# Instrumentation, Monitoring and Analysis of the Performance of a Type-A INSERT Wall Littleville, Alabama

Final Project Report

by

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# 1.0 INTRODUCTION

INSERT stands for In-Situ Earth Reinforcement Technique. It is called a Type "A" wall because they are usually used in areas where more traditional retaining walls have been used in the past and because of their distinctive cross-sectional appearance. In an insert wall, small diameter piles are used as reinforcing members and are inserted into the native soil or rock materials. The piles are formed by drilling a six to ten inch diameter hole into the native soils and filling it with grout. The reinforcement is provided by placing a steel pipe down the center of the hole and grouting it into place. These small diameter piles are commonly referred to as pin piles and have been used as conventional load bearing piles, but have more recently found acceptance in the slope stabilization field. The concept behind the use of pin piles for slope stability is that they form a composite structure with the in-situ earth which becomes a barrier to arrest actual or potential slope movements. The piles are usually socketed into an underlying rock formation which gives the wall more stability, and are arranged in a near vertical configuration, so that they can be connected together at the ground surface with a reinforced concrete cap. Due to the use of small diameter piles, relatively light weight drilling equipment can be used which allows these walls to be constructed on extremely steep slopes or in difficult terrain (Xanthakos, et.al., 1994).

The original design procedure for the insert wall was based on the gravity wall design procedures. Further research has shown that the effectiveness of the insert wall lies in the soil/pile interactions. A new design procedure has been developed where a stability analysis is performed to determine the increase in resistance along the slide plane that is needed to attain an adequate factor of safety. This resistance is then used to determine how many piles will be needed and at what spacing. These piles are designed, and checked against the potential of structural failure. If additional support is needed to provide an adequate factor of safety, then ground anchors can be added to the design. The spacing of the piles, as well as the type of soil the piles are placed in, is checked to ensure against a plastic failure of the soil around the piles. A reinforced concrete cap is used at the ground surface to connect the piles and ground anchors together and as a drilling template for pile and anchor placement (Xanthakos, et. al., 1994).

Once the insert wall design is completed the steps in the construction process are as follow. First, excavate the area for the concrete cap and place the reinforcing steel and drilling sleeves for the piles. The cap will serve as a drilling template. Next, the cap is poured and allowed to cure, then the piles are drilled and grouted using pressurized grout. If ground anchors are used, they are placed and tensioned. Lastly, the area is regraded and any repairs that need to be taken care of are done. An illustration depicting a typical cross-section is shown in Fig. 1.

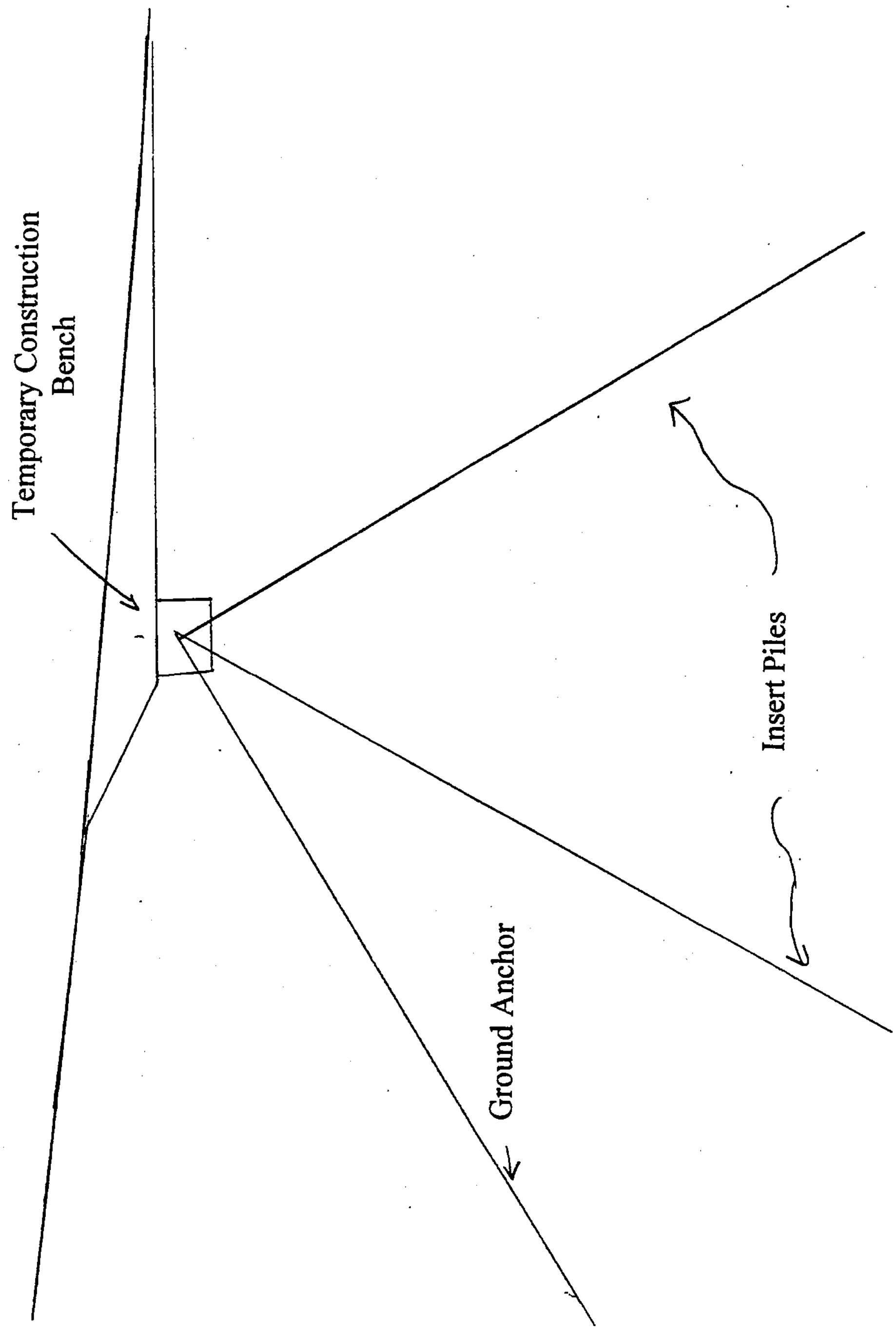



Figure 1: Typical cross-section of an INSERT wall. (Wolosick, 1994)

The purpose of this project is to assess the validity of the design approach and suggest improvements. This has been accomplished by attaching instruments to the piles and ground anchors and measuring the loads that are carried by the members. The movement of the slope was also monitored to determine the amount of movement needed to mobilize the forces in the members. The data gathered are then compared to the values calculated in the design procedure.

## 1.1 Site Conditions

The Insert wall in this study is located on a section of US Route 43, south of Florence, Alabama which has been the site of recurring landslides since the roadway was originally constructed in the 1950's. The area of the slide is a fill composed of sandstone and shale cut from the adjacent hillsides, and as such was extremely difficult to characterize during the initial geotechnical investigation of the slide. The underlying natural materials slope at a skew from west to east with the fill crossing a valley from north to south (see Fig. 2). The underlying stratum is a shale with some weathering along the top surface, which is believed to be the failure surface. Slope inclinometer measurements further corroborate this assumption. Groundwater, perched on this shale surface at levels typically only one to two meters above the surface of the shale, is believed to be contributing to the slope movement by promoting weathering of the shale and reducing effective stresses on the failure surface (see Fig. 3). For design, effective residual stress-strength parameters of the failure surface were estimated by back-analysis of the existing slope and failure surface (Brown and Wolosick, 1995).

Soon after construction of the original roadway was finished, stability problems arose with the result that the road surface dropped due to slides in the fill material. The initial repair technique was to fill the holes in and repave the road. Later in the 1970's, the roadway was widened and the slope flattened with the hope that it would relieve the problem. However, within a few years, rainy periods brought on the onset of more slope failures. Attempts were made to intercept the drainage and subsurface water by ditching up slope from the roadway, which proved temporarily helpful, but the movement again became progressively worse. When the recent repairs were initiated, the toe of the slope was approximately 250 to 300 feet down slope (to the east) from the roadway and the scarp of the slide covered an area of roadway surface which was approximately 600 to 700 feet long (Brown and Wolosick, 1995).

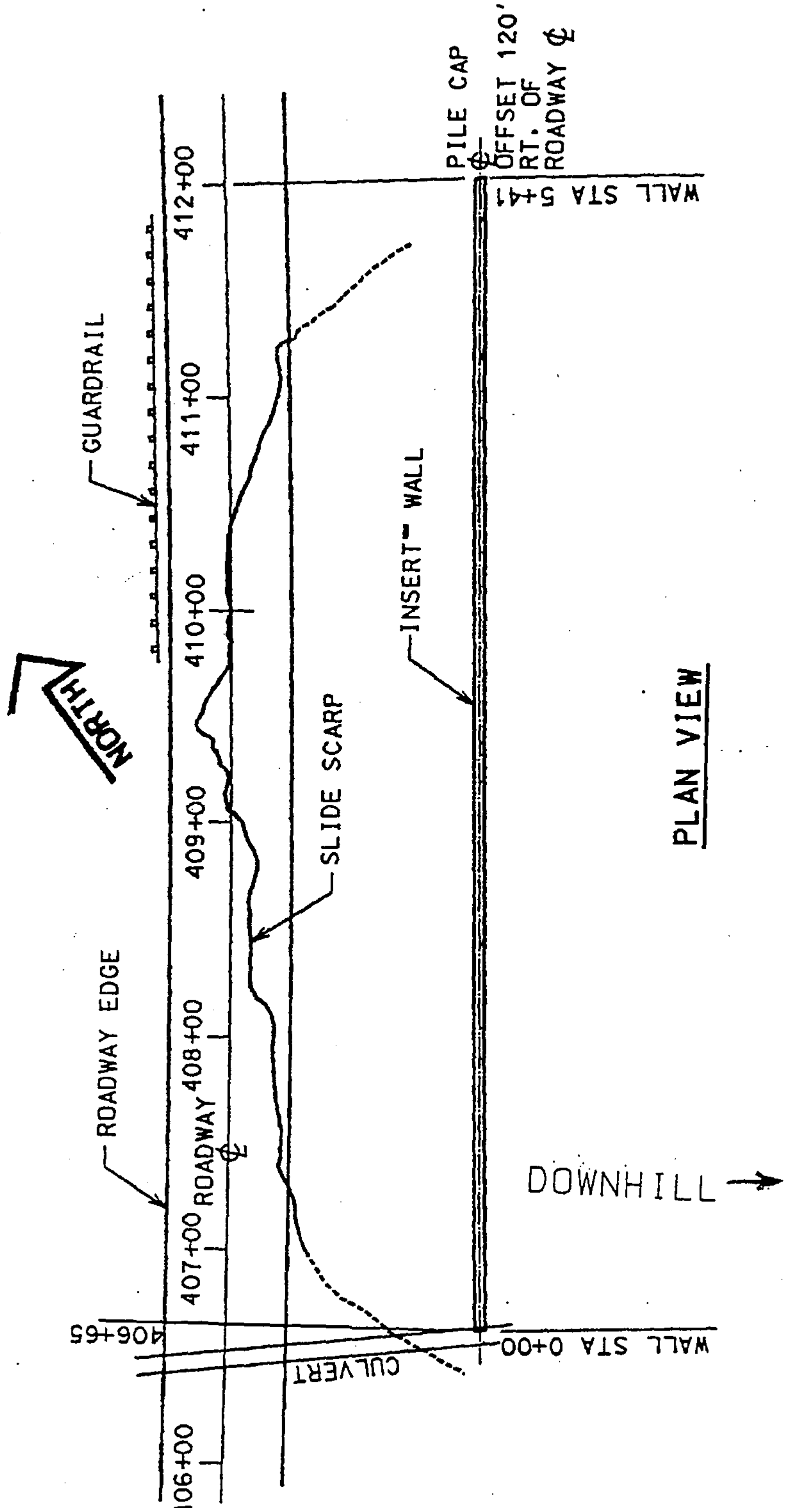



Figure 2: Plan view of the site. (Campbell, 1993)

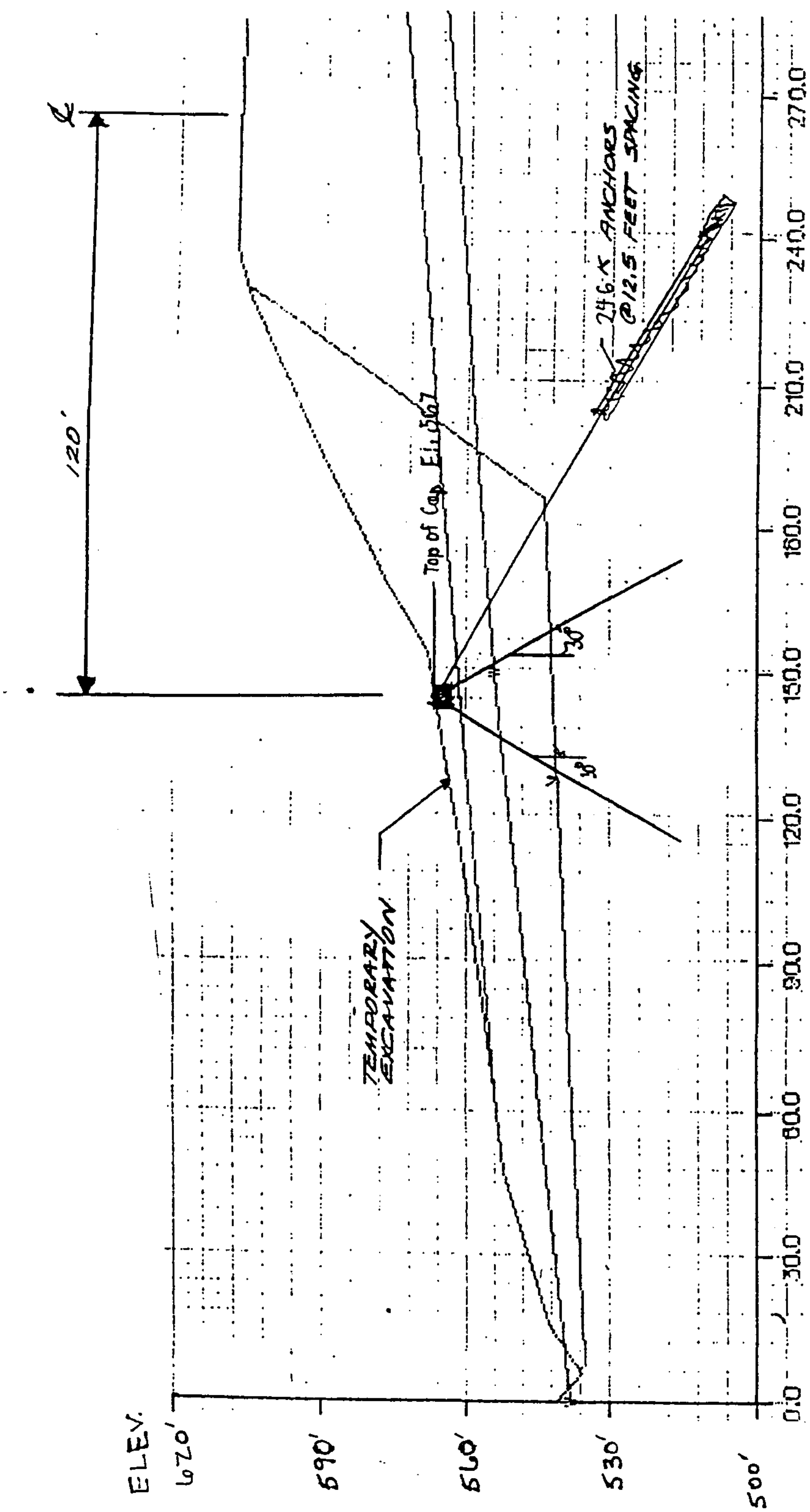



Figure 3: Typical cross-section of the slide. (Wolosick, 1994)

# 2.0 INSERT WALL DESIGN

The first step in the design process followed by D'Appalonia was to perform a back analysis of the existing failure using the code PCSTABL5 to determine the strength properties of the soils which would give a factor of safety of approximately one. Next the amount of force the wall would have to provide to reach the target factor of safety was determined from PCSTABL5 analyses. The INSERT wall was modeled for slope stability purposes using tieback forces and a thin, strong soil layer to model the pile resistance forces. The target factor of safety was in the range of 1.3 to 1.4 (Campbell, 1993).

## 2.1 Results of Back Analysis

The back analysis of the existing failure surface was performed at sections typical of four points along the length of the wall. The angle of internal friction for the fill material was estimated to be 30° and the angle for the natural material was back-calculated to be 16° at station 0+35 and 11° at stations 1+35, 2+35, and 3+35. A cohesion value of zero was used for both materials in the PCSTABL5 runs from which this data was derived. The unit weight of both materials was estimated to be 125 pcf. The failure surface was found to be at approximately an angle of ten degrees with the horizontal and located at an approximate elevation of 540 feet. Using these soil and groundwater values, the factors of safety for the existing conditions were computed to be:

| | | | |
|---|---|---|---|
| be: | FS = 0.996 | at | Sta. 0+35 |
| | FS = 0.993 | at | Sta. 1+35 |
| | FS = 0.997 | at | Sta. 2+35 |
| | FS = 1.001 | at | Sta. 3.35 |

These were calculated with an increased groundwater level such as that which drives the slope movement. This groundwater level was also used in the analysis of the slope with the INSERT wall in place to determine the final factor of safety (Campbell, 1993).

2.2 Pile Analysis with COM624

An average undrained shear strength was calculated for the soil from the failure plane to the top of the cap using a weighted average from the soil boring information. This value was found to be 2025 psf or approximately 14.1 psi. The unit weight of the soil was estimated to be 125 pcf and the angle of internal friction was assumed to be zero. Although slope stability computations utilize effective stress strength parameters, lateral load computations have been traditionally performed using empirical correlations of soil resistance with undrained shear strength. This data was used in COM624 to develop p-y curves to model the material above, and below, the slip plane as a stiff clay, above the water table with ks = 1000 psi and E50 = 0.005 (Reese and Sullivan, 1983). This p-y curve was then used in COM624 to conduct a lateral pile analysis with the assumption made that the pile acts as a free-headed pile at the slide surface. This is a reasonable assumption due to the fact that the slide surface is a great distance below the cap, and the shear load is transferred across the failure surface from the failure zone to the shale below the failure plane. The pile dimensions chosen were a 4.5 inch diameter by 0.3 inch thick pipe pile grouted into a six inch hole. This size pipe was chosen due to the availability of high strength oil well casings which could be recycled for use as INSERT pile reinforcing. The outer grout was assumed to crack under loading and therefore would not contribute significantly to the pile section modulus (EI). This led to an EI of 267,000 kip-in^2 to be used in the COM624 analysis which included the effects of the grout within the pipe. The pile was assumed to be 23 feet long and have a cross-sectional area of 28.3 square inches (Campbell, 1993).

The computer program STIFF1 was used to determine the ultimate bending capacity and bending stiffness of this proposed pile. It was found that the ultimate bending moment that the pile could withstand was 386 in-kip with zero axial load applied and a concrete strain of 0.003. With the addition of axial loads, the ultimate bending moments decreased slightly. COM624 was then used to determine a relationship between lateral loads, bending moments, and lateral deflections. This was accomplished by applying a series of lateral loads ranging from 10 to 35 kips to a pile with zero applied moment at the failure surface and zero axial load. This relationship was then used to determine the ultimate deflection at the pile top expected at the ultimate moment of 386 in-kip. Deflection to produce structural yield in bending was found to be 2.55 inches at a lateral load of 24 kips (Campbell, 1993).

## 2.3 Computing Axial and Shear Forces

The ultimate axial load that could be carried by the pile was that due to skin friction along the outside of the pile. Since the grout was to be placed under pressure, the bond with the pipe was assumed to be relatively good. The ultimate load that could be carried by skin friction was determined using the average cohesion of 2.03 ksf developed earlier. The formula used to determine the limiting axial capacity was simply the circumference times the length of the pile which could be counted on to provide support moving away from the failure plane times the cohesion. This value was found to be 86.1 kips (Campbell, 1993).

The piles were designed to be battered uphill and downhill which provided some additional capacity, due to the axial pile forces, with respect to the direction of movement. This additional capacity was determined using a procedure put forth by Poulos and Davis (1980, pp. 237-42), which relates axial and lateral forces with displacement for battered piles. Using this procedure it was determined that the batter angle should be 30 degrees in order for the axial capacity developed to remain under the 86 kip limit established earlier. Taking this batter angle into consideration and using the horizontal and vertical component of the limiting axial and lateral (transverse to pile axis) capacity, it was found that the pile would have an ultimate pile resistance in the horizontal direction of 57.7 kips and in the vertical direction of 58.4 kips. Taking the batter into account, this gives an axial resistance of 79.4 kips and a lateral resistance, transverse to the pile, of 20.8 kips. These values were then converted into a single beneficial force up slope which could be used in the slope analysis. The required ultimate wall capacity needed for the target factor of safety to be reached had been determined earlier for the four stations along the length of the wall using PCSTABL5. These values were compared to the beneficial force up slope determined by the Poulos and Davis procedure in order to compute the required pile spacing. The angle of the sliding zone in the vicinity of the proposed INSERT wall was assumed to be relatively flat and therefore the beneficial force up slope was considered to be the ultimate lateral pile resistance of 57.7 kips per pile. Why the failure surface angle of 10 degrees was not used to determine the beneficial force up slope was not mentioned. There was no report of ever having analyzed the slide again with the new beneficial force up slope of 57.7 kips per pile to determine a final factor of safety for the project (Campbell, 1993).

## 2.4 Ground Anchors Required

Once the above numbers were calculated, it was decided that the piles alone would not be able to stop the landslide and so ground anchors were added to the design to help take the lateral loads caused by the earth movement. A ground anchor is a steel tendon which has been grouted into sound rock and tensioned so that the soil above the rock is held in place. As the soil tries to move, the tendon goes into tension to keep the soil in place and prevent movement. The ground anchors were required to supply 19.7 kips per foot of INSERT wall at an angle of 30 degrees below the horizontal. This translated into an additional 17.1 kips of beneficial lateral force per foot of wall. It was decided that the ground anchors would be attached to the cap which then had to be analyzed for stability. First, the cap was analyzed to see if tensioning the anchors before the piles were installed would cause a bearing capacity failure in the soil above the cap. Both long term and short term loading conditions were analyzed to determine the effects of drained and undrained loading. It was determined that in either case, the stability of the cap would be marginal, at best, if the anchors were tensioned before the piles were installed. Next the capability of the cap and piles to withstand the anchor loads was checked, using the GROUP1 computer program. It was found that the highest stresses would be in the up slope piles, but that the stresses caused by the anchors were much smaller than the ultimate pile loads previously calculated, therefore, the piles could easily support the loads developed from the ground anchors (Campbell, 1993).

## 3.0 CONSTRUCTION AND MONITORING

The final design consisted of a 541 foot long INSERT wall with 432 drilled inserts and 44 ground anchors all connected into a reinforced concrete cap. Uphill and downhill inserts were placed on 33 inch centers typically with the insert from the opposite side slanting between the two inserts on center (see Fig. 4). The inserts were battered at an angle of 30° from the vertical uphill and downhill respectively except the downhill inserts at the ground anchor locations were battered at 20° from the vertical due to construction constraints. The insert pipes were 4.5 inch outside diameter steel pipes with a 0.3 inch wall thickness which had a minimum yield strength of 80 ksi. The ground anchors were placed at a distance of 12.5 feet center to center, all of which were battered in the uphill direction at an angle of 30° below the horizontal (see Fig. 5). The anchor tendons were a group of 0.6 inch diameter seven wire strands made of 270 kip grade, stress relieved or low relaxation steel. The design load for all the anchors was 246 kips. The ground anchors were constructed such that the lower 25 feet of the anchors were bonded into the rock and would supply the support against which the tendons would be stressed. The upper 55 feet were greased and sheathed such that the tendons could stretch, thereby supporting the load applied by the earth movement (see Fig. 6). The concrete cap has a minimum 28-day compressive strength of 4000 psi with

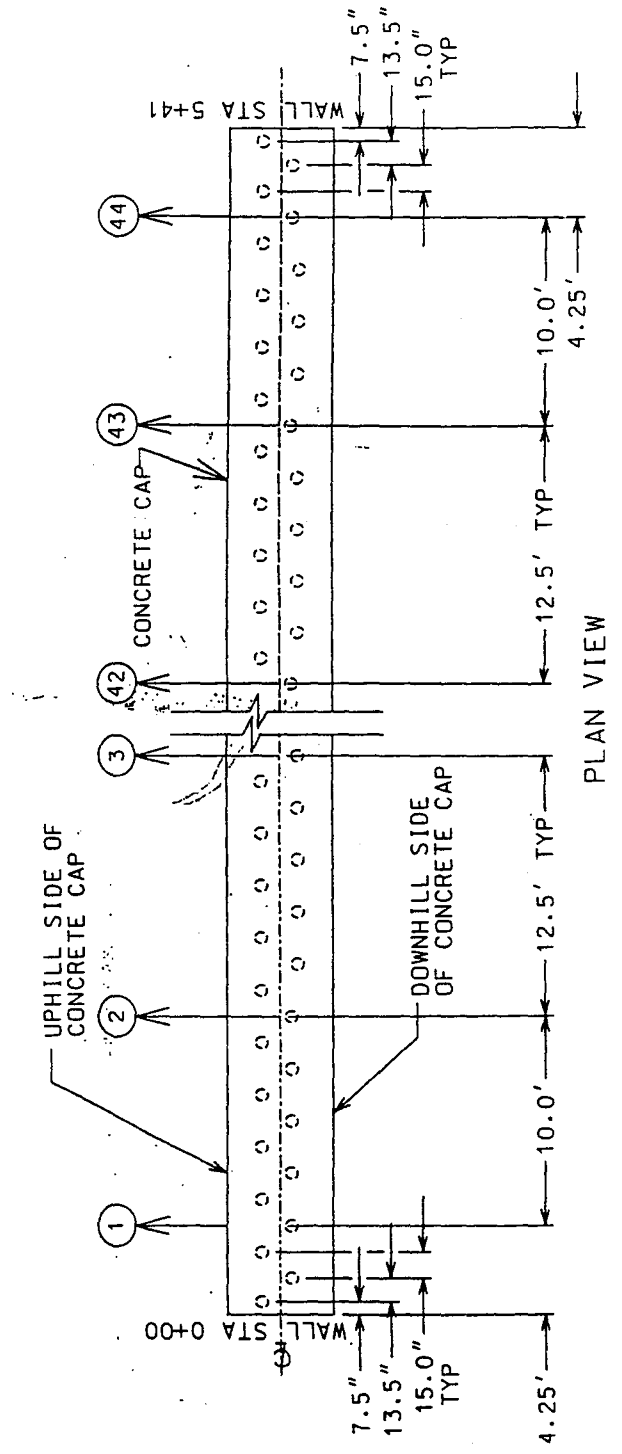



Figure 4: Insert Spacing Detail. (Wolosick, 1994)

