SOIL NAILING

OF A

BRIDGE FILL EMBANKMENT

CONSTRUCTION REPORT

FEDERAL HIGHWAY ADMINISTRATION EXPERIMENTAL FEATURES

OR 89-07

By

Claude T. Sakr, P.E.

OREGON DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION
BRIDGE SECTION

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ABSTRACT

Soil nailing as an alternative lateral earth support method has recently been introduced in Oregon to build the first permanent Soil-Nailed Wall on the State's Highway System.

The soil nailing technique was used for an underpass widening to provide for additional traveling lanes under the existing Oregon Slough Bridge in Portland, Oregon. The project required removal of the existing south end slope and the construction of a Soil-Nailed Wall in front of the pile-supported end bent to permanently retain the existing bridge fill embankment.

Construction and post-construction monitoring was performed to study the new wall's performance.

This is the first of a five-paper sequence describing the results of ODOT's extensive investigation into the soil nailing technique as an alternative to more conventional bridge embankment retention methods.

Based on the results of our study, it may be concluded that:

The soil nailing technique is a viable lateral earth support system to retain an existing bridge fill embankment and to allow for a roadway widening under a bridge. The selection of the soil nailing support system was based on economic considerations. This technique enabled (1) considerable cost savings to the owner, (2) the project to proceed without disrupting overhead bridge or adjacent roadway traffic, (3) the Contractor to work in low overhead clearance conditions, and (4) the Contractor to quickly alter his construction procedure to fit soil conditions.

1.0 INTRODUCTION

1.1 Background

Soil nailing, as an alternative lateral earth support system, has been used extensively in Europe to stabilize highway slopes and to support temporary and permanent vertical soil cuts.

The first soil nailing applications were linked to the development of the new Austrian Tunneling Method which considers the ground as a carrying, rather than a "to be carried" element, when properly assisted.

Soil nailing was first used in the United States during the temporary excavation for the foundation of the Good Samaritan Hospital Expansion in Portland, Oregon in 1976.

Research carried out in the early 1980s led to a better understanding of the mechanism of this technique. Since then it has been successfully used in the United States to support several temporary and permanent vertical earth cuts and most recently, to stabilize high slopes. In 1985, the first highway Soil-Nailed wall was used to temporarily support cuts up to 40 feet on the Federal Highway Administration's (FHWA) Cumberland Gap Tunnel project in Kentucky.

This embankment-support method is a soil improvement concept: It consists of placing passive (unstressed) steel bars in the in-situ soil to improve the shear strength of the reinforced soil by limiting decompression and dilation immediately after excavation. The reinforced soil body, given improved strength characteristics, becomes the prime structural element. The reinforced zone performs as a homogenous and resistant unit to support the unreinforced soil behind it, in a manner similar to a gravity wall.

Soil nailing is used in cut retention applications and consists of staged excavation from the 'top down'. The in-situ soil is reinforced with passive steel bars placed in drilled boreholes and grouted along their total length. The soil nails (steel bars) are installed in a lift-by-lift sequence as the excavation progresses. Nail spacing is designed so the material between the nails will arch and form a reinforced earth block. The outside facing of the structure which prevents relaxation or sloughing of the ground consists of a thin layer of shotcrete with wire mesh reinforcing.

Oregon's Interstate 5 Swift-Delta project (road widening under an existing bridge) required a 'top down' staged excavation technique. No consideration was given for a 'bottom up' conventional wall because it would have required an expensive temporary shoring system to retain the existing bridge end embankment.

A 'top down' staged excavation can also be accomplished by using a conventional Tied-back wall. The soil nailing technique was chosen for the following reasons:

1. Soil nailing, unlike Tied-back walls, requires no soldier pile installation; therefore, holes do not have to be made through the existing bridge deck to drive the pilings, and bridge traffic is not disrupted.

2. A Tied-back wall face must be designed to resist full design earth pressure; a Soil-Nailed wall face is designed to prevent local failure of the soil between the nails and does not play a major role in the overall structural stability of the wall.

3. Soil nails are installed at a far higher density than prestressed tieback anchors; the consequences of a unit failure are not necessarily severe.

4. Ease of construction and reduced construction time: soldier pile installation is not required, soil nails are not prestressed, and construction equipment is relatively small scale, mobile and quiet.

