | PIT #3 @ 3' | | 100 | 99 | 99 | 94 | 92 | 88 | 82 | 26 | NV | NP | A-2-4(0) | 4.4 | |
| | " @ 5' | | | | | | | 100 | 90 | 6 | NV | NP | A-3(0) | 4.4 | |
| | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | |

□ T-99 □ T-180
□ Rigid Pavement
□ Flexible Pavement

Notes and Samples by STAN SZABLIAK

## APPENDIX F. SOIL LABORATORY TEST RESULTS.

DEPARTMENT OF HIGHWAYS
STATE OF COLORADO
DIVISION OF HIGHWAYS
DOH Form No. 323
March, 1982

**LABORATORY REPORT ON ITEM 203**
(Embankment or Borrow)
□ PRELIMINARY
□ CONSTRUCTION

Field Sheet No. —
Project No. Research (Soil Anchors,
Location 1/4 mi. E. of Grizzly Ck
District 3 Date 8-15-83

| Test No. | Station and Log | Max. Size | Percent Passing 3 | 1 | 3/4 | 3/8 | #4 | #10 | #40 | #200 | LL | PI | Class, and Group Index | Moist % | R Value |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | PIT #4 @ 3' | | 100 | 99 | 99 | 95 | 93 | 86 | 74 | 95 | 23 | 3 | A-4(0) | 8.0 | |
| | " @ 5' | | | | | | | 100 | 99 | 24 | NV | NP | A-2-4(0) | 15.9 | |
| | " @ 8' | | | | | | | 100 | 99 | 4 | NV | NP | A-3(0) | 4.0 | |
| | " @ 10' | | | | | | | 100 | 97 | 9 | NV | NP | A-3(0) | 6.8 | |
| | | | | | | | | | | | | | | | |
| | PIT #5 @ 3' | | | 100 | 99 | 93 | 89 | 81 | 73 | 49 | 23 | 2 | A-4(0) | 6.8 | |
| | " @ 5' | | | | | | 100 | 99 | 98 | 18 | NV | NP | A-2-4(0) | 13.1 | |

□ T-99 □ T-180
□ Rigid Pavement
□ Flexible Pavement

Notes and Samples by STAN SZABELAK