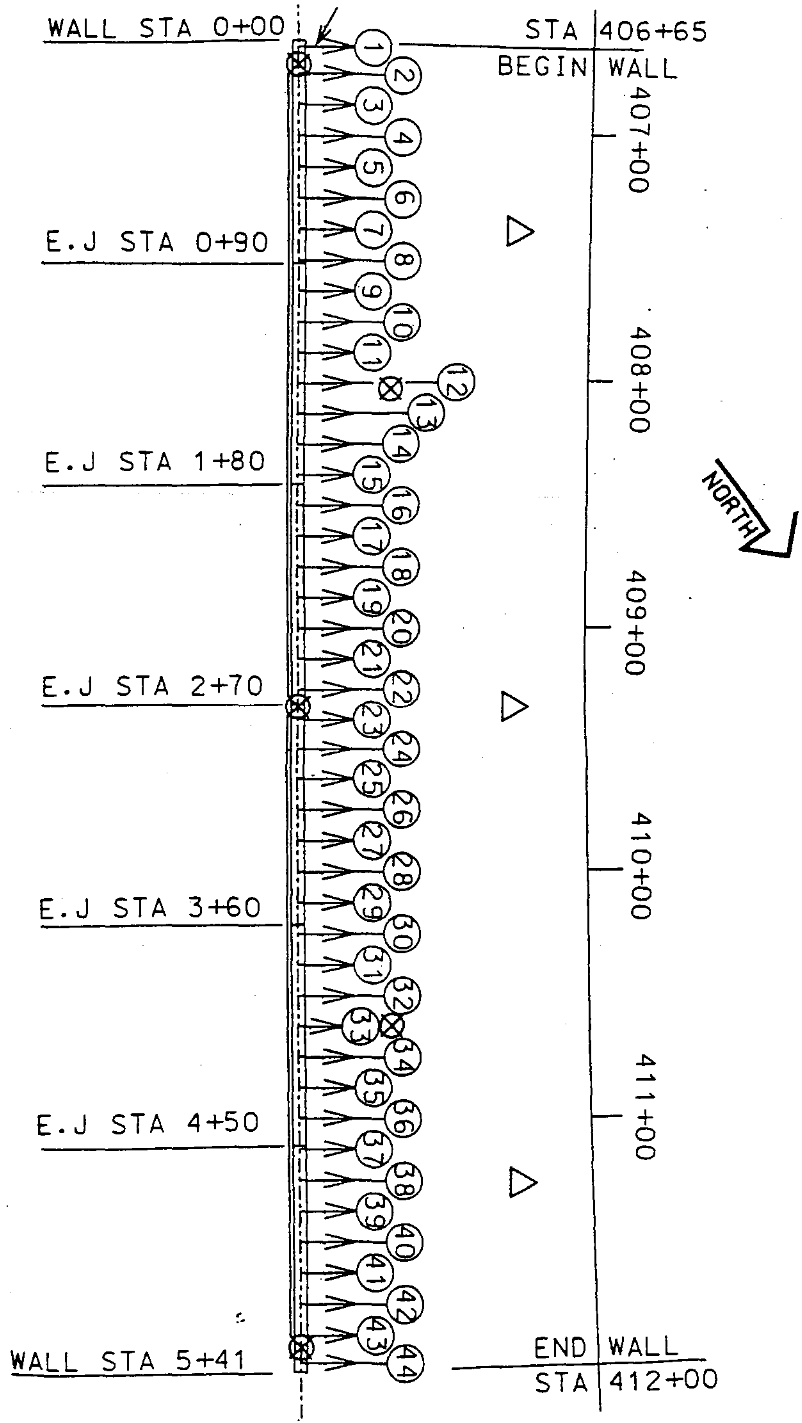



Figure 5: Ground Anchor Spacing Detail. (Wolosick, 1994)

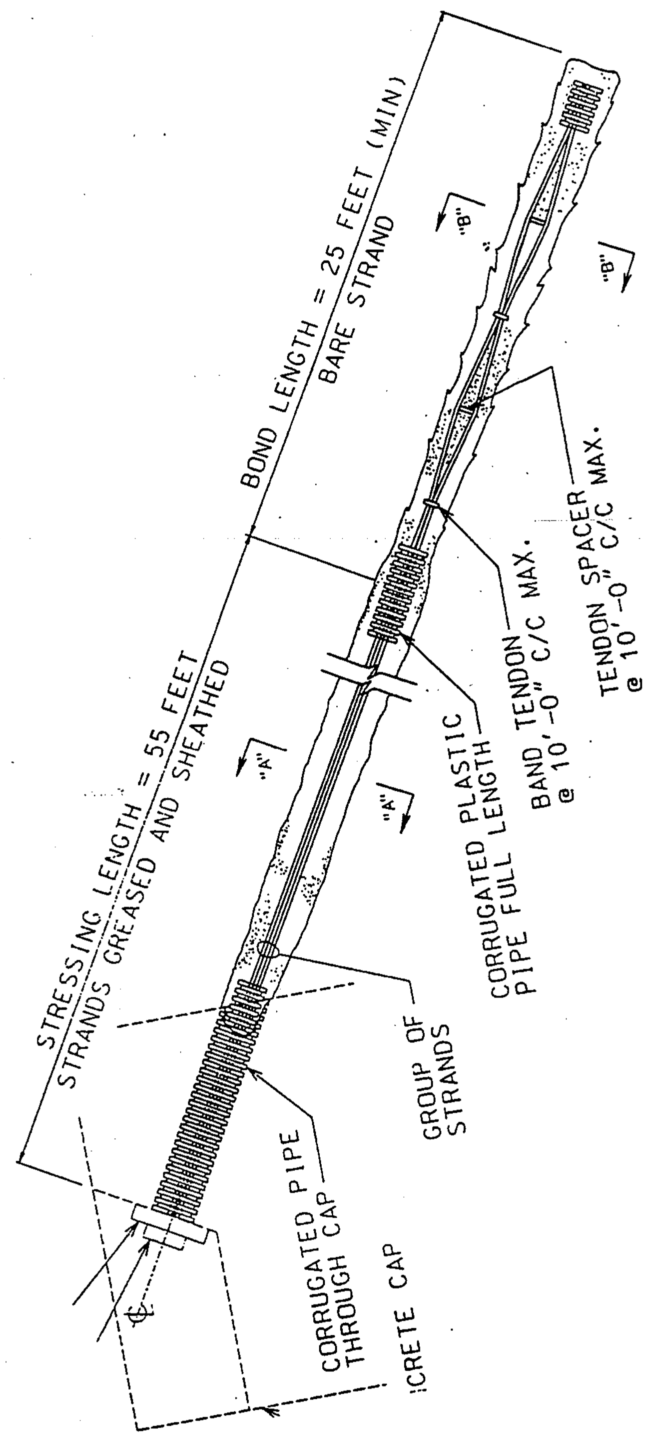



Figure 6: Cross-section of a Ground Anchor. (Wolosick, 1994)

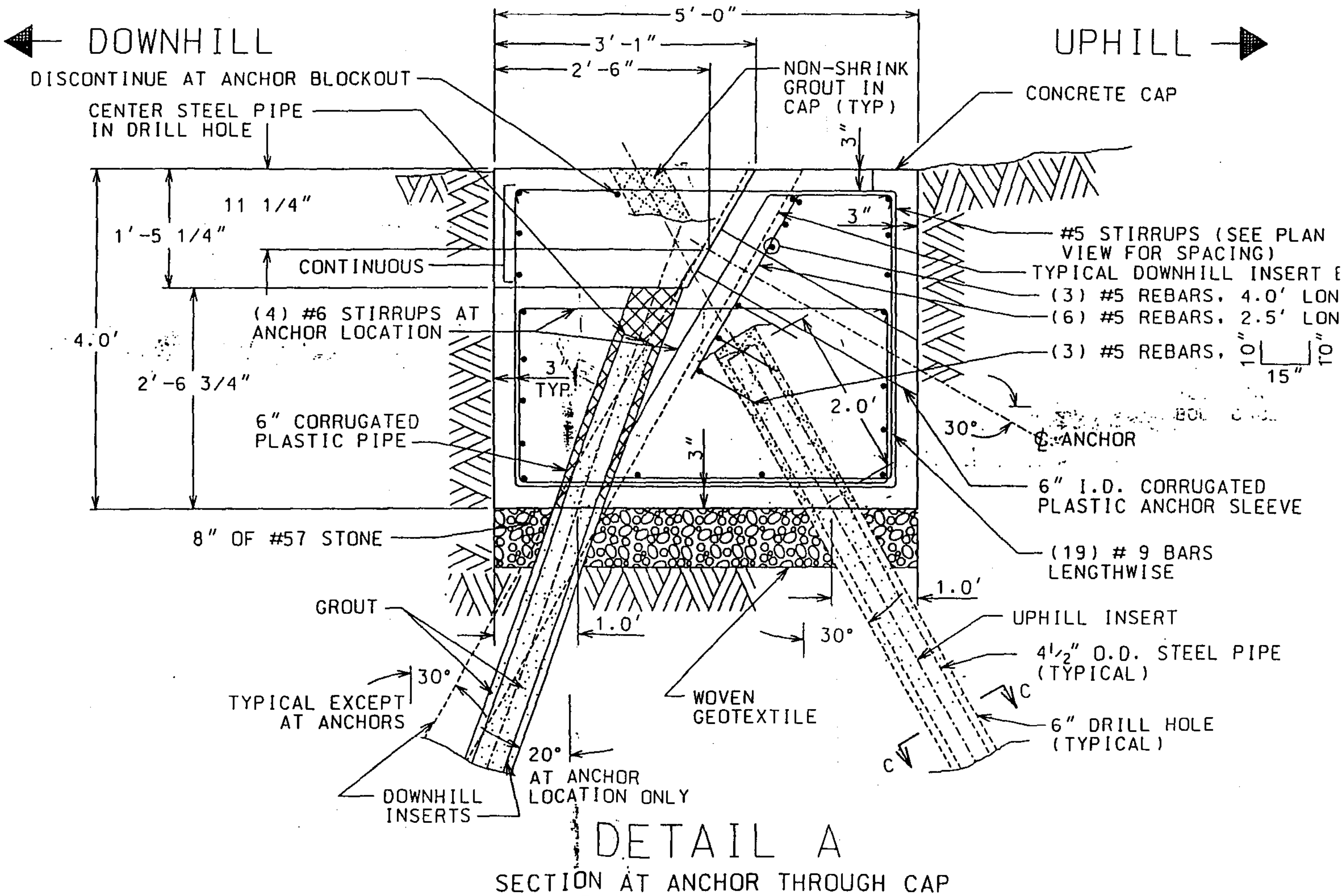



Figure 7: Cross-section of Concrete Cap. (Wolosick, 1994)

the reinforcement being grade 60 steel rebars. A cross-section through the cap showing the placement of the inserts and anchors within the cap and the cap reinforcement is shown in Figure 7.

3.1 Instrumentation Installed

All the internal monitoring equipment was of the vibrating wire type gage. These were desirable because such instruments are considered to be very stable over the long term. Resistance-type gages may have exhibited zero drift characteristics over the two year monitoring phase. The gages were wired into an automatic data logger which could further be attached to a modem enabling remote data acquisition. The internal monitoring equipment which was attached to the data logger included strain gages, load cells, settlement indicators, tiltmeters, and piezometers. These will each be discussed in the following sections along with the inclinometers which were installed, but required a site visit to be monitored.

3.11 Strain Gages

Instrumented piles were placed at stations 1+70 and 2+70 in both the uphill and downhill directions. The piles at station 2+70 were placed on August 5, 1994 and the piles at station 1+70 were placed on September 16, 1994. These piles were instrumented with strain gages which were placed on opposite sides of the pile at several points along the length of the pile so that the axial force and bending moment within the pile could be monitored. By using both strain gages at a point, the values could be averaged to cancel bending strains and compute the axial force being transferred to the pile, and the difference in readings used to cancel axial strains and determine the bending moment at that point. The strain gages were covered with a steel channel running the length of the pile to protect the gages from damage during construction. A cross-section of an instrumented pile is shown in Figure 8. All the strain gages were of the Geokon VSM-

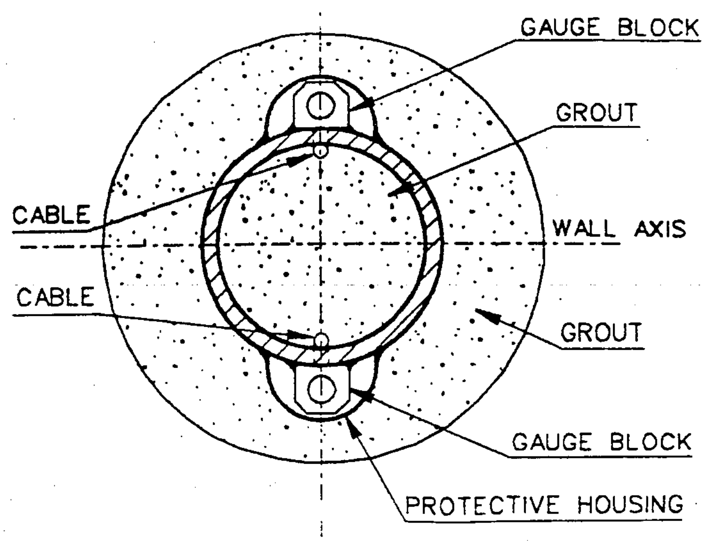



Figure 8: Cross-section of Instrumented Pile. (Wolosick, 1994)

4000 vibrating wire strain gage type which gives a direct measurement of strain in micro-inches/inch as a readout. The strain gages were welded to the pipe pile and then covered with the channel which was also welded to the pipe pile. Welding was omitted in the areas directly around the strain gages to avoid damaging the gages. The locations of the strain gages along the piles and the expected location relative to the failure surface is given in figures 9 and 10. After installation, the strain gages were read to get the zero readings for reference against all future readings.

The moment of inertia for this built up section was computed and verified in the field. Field verification was performed by hanging known weights at the third points of

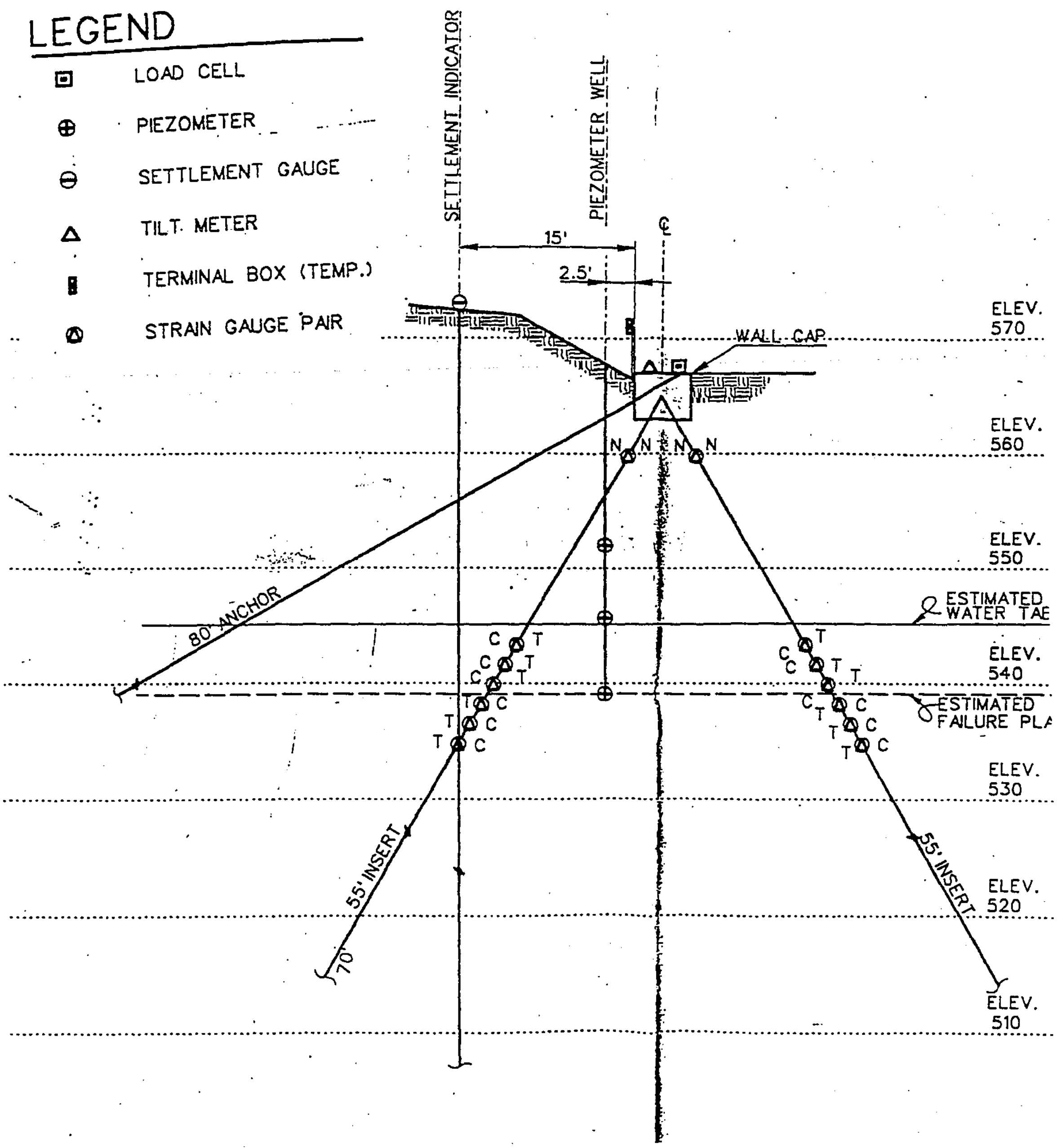



Figure 9: Cross-section of Instrumented Wall at Station 1+70. (Wolosick, 1994)

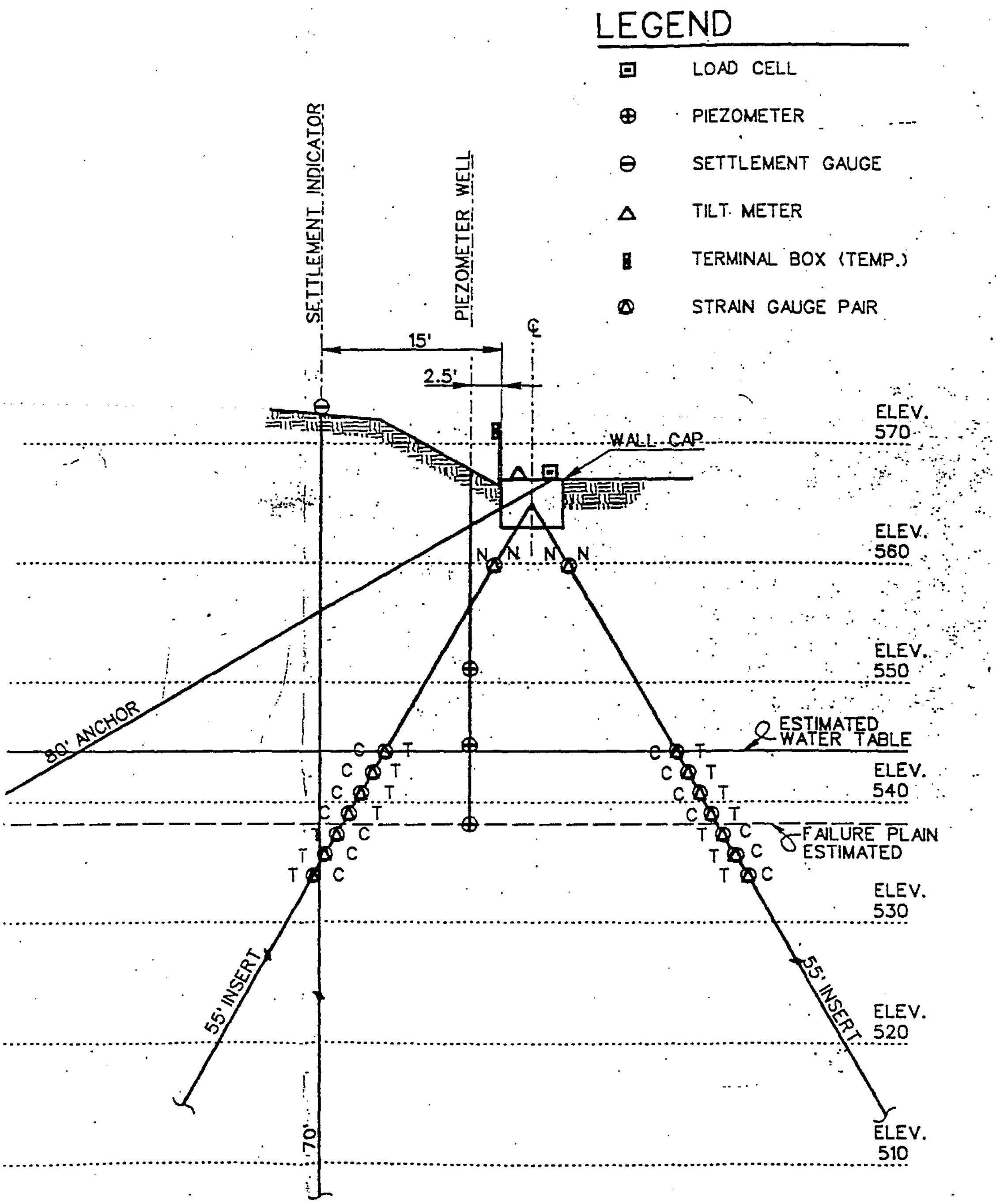



Figure 10: Cross-section of Instrumented Wall at Station 2+70. (Wolosick, 1994)

a simply supported pipe to achieve a constant known moment across a section of the pipe. The back-calculated I values were computed from the measured strain gage output using $\varepsilon = M^*c/I$. Values in the range of 7.5 to 8.4 were measured using this approach. An average value of 7.95 has been used in interpreting the bending moment value from strain measurements. The grout on the interior of the pipe was not counted on due to the large number of wires running up through the annulus, and the exterior grout was expected to crack rendering it insignificant with respect to section modulus.

3.12 Load Cells

The load cells were attached to the ground anchors so that the axial force transferred to the anchors could be determined and taken into consideration in the analysis of the effectiveness of the wall. The load cells were Geokon vibrating wire load cells, model 4900, which are designed primarily for tieback or rockbolting applications. The initial readings were taken on November 18, 1994, before the load cells were installed. These readings were then used to calibrate the subsequent readings and calculate the load. This calculation requires a gage factor which is multiplied by the difference in the current and initial readings for each gage in the load cell. There are three strain gages in each load cell so that the effects of eccentric loading is minimized. The sum of the values calculated for each gage is the total load experienced by the load cell and the ground anchor it is monitoring. The gage factor for these ground anchors is 0.0429 lbs/digit, with readings being taken in digits. The loadcells are located at stations 1+60, 1+72, 2+10, 2+23, 2+64, and 2+76.

3.13 Piezometers

Piezometers were placed at stations 1+70, 2+20, and 2+70, with three piezometers placed at each station at elevations of 17, 23, and 30 feet below the ground surface. These piezometers are monitored with Geokon vibrating wire piezometers, model 4500S-50, which contain vibrating wire pressure transducers. The piezometers at station 2+70 were installed on August 4, 1994, and the piezometers at stations 2+20 and 1+70 were installed on October 26, 1994. The piezometers were installed to measure the height of water which was perched above the weathered shale failure surface. The water pressure calculation requires a calibration factor which is unique to each piezometer and is given in Appendix A for each piezometer by station and elevation. Piezometer placement is illustrated in figures 9 and 10 for stations 1+70 and 2+70.

3.14 Settlement Indicators

Settlement indicators were placed at stations 1+70, 2+20, and 2+70 in order to monitor the soil above the slide plane for possible movement. The settlement indicator at station 2+70 was placed on August 4, 1994 and the ones at stations 1+70 and 2+20 were placed on October 26, 1994. These settlement indicators consisted of Geokon vibrating wire settlement sensors, model 4600-25. The settlement calculated is in inches and is dependent upon a calibration factor which is unique to each sensor. The calibration factor for the sensor at station 1+70 is 0.007244. The factor for station 2+20 is 0.007313 and the one for station 2+70 is 0.189189. These numbers are multiplied by the difference in the current reading and the initial reading to determine the amount of settlement which has occurred. Settlement indicator placement is illustrated in figures 9 and 10.

### 3.15 Tiltmeter

Tiltmeters were mounted on the cap at stations 1+70, 2+20, and 2+70 so that changes in tilt in the structure could be monitored. The tiltmeters were Geokon model 6301 uniaxial vibrating wire tiltmeters with a self-damping mechanism. The difference in readings is multiplied by a gage factor and then the inverse sine is taken of that value to determine the change in angle, in degrees. Due to the small angles involved in this calculation, the inverse sine of the number and the actual number are so close that the number was taken as the angle without applying the sine correction. The change in tilt is measured as a change in force caused by the rotation of the center of gravity of a mass suspended by the wire and an elastic hinge. The gage factors are 0.00003161 for station 1+70, 0.00003314 for station 2+20, and 0.00003302 for station 2+70. The tiltmeter at station 2+70 was placed on August 4, 1994 and the tiltmeters at stations 1+70 and 2+20 were set up on October 26, 1994. The tiltmeters were placed on top of the concrete cap before the final grading of the surface was completed.

### 3.16 Inclinometers

Inclinometers were installed in the cap as well as up slope and down slope from the wall at stations 1+70 and 2+70 so that movement of the slope could be monitored as construction progressed and as the wall began to take effect. The inclinometers were monitored on a weekly basis during the three month construction period, then monthly for a period of six months, and then bi-monthly for another year. The inclinometers required a site visit for each reading. The readings were taken using Slope Indicator's Digitilt Datamate and Inclinometer probe and the data reduced using Slope Indicator's DMM and DigiPro software packages. The inclinometer casings were installed in a vertical borehole which was drilled through the slide plane to a depth of 24 meters. Readings were taken from 23 meters to 1.5 meters at increments of 0.5 meters. The inclinometer probe measures the tilt of the casing which is caused by ground movement displacing the casing from its original position. This tilt is then converted to a lateral displacement which can be graphed. Some of the original inclinometer tubes sheared off as the slope continued to move during construction and had to be replaced. The original tubes placed at station 1+70 in the cap and downhill from the cap were placed on May 14, 1994 and remained usable. The tube at station 1+70 uphill from the cap had to be replaced on October 29, 1994. All of the tubes at station 2+70 had to be replaced during the course of construction. The tubes in the cap and downhill from the cap at station 2+70 were replaced on July 22, 1994 and the tube uphill from the cap was replaced on August 4, 1994.

3.17 Datalogger

All the internal monitoring equipment was wired into a Geokon multi-channel datalogger, model 8020 (Micro-10), which was programmed to read the instruments three times a day and store the information in a file which could be downloaded remotely with a modem. The datalogger is housed within a utility shed on the site uphill from the cap, between stations 1+70 and 2+70, and is supplied with electricity by a solar panel attached to the roof of the building. Prior to installation of the datalogger, the instruments were read intermittently for the first six to twelve weeks until the datalogger was installed. The actual data from all the instrumentation can be obtained on computer disk from Dr. Dan Brown at Auburn University.

3.2 Construction Procedure

The first step in constructing the INSERT wall was to excavate a working bench at the location where the concrete cap would be placed with enough extra room for maneuvering the drilling and grouting equipment. Next the form for the concrete cap was constructed and the reinforcing steel placed and tied. The corrugated plastic pipes used for aligning the insert piles and tieback anchors were also placed in their appropriate positions. The concrete cap was then poured and allowed to set. Next the insert pipe holes were drilled to the required depth through the corrugated pipe and concrete cap. Grout was pumped into the pile until the hole was filled to the surface, both inside and outside the pipe pile. The corrugated holes in the cap were filled with non-shrink grout. Piles were constructed, generally, from station 5+41 to station 0+00, or north to south. Once all the insert piles were drilled and placed, the ground anchors were drilled and a 4.5 inch diameter casing advanced through the corrugated pipe sleeve and concrete cap into the soil to the required depth. This was accomplished by using air and/or water as a flushing medium. The ground anchors were then installed and stressed to the appropriate lock-off load. The anchor blockouts in the concrete cap were filled with non-shrink grout and allowed to set. The site was then regraded to cover the pile cap, allowing for proper drainage. The construction process began in June 1994 and the contractor left the completed project on December 15, 1994.

Several unanticipated factors affected the construction of this structure. First was the fact that water had to be used to drill the holes for the piles and ground anchors. These were designed to be air jettisoned, but due to the stiffness and/or cohesiveness of the soil, water had to be used to get the piles into the shale strata. This water further aggravated the slope movement problems, as significant movements were observed during construction. With the slope moving faster there was a fear that the first piles would be sheared off before enough of the piles could be placed to sufficiently slow the slope movement down. As the piles were placed and grouted, a much larger amount of grout was used than was expected, undoubtedly due to loss into the rock fill.

Most of the regular piles were placed by drilling the hole, removing the drill string to recover the drill bit, reinserting the pipe and grouting the hole through the pipe. When the instrumented piles were placed, the drill string had to be fully removed and grout placed before the instrumented pipe could be placed. This different procedure was due to the channels running down the length of the piles and the gage lead wires inside the pipe. On occasion with the drill string removed, the hole collapsed before the instrumented piles could be placed, which caused a major construction headache.

# 4.0 MEASUREMENT AND REDUCTION OF DATA

## 4.1 Slope Movement Data

Inclinometers were placed in the slope before construction of the wall so that a base line reading of how the slope moved could be recorded, as well as to enable the gathering of information about slope movements as construction progressed. The first inclinometer tubes placed in the slope sheared off and eventually had to be replaced as construction progressed. There were 15 mm of slope movement before construction of the piles began. As construction continued, another 7 mm of movement was measured. Since the end of construction in Dec. of 1994, slope movement has been approximately 1.5 mm at the cap and 2 mm above and below the cap. The information from the inclinometers have shown that the movement of the slope has basically stopped since the wall was constructed, as can be seen in Figures 11 through 13. The settlement and tilt meters confirm this leveling off of movement. When the inclinometer data is reduced via the computer software, the resulting graphs give the appearance that there is movement below the slip plane. The slip plane is approximately 13 m below the ground surface on the weathered shale surface. Below this level is the natural shale bedrock in which no slope movement can occur. For this reason, all graphs have been corrected by a zero offset so that no movement is shown below the slip plane.