5. Cost saving is typically 10 to 30 percent relative to a Tied-back wall.

The I-5 Soil-Nailed wall was the first permanent nailed wall used on a highway project within Oregon. Since this technique is relatively new, the project was defined as experimental. Construction and post-construction monitoring was required to study the performance of the new wall.

1.2 Objectives and Scope

This project calls for the removal of the existing south end soil berm and the construction of a Soil-Nailed wall to permanently retain the fill behind the pile-supported abutment at the south end of the Oregon Slough Bridge.

The purpose of this experimental features project is twofold:

1. To implement an instrumentation program to improve our understanding of soil nailing performance under actual service conditions; a performance not well predicted by the limit equilibrium analysis (at failure conditions) currently used to design Soil-Nailed walls; and,

2. To evaluate the influence of the existing piles on the behavior of the Soil-Nailed wall.

To accomplish the project objectives, construction and post-construction monitoring will be performed for the first two years after wall construction has been completed.

The project reporting was subdivided into five reports for purposes of organization. These interim reports will be published during the first two years after completion of construction . A final report will be published in 1993. Those reports will address the following tasks:

REPORT 1 - Will cover the construction and the short-term performance of the new Soil-Nailed wall; construction problems will be discussed; conclusions and recommendations to update the standard specifications will be presented.

REPORT 2 - The instrumentation program, implemented during the construction of the wall, will be discussed in detail; the instrumentation data at two vertical cross sections will be presented and data interpretation discussed in detail; the performance predicted by the original design methodology will be compared critically to the measured performance; conclusions and recommendations will be presented.

REPORT 3 - In-situ pullout test results will be presented; the frictional resistance at the Soil-Nail interface will be determined from pullout test results and compared to corresponding in-situ pressuremeter test results.

REPORT 4 - The full-scale instrumentation program demonstrates a measured performance under actual service conditions. Currently available design methods that allow an estimate of the maximum tension generated in the nails under working conditions will be evaluated; in particular, the KINEMATICAL method of design (Juran and Elias) and the CALTRANS method of design will be compared critically to the measured performance.

REPORT 5 - The staged excavation in front of the pile-supported abutment will be analyzed using a computer model to predict the stress variation in the existing piles; the predicted results will be compared to the measured performance. The influence of the existing pile bent on the behavior of the Soil-Nailed wall will be evaluated to better our understanding of soil-structural interaction in the Soil-Nailed retaining system.

2.0 PROJECT LOCATION

This project was located approximately seven miles north of Portland, Oregon on the Interstate 5 Freeway at MP 307.46. The purpose of this project was to widen and lower the grade of the Swift Highway under the south end of the existing Oregon Slough Bridge.

The project was an integral part of the Swift Interchange - Delta Park Interchange reconstruction scheme jointly funded by the State of Oregon and the Federal Highway Administration (FHWA).

3.0 PROJECT DESCRIPTION

The work consisted of staged excavation from the top down of the existing south end berm in front of the pile-supported bridge abutment and the construction of a permanent Soil-Nailed wall.

The total wall length was 256 feet of which 165 feet was located under the Oregon Slough Bridge. The maximum wall height was 19 feet. The total wall surface area was 4,105 square feet. The contract plans wall thickness was eight inches and the permanent facing was exposed shotcrete with a Class I finish and an architectural treatment consisting of vertical and horizontal scoring. See Appendix E for contract plans.

The soil nailing construction sequence consisted of stage excavating from the existing ground surface down to the layer limits shown in the plans, placing preformed permeable drainage fabric and welded wire fabric, applying air blown structural shotcrete, drilling holes at the inclination shown in the plans, and placing and grouting steel bars (soil nails).

The Contractor selected the nail installation method, the maximum hole diameter, and the grouting method.

Preproduction testing was performed on soil nails installed with the proposed production drilling and nail installation method. The purpose of the tests (three successful preproduction tests were required) was to verify the Contractor's drilling procedure, proposed maximum hole diameter and the grouting method. The intent of the tests was to stress the bond between the grout and the surrounding soil to a total load equal to the minimum pullout resistance (kips) shown on the plans. The Engineer evaluated the test results to determine the suitability of the Contractor's proposed drilling and grouting procedures.

Production testing was performed to demonstrate that the minimum required pullout resistances (kips) shown in the plans were being developed. Ten percent of the nails in each shotcrete lift were tested. Production test nails could be either sacrificial or used as permanent production nails.