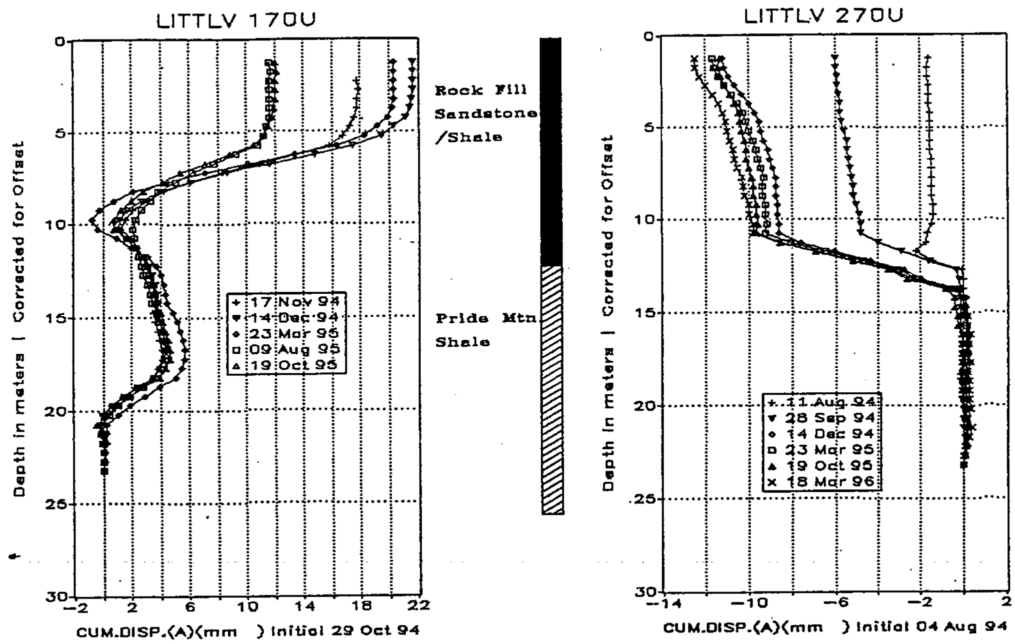



Fig. 11: Slope Inclinometer Data, uphill tubes

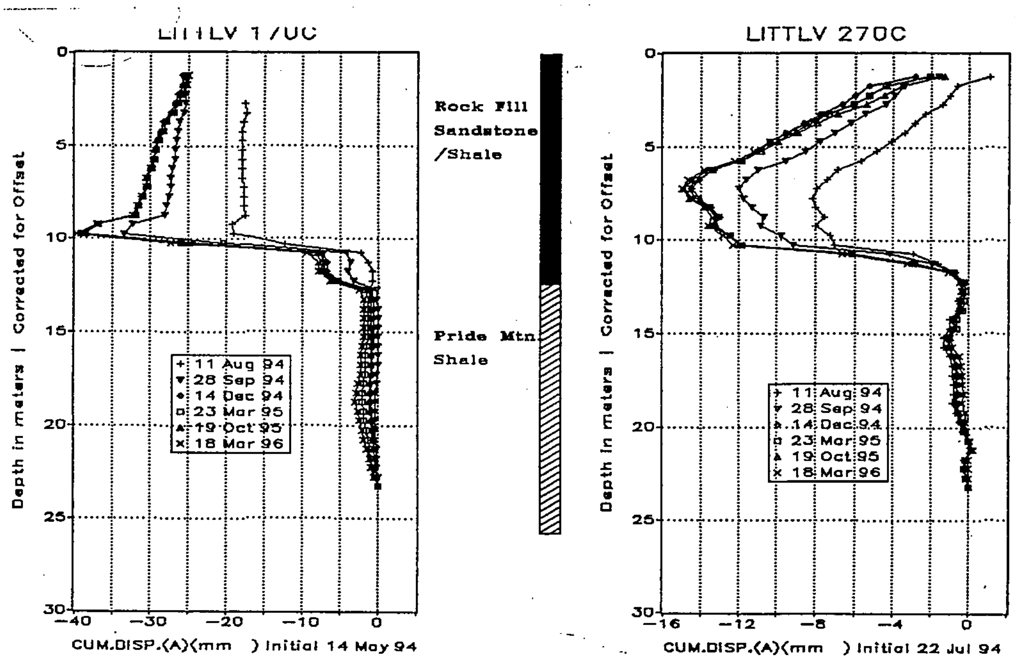



Figure 12: Slope Inclinometer Data, tubes at the cap

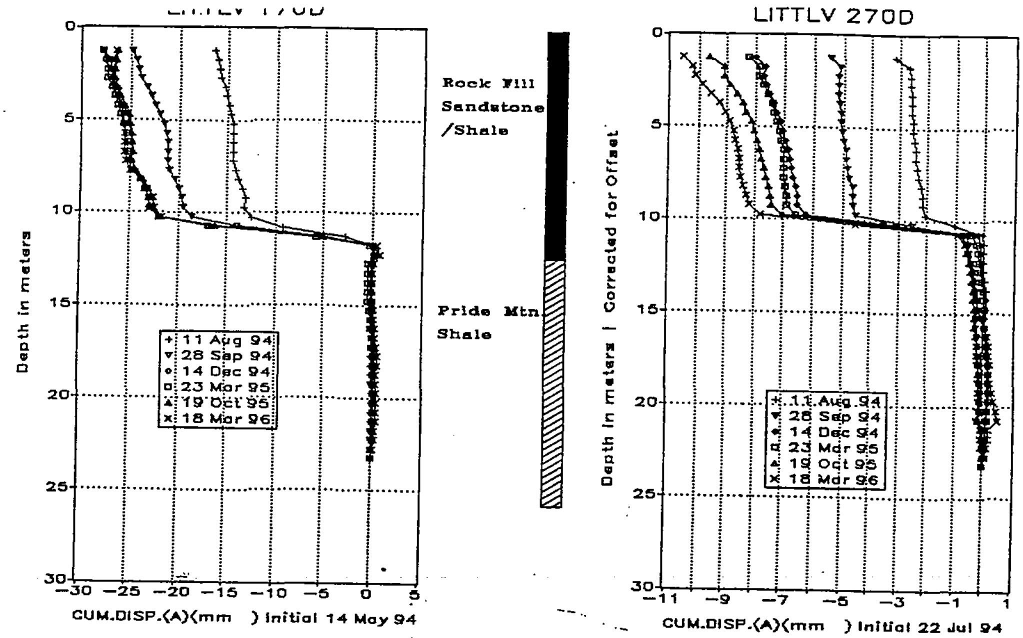



Figure 13: Slope Inclinometer Data, downhill tubes

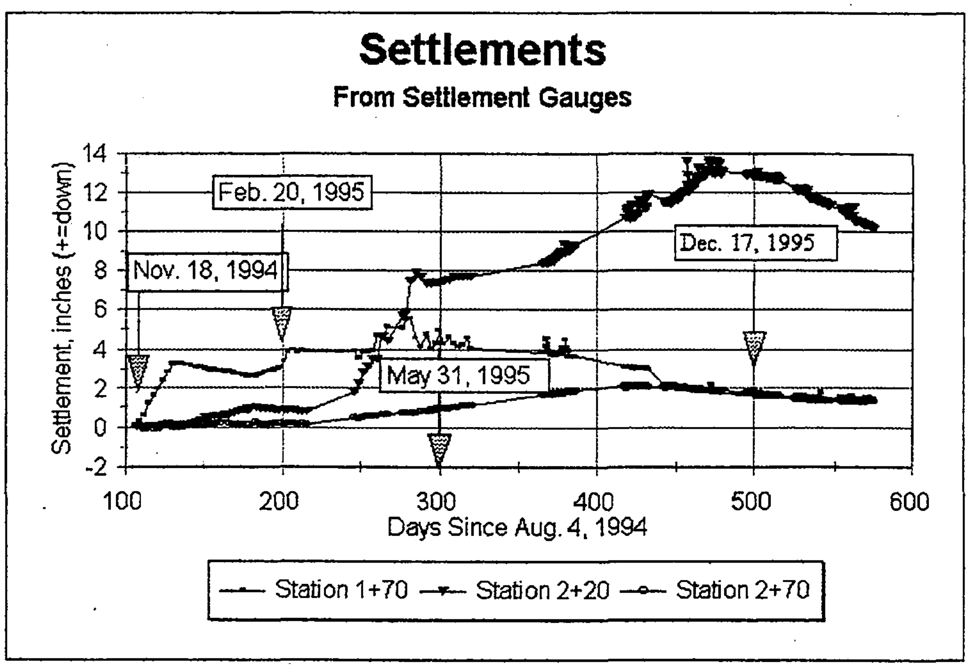



Figure 14: Settlement Data

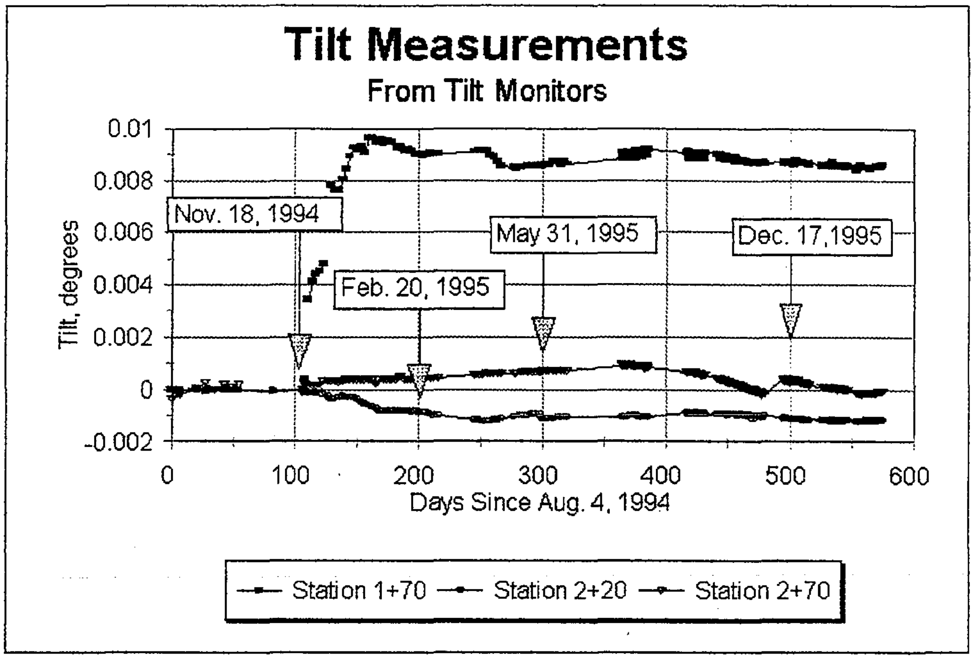



Figure 15: Tiltmeter Data

The ground anchors were tensioned on November 18, 1994 which is evident in the data from the settlement indicators and the tiltmeters (fig 14 and 15). Some of the readings look strange and do not match the other data which implies that in future endeavors redundant gages should be used. Also the tiltmeters should not be mounted on the concrete cap which will be subject to movement due to the ground anchors being jacked against it, thus jerking the cap and the tiltmeter. A better placement might be to mount them on a ground monument which will not be moved during the construction process.

4.2 Piezometer Data

The piezometer levels remained relatively stable for the first year after being placed and have since shown the heavy rains which have taken place during the fall and winter of 1995-96 (see fig. 16). Even with these elevated amounts of rainfall, no slope movement has been detected in the inclinometer data.

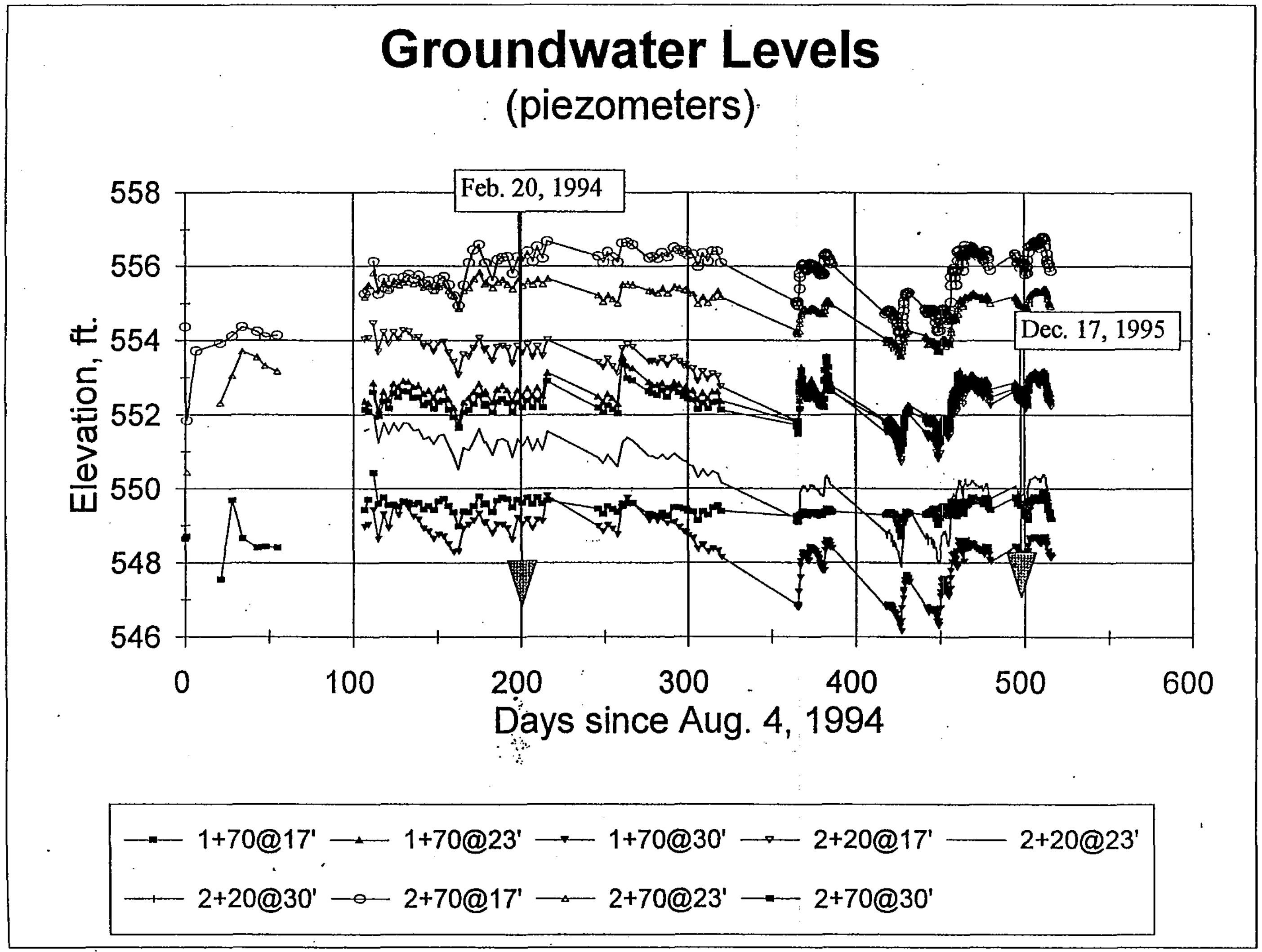



Figure 16: Piezometer Data

4.3 Stress Measurement Data

Strain gages were placed at eight points along the length of the pile. The first set of strain gages is located just below the pile cap, well away from any bending near the failure surface, but subject to forces from the cap. The other seven sets are all centered around the slip plane so that the full extent of pile reaction could be recorded. The sets of strain gages are located on opposite sides of the pile with steel channels running the length of the pile to protect the gages from damage during construction. The values from the gages are averaged and converted to an axial force along the pile. The uphill pile is in tension and the downhill pile is in compression. The difference in the strain gage values is converted into a bending moment along the pile. The axial force and bending moments are calculated by the following equations:

Axial Force = $\varepsilon$*A*E    Bending Moment = $\varepsilon$*E*I/c

where $\varepsilon$ = avg. of strains for axial and difference in strains for bending
A = cross-sectional area of the pile
E = modulus of elasticity for steel
I = moment of inertia for the member
c = distance to the centroidal axis of the member.

Load cells were placed on some of the ground anchors so that the applied load could be monitored as the slope movement continued. The lock off load for the ground anchors was approximately 2/3 of the design load.

4.4 Direct Shear Testing

In order to determine the actual strength properties of the soil along the slip plane, samples were taken from the toe of the slope in the weathered shale strata. These samples were obtained by digging through the overburden down to the strata and then

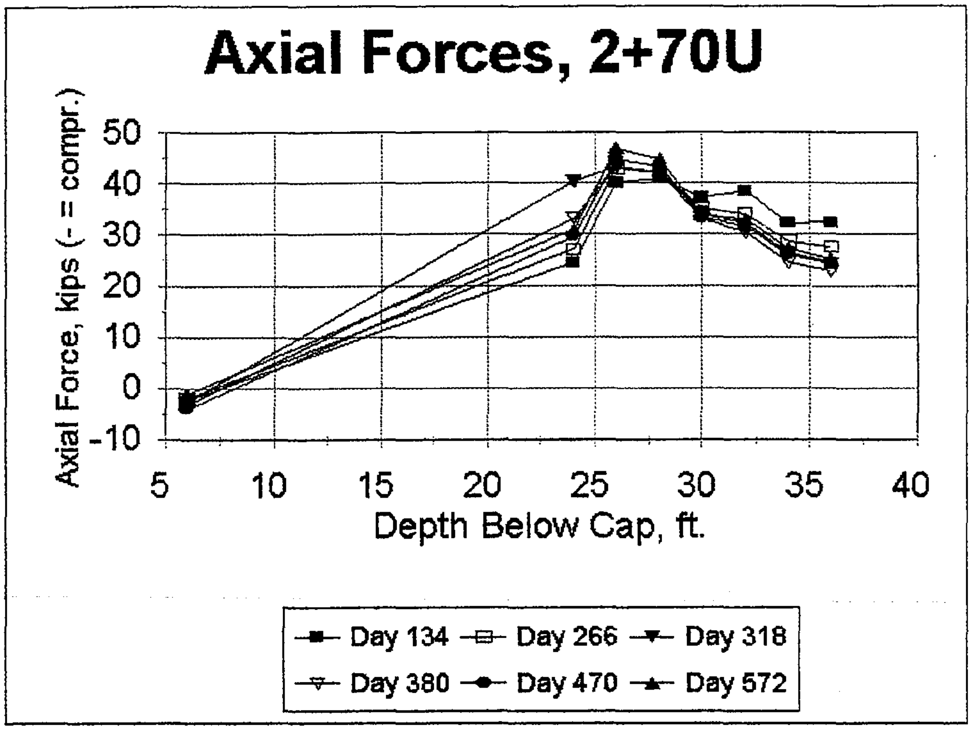



Fig. 17: Axial forces measured on the uphill pile at Sta. 2+70

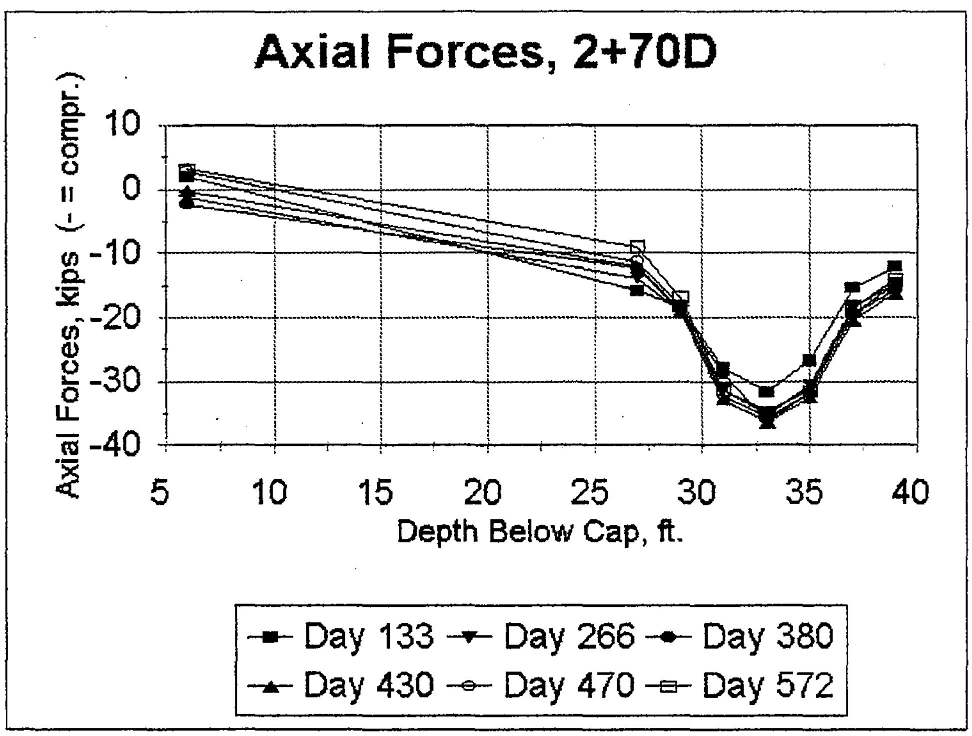



Fig. 18: Axial forces measured on the downhill pile at Sta. 2+70

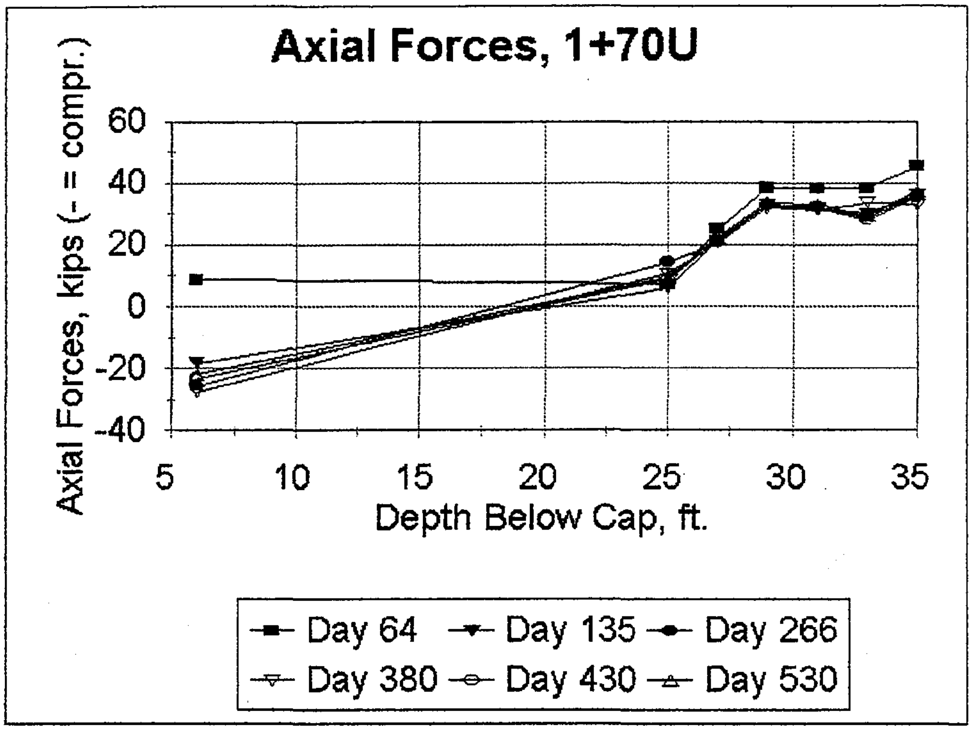



Figure 19: Axial Forces in the Uphill Pile at Sta. 1+70

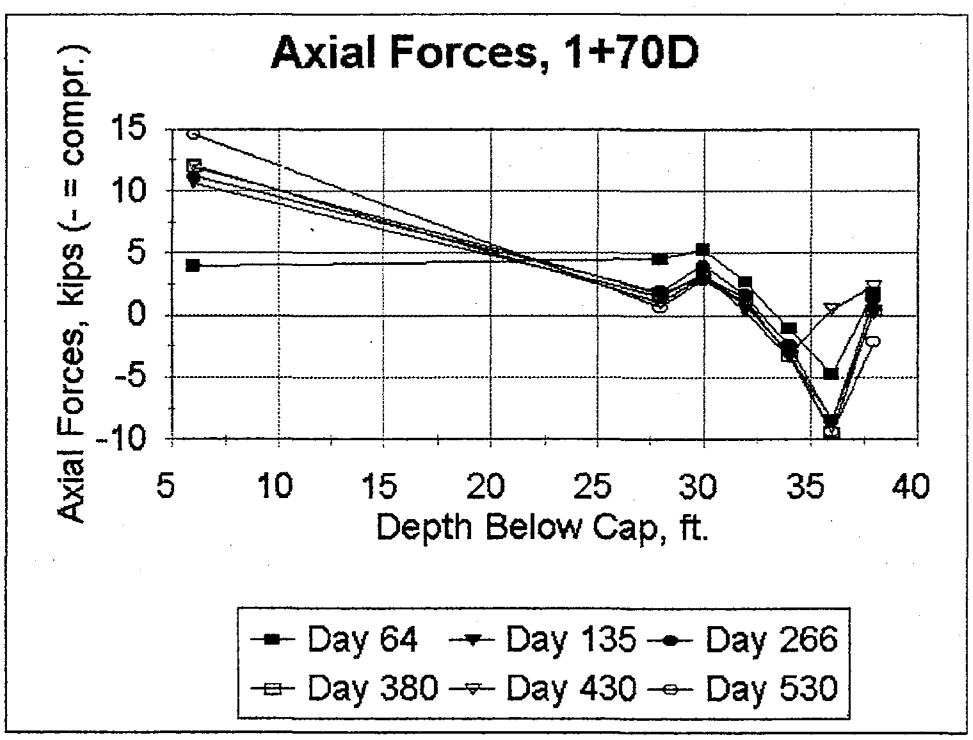



Figure 20: Axial Forces in the Downhill Pile at Sta. 1+70

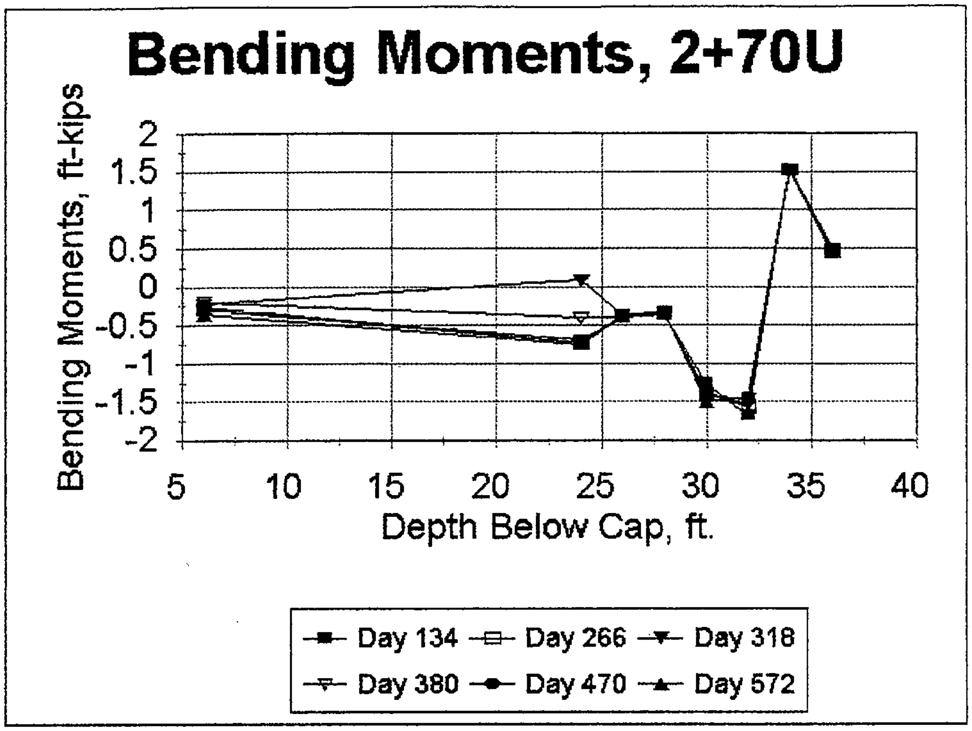



Figure 21: Bending Moments Along the Uphill Pile at Sta. 2+70

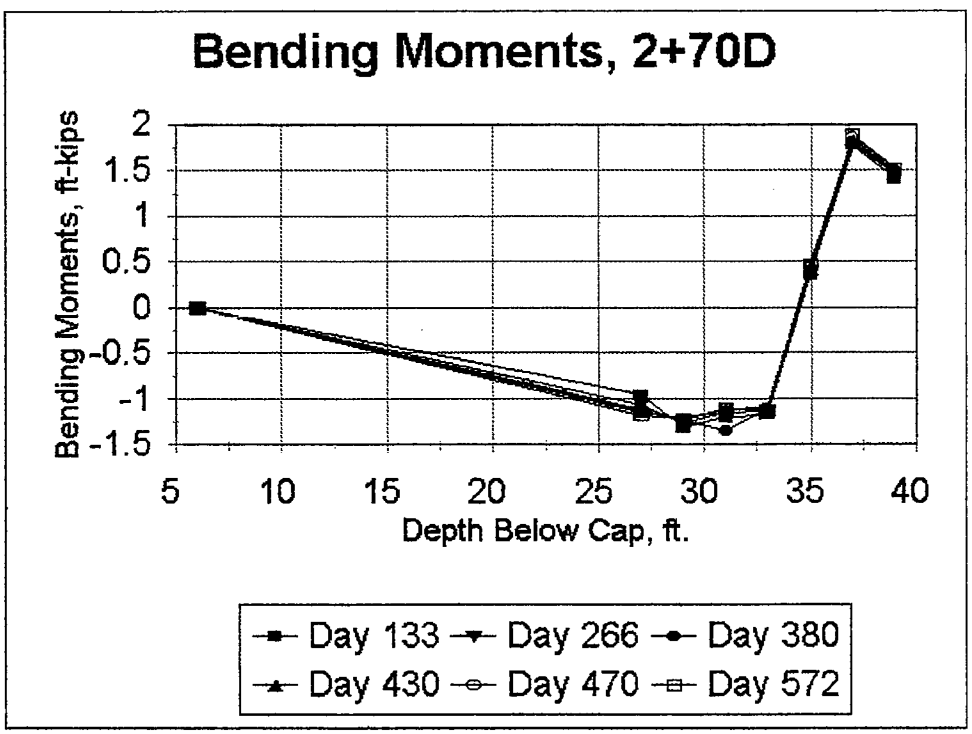



Figure 22: Bending Moments Along the Downhill Pile at Sta. 2+70

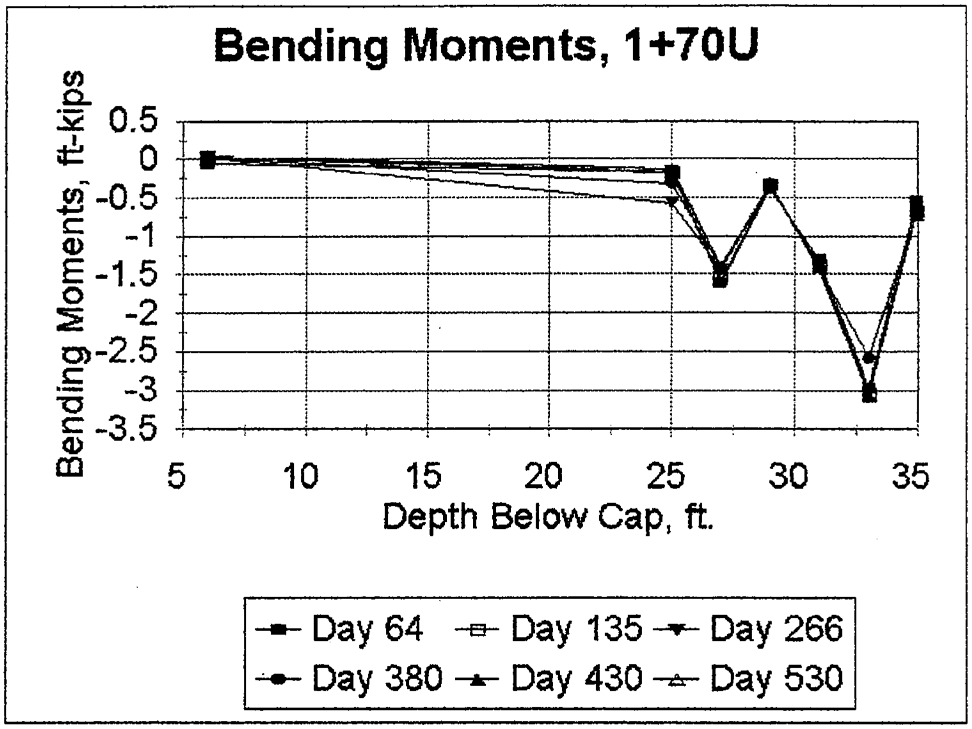



Figure 23: Bending Moments Along the Uphill Pile at Sta. 1+70

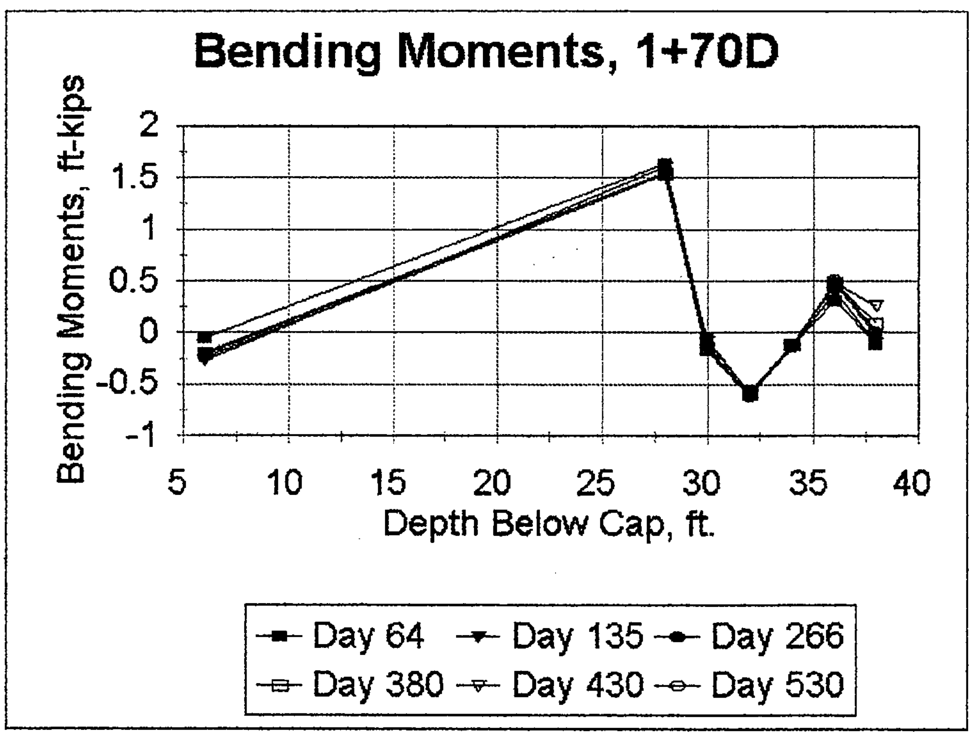



Figure 24: Bending Moments Along the Downhill Pile at Sta. 1+70

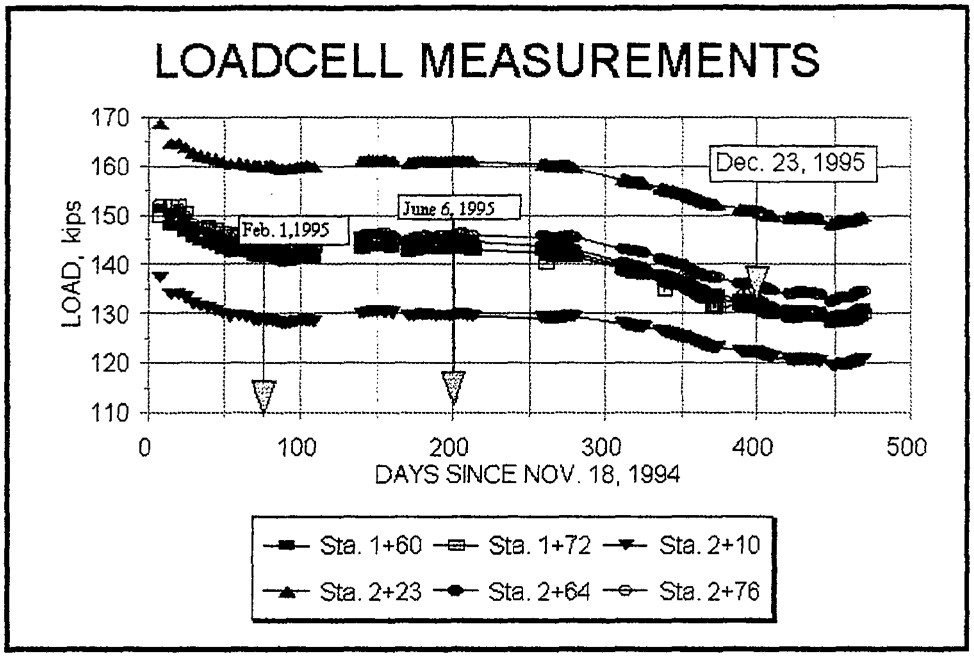



Figure 25: Loads Measured in the Ground Anchors

hand carving the samples out of the weathered shale. The samples were sealed in concrete cylinder tubes with wax and double ziplock bags and brought back to the lab for testing. The samples were then hand trimmed using a 2.5 inch diameter cutting ring. The samples were 0.75 inches tall and were extruded from the cutting ring into the direct shear machine. The samples were tested at a range of vertical stresses which could then be used to determine the Mohr-Coulomb envelope for the soil. Some of the samples were tested for residual strength and the results of all these tests are graphed in Fig. 26. The residual friction angle found by graphing the values of normal and shear stress was approximately 9.5°. This value was then used to draw a best fit line through the peak shear stress values and a cohesion value of two psi was determined. The data from direct shear testing were erratic, likely due to the heavily fissured structure of the soil samples. The direct shear testing probably overestimated the peak strength of the soil

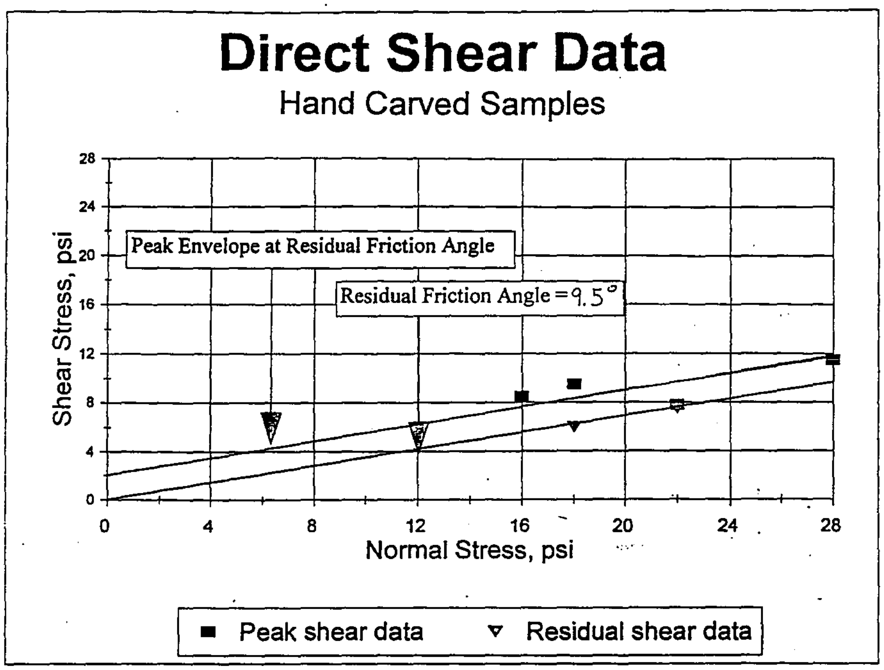



Figure 26: Results of Direct Shear Testing

in the field due to the shear plane being forced to follow a particular path in the machine which would not happen in the field. In the field the shear stress would follow the path of least resistance which would be along any fissures or slickenslides which were close by. Therefore, the residual value of the friction angle was used to determine the value of cohesion for the soil. The value of 11° determined by back-calculating the strengths along the shear surface with cohesion equal to zero was a residual value, since the slide has existed for a number of years.

## 4.5 Observations

The data obtained from the strain gages at station 1+70 have been strange and unreliable from the beginning, therefore all analyses have been focused on the more reliable data from station 2+70. The bending moment readings were double integrated over the length of the pile to determine the relative deflection. The deflection for the uphill pile at 2+70 was calculated to be 0.068 inches or 1.73 mm and the deflection for the downhill pile was 0.38 inches or 9.65 mm. The comparative value from the inclinometer data was 2 mm for both the uphill and downhill pile as noted earlier.

In looking at successive inclinometer readings, it would appear that there has been movement back up slope in the upper five to seven meters. This would be reasonable if the ground anchors are being put into tension as the slope moves downhill. The tension force would keep the cap block from moving downhill with the rest of the slope which would appear in the inclinometer readings as movement up slope. If the soil crept up slope, then the tendons would relax a little as seen in the loadcell measurements which have recorded a decrease in measured load since construction ended. This relaxation of load may be attributable to the stretching of the ground anchor tendons or the tendons may have pulled the cap uphill.

The axial strains in the piles were integrated to determine the amount of deflection in the axial direction. The axial displacement was computed to be 0.04 inches or 1 mm for the uphill pile at station 2+70 that is in tension. The downhill pile (which is in compression) was estimated to have 0.03 inches or 0.76 mm of axial compression.

It appears that the uphill pile is carrying more axial force than the downhill pile at both station 1+70 and 2+70. This trend can also be seen in the bending moment plots which show that the uphill pile is carrying more force. The shape of the bending moment curve for the uphill pile shows a sharper rebound in bending moment than the downhill pile which is believed to be representative of the rest of the piles. Of course, it is always possible that there is an obstruction next to the downhill pile which is affecting the bending strains. There is also an obvious moment reversal along the pile at approximately the location of the failure plane which is to be expected due to the distribution of stresses caused by the movement of the slope. The distribution of axial load away from the failure plane, as seen at station 2+70, is due to this stress build up with slope movement and would be expected in this situation.

# 5.0 ANALYSIS OF DATA

The purpose of this project was to evaluate the validity of the design approach and suggest improvements to that design procedure. The forces used in the design calculations and for factor of safety calculations were at failure (limit) conditions. The forces measured and analyzed represent forces mobilized at the current state of equilibrium (which is hopefully not a failure condition). Thus the analyses are directed toward the mobilized forces and utilize these values to try to project what forces could ultimately be supported by the pile.

## 5.1 Shear Forces Determined

The information that the instrumentation could not provide directly was how much shear force was being transferred to the pile by the slip plane. Estimates of shear transverse to the pile axis were determined by using the computer software COM624 to match the bending moments found by the strain gages. The pile was modeled as two free headed piles with the respective soil conditions found above and below the slip plane. A negative moment was applied to the pile in the fill material and a positive moment of the same magnitude was applied to the lower portion of the pile. The shear and moment applied to the two pile halves was varied until the computed slope at the pile heads matched and the moment versus depth curve fit the data reasonably well. The respective axial force for the uphill and downhill piles was used, which meant using a negative axial force or tension force for the uphill pile . The uphill pile was interpreted in this manner to have a shear of 2.68 kips and the downhill pile interpreted to have a shear of 2.52 kips. The shape of the moment versus depth curve on the downhill pile at station 2+70 may be suspect but was assumed to have the same shape as the uphill pile at station 2+70 (see figs. 27 and 28). The downhill pile may be stiffer due to the grout contributing to a greater section modulus, EI,. Since the pile is in compression, the grout would not be expected to crack and would be available to strengthen the pile. The COM624 output can be found in Appendix B.

5.2 Computation of Resisting Forces

The transverse shear forces developed from COM624 were used to determine the forces supplied by the piles along the slip plane and perpendicular to the slip plane. The axial force found by the strain gage plots was also used in this determination. The slip plane is believed to act at an angle of 10° with the horizontal and the piles are battered at an angle of 30° from vertical uphill and downhill respectively. The ground anchors are battered at an angle of 30° with the horizontal (see Fig. 29). The resultant force on the uphill pile acting parallel to the slip plane was found to be +17.2 kips and the force perpendicular to the slip plane was +39.5 kips, with positive forces denoting shear in a direction opposing slope movement and increasing compression force normal to the slip plane. For the downhill pile, the slip plane shear force was +24.4 kips and the perpendicular force was -25.2 kips. The forces due to the ground anchors was found to

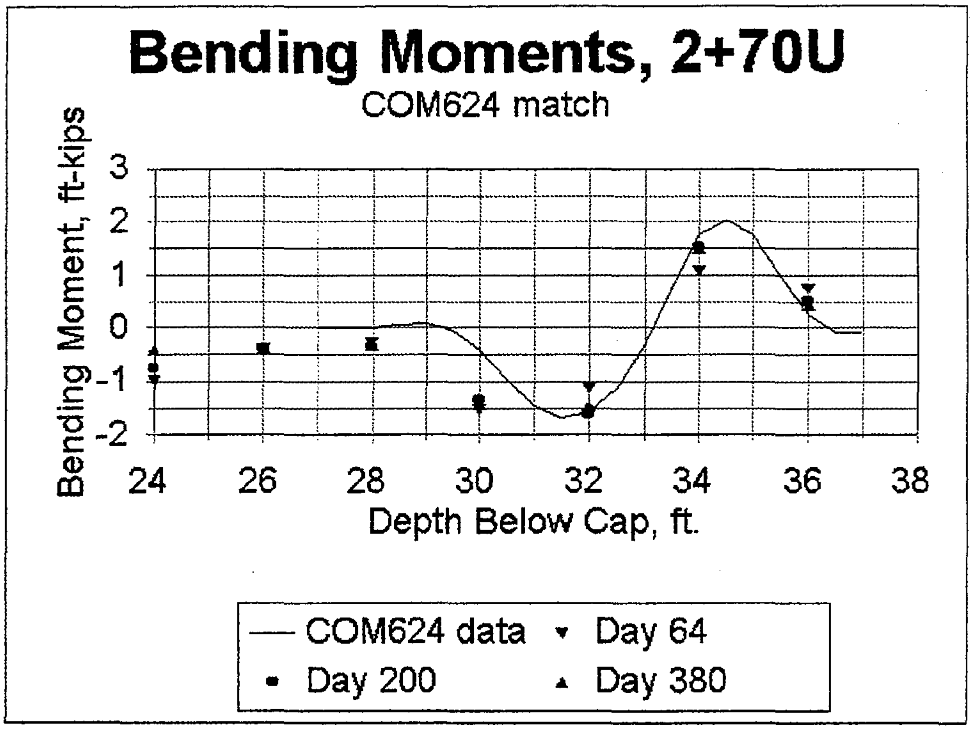



Figure 27: Bending Moments Matched Using COM624 for Uphill Pile

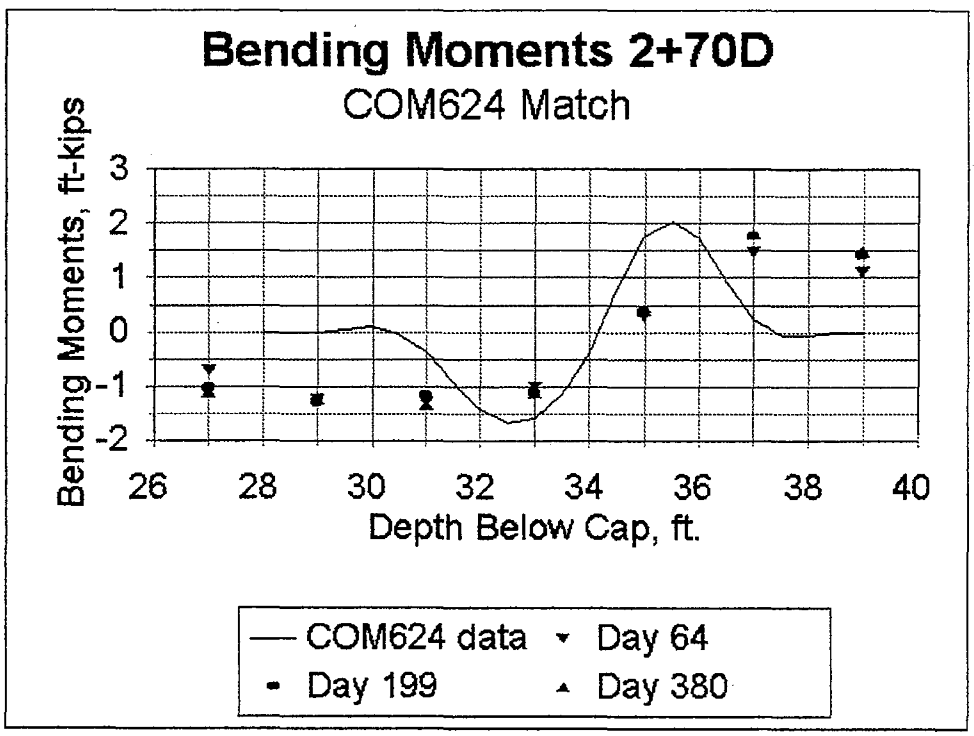



Figure 28: Bending Moments Matched Using COM624 for Downhill Pile

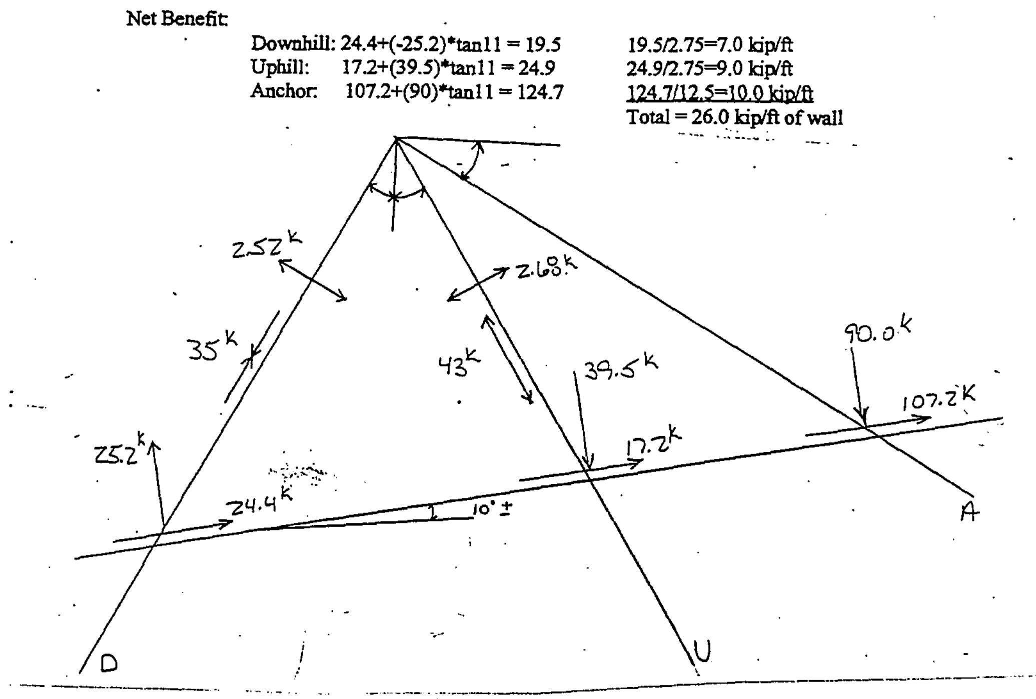



Figure 29: Force Distribution in the Structure

be +107.2 kips along the slip plane and +90 kips perpendicular to the slip plane. Note that there are approximately 4.5 piles for every ground anchor along the wall.

Once all three member types were analyzed, and the compression in the downhill pile was accounted for, the wall directly produced 26 kips of beneficial force up slope per foot of wall. The ground anchors provided 38.5% of this force with 10 kips, the uphill piles provided 34.6% of the force at 9 kips and the downhill piles provided the other 26.9% with 7 kips (see Fig. 29). Included in these forces are the values of the forces acting normal to the failure plane multiplied by the tangent of the friction angle, 11°. These were included in the beneficial force upslope because there is not a distinct failure plane but the failure is believed to occur over a section of soil approximately one to two foot thick, in which case the friction force developed along the pile in this region would help to resist slope movement downhill. As mentioned, the uphill piles are carrying more of the load than the downhill piles.

5.3 Check of Calculated Shear

The Poulos and Davis procedure for analyzing battered piles was applied to both the uphill and downhill piles at station 2+70. The batter angle for each pile was taken with respect to the slip plane which is inclined at 10° from the horizontal. The lateral displacement values used for this computation were 0.34" for the uphill pile and 0.20" for the downhill pile. These values were based on inclinometer readings at the uphill and downhill positions. The inclinometers measure movement differential to the vertical direction, but the piles are battered at a 30° angle with the vertical and therefore movements had to be corrected to correspond with the angle of the battered pile. The value for EI was that used in the COM624 input, which was 230,550 k-in^2. This value is for steel only with I = 7.95 in^4 which was found by experiment in the field before the piles were placed. The concrete or grout was not considered in this part of the calculations, but the diameter of the pile was considered to be six inches which would include the grout. This is a possible source of error but is believed to be insignificant due to the empirical nature of the Poulos and Davis procedure. Many of the values for the procedure must be read from log or semi-log plots which are difficult to read for specific numbers within an order of magnitude.

Using the Poulos and Davis procedure with the displacements described above, the axial force along the uphill pile was found to be 18.6 kips and the shear force was 1 kip. For the downhill pile the computed axial force was 9 kips and the shear force was again 1 kip. These forces are well below the values given by the strain gages or the COM624 backfitted results. The value for $E_{soil}$ was taken as the value used in the original design, equal to 0.436 ksi. Using a range of values of soil elastic modulus, and values of $E_{soil}$ in the range of 0.847 ksi to 1.135 ksi provided better agreement with observations of loads in the uphill and downhill piles, respectively. Using these values of $E_{soil}$ , the uphill pile was computed to carry 48.3 kips of axial force along the pile and 2.38 kips of shear force, and the downhill pile was calculated to provide 23.2 kips axial and 2.62 kips shear. In summary, the procedure can provide a reasonable estimate of the ratio of axial to shear force, but is very sensitive to the value used for $E_{soil}$ which is very difficult to determine with any degree of accuracy. It is also sensitive to the displacement used; the original computations at a shear displacement of 2.55" suggested an axial to shear ration which was much lower, at about 3.8:1.

5.4 Factor of Safety Determined with Mobilized Pile Forces

The original design called for a factor of safety after construction of 1.3. This assumed that enough slope movement would occur to fully mobilize the strength capacities of the respective members. The computer program STABL5M was used at four positions along the slope where appropriate data was available to model the slope. The program was first used to find the critical failure surface. This failure surface was then used as input along with a one foot layer of soil with a strength corresponding to the strength mobilized by the wall and tieback forces equaling the actual forces to be produced. The program was run over and over again changing the strength value of the one foot layer until the strength of the soil layer was enough to produce the target factor of safety. As mentioned previously, this strength was then used to design the piles to be placed. This resulted in factors of safety ranging from 1.296 at Sta. 2+35 to 1.305 at Sta 1+35 and 1.310 at Sta. 0+35 and 3+35. These same slope geometries were used to determine the factor of safety for the slope as it stands now with the currently mobilized forces. These were found to be:

| | | | |
|---|---|---|---|
| Mobilized FS = | 1.17 | at | Sta. 0+35 |
| Mobilized FS = | 1.13 | at | Sta. 1+35 |
| Mobilized FS = | 1.12 | at | Sta. 2+35 |
| Mobilized FS = | 1.13 | at | Sta. 3+35 |

Note that this computed "Mobilized FS" does not represent a factor of safety against a limit state of failure, but rather represents the ratio of the limit state resistance in the soil plus the mobilized wall force to that without the wall forces.

5.5 Ultimate Factor of Safety Calculated

The computer program COM624 was then used to determine the maximum shear and axial force combination the piles could withstand before the designed maximum moment was reached, or the pile failed. It was determined that the mobilized axial load capacity of 86.1 kips of skin friction would be reached before the pile failed in bending due to shear. Using the ratio of axial load to shear load developed for the present loading situation, a maximum shear force which the pile could be expected to experience at an axial load of 86.1 kips was calculated. This shear force was found to be 6.2 kips for the downhill pile and 5.37 kips for the uphill pile. The proportion of ultimate forces to present forces for the uphill piles was 2.00 and the proportion for the downhill pile was 2.46. The uphill pile has a lower proportion due to the larger mobilized shear force it is experiencing at present, i.e., the uphill pile is mobilizing its capacity at a lesser deflection than the downhill pile. The four slope geometries which were used in the design and analysis were again used with the new values of beneficial strength and tieback loads. The tieback load used was the design load of 246 kips. The factors of safety found by using the ultimate computed forces are:

| | | | |
|---|---|---|---|
| FS = | 1.36 | at | Sta. 0+35 |
| FS = | 1.27 | at | Sta. 1+35 |
| FS = | 1.25 | at | Sta. 2+35 |
| FS = | 1.28 | at | Sta. 3+35 |

These values are close to the target factor of safety of 1.3.