The contract work scope also consisted of furnishing all instruments, tools, materials and labor and performing all tests necessary to install instruments at two instrumented wall cross sections. The instrumentation scheme for each cross section consisted of:

1. Strain gauges welded to the nails
2. One load cell at each of three nail layers
3. Earth pressure cells
4. Two inclinometers installed to monitor wall deflections; three tiltmeters and one extensometer installed to monitor the existing bridge pile cap rotation and deflection respectively as excavation progressed.

For a detailed description of the instrumentation program, please refer to Appendix D.

The overall project cost was estimated at $26,625,358 which included the reconstruction of the Swift and the Delta Park Interchanges. The construction cost of the Soil-Nailed wall portion of the project was $242,195. The instrumentation cost was an additional $10,000. The project was awarded on April 13, 1990 to the joint venture of Kiewit-Marmolejo.

4.0 SITE AND SUBSURFACE CONDITIONS

An exploration and testing program was undertaken to evaluate subsurface materials and soil standup time along the proposed wall alignment. The exploration program consisted of rotary drilling, test pit excavation, field including in-situ torvane testing and laboratory soil tests. The exploration program was limited to the wall portions located east and west of the Oregon Slough Bridge. The embankment material properties were available from an earlier report prepared when the bridge was constructed earlier.

The subsurface material consisted of approximately 35 feet of backfill material overlying 115 feet of unconsolidated alluvial sediments overlying the cemented gravels of the Troutdale Formation. The proposed wall construction required locating the permanent nails in the backfill material.

The backfill material encountered in the borings and test pits were divided into two soil units, A1 and A2. These units identified from the existing ground surface down were:

1. Soil unit A1 was silty fine sand with varying amounts of rock and debris. East of the bridge abutment, seven feet of silty fine sand with 40 percent rounded gravels up to two inches in diameter was encountered. West of the bridge abutment, six feet of silty fine sand was encountered with concrete pieces, rock boulders and metal debris up to two feet in diameter. Soil unit A1 was not continuous under the bridge abutment.

   East and west of the bridge abutment, the soil was dry to moist and generally loose. Sloughing of the top six feet of the test pits occurred during excavation due to the looseness of the soil and the large concrete pieces and rock boulders that were dislodged by the backhoe.

2. Soil unit A2 consisted of a clayey silt (ML) layer overlying clean fine sand (SP). The silt layer was encountered west of the bridge abutment and was four feet thick. SPT blow counts ranged from four to six blows per foot (Bpf) and torvane values of 0.5 to 0.75 tsf. Moisture content ranged from 33 to 40 percent. The material was easily excavated and held a vertical slope for the three-day standup test period.

   The underlying clean fill sand was continuous under the bridge abutment and was encountered along the entire wall. The sand was medium dense, damp to moist, poorly graded to uniform. SPT counts ranged from 11 to 28 Bpf with a general increase in Bpf with increased depth. Direct shear testing of the sand showed a phi angle of 32°. Minor sand sloughing occurred in the test pits after three days but did not significantly affect the test pit sides.

Ground water was encountered in three borings and in one test pit. The elevation of the ground water was determined to be five to six feet below the bottom of the wall base and was not expected be encountered during wall excavation. No evidence of a perched water condition was noted.

The exploration and testing program was compiled in a geological report and made available to all bidders. Included in the geological report were the following construction considerations that were deemed essential:

a. Soil unit A1 was susceptible to sloughing, and there was a possibility of encountering +two-foot size pieces of concrete and other debris at the cut face.

b. The wall construction in soil unit A2 (clean sand) should proceed in a timely manner to reduce the risk of sloughing. Drying and the resulting reduction of apparent cohesion in the sand will occur quickly in dry weather. Special treatment of the sand layer may be necessary to keep the open face from sloughing following the excavation of a lift. A thin temporary layer of shotcrete may have to be applied immediately after excavating the face in the event the open cut was to remain open for more than one day prior to soil nailing.

5.0 PREQUALIFICATION OF CONTRACTOR AND CONTRACTOR'S PERSONNEL

The overall project was awarded to the joint venture of Kiewit-Marmolejo, General Contractors, who subcontracted Donald B. Murphy Contractor's Inc. (DBM) to construct the Soil-Nailed wall portion of the project.