# 6.0 LOCATION OF THE WALL AND GEOMETRIC ARRANGEMENT OF THE PILES

During the course of this research, some potential considerations have come to light as a result of the use of a limit equilibrium approach to slope stability to model a wall of this type. The stability analyses are performed using a method of slices as characterized by PCSTABL5. The discussions which follow also provide some insight into the appropriate placement of a relatively flexible "wall" system such as the INSERT wall system and the appropriate geometry of the piles. For purposes of this discussion, the code PCSTABL or PCSTABL5 is referenced; however, this code is typical of a code used for limit equilibrium analysis of a slope failure using the method of slices and some set of assumptions regarding interslice forces and the comments are intended to generally apply to any such slope stability analysis method.

## 6.1 Location of the INSERT Wall

For analysis of the slide, it has been a common practice to use a version of the PCSTABL computer code with the wall represented by a thin layer of high shear strength. This practice does not reflect the method by which the pile capacity is estimated, but rather the method in which the pile capacity is incorporated into the overall analysis of the slope stability problem. An examination of the interslice forces resulting from such an analysis can be helpful in understanding the problem and considerations resulting from various locations up or downslope in which an INSERT wall might be placed.

Analyses of a hypothetical potential failure surface on a steep slope (shown on Figure 30) have been performed using a force equilibrium procedure available in PCSTABL5 in which interslice forces are assumed to be horizontal (as per the "Janbu" method in PCSTABL5). For this simple example, the soil is taken as a cohesionless material with $\phi=30°$, no pore water pressure, and the wall modeled as a one foot wide slice with a cohesion of 11 ksf (to produce a wall with a resistance of 11 kips/foot parallel to the failure surface). Table 1 illustrates in spreadsheet form the computed forces on each slice, with the interslice forces presented graphically on Figure 31. These two analyses differ only in the placement of the wall along the slope; the data on Table 1a and Figure 31a are for the wall placed near the top of the slope and the data on Table 1b and Figure 31b are for the wall placed near the toe of the slope.

The interslice forces shown on Figure 31 demonstrate dramatically the effect of placing a wall too high on the failure surface. The side forces for the wall placed near the top of the slope on Figure 31 actually go into tension in the region immediately below the wall. This condition is unlikely to be sustained in a real situation because the soil would be unable to sustain tension; furthermore, the wall would be required to exert tension on the soil immediately below the wall for the results of this calculation to be meaningful! In this case, with a steep slope and a very low factor of safety, it is evident that a failure in the soil downslope below the wall would be likely. Such a condition would leave the INSERT wall exposed and subject to erosion and a generally undesirable situation even if the critical portion of the slope were above the INSERT wall.

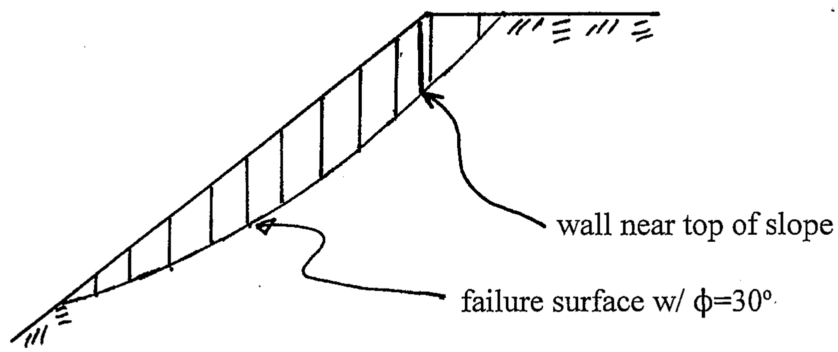
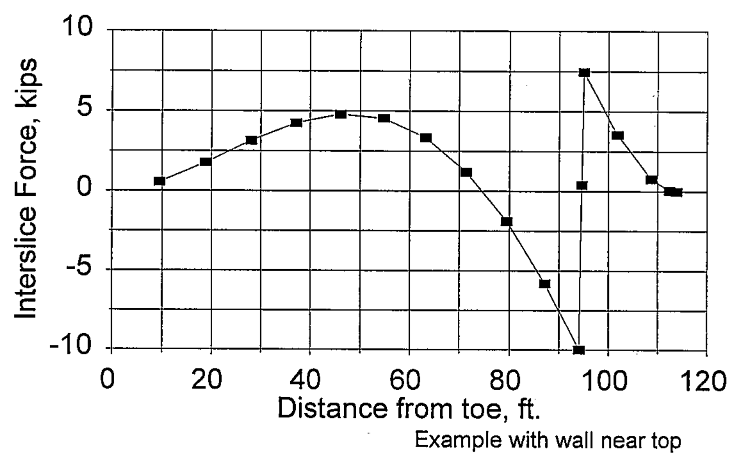



Figure 30 Schematic Diagram of Example Slope Problem



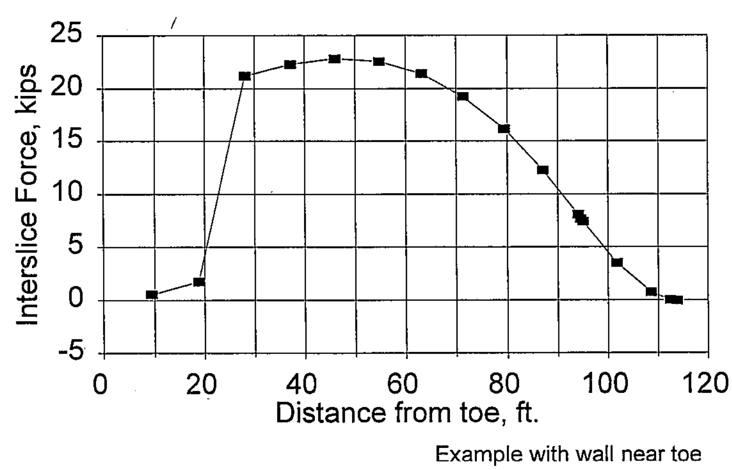




Figure 31 Interslice Forces a) with wall near TOP of slope, b) with wall near TOE of slope

Table 1a) Example calculations with wall near TOP of slope — Theta = 0

wall simulated by slices 12 and 13 — cbar = 11ksf r sub u = 0 — Trial F= 1.04

| slice # | delta x | x @ ctr | W, kips | alpha | W sin alpha | c | u(dx) | phi | Nsub0 | Nsub1 | Nsub2 | Nsub3 | Nsub4 | delta E | Cumm E | x |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 9.53 | 65.775 | 2.7041 | 17.62716 | 0.818859 | | 0 | 30 | -0.20181 | -0.54571 | 0 | 0 | 1 | -0.54571 | 0.545706 | 9.53 |
| 2 | 9.4 | 75.24 | 7.6961 | 20.05845 | 2.639596 | | 0 | 30 | -0.15799 | -1.21593 | 0 | 0 | 1 | -1.21593 | 1.76164 | 18.93 |
| 3 | 9.23 | 84.555 | 11.9249 | 22.48858 | 4.561262 | | 0 | 30 | -0.11478 | -1.3688 | 0 | 0 | 1 | -1.3688 | 3.130437 | 28.16 |
| 4 | 9.07 | 93.705 | 15.3892 | 24.91794 | 6.48377 | | 0 | 30 | -0.07201 | -1.10815 | 0 | 0 | 1 | -1.10815 | 4.238587 | 37.23 |
| 5 | 8.89 | 102.685 | 18.096 | 27.34963 | 8.313642 | | 0 | 30 | -0.02945 | -0.53296 | 0 | 0 | 1 | -0.53296 | 4.771546 | 46.12 |
| 6 | 8.68 | 111.47 | 20.0611 | 29.78132 | 9.964162 | | 0 | 30 | 0.012999 | 0.260766 | 0 | 0 | 1 | 0.260766 | 4.510781 | 54.8 |
| 7 | 8.46 | 120.04 | 21.3084 | 32.21182 | 11.35845 | | 0 | 30 | 0.055475 | 1.18208 | 0 | 0 | 1 | 1.18208 | 3.3287 | 63.26 |
| 8 | 8.23 | 128.385 | 21.8702 | 34.64263 | 12.43224 | | 0 | 30 | 0.098157 | 2.146716 | 0 | 0 | 1 | 2.146716 | 1.181984 | 71.49 |
| 9 | 7.98 | 136.49 | 21.7862 | 37.07379 | 13.13365 | | 0 | 30 | 0.141203 | 3.07627 | 0 | 0 | 1 | 3.07627 | -1.89429 | 79.47 |
| 10 | 7.71 | 144.335 | 21.1039 | 39.50361 | 13.42475 | | 0 | 30 | 0.184744 | 3.898809 | 0 | 0 | 1 | 3.898809 | -5.79309 | 87.18 |
| 11 | 6.84 | 152.09 | 18.2192 | 41.99216 | 12.18916 | | 0 | 30 | 0.230051 | 4.19135 | 0 | 0 | 1 | 4.19135 | -9.98445 | 94.02 |
| 12 | 0.6 | 155.33 | 1.6387 | 43.06089 | 1.118863 | 11 | 0 | | 0.934504 | 1.531371 | 11.88823 | 0 | 1 | -10.3569 | 0.372417 | 94.62 |
| 13 | 0.4 | 155.83 | 0.9361 | 43.22747 | 0.641131 | 11 | 0 | | 0.939965 | 0.879901 | 7.9688 | 0 | 1 | -7.0889 | 7.461315 | 95.02 |
| 14 | 6.75 | 159.38 | 14.2939 | 44.42364 | 10.00512 | | 0 | 30 | 0.275202 | 3.933716 | 0 | 0 | 1 | 3.933716 | 3.527599 | 101.77 |
| 15 | 6.85 | 166.205 | 8.5695 | 46.79546 | 6.246427 | | 0 | 30 | 0.320273 | 2.744579 | 0 | 0 | 1 | 2.744579 | 0.783021 | 108.62 |
| 16 | 3.91 | 171.58 | 2.0378 | 48.73948 | 1.531852 | | 0 | 30 | 0.358109 | 0.729754 | 0 | 0 | 1 | 0.729754 | 0.053267 | 112.53 |
| 17 | 1.4 | 174.24 | 0.1801 | 49.72977 | 0.137417 | | 0 | 30 | 0.377732 | 0.06803 | 0 | 0 | 1 | 0.06803 | -0.01476 | 113.93 |

Table 1b) Example calculations with wall near TOE of slope — Theta = 0

wall simulated by slice 3 — Trial F= 1.04

| slice # | delta x | x @ ctr | W, kips | alpha | W sin alpha | c | u(dx) | phi | Nsub0 | Nsub1 | Nsub2 | Nsub3 | Nsub4 | delta E | Cumm E | x |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 9.53 | 65.775 | 2.7041 | 17.62716 | 0.818859 | | 0 | 30 | -0.20181 | -0.54571 | 0 | 0 | 1 | -0.54571 | 0.545706 | 9.53 |
| 2 | 9.4 | 75.24 | 7.6961 | 20.05845 | 2.639596 | | 0 | 30 | -0.15799 | -1.21593 | 0 | 0 | 1 | -1.21593 | 1.76164 | 18.93 |
| 3 | 9.23 | 84.555 | 11.9249 | 22.48858 | 4.561262 | 0.243 | 0 | 30 | -0.11478 | -1.3688 | 18.04559 | 0 | 1 | -19.4144 | 21.17603 | 28.16 |
| 4 | 9.07 | 93.705 | 15.3892 | 24.91794 | 6.48377 | | 0 | 30 | -0.07201 | -1.10815 | 0 | 0 | 1 | -1.10815 | 22.28418 | 37.23 |
| 5 | 8.89 | 102.685 | 18.096 | 27.34963 | 8.313642 | | 0 | 30 | -0.02945 | -0.53296 | 0 | 0 | 1 | -0.53296 | 22.81714 | 46.12 |
| 6 | 8.68 | 111.47 | 20.0611 | 29.78132 | 9.964162 | | 0 | 30 | 0.012999 | 0.260766 | 0 | 0 | 1 | 0.260766 | 22.55637 | 54.8 |
| 7 | 8.46 | 120.04 | 21.3084 | 32.21182 | 11.35845 | | 0 | 30 | 0.055475 | 1.18208 | 0 | 0 | 1 | 1.18208 | 21.37429 | 63.26 |
| 8 | 8.23 | 128.385 | 21.8702 | 34.64263 | 12.43224 | | 0 | 30 | 0.098157 | 2.146716 | 0 | 0 | 1 | 2.146716 | 19.22757 | 71.49 |
| 9 | 7.98 | 136.49 | 21.7862 | 37.07379 | 13.13365 | | 0 | 30 | 0.141203 | 3.07627 | 0 | 0 | 1 | 3.07627 | 16.1513 | 79.47 |
| 10 | 7.71 | 144.335 | 21.1039 | 39.50361 | 13.42475 | | 0 | 30 | 0.184744 | 3.898809 | 0 | 0 | 1 | 3.898809 | 12.25249 | 87.18 |
| 11 | 6.84 | 152.09 | 18.2192 | 41.99216 | 12.18916 | | 0 | 30 | 0.230051 | 4.19135 | 0 | 0 | 1 | 4.19135 | 8.061144 | 94.02 |
| 12 | 0.6 | 155.33 | 1.6387 | 43.06089 | 1.118863 | | 0 | 30 | 0.249778 | 0.409312 | 0 | 0 | 1 | 0.409312 | 7.651832 | 94.62 |
| 13 | 0.4 | 155.83 | 0.9361 | 43.22747 | 0.641131 | | 0 | 30 | 0.252869 | 0.236711 | 0 | 0 | 1 | 0.236711 | 7.415121 | 95.02 |
| 14 | 6.75 | 159.38 | 14.2939 | 44.42364 | 10.00512 | | 0 | 30 | 0.275202 | 3.933716 | 0 | 0 | 1 | 3.933716 | 3.481405 | 101.77 |
| 15 | 6.85 | 166.205 | 8.5695 | 46.79546 | 6.246427 | | 0 | 30 | 0.320273 | 2.744579 | 0 | 0 | 1 | 2.744579 | 0.736827 | 108.62 |
| 16 | 3.91 | 171.58 | 2.0378 | 48.73948 | 1.531852 | | 0 | 30 | 0.358109 | 0.729754 | 0 | 0 | 1 | 0.729754 | 0.007073 | 112.53 |
| 17 | 1.4 | 174.24 | 0.1801 | 49.72977 | 0.137417 | | 0 | 30 | 0.377732 | 0.06803 | 0 | 0 | 1 | 0.06803 | -0.06096 | 113.93 |

Table 1 Spreadsheet of Slope Stability Calculations for Circular Arc with Wall: a) near TOP of slope, b) near TOE of slope

The alternative condition with the wall placed more near the toe of the slope is presented on Table 1b and Figure 31b. For this condition, the interslice forces are in compression throughout the slope and the wall is effective at supporting the majority of the soil within the failure zone. Of course, it would still be prudent for an engineer to evaluate potential failure surface above the INSERT wall in this case, but for the particular failure surface analyzed, it is expected that the use of a single high strength slice to model the pile wall is unlikely to lead to serious computational errors.

The hypothetical case shown and described above is used to demonstrate the following points:

1) the location of the wall along the slope can be an important consideration from the standpoint of overall stability, and it is risky to place the wall too high on the slope, and

2) the use of a single high strength slice within the slope to model the pile resistance for purposes of slope stability must be done with care, as tensile interslice forces can develop which would be unrealistic and produce computational errors.

Note that these conditions are not considered to be a concern with the Littleville project as the slope is relatively flat and the INSERT wall is placed well downslope along the failure surface. The more critical conditions would likely exist where a steep slope was present with a relatively low factor of safety after placement of the wall.

6.2 Pile Geometry

In evaluating the pile axial forces (which contributed the majority of resisting force) on the Littleville project, it is noted that some piles on this project act in tension and some in compression due to the pile batter with respect to the failure surface. Of course, the Littleville slide is on a relatively flat failure surface estimated at around 10° from horizontal. On a steeply sloping failure surface with relatively little batter (from vertical) in the piles, it may be possible that all of the piles could end up battered to act in compression with respect to the failure surface. In such a situation, without a tieback anchor or other member acting in tension, the group of piles representing the INSERT wall could be left in an unstable position as shown on Figure 32. In this case, the majority of the resistance to overturning of the group is provided by the bending resistance of the individual piles themselves. Such a situation is to be avoided. In addition, the movement associated with the mobilization of compression in the piles would tend to "lift" the soil off the failure surface and thus <u>reduce</u> the effective normal stress on the failure surface. A reduction in normal stress on the failure surface would reduce the shear capacity and be detrimental to stabilizing the slide.

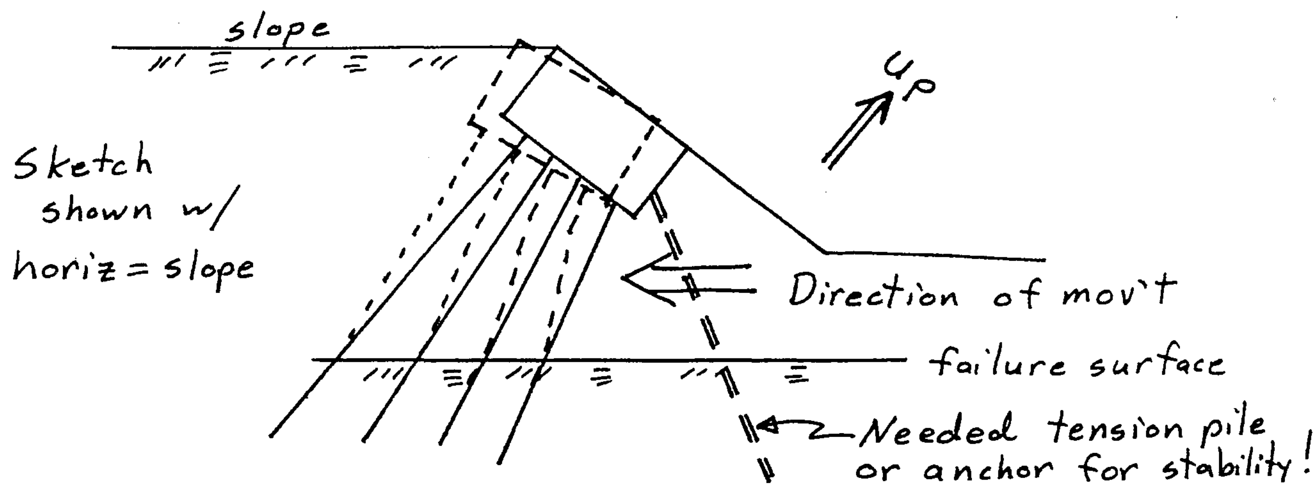



Figure 32 Batter Pile Group on a Steeply Sloping Shear Surface

On an alternate note, it would appear that a slope stabilization scheme in which all of the piles were battered with respect to the failure surface so as to act in tension would produce a stable and possibly efficient configuration. The movement associated with the mobilization of tension in the piles would tend to "pull" the soil within the failure zone down onto the failure surface and thus increase the effective normal stress on the failure surface. An increase in normal stress on the failure surface would increase the shear capacity and provide an additional contribution to stabilizing the slide. It would appear that such a design might even be effectively accomplished without the use of a pilecap and perhaps by using a series of rows of "tensile" piles at various elevations along the failure surface. For the conditions at Littleville, consider the case of uphill piles only, battered at an angle of 45° with the same mobilized forces of 43 kips axial load and 2.7 kips transverse to the pile. These resolve into a +36.8 kip force perpendicular to the slip plane and a +26.9 kip force along the slip plane. At 2.5' center to center spacing, to get the 26 kip/foot beneficial force provided by the present structure, 1.91 piles would be needed. Thus, two rows of uphill battered piles at 45° and 2.5' center to center spacing would (in principal) provide as much stabilizing benefit as much as the pair of battered piles and the anchors presently employed (provided the same full capacity was reached).

## 6.3 Summary

In summary, this chapter has provided a consideration of the effects of wall location, interslice forces in slope stability analysis, pile geometry, and alternative pile arrangements. A thorough and careful review of the considerations outlined may serve to result in more efficient, safe, and economical designs on future projects.

## 7.0 ADDITIONAL MONITORING

After completion of the project monitoring period, analyses of the data, and preparation of the bulk of this report, the data logging has continued to obtain long term measurements of performance. In addition, inclinometer readings were taken during November, 1996. The results of these measurements are presented on Figures 34 through 48, with the most recent data representing December, 1996 in all cases. Some cracking in the roadway above the slide repair was noted during November, 1996. However, the inclinometer data and the measurements from other instrumentation suggest that there has been little movement of the slide itself. Some cracking in the roadway may be related to reflective cracking from the older repaired surface below. Note that a few gauges (tiltmeter, settlement) appear to be producing erratic readings which are not correlating with other measurements.

In summary, the ongoing monitoring since completion of the analyses for this report appear to indicate that the slide remains stable, with very small additional movements or changes in pile and anchor forces. Monitoring will continue for the next few years as a check for any significant changes.

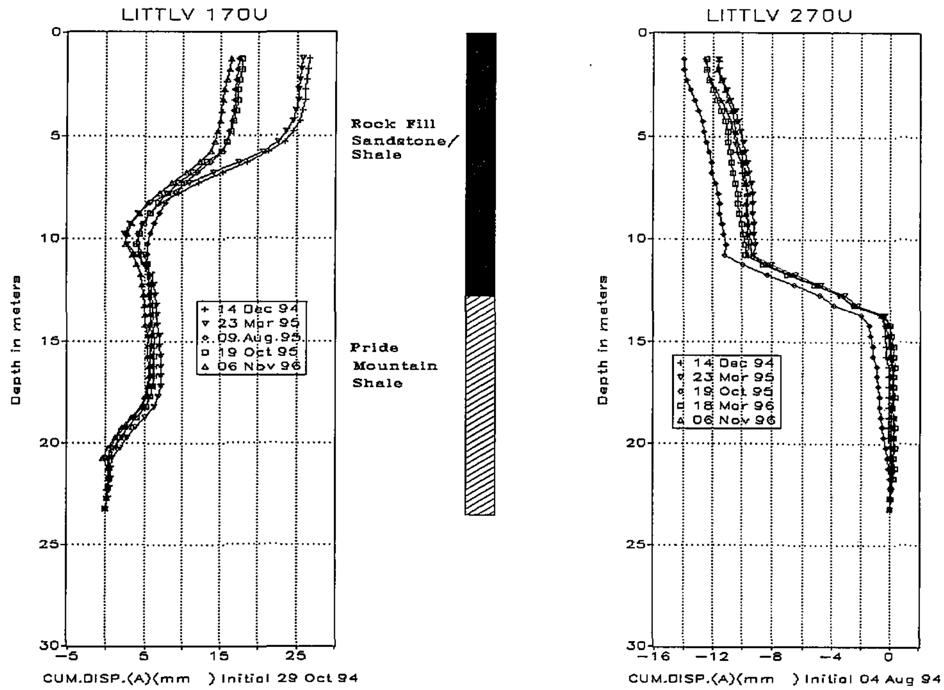



Figure 34: Slope Inclinometer Data, uphill tubes

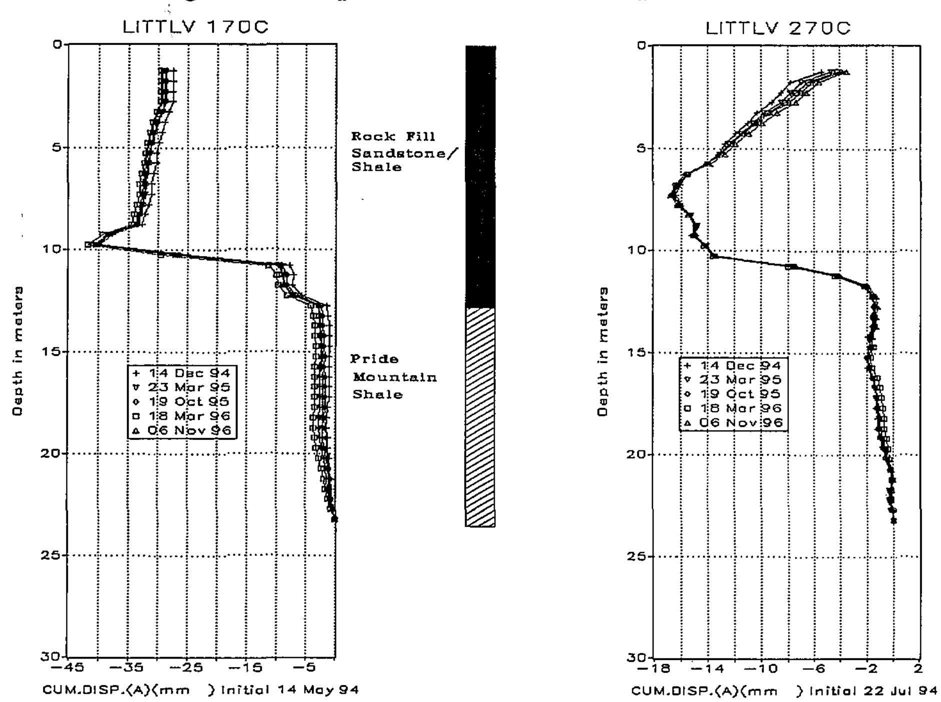



Figure 35: Slope Inclinometer Data, tubes at the cap

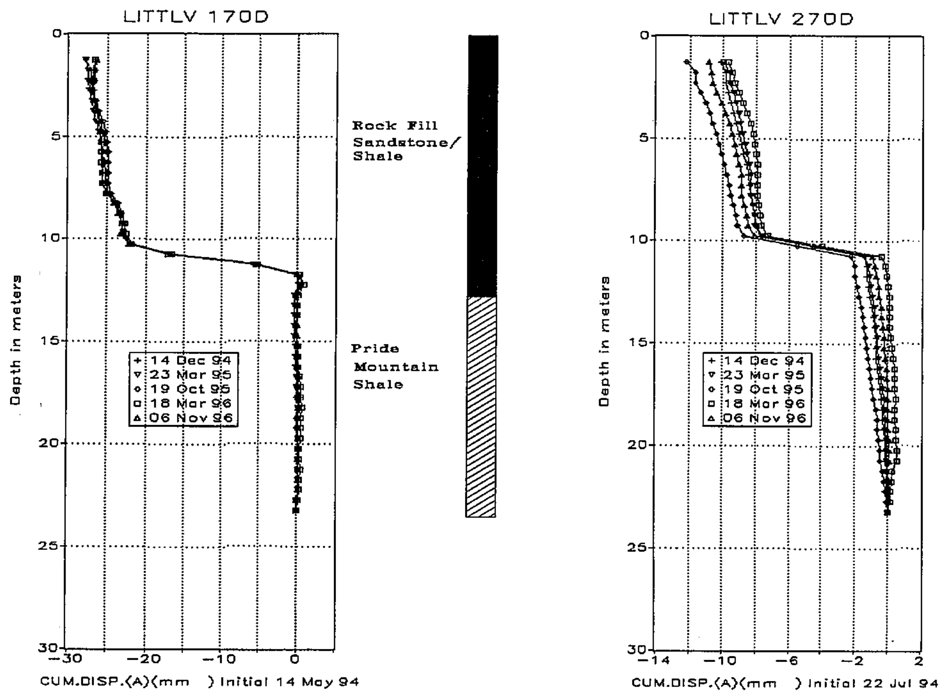



Figure 36: Slope Inclinometer Data, downhill tubes

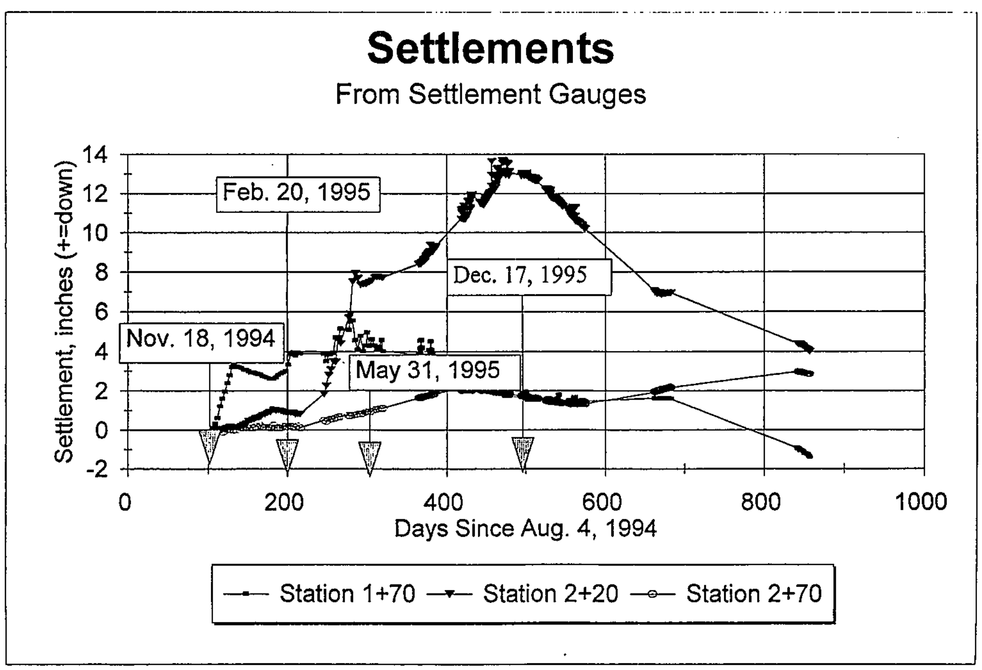



Figure 37: Settlement Data

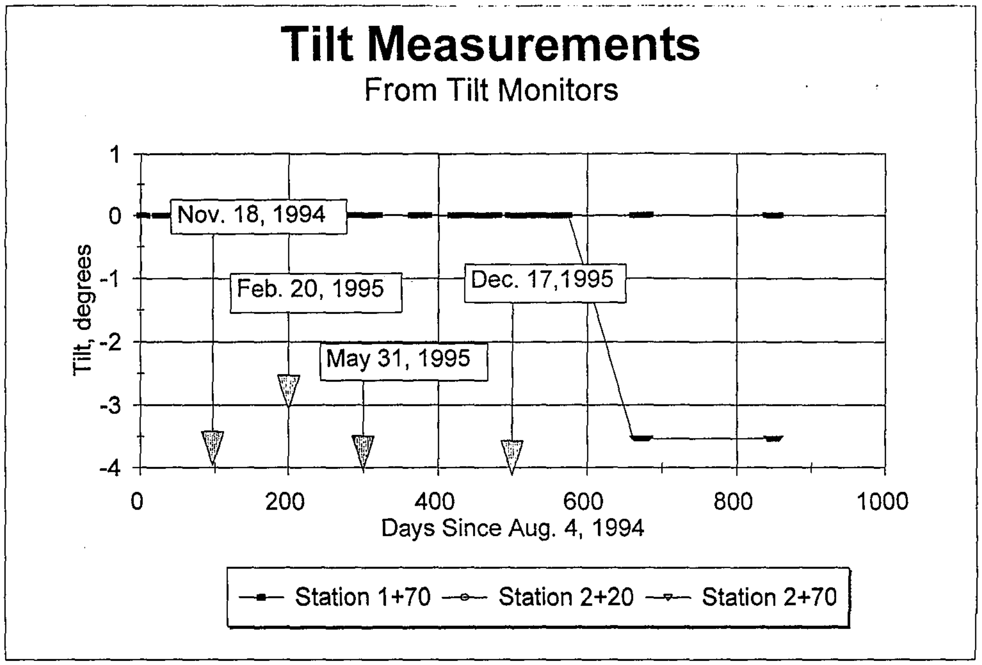



Figure 38: Tiltmeter Data

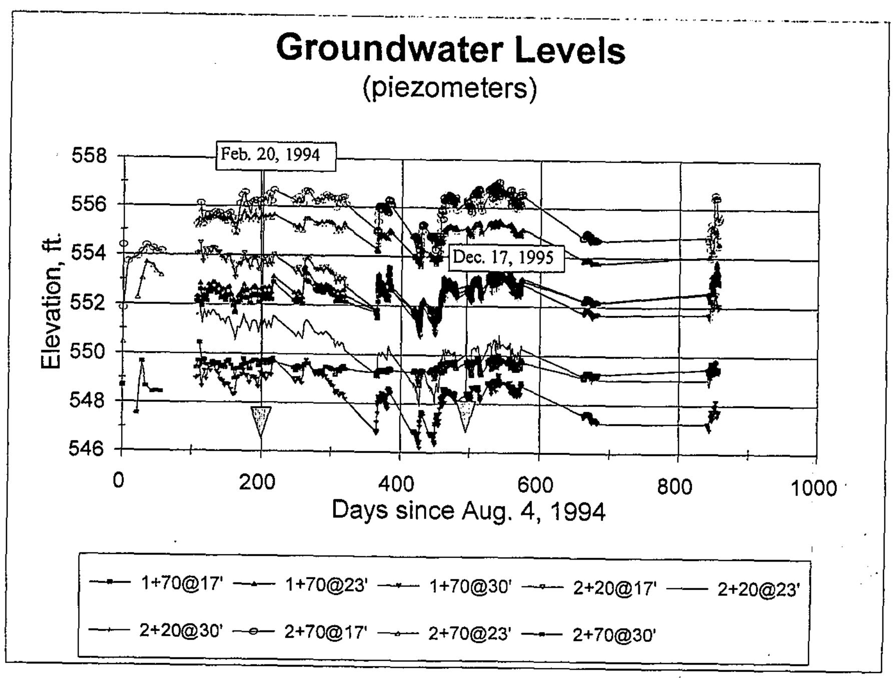



Figure 39: Piezometer Data

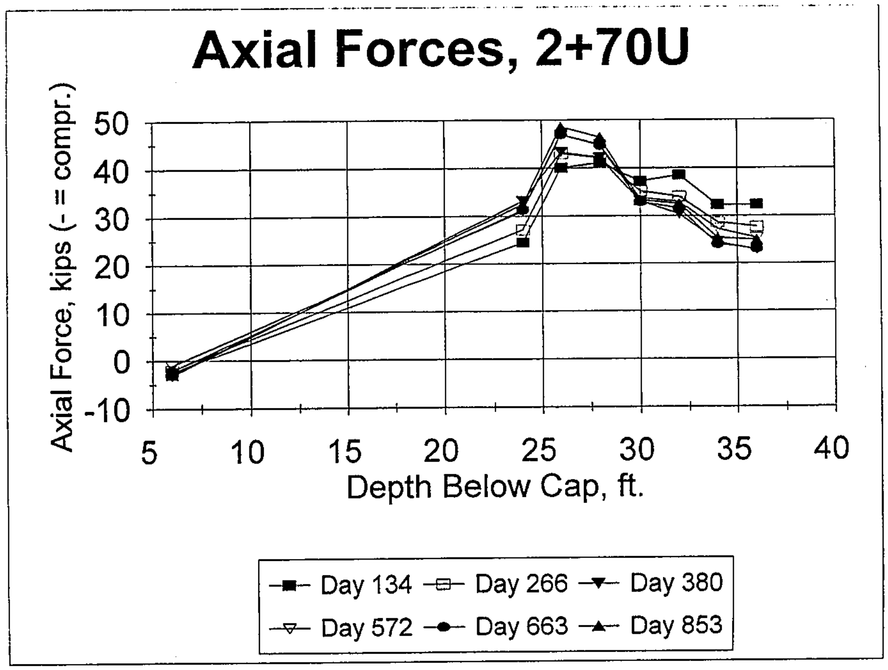



Figure 40: Axial forces in the uphill pile at Sta. 2+70

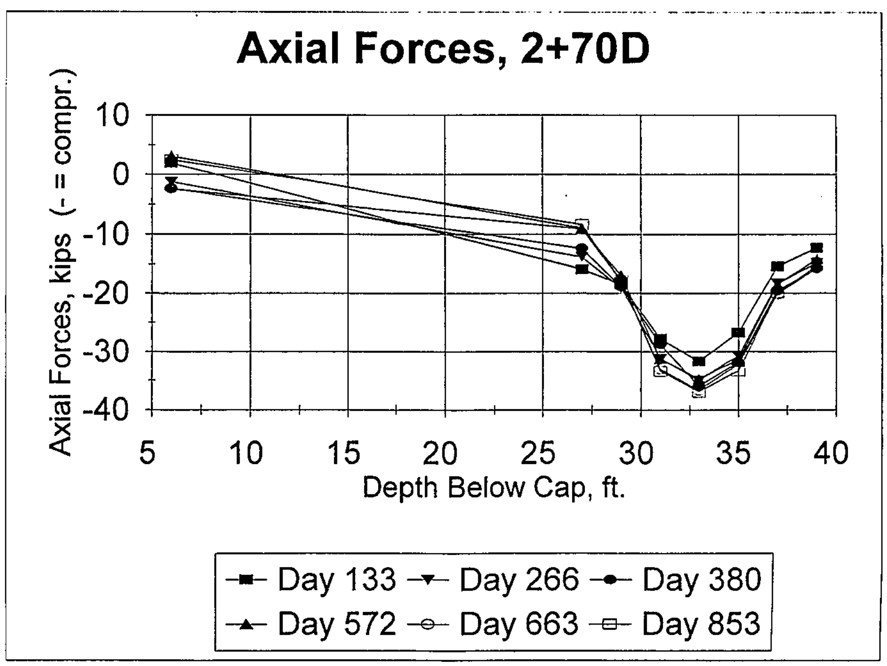



Figure 41: Axial forces in the downhill pile at Sta. 2+70

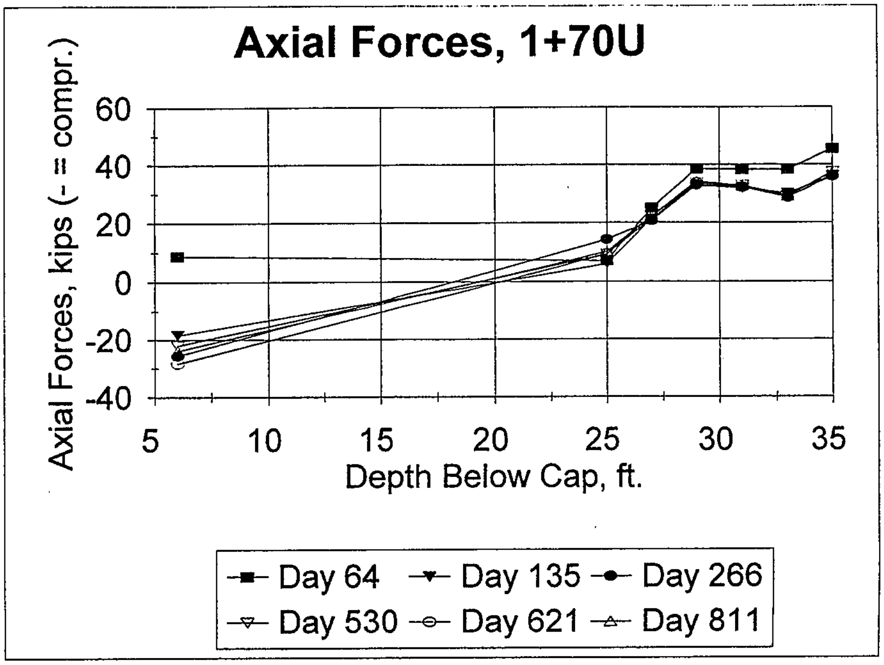



Figure 42: Axial forces in the uphill pile at Sta. 1+70

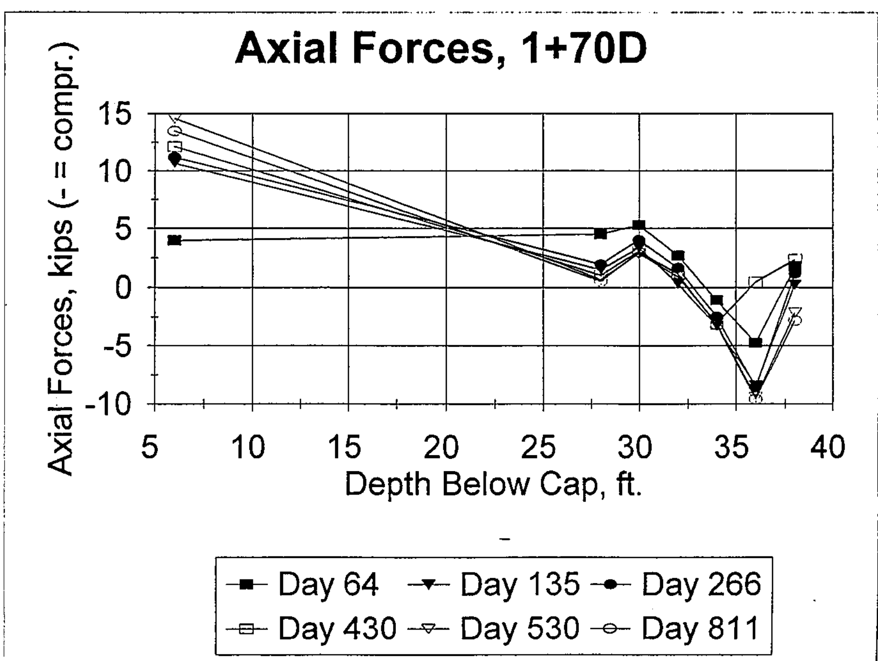



Figure 43: Axial forces in the downhill pile at Sta. 1+70

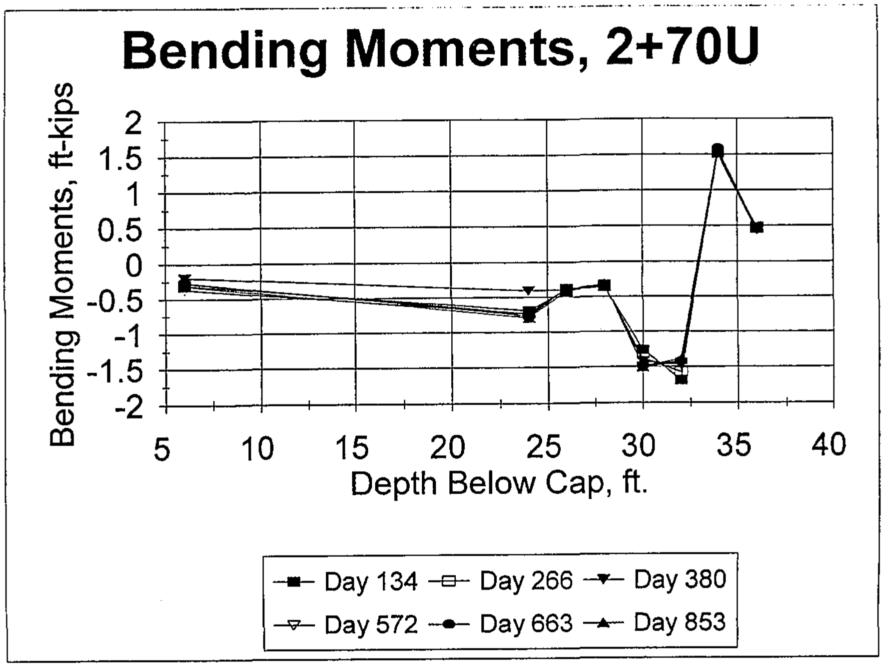



Figure 44: Bending Moments in the uphill pile at Sta. 2+70

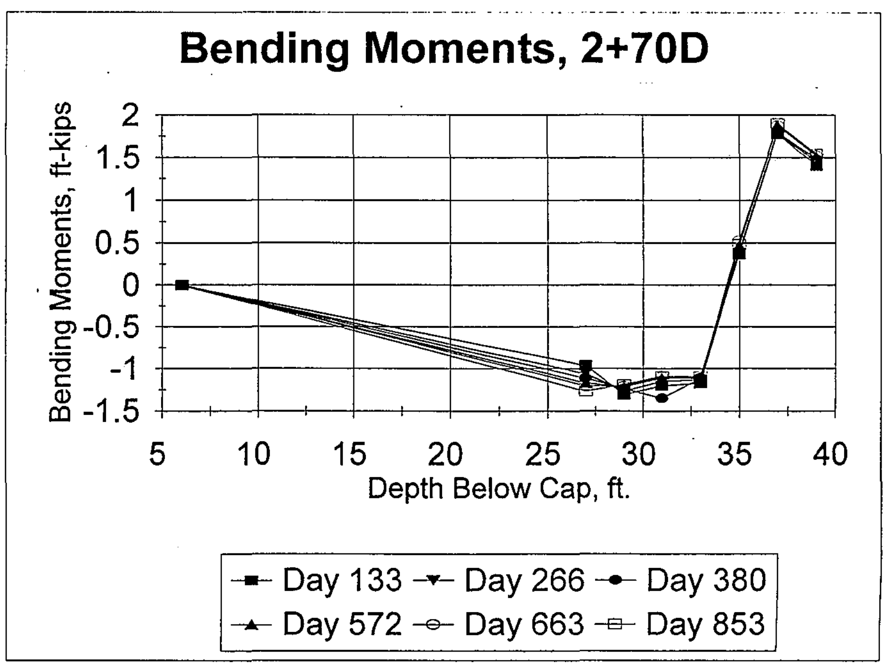



Figure 45: Bending Moments in the downhill pile at Sta. 2+70

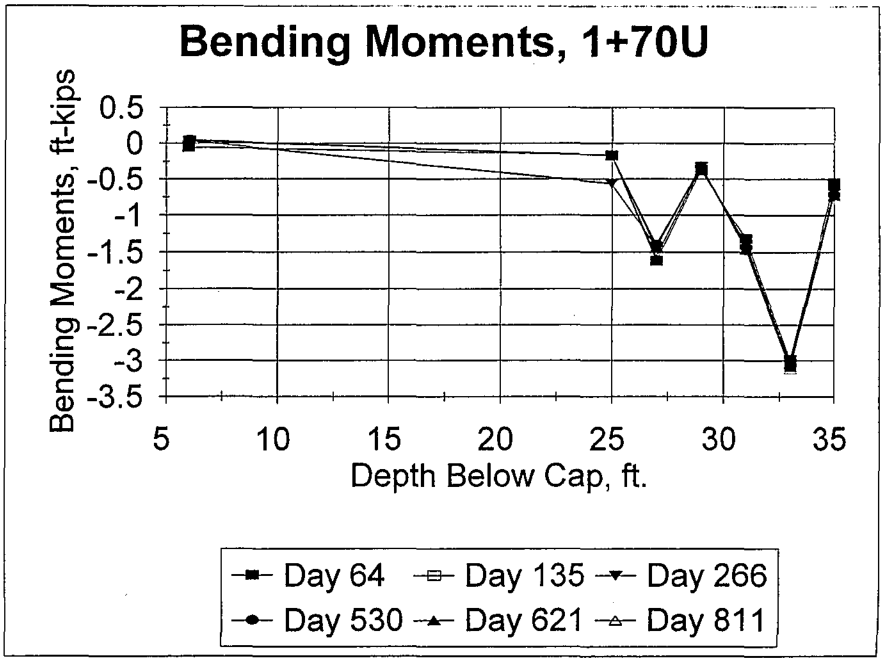



Figure 46: Bending Moments in the uphill pile at Sta. 1+70

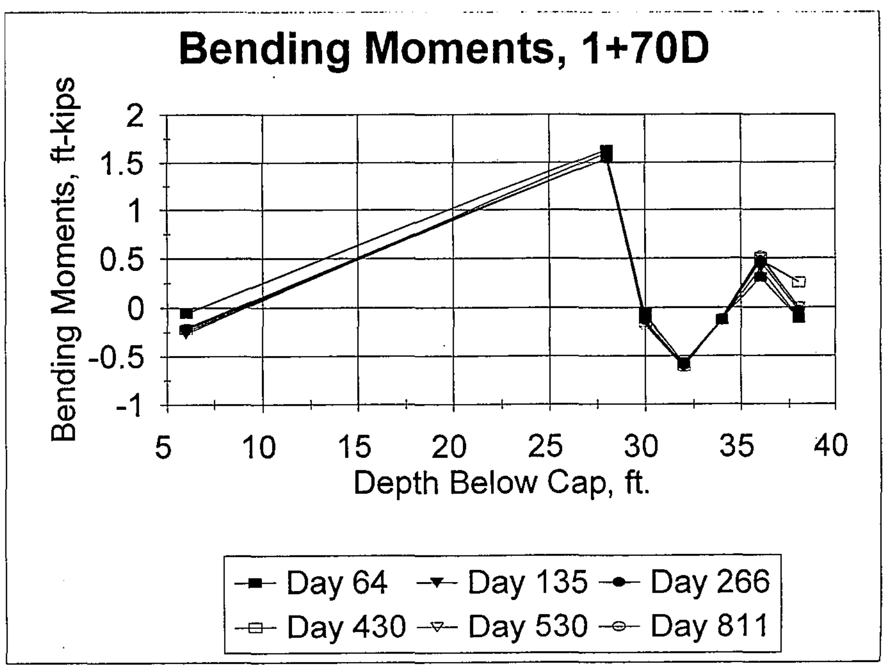



Figure 47: Bending Moments in the downhill pile at Sta. 1+70

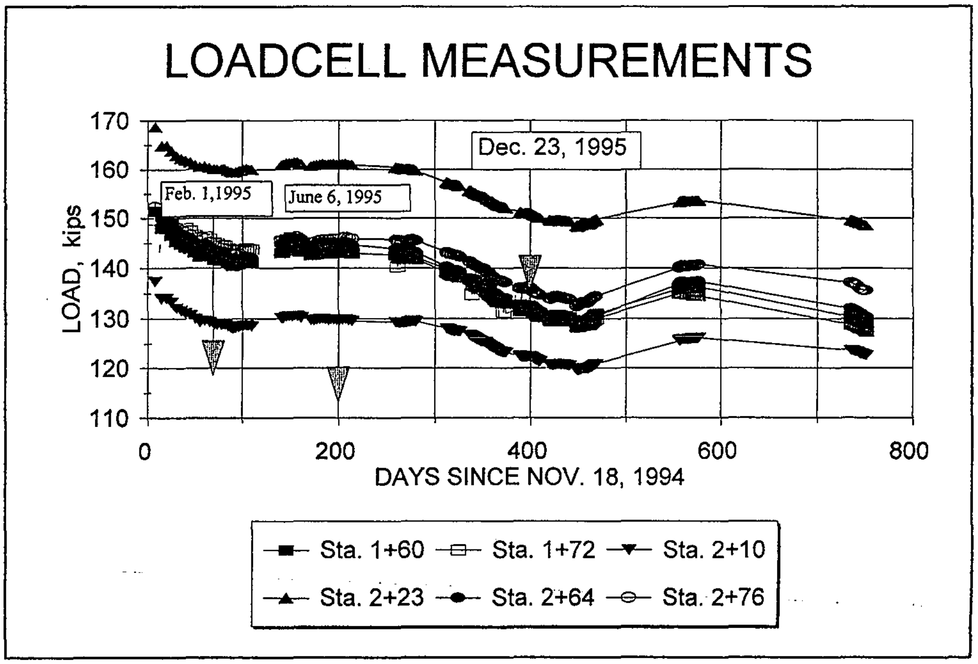



Figure 48: Loads measured in the ground anchors

## 8.0 SUMMARY AND CONCLUSIONS

For a period of over two years after construction of the INSERT wall in Littleville, AL, data were obtained on the movement of the slope, pore water pressures, and stresses developed in the piles and anchors. Most of the data were helpful in understanding the behavior of this structure, although some measurements appeared to be confusing or erratic such as the strain gage data at station 1+70. The inclinometer data in particular have shown that the structure has halted the movement of the slope (through two wet winters) as was intended. Analyses of the strain gage data imply that this was accomplished without approaching potential failure conditions in the piles and without further stressing of the ground anchors. The forces mobilized in the piles were found to be below the estimated ultimate loads by a factor of approximately two, and the ultimate factor of safety was calculated to be approximately 1.3 as originally estimated. The values of axial and shear force found by strain gages and interpreted using COM624 suggest a ratio of mobilized axial to shear force of 16:1 in the uphill pile and 14:1 in the downhill pile. The original design is based on an axial to shear force ratio at failure of 3.8:1, much lower than the presently mobilized ratios. These figures imply that the stabilizing effect from the piles is more closely related to axial forces and the batter angle than was expected in the original design procedure. Anchor loads have diminished slightly from the lock-off load due to creep uphill, movements which have also been measured in the inclinometer data. The piezometers have shown relatively stable water levels over the two plus years with a fluctuation of approximately two to three feet. The measured direct shear values for peak and residual shear strength appear reasonably close to those back-calculated from PCSTABL5, although the direct shear test measurements are somewhat variable due to the highly fissured structure of the weathered shale as observed during sample excavation in the field.

## 8.1 Needed Research

The data and conclusions presented in this report provide a basis for understanding the use of small diameter piles in slope remediation, especially with respect to the Nicholson INSERT Type-A process. More data are needed to allow generalizations, especially data on pile forces in different soil types, data without complications due to grouting in the rock fill and its possible effect on overall drainage and stability, and data gathered where no anchors are used so that the amount of movement needed to stabilize the slope can actually be measured. Further investigation is warranted in this developing field of slope remediation.

# REFERENCES

Brown, D. and J. Wolosick. "Instrumentation and Measurements of a Type-A INSERT Wall". ASCE Civil Engineering. July 1995, pp. 72-3.

Campbell, B.D. Design Sketches and Calculation Brief Final Design Services for Proposed Type "A" INSERT Wall. Monroeville, Penn.: D'Appolonia, 1993.

Poulos, H.G. and E.H. Davis. Pile Foundation Analysis and Design. New York: John Wiley and Sons, 1980.

Reese, L.C. and W.R. Sullivan. "Documentation for Computer Program COM624, Parts I and II, Analysis of Stresses and Deflections for Laterally Loaded Piles, Including Generation of py Curves". 1983.

Wolosick, J. "Nicholson Type 'A' INSERT (SM) Wall - Plan View, Wall Section, and Notes". Alpharetta, GA: Nicholson Construction Geotechnical Specialists, January 1994.

Xanthakos, P.P., L.W. Abramson, and D.A. Bruce. Ground Control and Improvement. New York: John Wiley and Sons, 1994.