DBM subquote of $171,806 ($41.85 per square foot of wall) was adjusted by Kiewit-Marmolejo prior to bid time to $242,195 ($59 per square foot of wall). DBM's subquote for the instrumentation was an additional $10,000. The second and third low bidders' subquotes for the instrumentation were $70,000 and $31,650, respectively.

Subsection 614.02 of the Supplemental Standard Specifications (Appendix D) required that the Contractor performing the soil nailing work to have successfully completed at least five projects in the last three years involving construction of earth reinforced walls using permanent soil nails.

In accordance with Subsection 614.02, the prequalification submittal for DBM was reviewed, and the references on DBM's list of current and past soil nailing projects were contacted. The respondents all indicated that soil nailing was used on their projects as a temporary excavation support system.

Based on our findings, we considered DBM not conforming to the requirements of Subsection 614.02; subsequently their qualifications were rejected as a subcontractor for the Soil-Nailed wall.

Following the disqualification of DBM, Kiewit-Marmolejo subcontracted Schnabel Foundation Co. to perform the soil nailing work.

The prequalification submittal from Schnabel Foundation Co. was considered to conform to Subsection 614.02. Schnabel's staff qualifications satisfactorily met the intent of the specifications, and they were qualified to perform the soil nailing work. Schnabel Foundation Co. subcontracted Johnson Western Gunite Company to perform the structural shotcrete portion of the Soil-Nailed wall project.

6.0 CONTRACTOR'S SUBMITTAL FOR SOIL NAILING CONSTRUCTION PROCEDURES

Subsection 614.03 of the Supplemental Standard Specifications required that the specialty Contractor submit the following, in writing, for the Engineer's approval not less than 15 working days prior to start of wall excavation:

1. A detailed construction sequence,
2. A proposed method of excavation,
3. The proposed drilling method and equipment to be used,
4. The proposed hole diameter,
5. Grout and shotcrete mix designs,
6. Procedures for placing the grout; and,
7. The soil nail testing method proposed to be followed.

Schnabel's initial submittal called for the following: (Subsequent changes made are italicized after each item.)

1. To use a Krupp crawler wagon drill Model DHR80A to install the soil nails. This machine is designed to work in low overhead clearance, which was a requirement for the project site.

2. To use Schnabel Foundation Co.'s patented tieback corrosion protection system, which utilizes a combination of epoxy and heat-shrink sleeve protection. The latter is a heat-shrinkable tube applied over the epoxy-coated nail near the head of the nail. The heat-shrinkable polyethylene tube is internally coated with a thixotropic sealant and is manufactured by Raychem Corporation. The tubing is applied by sliding it over the epoxy-coated nail. When the tubing is in position, it is heated to an excess of 250°F. It then shrinks and tightly encapsulates the nail. When the tubing shrinks, the sealant is forced around the deformations of the nail. (Contract plans called for the use of epoxy-coated rebar.)

   The use of a heat-shrink sleeve over the epoxy-coated nail near the nail head was not allowed. The information on the heat shrink sleeve did not clearly demonstrate that it would not act as a bond breaker between the nail and the grout.

3. The soil nails were to be epoxy-coated Dywidag bars conforming to ASTM A322 and ASTM A29. (Contract plans called for regular epoxy-coated rebar threaded on one end a minimum of six inches. See Appendix E.)

   Epoxy-coated Dywidag nails were approved since the substitution was at no cost to the State. The rolled deformations along their entire length offered the advantage of serving as continuous threads and permit anchorage or coupling hardware to be screwed onto the bars at any location.

4. To install the soil nails in five-inch drill holes, providing a two-inch grout cover around the epoxy-coated nail. (Contract plans required a minimum three-inch grout cover.)

The installation of soil nails in five-inch drill holes was approved provided the Contractor:
a. pressure grout the hole and
b. demonstrate that he could achieve the minimum pullout resistance shown on the contract plans.

The approval of the change from a seven-inch drill hole to a five-inch drill hole was based on existing literature indicating that pressure-injected grout increases the pullout capacity of tieback anchors installed in porous cohesionless soil. Any theoretical decrease in the soil-grout bond stress attributed to the use of a smaller borehole diameter was expected to be offset by the increase in the bond stress associated with pressure grouting.