# APPENDIX A

## Calibration Factors for Instrumentation

Piezometers

| | |
|---|---|
| 1+70-PZ17 | 0.01879 (PSI/Digit) |
| 1+70-PZ23 | 0.01910 |
| 1+70-PZ30 | 0.01876 |
| 2+20-PZ17 | 0.01802 |
| 2+20-PZ23 | 0.01913 |
| 2+20-PZ30 | 0.01839 |
| 2+70-PZ17 | 0.01931 |
| 2+70-PZ23 | 0.01900 |
| 2+70-PZ30 | 0.01945 |

Settlement Indicators

| | |
|---|---|
| 1+70 | 0.189189 (inches/Digit) |
| 2+20 | 0.190476 |
| 2+70 | 0.189189 |

Tiltmeters

| | |
|---|---|
| 1+70 | 0.00003161 |
| 2+20 | 0.00003314 |
| 2+70 | 0.00003302 |

Loadcells

| | |
|---|---|
| All | -0.0429 |

APPENDIX B

COM624 and PCSTABL5M Outputs for
Presently Carried Loads

```
Landslide on US 43, Littleville, 2+70u, top, Present Loads


UNITS--ENGL

                    I N P U T   I N F O R M A T I O N
                    *********************************


THE LOADING IS STATIC


PILE GEOMETRY AND PROPERTIES

    PILE LENGTH                         =     120.00 IN
    MODULUS OF ELASTICITY OF PILE       =   .100E+07 LBS/IN**2
                1 SECTION(S)

                  X           DIAMETER    MOMENT OF        AREA
                                           INERTIA
                 IN              IN         IN**4          IN**2
                .00
                               6.000       .231E+03       .283E+02
              120.00


SOILS INFORMATION

    X-COORDINATE AT THE GROUND SURFACE     =         .00 IN

    SLOPE ANGLE AT THE GROUND SURFACE      =         .00 DEG.

    1 LAYER(S) OF SOIL

    LAYER  1
    THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
    X AT THE TOP OF THE LAYER         =          .00 IN
    X AT THE BOTTOM OF THE LAYER      =       120.00 IN
    VARIATION OF SOIL MODULUS, k      =     .100E+04 LBS/IN**3


    DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
                       2 POINTS
               X,IN     WEIGHT,LBS/IN**3
                .00         .72E-01
             120.00         .72E-01

    DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
                       2 POINTS
               X,IN        C,LBS/IN**2    PHI,DEGREES     E50
                .00         .150E+02          .000      .100E-01
             120.00         .150E+02          .000      .100E-01


FINITE DIFFERENCE PARAMETERS
    NUMBER OF PILE INCREMENTS                              =         40
    TOLERANCE ON DETERMINATION OF DEFLECTIONS              =   .100E-03 IN
    MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =        100
    MAXIMUM ALLOWABLE DEFLECTION                           =    .60E+02 IN

INPUT CODES
    OUTPT  =  1
    KCYCL  =  1
    KBC    =  1
    KPYOP  =  1
    INC    =  2
```

```
Landslide on US 43, Littleville, 2+70u, top

 UNITS--ENGL

                   O U T P U T   I N F O R M A T I O N
                   ***********************************

 GENERATED P-Y CURVES

     THE NUMBER OF CURVE IS                         =  3

     THE NUMBER OF POINTS ON EACH CURVE             = 17

      DEPTH BELOW GS      DIAM        C        CAVG       GAMMA        E50
            IN             IN     LBS/IN**2 LBS/IN**2 LBS/IN**3
           .00            6.000     .2E+02    .2E+02     .7E-01     .100E-01
                                  Y                   P
                                  IN               LBS/IN
                                 .000                .000
                                 .160             137.196
                                 .320             163.154
                                 .480             180.560
                                 .640             194.024
                                 .800             205.156
                                 .960             214.723
                                1.120             223.160
                                1.280             230.735
                                1.440             237.630
                                1.600             243.973
                                1.760             249.856
                                1.920             255.350
                                2.080             260.511
                                2.240             265.383
                                2.400             270.000
                                3.000             270.000

      DEPTH BELOW GS      DIAM        C        CAVG       GAMMA        E50
            IN             IN     LBS/IN**2 LBS/IN**2 LBS/IN**3
          60.00           6.000     .2E+02    .2E+02     .7E-01     .100E-01
                                  Y                   P
                                  IN               LBS/IN
                                 .000                .000
                                 .160             379.026
                                 .320             450.741
                                 .480             498.827
                                 .640             536.024
                                 .800             566.777
                                 .960             593.208
                                1.120             616.515
                                1.280             637.444
                                1.440             656.493
                                1.600             674.015
                                1.760             690.268
                                1.920             705.448
                                2.080             719.706
                                2.240             733.165
                                2.400             745.920
                                3.000             745.920

      DEPTH BELOW GS      DIAM        C        CAVG       GAMMA        E50
            IN             IN     LBS/IN**2 LBS/IN**2 LBS/IN**3
         120.00           6.000     .2E+02    .2E+02     .7E-01     .100E-01
                                  Y                   P
                                  IN               LBS/IN
                                 .000                .000
                                 .160             411.588
```

| | |
|---|---|
| .320 | 489.463 |
| .480 | 541.680 |
| .640 | 582.073 |
| .800 | 615.467 |
| .960 | 644.169 |
| 1.120 | 669.479 |
| 1.280 | 692.205 |
| 1.440 | 712.891 |
| 1.600 | 731.918 |
| 1.760 | 749.567 |
| 1.920 | 766.051 |
| 2.080 | 781.534 |
| 2.240 | 796.149 |
| 2.400 | 810.000 |
| 3.000 | 810.000 |

---------- *** ----------

PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .268E+04 LBS
APPLIED MOMENT AT PILE HEAD = -.938E+04 LBS-IN
AXIAL LOAD AT PILE HEAD = -.430E+05 LBS

| X | DEFLECTION | MOMENT | TOTAL STRESS | SHEAR | SOIL RESIST | FLEXURAL RIGIDITY |
|---|---|---|---|---|---|---|
| IN | IN | LBS-IN | LBS/IN**2 | LBS | LBS/IN | LBS-IN**2 |
| ***** | ********** | ********** | ********** | ********** | ********** | ********** |
| .00 | .542E-01 | -.937E+04 | .164E+04 | .258E+04 | .105E+03 | .231E+09 |
| 6.00 | .403E-01 | .415E+04 | .157E+04 | .202E+04 | .114E+03 | .231E+09 |
| 12.00 | .271E-01 | .136E+05 | .170E+04 | .132E+04 | .119E+03 | .231E+09 |
| 18.00 | .160E-01 | .189E+05 | .176E+04 | .603E+03 | .118E+03 | .231E+09 |
| 24.00 | .775E-02 | .200E+05 | .178E+04 | -.824E+02 | .110E+03 | .231E+09 |
| 30.00 | .261E-02 | .174E+05 | .175E+04 | -.692E+03 | .922E+02 | .231E+09 |
| 36.00 | .153E-03 | .116E+05 | .167E+04 | -.114E+04 | .493E+02 | .231E+09 |
| 42.00 | -.500E-03 | .486E+04 | .158E+04 | -.923E+03 | -.727E+02 | .231E+09 |
| 48.00 | -.369E-03 | .634E+03 | .153E+04 | -.487E+03 | -.728E+02 | .231E+09 |
| 54.00 | -.113E-03 | -.105E+04 | .153E+04 | -.983E+02 | -.584E+02 | .231E+09 |
| 60.00 | -.150E-05 | -.730E+03 | .153E+04 | .142E+03 | -.226E+02 | .231E+09 |
| 66.00 | .219E-05 | .142E+02 | .152E+04 | .466E+02 | .250E+02 | .231E+09 |
| 72.00 | -.452E-08 | .326E+01 | .152E+04 | -.835E+01 | -.552E+01 | .231E+09 |
| 78.00 | .624E-15 | -.719E-04 | .152E+04 | .193E-01 | .147E-01 | .231E+09 |
| 84.00 | -.861E-22 | .996E-11 | .152E+04 | -.266E-08 | -.202E-08 | .231E+09 |
| 90.00 | .119E-28 | -.138E-17 | .152E+04 | .367E-15 | .277E-15 | .231E+09 |
| 96.00 | -.164E-35 | .191E-24 | .152E+04 | -.506E-22 | -.381E-22 | .231E+09 |
| 102.00 | .226E-42 | -.264E-31 | .152E+04 | -.881E-32 | .523E-29 | .231E+09 |
| 108.00 | -.312E-49 | .366E-38 | .152E+04 | .122E-38 | -.719E-36 | .231E+09 |
| 114.00 | .430E-56 | -.506E-45 | .152E+04 | -.169E-45 | .989E-43 | .231E+09 |
| 120.00 | -.119E-62 | .000E+00 | .152E+04 | .000E+00 | -.273E-49 | .231E+09 |

COMPUTED LATERAL FORCE AT PILE HEAD = .26750E+04 LBS
COMPUTED MOMENT AT PILE HEAD = -.93750E+04 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -.22299E-02

THE OVERALL MOMENT IMBALANCE = -.167E-08 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = .249E-10 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .542E-01 IN
MAXIMUM BENDING MOMENT = .200E+05 IN-LBS
MAXIMUM TOTAL STRESS = .178E+04 LBS/IN**2
MAXIMUM SHEAR FORCE = .313E+04 LBS
NO. OF ITERATIONS = 12
MAXIMUM DEFLECTION ERROR = .588E-04 IN

S U M M A R Y   T A B L E
*************************

| LATERAL LOAD (LBS) | BOUNDARY CONDITION BC2 | AXIAL LOAD (LBS) | YT (IN) | ST (IN/IN) | MAX. MOMENT (IN-LBS) | MAX. STRESS (LBS/IN**2) |
|---|---|---|---|---|---|---|
| .268E+04 | -.938E+04 | -.430E+05 | .542E-01 | -.223E-02 | .200E+05 | .178E+04 |

Landslide on US 43, Littleville, 2+70u, bottom, Present Loads

UNITS--ENGL

I N P U T   I N F O R M A T I O N
*********************************

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 120.00 IN
MODULUS OF ELASTICITY OF PILE = .100E+07 LBS/IN**2
1 SECTION(S)

| X | DIAMETER | MOMENT OF INERTIA | AREA |
|---|---|---|---|
| IN | IN | IN**4 | IN**2 |
| .00 | | | |
| | 6.000 | .231E+03 | .283E+02 |
| 120.00 | | | |

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = .00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
X AT THE TOP OF THE LAYER = .00 IN
X AT THE BOTTOM OF THE LAYER = 120.00 IN
VARIATION OF SOIL MODULUS, k = .100E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

| X,IN | WEIGHT,LBS/IN**3 |
|---|---|
| .00 | .72E-01 |
| 120.00 | .72E-01 |

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

| X,IN | C,LBS/IN**2 | PHI,DEGREES | E50 |
|---|---|---|---|
| .00 | .365E+02 | .000 | .100E-01 |
| 120.00 | .365E+02 | .000 | .100E-01 |

FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 40
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .60E+02 IN

INPUT CODES
OUTPT = 1
KCYCL = 1
KBC = 1
KPYOP = 1
INC = 2

Landslide on US 43, Littleville, 2+70u, bot

UNITS--ENGL

OUTPUT INFORMATION

GENERATED P-Y CURVES

THE NUMBER OF CURVE IS = 3

THE NUMBER OF POINTS ON EACH CURVE = 17

| DEPTH BELOW GS | DIAM | C | CAVG | GAMMA | E50 |
|---|---|---|---|---|---|
| IN | IN | LBS/IN**2 | LBS/IN**2 | LBS/IN**3 | |
| .00 | 6.000 | .4E+02 | .4E+02 | .7E-01 | .100E-01 |

| Y<br>IN | P<br>LBS/IN |
|---|---|
| .000 | .000 |
| .160 | 333.843 |
| .320 | 397.009 |
| .480 | 439.362 |
| .640 | 472.126 |
| .800 | 499.212 |
| .960 | 522.493 |
| 1.120 | 543.022 |
| 1.280 | 561.455 |
| 1.440 | 578.233 |
| 1.600 | 593.667 |
| 1.760 | 607.982 |
| 1.920 | 621.352 |
| 2.080 | 633.911 |
| 2.240 | 645.765 |
| 2.400 | 657.000 |
| 3.000 | 657.000 |

| DEPTH BELOW GS | DIAM | C | CAVG | GAMMA | E50 |
|---|---|---|---|---|---|
| IN | IN | LBS/IN**2 | LBS/IN**2 | LBS/IN**3 | |
| 60.00 | 6.000 | .4E+02 | .4E+02 | .7E-01 | .100E-01 |

| Y<br>IN | P<br>LBS/IN |
|---|---|
| .000 | .000 |
| .160 | 903.419 |
| .320 | 1074.353 |
| .480 | 1188.967 |
| .640 | 1277.628 |
| .800 | 1350.927 |
| .960 | 1413.928 |
| 1.120 | 1469.481 |
| 1.280 | 1519.364 |
| 1.440 | 1564.768 |
| 1.600 | 1606.532 |
| 1.760 | 1645.271 |
| 1.920 | 1681.453 |
| 2.080 | 1715.439 |
| 2.240 | 1747.517 |
| 2.400 | 1777.920 |
| 3.000 | 1777.920 |

| DEPTH BELOW GS | DIAM | C | CAVG | GAMMA | E50 |
|---|---|---|---|---|---|
| IN | IN | LBS/IN**2 | LBS/IN**2 | LBS/IN**3 | |
| 120.00 | 6.000 | .4E+02 | .4E+02 | .7E-01 | .100E-01 |

| Y<br>IN | P<br>LBS/IN |
|---|---|
| .000 | .000 |

| | |
|---|---|
| .160 | 1001.530 |
| .320 | 1191.026 |
| .480 | 1318.087 |
| .640 | 1416.377 |
| .800 | 1497.636 |
| .960 | 1567.479 |
| 1.120 | 1629.065 |
| 1.280 | 1684.365 |
| 1.440 | 1734.700 |
| 1.600 | 1781.000 |
| 1.760 | 1823.946 |
| 1.920 | 1864.057 |
| 2.080 | 1901.733 |
| 2.240 | 1937.295 |
| 2.400 | 1971.000 |
| 3.000 | 1971.000 |

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .268E+04 LBS
APPLIED MOMENT AT PILE HEAD = .938E+04 LBS-IN
AXIAL LOAD AT PILE HEAD = -.430E+05 LBS

| X IN | DEFLECTION IN | MOMENT LBS-IN | TOTAL STRESS LBS/IN**2 | SHEAR LBS | SOIL RESIST LBS/IN | FLEXURAL RIGIDITY LBS-IN**2 |
|---|---|---|---|---|---|---|
| ***** | ********** | ********** | ********** | ********** | ********** | ********** |
| .00 | .292E-01 | .938E+04 | .164E+04 | .258E+04 | .218E+03 | .231E+09 |
| 6.00 | .168E-01 | .209E+05 | .179E+04 | .135E+04 | .223E+03 | .231E+09 |
| 12.00 | .759E-02 | .247E+05 | .184E+04 | .527E+02 | .209E+03 | .231E+09 |
| 18.00 | .217E-02 | .212E+05 | .179E+04 | -.110E+04 | .172E+03 | .231E+09 |
| 24.00 | -.813E-05 | .117E+05 | .167E+04 | -.169E+04 | -.456E+02 | .231E+09 |
| 30.00 | -.336E-03 | .314E+04 | .156E+04 | -.104E+04 | -.133E+03 | .231E+09 |
| 36.00 | -.126E-03 | -.799E+03 | .153E+04 | -.304E+03 | -.114E+03 | .231E+09 |
| 42.00 | -.254E-05 | -.855E+03 | .153E+04 | .172E+03 | -.500E+02 | .231E+09 |
| 48.00 | .301E-06 | .430E+02 | .152E+04 | .303E+02 | .297E+02 | .231E+09 |
| 54.00 | -.153E-11 | -.101E-02 | .152E+04 | -.129E+01 | -.179E+01 | .231E+09 |
| 60.00 | .963E-19 | .559E-10 | .152E+04 | .653E-05 | .907E-05 | .231E+09 |
| 66.00 | -.571E-26 | -.282E-17 | .152E+04 | -.411E-12 | -.571E-12 | .231E+09 |
| 72.00 | .325E-33 | .129E-24 | .152E+04 | .244E-19 | .339E-19 | .231E+09 |
| 78.00 | -.184E-40 | -.544E-32 | .152E+04 | -.213E-32 | -.193E-26 | .231E+09 |
| 84.00 | .105E-47 | .202E-39 | .152E+04 | .852E-40 | .109E-33 | .231E+09 |
| 90.00 | -.596E-55 | -.536E-47 | .152E+04 | -.281E-47 | -.622E-41 | .231E+09 |
| 96.00 | .339E-62 | -.428E-55 | .152E+04 | .436E-55 | .354E-48 | .231E+09 |
| 102.00 | -.193E-69 | .222E-61 | .152E+04 | .411E-62 | -.201E-55 | .231E+09 |
| 108.00 | .109E-76 | -.238E-68 | .152E+04 | -.608E-69 | .114E-62 | .231E+09 |
| 114.00 | -.622E-84 | .199E-75 | .152E+04 | .558E-76 | -.649E-70 | .231E+09 |
| 120.00 | .707E-91 | .000E+00 | .152E+04 | .000E+00 | .738E-77 | .231E+09 |

COMPUTED LATERAL FORCE AT PILE HEAD = .26750E+04 LBS
COMPUTED MOMENT AT PILE HEAD = .93750E+04 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -.22320E-02

THE OVERALL MOMENT IMBALANCE = .392E-09 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -.405E-10 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .292E-01 IN
MAXIMUM BENDING MOMENT = .247E+05 IN-LBS
MAXIMUM TOTAL STRESS = .184E+04 LBS/IN**2
MAXIMUM SHEAR FORCE = .258E+04 LBS

NO. OF ITERATIONS = 13
MAXIMUM DEFLECTION ERROR = .809E-04 IN

S U M M A R Y T A B L E
************************

| LATERAL LOAD (LBS) | BOUNDARY CONDITION BC2 | AXIAL LOAD (LBS) | YT (IN) | ST (IN/IN) | MAX. MOMENT (IN-LBS) | MAX. STRESS (LBS/IN**2) |
|---|---|---|---|---|---|---|
| .268E+04 | .938E+04 | -.430E+05 | .292E-01 | -.223E-02 | .247E+05 | .184E+04 |

Landslide on US 43, Littleville, 2+70d, top, Present Loads

UNITS--ENGL

I N P U T   I N F O R M A T I O N
*********************************

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 120.00 IN
MODULUS OF ELASTICITY OF PILE = .100E+07 LBS/IN**2
1 SECTION(S)

| X IN | DIAMETER IN | MOMENT OF INERTIA IN**4 | AREA IN**2 |
|---|---|---|---|
| .00 | | | |
| | 6.000 | .231E+03 | .283E+02 |
| 120.00 | | | |

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = .00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
X AT THE TOP OF THE LAYER = .00 IN
X AT THE BOTTOM OF THE LAYER = 120.00 IN
VARIATION OF SOIL MODULUS, k = .100E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

| X,IN | WEIGHT,LBS/IN**3 |
|---|---|
| .00 | .72E-01 |
| 120.00 | .72E-01 |

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

| X,IN | C,LBS/IN**2 | PHI,DEGREES | E50 |
|---|---|---|---|
| .00 | .150E+02 | .000 | .100E-01 |
| 120.00 | .150E+02 | .000 | .100E-01 |

FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 40
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-03 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .60E+02 IN

INPUT CODES
OUTPT = 1

```
KCYCL  =  1
KBC    =  1
KPYOP  =  1
INC    =  2


Landslide on US 43, Littleville, 2+70d, top

UNITS--ENGL


                 O U T P U T   I N F O R M A T I O N
                 ***********************************


GENERATED P-Y CURVES

    THE NUMBER OF CURVE IS                         =  3

    THE NUMBER OF POINTS ON EACH CURVE             = 17

     DEPTH BELOW GS      DIAM       C        CAVG      GAMMA         E50
           IN             IN    LBS/IN**2 LBS/IN**2 LBS/IN**3
           .00           6.000   .2E+02    .2E+02    .7E-01     .100E-01
                                  Y                   P
                                  IN                LBS/IN
                                 .000                 .000
                                 .160              137.196
                                 .320              163.154
                                 .480              180.560
                                 .640              194.024
                                 .800              205.156
                                 .960              214.723
                                1.120              223.160
                                1.280              230.735
                                1.440              237.630
                                1.600              243.973
                                1.760              249.856
                                1.920              255.350
                                2.080              260.511
                                2.240              265.383
                                2.400              270.000
                                3.000              270.000

     DEPTH BELOW GS      DIAM       C        CAVG      GAMMA         E50
           IN             IN    LBS/IN**2 LBS/IN**2 LBS/IN**3
         60.00           6.000   .2E+02    .2E+02    .7E-01     .100E-01
                                  Y                   P
                                  IN                LBS/IN
                                 .000                 .000
                                 .160              379.026
                                 .320              450.741
                                 .480              498.827
                                 .640              536.024
                                 .800              566.777
                                 .960              593.208
                                1.120              616.515
                                1.280              637.444
                                1.440              656.493
                                1.600              674.015
                                1.760              690.268
                                1.920              705.448
                                2.080              719.706
                                2.240              733.165
                                2.400              745.920
                                3.000              745.920

     DEPTH BELOW GS      DIAM       C        CAVG      GAMMA         E50
```

| | | | | | |
|---|---|---|---|---|---|
| IN | IN | LBS/IN**2 | LBS/IN**2 | LBS/IN**3 | |
| 120.00 | 6.000 | .2E+02 | .2E+02 | .7E-01 | .100E-01 |

| Y | P |
|---|---|
| IN | LBS/IN |
| .000 | .000 |
| .160 | 411.588 |
| .320 | 489.463 |
| .480 | 541.680 |
| .640 | 582.073 |
| .800 | 615.467 |
| .960 | 644.169 |
| 1.120 | 669.479 |
| 1.280 | 692.205 |
| 1.440 | 712.891 |
| 1.600 | 731.918 |
| 1.760 | 749.567 |
| 1.920 | 766.051 |
| 2.080 | 781.534 |
| 2.240 | 796.149 |
| 2.400 | 810.000 |
| 3.000 | 810.000 |

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .252E+04 LBS
APPLIED MOMENT AT PILE HEAD = -.938E+04 LBS-IN
AXIAL LOAD AT PILE HEAD = .350E+05 LBS

| X | DEFLECTION | MOMENT | TOTAL STRESS | SHEAR | SOIL RESIST | FLEXURAL RIGIDITY |
|---|---|---|---|---|---|---|
| IN | IN | LBS-IN | LBS/IN**2 | LBS | LBS/IN | LBS-IN**2 |
| ***** | ********** | ********** | ********** | ********** | ********** | ********** |
| .00 | .531E-01 | -.937E+04 | .136E+04 | .260E+04 | .104E+03 | .231E+09 |
| 6.00 | .393E-01 | .431E+04 | .129E+04 | .187E+04 | .114E+03 | .231E+09 |
| 12.00 | .262E-01 | .139E+05 | .142E+04 | .117E+04 | .118E+03 | .231E+09 |
| 18.00 | .152E-01 | .192E+05 | .149E+04 | .465E+03 | .117E+03 | .231E+09 |
| 24.00 | .719E-02 | .202E+05 | .150E+04 | -.209E+03 | .108E+03 | .231E+09 |
| 30.00 | .227E-02 | .172E+05 | .146E+04 | -.802E+03 | .892E+02 | .231E+09 |
| 36.00 | .505E-05 | .111E+05 | .138E+04 | -.118E+04 | .226E+02 | .231E+09 |
| 42.00 | -.538E-03 | .444E+04 | .129E+04 | -.896E+03 | -.739E+02 | .231E+09 |
| 48.00 | -.364E-03 | .383E+03 | .124E+04 | -.455E+03 | -.724E+02 | .231E+09 |
| 54.00 | -.104E-03 | -.113E+04 | .125E+04 | -.687E+02 | -.569E+02 | .231E+09 |
| 60.00 | -.934E-07 | -.663E+03 | .125E+04 | .149E+03 | -.132E+02 | .231E+09 |
| 66.00 | .188E-05 | .249E+02 | .124E+04 | .398E+02 | .242E+02 | .231E+09 |
| 72.00 | -.418E-08 | .165E+01 | .124E+04 | -.755E+01 | -.545E+01 | .231E+09 |
| 78.00 | .578E-15 | -.133E-04 | .124E+04 | .178E-01 | .139E-01 | .231E+09 |
| 84.00 | -.799E-22 | .185E-11 | .124E+04 | -.247E-08 | -.192E-08 | .231E+09 |
| 90.00 | .110E-28 | -.257E-18 | .124E+04 | .341E-15 | .266E-15 | .231E+09 |
| 96.00 | -.153E-35 | .357E-25 | .124E+04 | -.471E-22 | -.367E-22 | .231E+09 |
| 102.00 | .211E-42 | -.497E-32 | .124E+04 | -.165E-32 | .507E-29 | .231E+09 |
| 108.00 | -.292E-49 | .690E-39 | .124E+04 | .229E-39 | -.700E-36 | .231E+09 |
| 114.00 | .403E-56 | -.958E-46 | .124E+04 | -.318E-46 | .966E-43 | .231E+09 |
| 120.00 | -.112E-62 | .000E+00 | .124E+04 | .000E+00 | -.267E-49 | .231E+09 |

COMPUTED LATERAL FORCE AT PILE HEAD = .25200E+04 LBS
COMPUTED MOMENT AT PILE HEAD = -.93750E+04 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -.22168E-02

THE OVERALL MOMENT IMBALANCE = .757E-09 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -.204E-09 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .531E-01 IN
MAXIMUM BENDING MOMENT = .202E+05 IN-LBS
MAXIMUM TOTAL STRESS = .150E+04 LBS/IN**2
MAXIMUM SHEAR FORCE = .317E+04 LBS

NO. OF ITERATIONS = 12

MAXIMUM DEFLECTION ERROR = .918E-04 IN

S U M M A R Y   T A B L E
*************************

| LATERAL LOAD (LBS) | BOUNDARY CONDITION BC2 | AXIAL LOAD (LBS) | YT (IN) | ST (IN/IN) | MAX. MOMENT (IN-LBS) | MAX. STRESS (LBS/IN**2) |
|---|---|---|---|---|---|---|
| .252E+04 | -.938E+04 | .350E+05 | .531E-01 | -.222E-02 | .202E+05 | .150E+04 |

Landslide on US 43, Littleville, 2+70d, bottom, Present Loads

UNITS--ENGL

I N P U T   I N F O R M A T I O N
*********************************

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 120.00 IN
MODULUS OF ELASTICITY OF PILE = .100E+07 LBS/IN**2
1 SECTION(S)

| X IN | DIAMETER IN | MOMENT OF INERTIA IN**4 | AREA IN**2 |
|---|---|---|---|
| .00 | | | |
| | 6.000 | .231E+03 | .283E+02 |
| 120.00 | | | |

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = .00 IN

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

1 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
X AT THE TOP OF THE LAYER = .00 IN
X AT THE BOTTOM OF THE LAYER = 120.00 IN
VARIATION OF SOIL MODULUS, k = .100E+04 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
2 POINTS

| X,IN | WEIGHT,LBS/IN**3 |
|---|---|
| .00 | .72E-01 |
| 120.00 | .72E-01 |

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
2 POINTS

| X,IN | C,LBS/IN**2 | PHI,DEGREES | E50 |
|---|---|---|---|
| .00 | .365E+02 | .000 | .100E-01 |
| 120.00 | .365E+02 | .000 | .100E-01 |

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 FINITE DIFFERENCE PARAMETERS
     NUMBER OF PILE INCREMENTS                                =          40
     TOLERANCE ON DETERMINATION OF DEFLECTIONS                =     .100E-03 IN
     MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS =         100
     MAXIMUM ALLOWABLE DEFLECTION                             =     .60E+02 IN

 INPUT CODES
     OUTPT  =  1
     KCYCL  =  1
     KBC    =  1
     KPYOP  =  1
     INC    =  2

Landslide on US 43, Littleville, 2+70d, bot

 UNITS--ENGL

                       O U T P U T   I N F O R M A T I O N
                       ***********************************

 GENERATED P-Y CURVES

     THE NUMBER OF CURVE IS                          =  3

     THE NUMBER OF POINTS ON EACH CURVE              = 17

      DEPTH BELOW GS      DIAM        C        CAVG       GAMMA        E50
            IN            IN     LBS/IN**2 LBS/IN**2 LBS/IN**3
           .00           6.000     .4E+02    .4E+02     .7E-01     .100E-01
                                    Y                    P
                                    IN                 LBS/IN
                                   .000                 .000
                                   .160              333.843
                                   .320              397.009
                                   .480              439.362
                                   .640              472.126
                                   .800              499.212
                                   .960              522.493
                                  1.120              543.022
                                  1.280              561.455
                                  1.440              578.233
                                  1.600              593.667
                                  1.760              607.982
                                  1.920              621.352
                                  2.080              633.911
                                  2.240              645.765
                                  2.400              657.000
                                  3.000              657.000

      DEPTH BELOW GS      DIAM        C        CAVG       GAMMA        E50
            IN            IN     LBS/IN**2 LBS/IN**2 LBS/IN**3
         60.00           6.000     .4E+02    .4E+02     .7E-01     .100E-01
                                    Y                    P
                                    IN                 LBS/IN
                                   .000                 .000
                                   .160              903.419
                                   .320             1074.353
                                   .480             1188.967
                                   .640             1277.628
                                   .800             1350.927
                                   .960             1413.928
                                  1.120             1469.481
                                  1.280             1519.364
                                  1.440             1564.768
```

| | |
|---|---|
| 1.600 | 1606.532 |
| 1.760 | 1645.271 |
| 1.920 | 1681.453 |
| 2.080 | 1715.439 |
| 2.240 | 1747.517 |
| 2.400 | 1777.920 |
| 3.000 | 1777.920 |

| DEPTH BELOW GS | DIAM | C | CAVG | GAMMA | E50 |
|---|---|---|---|---|---|
| IN | IN | LBS/IN**2 | LBS/IN**2 | LBS/IN**3 | |
| 120.00 | 6.000 | .4E+02 | .4E+02 | .7E-01 | .100E-01 |

| Y | P |
|---|---|
| IN | LBS/IN |
| .000 | .000 |
| .160 | 1001.530 |
| .320 | 1191.026 |
| .480 | 1318.087 |
| .640 | 1416.377 |
| .800 | 1497.636 |
| .960 | 1567.479 |
| 1.120 | 1629.065 |
| 1.280 | 1684.365 |
| 1.440 | 1734.700 |
| 1.600 | 1781.000 |
| 1.760 | 1823.946 |
| 1.920 | 1864.057 |
| 2.080 | 1901.733 |
| 2.240 | 1937.295 |
| 2.400 | 1971.000 |
| 3.000 | 1971.000 |

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PILE LOADING CONDITION

LATERAL LOAD AT PILE HEAD = .252E+04 LBS
APPLIED MOMENT AT PILE HEAD = .938E+04 LBS-IN
AXIAL LOAD AT PILE HEAD = .350E+05 LBS

| X | DEFLECTION | MOMENT | TOTAL STRESS | SHEAR | SOIL RESIST | FLEXURAL RIGIDITY |
|---|---|---|---|---|---|---|
| IN | IN | LBS-IN | LBS/IN**2 | LBS | LBS/IN | LBS-IN**2 |
| ***** | ********** | ********** | ********** | ********** | ********** | ********** |
| .00 | .283E-01 | .937E+04 | .136E+04 | .260E+04 | .217E+03 | .231E+09 |
| 6.00 | .161E-01 | .210E+05 | .151E+04 | .121E+04 | .220E+03 | .231E+09 |
| 12.00 | .715E-02 | .246E+05 | .156E+04 | -.756E+02 | .206E+03 | .231E+09 |
| 18.00 | .195E-02 | .208E+05 | .151E+04 | -.120E+04 | .168E+03 | .231E+09 |
| 24.00 | -.609E-04 | .110E+05 | .138E+04 | -.171E+04 | -.746E+02 | .231E+09 |
| 30.00 | -.323E-03 | .274E+04 | .127E+04 | -.992E+03 | -.131E+03 | .231E+09 |
| 36.00 | -.110E-03 | -.905E+03 | .125E+04 | -.257E+03 | -.110E+03 | .231E+09 |
| 42.00 | -.532E-06 | -.740E+03 | .125E+04 | .179E+03 | -.370E+02 | .231E+09 |
| 48.00 | .137E-06 | .478E+02 | .124E+04 | .205E+02 | .242E+02 | .231E+09 |
| 54.00 | -.115E-12 | -.266E-02 | .124E+04 | -.586E+00 | -.980E+00 | .231E+09 |
| 60.00 | .723E-20 | .172E-09 | .124E+04 | .490E-06 | .817E-06 | .231E+09 |
| 66.00 | -.429E-27 | -.105E-16 | .124E+04 | -.309E-13 | -.515E-13 | .231E+09 |
| 72.00 | .244E-34 | .601E-24 | .124E+04 | .183E-20 | .305E-20 | .231E+09 |
| 78.00 | -.139E-41 | -.340E-31 | .124E+04 | -.114E-31 | -.174E-27 | .231E+09 |
| 84.00 | .788E-49 | .193E-38 | .124E+04 | .643E-39 | .986E-35 | .231E+09 |
| 90.00 | -.448E-56 | -.109E-45 | .124E+04 | -.364E-46 | -.561E-42 | .231E+09 |
| 96.00 | .255E-63 | .617E-53 | .124E+04 | .206E-53 | .319E-49 | .231E+09 |
| 102.00 | -.145E-70 | -.349E-60 | .124E+04 | -.117E-60 | -.181E-56 | .231E+09 |
| 108.00 | .823E-78 | .198E-67 | .124E+04 | .660E-68 | .103E-63 | .231E+09 |
| 114.00 | -.468E-85 | -.112E-74 | .124E+04 | -.373E-75 | -.585E-71 | .231E+09 |
| 120.00 | .532E-92 | .000E+00 | .124E+04 | .000E+00 | .665E-78 | .231E+09 |

COMPUTED LATERAL FORCE AT PILE HEAD = .25200E+04 LBS
COMPUTED MOMENT AT PILE HEAD = .93750E+04 IN-LBS
COMPUTED SLOPE AT PILE HEAD = -.21958E-02

```
THE OVERALL MOMENT IMBALANCE               =  -.185E-09 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE        =   .469E-11 LBS
```

OUTPUT SUMMARY

```
PILE HEAD DEFLECTION    =   .283E-01 IN
MAXIMUM BENDING MOMENT =   .246E+05 IN-LBS
MAXIMUM TOTAL STRESS    =   .156E+04 LBS/IN**2
MAXIMUM SHEAR FORCE     =   .260E+04 LBS

NO. OF ITERATIONS           =          14
MAXIMUM DEFLECTION ERROR   =    .665E-04 IN
```

S U M M A R Y   T A B L E
*************************

| LATERAL LOAD (LBS) | BOUNDARY CONDITION BC2 | AXIAL LOAD (LBS) | YT (IN) | ST (IN/IN) | MAX. MOMENT (IN-LBS) | MAX. STRESS (LBS/IN**2) |
|---|---|---|---|---|---|---|
| .252E+04 | .938E+04 | .350E+05 | .283E-01 | -.220E-02 | .246E+05 | .156E+04 |

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

```
Run Date:                  4-4-96
Time of Run:               1000
Run By:                    kc
Input Data Filename:       slope.in
Output Filename:           slope.out
```

PROBLEM DESCRIPTION   Littleville Landslide Slope Analysis - Present Loads

BOUNDARY COORDINATES
13 Top   Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 41.00 | 6.00 | 35.50 | 2 |
| 2 | 6.00 | 35.50 | 16.00 | 42.50 | 2 |
| 3 | 16.00 | 42.50 | 48.00 | 52.00 | 2 |
| 4 | 48.00 | 52.00 | 64.00 | 54.00 | 2 |
| 5 | 64.00 | 54.00 | 116.00 | 61.50 | 1 |
| 6 | 116.00 | 61.50 | 154.00 | 68.00 | 1 |
| 7 | 154.00 | 68.00 | 169.00 | 77.00 | 1 |
| 8 | 169.00 | 77.00 | 227.00 | 105.00 | 1 |
| 9 | 227.00 | 105.00 | 233.50 | 107.00 | 1 |
| 10 | 233.50 | 107.00 | 235.00 | 107.50 | 1 |
| 11 | 235.00 | 107.50 | 238.50 | 108.00 | 1 |
| 12 | 238.50 | 108.00 | 259.00 | 108.00 | 1 |
| 13 | 259.00 | 108.00 | 320.50 | 105.50 | 1 |
| 14 | 64.00 | 54.00 | 320.50 | 77.00 | 2 |
| 15 | 144.00 | 50.00 | 145.00 | 50.00 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 16.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 16000.0 | 16.0 | .00 | .0 | 1 |

```
1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
       Unit Weight of Water =  62.40

Piezometric Surface No.  1 Specified by  3 Coordinate Points


  Point        X-Water      Y-Water
   No.           (ft)         (ft)
    1             .00        38.00
    2          150.00        54.00
    3          320.50        68.00


TIEBACK LOAD(S)
    1 Tieback Load(s) Specified

Tieback   X-Pos     Y-Pos     Load      Spacing  Inclination  Length
  No.      (ft)      (ft)     (lbs)      (ft)       (deg)       (ft)
   1     134.00     64.58   140000.0     12.5       30.00       80.0


NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks
       Assuming A Uniform Distribution Of Load Horizontally Between
       Individual Tiebacks.

Trial Failure Surface Specified By  5 Coordinate Points

  Point        X-Surf       Y-Surf
   No.          (ft)         (ft)
    1            6.51        35.86
    2            7.43        35.16
    3          186.83        43.64
    4          227.12       102.00
    5          229.25       105.69


Factor Of Safety For The Preceding Specified Surface =  1.173


            Y           A     X     I     S            F     T

            .00     40.06     80.13    120.19    160.25    200.31
X       .00 +--------W*---------+---------+---------+---------+
            -         *
            -           *
            -
            -
            -
      40.06 +
            -             *
            -
            -
            -             *
            -
A     80.13 +
            -
            -
            -
            -               *
X    120.19 +
            -                1
            -                T
            -
            -           *W
            -                 *
I    160.25 +
            -                   *
            -
            -
            -           S
```

```
             -
S   200.31 +      T
             -    1
             -
             -
             -                     S*
             -                       *
    240.38 +                         *
             -
             -
             -                       *
             -
             -
F   280.44 +
             -
             -
             -
             -
             -
T   320.50 +              W *      *
```

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

| | |
|---|---|
| Run Date: | 4-4-96 |
| Time of Run: | 1000 |
| Run By: | kc |
| Input Data Filename: | slope2.in |
| Output Filename: | slope2.out |

PROBLEM DESCRIPTION   LITTLVILLE LANDSLIDE SLOPE ANALYSIS - PRESENT LOADS

BOUNDARY COORDINATES
10 Top Boundaries
14 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 33.00 | 3.00 | 28.00 | 2 |
| 2 | 3.00 | 28.00 | 19.00 | 31.00 | 2 |
| 3 | 19.00 | 31.00 | 30.00 | 37.00 | 2 |
| 4 | 30.00 | 37.00 | 60.00 | 38.00 | 2 |
| 5 | 60.00 | 38.00 | 167.00 | 67.00 | 1 |
| 6 | 167.00 | 67.00 | 257.00 | 76.00 | 1 |
| 7 | 257.00 | 76.00 | 300.00 | 101.00 | 1 |
| 8 | 300.00 | 101.00 | 311.00 | 104.00 | 1 |
| 9 | 311.00 | 104.00 | 340.00 | 104.00 | 1 |
| 10 | 340.00 | 104.00 | 379.00 | 103.00 | 1 |
| 11 | 60.00 | 38.00 | 147.00 | 43.00 | 2 |
| 12 | 147.00 | 43.00 | 206.00 | 57.00 | 2 |
| 13 | 206.00 | 57.00 | 379.00 | 60.00 | 2 |
| 14 | 220.00 | 50.00 | 221.00 | 50.00 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 16000.0 | 11.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 4 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 31.00 |
| 2 | 19.00 | 31.00 |
| 3 | 236.00 | 49.00 |
| 4 | 379.00 | 51.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|

```
   1      210.00     71.30   140000.0     12.5      30.00       80.0
```

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks.

Trial Failure Surface Specified By 6 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 19.78 | 31.43 |
| 2 | 19.82 | 31.39 |
| 3 | 236.08 | 41.00 |
| 4 | 286.74 | 42.46 |
| 5 | 330.04 | 103.90 |
| 6 | 330.10 | 104.00 |

Factor Of Safety For The Preceding Specified Surface = 1.126

```
           Y            A     X     I     S            F     T

            .00     47.38     94.75    142.13    189.50    236.88
X       .00 +-----**--+---------+---------+---------+---------+
            -
            -      *
            -      S
            -       *
            -
      47.38 +
            -
            -       *
            -
            -
            -
A     94.75 +
            -
            -
            -
            -
            -
X    142.13 +
            -        *
            -
            -             *
            -
            -
I    189.50 +
            -
            -           *  1
            -              T
            -          *
            -
S    236.88 +        SW
            -
            -
            -               *
            -
            -      T
     284.25 +      1 S
            -
            -                    *
            -                     *
            -
            -
F    331.63 +                     S
            -                     *
            -
            -
            -
            -
T    379.00 +          W *        *
```

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

| | |
|---|---|
| Run Date: | 4-4-96 |
| Time of Run: | 1000 |
| Run By: | kc |
| Input Data Filename: | slope3.in |
| Output Filename: | slope3.out |

PROBLEM DESCRIPTION   LITTLEVILLE LANDSLIDE SLOPE ANALYSIS - PRESENT LOADS

BOUNDARY COORDINATES
11 Top Boundaries
13 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 29.00 | 35.00 | 23.00 | 2 |
| 2 | 35.00 | 23.00 | 70.00 | 30.00 | 2 |
| 3 | 70.00 | 30.00 | 80.00 | 37.00 | 2 |
| 4 | 80.00 | 37.00 | 125.00 | 40.00 | 2 |
| 5 | 125.00 | 40.00 | 187.00 | 60.00 | 1 |
| 6 | 187.00 | 60.00 | 210.00 | 65.00 | 1 |
| 7 | 210.00 | 65.00 | 310.00 | 76.00 | 1 |
| 8 | 310.00 | 76.00 | 357.00 | 99.00 | 1 |
| 9 | 357.00 | 99.00 | 367.00 | 101.00 | 1 |
| 10 | 367.00 | 101.00 | 400.00 | 102.00 | 1 |
| 11 | 400.00 | 102.00 | 433.00 | 101.00 | 1 |
| 12 | 125.00 | 40.00 | 433.00 | 74.00 | 2 |
| 13 | 279.00 | 47.00 | 280.00 | 47.00 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 16000.0 | 11.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 4 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 29.00 |
| 2 | 63.00 | 29.00 |
| 3 | 296.00 | 50.00 |
| 4 | 433.00 | 57.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|
| 1 | 268.00 | 71.38 | 140000.0 | 12.5 | 30.00 | 80.0 |

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks.

Trial Failure Surface Specified By 6 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 63.30 | 28.66 |
| 2 | 64.14 | 26.18 |
| 3 | 68.00 | 23.00 |
| 4 | 372.00 | 49.01 |
| 5 | 400.00 | 101.90 |
| 6 | 400.06 | 102.00 |

Factor Of Safety For The Preceding Specified Surface = 1.119

```
           Y            A     X     I     S            F     T

            .00     54.13    108.25    162.38    216.50    270.63
X        .00 +----*----+---------+---------+---------+---------+
            -
            -
            -
            -   *
            -
      54.13 +
            -    S
            -   S *
            -      *
            -
            -
A     108.25 +
            -
            -      *
            -
            -
            -
X     162.38 +
            -
            -
            -          *
            -
            -           *
I     216.50 +
            -
            -
            -
            -
            -            1
S     270.63 +            T
            -        *
            -
            -        W
            -             *
            -
      324.75 +
            -     T
            -     1
            -
            -                 *
            -        S         *
F     378.88 +
            -
            -                  *
            -
            -
```

T 433.00 + W * *

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

| | |
|---|---|
| Run Date: | 4-4-96 |
| Time of Run: | 1000 |
| Run By: | kc |
| Input Data Filename: | slope4.in |
| Output Filename: | slope4.out |

PROBLEM DESCRIPTION LITTLEVILLE LANDSLIDE SLOPE ANALYSIS - PRESENT LOADS

BOUNDARY COORDINATES
15 Top Boundaries
17 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 27.00 | 10.00 | 22.50 | 2 |
| 2 | 10.00 | 22.50 | 22.00 | 23.00 | 2 |
| 3 | 22.00 | 23.00 | 44.00 | 26.00 | 2 |
| 4 | 44.00 | 26.00 | 65.00 | 41.50 | 2 |
| 5 | 65.00 | 41.50 | 116.00 | 45.00 | 2 |
| 6 | 116.00 | 45.00 | 131.00 | 48.00 | 1 |
| 7 | 131.00 | 48.00 | 178.00 | 57.00 | 1 |
| 8 | 178.00 | 57.00 | 270.00 | 77.00 | 1 |
| 9 | 270.00 | 77.00 | 295.50 | 79.50 | 1 |
| 10 | 295.50 | 79.50 | 359.50 | 96.00 | 1 |
| 11 | 359.50 | 96.00 | 364.50 | 97.00 | 1 |
| 12 | 364.50 | 97.00 | 372.50 | 98.00 | 1 |
| 13 | 372.50 | 98.00 | 400.50 | 99.00 | 1 |
| 14 | 400.50 | 99.00 | 431.50 | 98.00 | 1 |
| 15 | 431.50 | 98.00 | 435.50 | 97.50 | 1 |
| 16 | 116.00 | 45.00 | 435.50 | 75.00 | 2 |
| 17 | 280.00 | 41.50 | 281.00 | 41.50 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 16000.0 | 11.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 4 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 30.00 |
| 2 | 44.00 | 30.00 |
| 3 | 268.00 | 53.00 |
| 4 | 435.50 | 68.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|
| 1 | 272.00 | 77.20 | 140000.0 | 12.5 | 30.00 | 80.0 |

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks

Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks.

Trial Failure Surface Specified By 5 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 46.22 | 27.64 |
| 2 | 47.00 | 27.00 |
| 3 | 350.02 | 45.03 |
| 4 | 400.40 | 98.00 |
| 5 | 400.97 | 98.98 |

Factor Of Safety For The Preceding Specified Surface = 1.134

```
           Y            A     X     I     S            F     T

            .00     54.44    108.88    163.31    217.75    272.19
X       .00 +----*W---+---------+---------+---------+---------+
            -   *
            -   *
            -
            -
            -    *W
      54.44 +
            -       *
            -
            -
            -
            -
A    108.88 +
            -       *
            -        *
            -
            -
            -
X    163.31 +
            -
            -         *
            -
            -
            -
I    217.75 +
            -
            -
            -
            -
            -             1
S    272.19 +         W   *
            -       *
            -
            -              *
            -
            -
     326.63 +
            -
            -      T
            -      1S
            -                 *
            -                 *
F    381.06 +
            -
            -                 *
            -
            -
            -
T    435.50 +           W *   *
```

APPENDIX C

PCSTABL5M Outputs for Ultimate Loads

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

| | |
|---|---|
| Run Date: | 4-4-96 |
| Time of Run: | 1010 |
| Run By: | kc |
| Input Data Filename: | slopeu.in |
| Output Filename: | slopeu.out |

PROBLEM DESCRIPTION   Littleville Landslide Slope Analysis - Ultimates

BOUNDARY COORDINATES
13 Top Boundaries
15 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 41.00 | 6.00 | 35.50 | 2 |
| 2 | 6.00 | 35.50 | 16.00 | 42.50 | 2 |
| 3 | 16.00 | 42.50 | 48.00 | 52.00 | 2 |
| 4 | 48.00 | 52.00 | 64.00 | 54.00 | 2 |
| 5 | 64.00 | 54.00 | 116.00 | 61.50 | 1 |
| 6 | 116.00 | 61.50 | 154.00 | 68.00 | 1 |
| 7 | 154.00 | 68.00 | 169.00 | 77.00 | 1 |
| 8 | 169.00 | 77.00 | 227.00 | 105.00 | 1 |
| 9 | 227.00 | 105.00 | 233.50 | 107.00 | 1 |
| 10 | 233.50 | 107.00 | 235.00 | 107.50 | 1 |
| 11 | 235.00 | 107.50 | 238.50 | 108.00 | 1 |
| 12 | 238.50 | 108.00 | 259.00 | 108.00 | 1 |
| 13 | 259.00 | 108.00 | 320.50 | 105.50 | 1 |
| 14 | 64.00 | 54.00 | 320.50 | 77.00 | 2 |
| 15 | 144.00 | 50.00 | 145.00 | 50.00 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 16.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 35300.0 | 16.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 3 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 38.00 |
| 2 | 150.00 | 54.00 |
| 3 | 320.50 | 68.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|
| 1 | 134.00 | 64.58 | 246000.0 | 12.5 | 30.00 | 80.0 |

```
NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks
       Assuming A Uniform Distribution Of Load Horizontally Between
       Individual Tiebacks.


Trial Failure Surface Specified By  5 Coordinate Points

  Point        X-Surf        Y-Surf
   No.          (ft)          (ft)
    1           6.51          35.86
    2           7.43          35.16
    3         186.83          43.64
    4         227.12         102.00
    5         229.25         105.69

Factor Of Safety For The Preceding Specified Surface =  1.362

            Y            A     X     I     S            F     T

             .00     40.06     80.13    120.19    160.25   200.31
X        .00 +--------W*---------+---------+---------+---------+
             -        *
             -          *
             -
             -
             -
       40.06 +
             -            *
             -
             -
             -            *
             -
A      80.13 +
             -
             -
             -
             -
             -              *
X     120.19 +
             -               1
             -               T
             -
             -           *W
             -                *
I     160.25 +
             -                 *
             -
             -
             -          S
             -
S     200.31 +     T
             -     1
             -
             -
             -                        S*
             -                          *
      240.38 +                          *
             -
             -
             -                          *
             -
F     280.44 +
             -
             -
             -
             -
```

```
T    320.50 +                  W *      *
```

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

| | |
|---|---|
| Run Date: | 4-4-96 |
| Time of Run: | 1010 |
| Run By: | kc |
| Input Data Filename: | slopeu2.in |
| Output Filename: | slopeu2.out |

PROBLEM DESCRIPTION   LITTLVILLE LANDSLIDE SLOPE ANALYSIS - ULTIMATES

BOUNDARY COORDINATES
10 Top Boundaries
14 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 33.00 | 3.00 | 28.00 | 2 |
| 2 | 3.00 | 28.00 | 19.00 | 31.00 | 2 |
| 3 | 19.00 | 31.00 | 30.00 | 37.00 | 2 |
| 4 | 30.00 | 37.00 | 60.00 | 38.00 | 2 |
| 5 | 60.00 | 38.00 | 167.00 | 67.00 | 1 |
| 6 | 167.00 | 67.00 | 257.00 | 76.00 | 1 |
| 7 | 257.00 | 76.00 | 300.00 | 101.00 | 1 |
| 8 | 300.00 | 101.00 | 311.00 | 104.00 | 1 |
| 9 | 311.00 | 104.00 | 340.00 | 104.00 | 1 |
| 10 | 340.00 | 104.00 | 379.00 | 103.00 | 1 |
| 11 | 60.00 | 38.00 | 147.00 | 43.00 | 2 |
| 12 | 147.00 | 43.00 | 206.00 | 57.00 | 2 |
| 13 | 206.00 | 57.00 | 379.00 | 60.00 | 2 |
| 14 | 220.00 | 50.00 | 221.00 | 50.00 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 35300.0 | 11.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 4 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 31.00 |
| 2 | 19.00 | 31.00 |
| 3 | 236.00 | 49.00 |
| 4 | 379.00 | 51.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback | X-Pos | Y-Pos | Load | Spacing | Inclination | Length |
|---|---|---|---|---|---|---|

```
  No.      (ft)      (ft)      (lbs)      (ft)       (deg)       (ft)
   1     210.00     71.30   246000.0     12.5        30.00        80.0
```

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks
       Assuming A Uniform Distribution Of Load Horizontally Between
       Individual Tiebacks.

Trial Failure Surface Specified By 6 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 19.78 | 31.43 |
| 2 | 19.82 | 31.39 |
| 3 | 236.08 | 41.00 |
| 4 | 286.74 | 42.46 |
| 5 | 330.04 | 103.90 |
| 6 | 330.10 | 104.00 |

Factor Of Safety For The Preceding Specified Surface = 1.268

```
           Y           A     X     I     S           F     T

             .00     47.38     94.75    142.13    189.50    236.88
X        .00 +-----**--+---------+---------+---------+---------+
             -
             -      *
             -      S
             -       *
             -
       47.38 +
             -
             -       *
             -
             -
             -
A      94.75 +
             -
             -
             -
             -
             -
X     142.13 +
             -        *
             -
             -             *
             -
             -
I     189.50 +
             -
             -           *  1
             -              T
             -          *
             -
S     236.88 +        SW
             -
             -
             -               *
             -
             -      T
      284.25 +      1 S
             -
             -                    *
             -                     *
             -
             -
F     331.63 +                     S
             -                     *
```

```
                   -
                   -
                   -
                   -
T    379.00 +          W *        *
```

```
                     ** PCSTABL5M **

                           by
                    Purdue University

                --Slope Stability Analysis--
            Simplified Janbu, Simplified Bishop
               or Spencer`s Method of Slices

Run Date:                4-4-96
Time of Run:             1010
Run By:                  kc
Input Data Filename:     slopeu3.in
Output Filename:         slopeu3.out

 PROBLEM DESCRIPTION   LITTLEVILLE LANDSLIDE SLOPE ANALYSIS - ULTIMATES

BOUNDARY COORDINATES
   11 Top   Boundaries
   13 Total Boundaries
```

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 29.00 | 35.00 | 23.00 | 2 |
| 2 | 35.00 | 23.00 | 70.00 | 30.00 | 2 |
| 3 | 70.00 | 30.00 | 80.00 | 37.00 | 2 |
| 4 | 80.00 | 37.00 | 125.00 | 40.00 | 2 |
| 5 | 125.00 | 40.00 | 187.00 | 60.00 | 1 |
| 6 | 187.00 | 60.00 | 210.00 | 65.00 | 1 |
| 7 | 210.00 | 65.00 | 310.00 | 76.00 | 1 |
| 8 | 310.00 | 76.00 | 357.00 | 99.00 | 1 |
| 9 | 357.00 | 99.00 | 367.00 | 101.00 | 1 |
| 10 | 367.00 | 101.00 | 400.00 | 102.00 | 1 |
| 11 | 400.00 | 102.00 | 433.00 | 101.00 | 1 |
| 12 | 125.00 | 40.00 | 433.00 | 74.00 | 2 |
| 13 | 279.00 | 47.00 | 280.00 | 47.00 | 3 |

```
ISOTROPIC SOIL PARAMETERS
  3 Type(s) of Soil
```

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 35300.0 | 11.0 | .00 | .0 | 1 |

```
 1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
        Unit Weight of Water =   62.40

 Piezometric Surface No.   1 Specified by   4 Coordinate Points
```

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 29.00 |
| 2 | 63.00 | 29.00 |
| 3 | 296.00 | 50.00 |
| 4 | 433.00 | 57.00 |

```
TIEBACK LOAD(S)
     1 Tieback Load(s) Specified
```

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|
| 1 | 268.00 | 71.38 | 246000.0 | 12.5 | 30.00 | 80.0 |

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks.

Trial Failure Surface Specified By 6 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 63.30 | 28.66 |
| 2 | 64.14 | 26.18 |
| 3 | 68.00 | 23.00 |
| 4 | 372.00 | 49.01 |
| 5 | 400.00 | 101.90 |
| 6 | 400.06 | 102.00 |

Factor Of Safety For The Preceding Specified Surface = 1.249

```
           Y            A     X     I     S            F     T

            .00     54.13    108.25    162.38    216.50    270.63
X        .00 +----*----+---------+---------+---------+---------+
            -
            -
            -
            -   *
            -
      54.13 +
            -    S
            -   S *
            -      *
            -
            -
A    108.25 +
            -
            -      *
            -
            -
            -
X    162.38 +
            -
            -
            -          *
            -
            -           *
I    216.50 +
            -
            -
            -
            -
            -            1
S    270.63 +            T
            -        *
            -
            -        W
            -             *
            -
     324.75 +
            -     T
            -     1
```

```
             -
             -              *
             -       S       *
F    378.88 +
             -
             -               *
             -
             -
             -
T    433.00 +          W  *    *
```

** PCSTABL5M **

by
Purdue University

--Slope Stability Analysis--
Simplified Janbu, Simplified Bishop
or Spencer`s Method of Slices

Run Date: 4-4-96
Time of Run: 1010
Run By: kc
Input Data Filename: slopeu4.in
Output Filename: slopeu4.out

PROBLEM DESCRIPTION LITTLEVILLE LANDSLIDE SLOPE ANALYSIS - ULTIMATES

BOUNDARY COORDINATES
15 Top Boundaries
17 Total Boundaries

| Boundary No. | X-Left (ft) | Y-Left (ft) | X-Right (ft) | Y-Right (ft) | Soil Type Below Bnd |
|---|---|---|---|---|---|
| 1 | .00 | 27.00 | 10.00 | 22.50 | 2 |
| 2 | 10.00 | 22.50 | 22.00 | 23.00 | 2 |
| 3 | 22.00 | 23.00 | 44.00 | 26.00 | 2 |
| 4 | 44.00 | 26.00 | 65.00 | 41.50 | 2 |
| 5 | 65.00 | 41.50 | 116.00 | 45.00 | 2 |
| 6 | 116.00 | 45.00 | 131.00 | 48.00 | 1 |
| 7 | 131.00 | 48.00 | 178.00 | 57.00 | 1 |
| 8 | 178.00 | 57.00 | 270.00 | 77.00 | 1 |
| 9 | 270.00 | 77.00 | 295.50 | 79.50 | 1 |
| 10 | 295.50 | 79.50 | 359.50 | 96.00 | 1 |
| 11 | 359.50 | 96.00 | 364.50 | 97.00 | 1 |
| 12 | 364.50 | 97.00 | 372.50 | 98.00 | 1 |
| 13 | 372.50 | 98.00 | 400.50 | 99.00 | 1 |
| 14 | 400.50 | 99.00 | 431.50 | 98.00 | 1 |
| 15 | 431.50 | 98.00 | 435.50 | 97.50 | 1 |
| 16 | 116.00 | 45.00 | 435.50 | 75.00 | 2 |
| 17 | 280.00 | 41.50 | 281.00 | 41.50 | 3 |

ISOTROPIC SOIL PARAMETERS
3 Type(s) of Soil

| Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 125.0 | 125.0 | .0 | 30.0 | .00 | .0 | 1 |
| 2 | 125.0 | 125.0 | .0 | 11.0 | .00 | .0 | 1 |
| 3 | 125.0 | 125.0 | 35300.0 | 11.0 | .00 | .0 | 1 |

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED
Unit Weight of Water = 62.40

Piezometric Surface No. 1 Specified by 4 Coordinate Points

| Point No. | X-Water (ft) | Y-Water (ft) |
|---|---|---|
| 1 | .00 | 30.00 |
| 2 | 44.00 | 30.00 |
| 3 | 268.00 | 53.00 |
| 4 | 435.50 | 68.00 |

TIEBACK LOAD(S)
1 Tieback Load(s) Specified

| Tieback No. | X-Pos (ft) | Y-Pos (ft) | Load (lbs) | Spacing (ft) | Inclination (deg) | Length (ft) |
|---|---|---|---|---|---|---|
| 1 | 272.00 | 77.20 | 246000.0 | 12.5 | 30.00 | 80.0 |

NOTE - An Equivalent Line Load Is Calculated For Each Row Of Tiebacks Assuming A Uniform Distribution Of Load Horizontally Between Individual Tiebacks.

Trial Failure Surface Specified By 5 Coordinate Points

| Point No. | X-Surf (ft) | Y-Surf (ft) |
|---|---|---|
| 1 | 46.22 | 27.64 |
| 2 | 47.00 | 27.00 |
| 3 | 350.02 | 45.03 |
| 4 | 400.40 | 98.00 |
| 5 | 400.97 | 98.98 |

Factor Of Safety For The Preceding Specified Surface = 1.277

```
           Y            A     X     I     S            F      T

            .00     54.44    108.88    163.31    217.75    272.19
X       .00 +----*W---+---------+---------+---------+---------+
            -   *
            -   *
            -
            -
            -    *W
      54.44 +
            -       *
            -
            -
            -
            -
A    108.88 +
            -       *
            -        *
            -
            -
            -
X    163.31 +
            -
            -         *
            -
            -
            -
I    217.75 +
            -
            -
            -
            -             1
S    272.19 +         W   *
            -       *
            -
            -              *
            -
```

```
            -
     326.63 +
            -
            -      T
            -      1S
            -                 *
            -                 *
F    381.06 +
            -
            -                 *
            -
            -
            -
T    435.50 +          W *    *
```