5. To drill the holes using the "cased hole drilling method" which consisted of an outer temporary casing and an inner drill steel advanced simultaneously. The temporary outer casing was used to prevent the nail hole from collapsing following the completion of drilling. Soil cuttings were to be removed by water, or a mixture of air and water through the annular space between the outer casing and inner drill steel. After the casing and drill steel had reached the required depth, the inner drill steel was to be removed and the centralized nail inserted in the nailhole. The grouting was to be performed through the outer casing and the hole pressure grouted while extracting the casing. (Contract plans did not require pressure grouting the borehole.)

6. The Preproduction Test Nails were sacrificial according to the contract plans.

7. The Production Test Nails were to be used as permanent Production Nails following the successful stressing of the bond between the grout and the surrounding soil to a total load equal to the minimum pullout resistance (kips) shown in the plans. The Contractor later agreed to consider the Production Test Nails as sacrificial nails as per Item 8.

8. The Preproduction and Production Test requirements of a no-load zone (ungrounted test length) and a bond zone (grouted length) were to be met by providing a grout bond breaker over the entire unbonded test length and by simultaneously grouting the bond length and unbonded length on the Preproduction and Production Test Nails.

   Grouting the bonded and unbonded length of the test nail was to prevent the sides of the hole in the ungrouted zone from collapsing after removal of the temporary casing and during the wait period required for the grout to set in the bonded zone prior to testing. The proposed complete removal of the temporary casing was to prevent the grout from seizing of the casing in the ground.

The use of a grout-bond breaker over the entire unbonded length (ungrouted test length) of the Production Test Nails and simultaneously grouting the bond and unbonded length was approved provided the Contractor agree to such Production Test Nails being considered sacrificial nails. Using a grout-bond breaker on the permanent Production Nails was rejected on the grounds that it would prevent the soil nails and the in-situ soil from acting as a homogeneous and resistant unit to support the unreinforced ground behind. Conventional soil nails are bonded full length. Further, it was felt that placing the bond breaker on 10 percent of the permanent Production Nails would have an adverse effect on the long-term performance of the wall.

The Contractor agreed to consider the Production Test Nails as sacrificial (28 nails total).

9. The first structural shotcrete application was to be applied as the excavation proceeds downward. The second and third shotcrete applications were to be placed from the bottom to the top after subgrade (bottom of wall) was reached. (Contract plans required placing both the first and second shotcrete applications as the excavation proceeded downward.)

   The Contractor's stated purpose to delay the placement of the second shotcrete application (three-inches thick) until after subgrade was reached was due to concerns that a total of 6.5 inches of shotcrete in the first and second shotcrete applications, plus one to three inches of additional shotcrete due to overbreak, would induce large bending stresses at the head of the soil nails during excavation.

   The Contractor's proposal to place the first shotcrete application as excavation proceeded and to place the second and third applications from the bottom to the top after subgrade was reached was approved provided the second and third applications were applied in **ONE** *lift from the bottom to the top. Otherwise, the contractor was required to wait a minimum of 12 hours following the placement of the second shotcrete application. This wait period was recommended when the final shotcrete lift was to be finished by wood or steel trowel. The final lift finishing process may cause sags, cracks and tearing of the partially set second shotcrete lift.*

10. At each excavation lift, the first structural shotcrete application was to be placed prior to or after drilling and pressure grouting the soil nails. (Contract plans allowed nail installation prior to shotcreting provided the Contractor could demonstrate that sufficient duration of soil standup was achievable.)

    The proposal to place the first structural shotcrete application prior to soil nail installation was approved provided the Contractor placed blockouts at the nail locations prior to placing the first shotcrete layer. Otherwise, a five-day minimum wait was required before any soil nail drilling could be performed; this waiting period was to minimize the potential of the green shotcrete being cracked and structurally damaged by the drilling operation.

11. All soil nails, except the bottom most row, were to be installed at approximately 15° to the horizontal. The bottom row in all zones were to be installed at 25° to the horizontal. (Contract plans called for all nails to be installed at 15° to the horizontal.)

    The Contractor's stated purpose to place the bottom row at 25° was to enhance the wall's performance. This change would place a portion of the nail below the subgrade level and would increase the resistance against sliding when considering external stability.

    Placing the bottom most nail layer at 25° was approved.

12. The welded steel wire fabric reinforcing were to be W20 on six-inch centers. (Contract plans called for W20 on four-inch centers.) The Contractor's stated purpose for the substitution was the unavailability of the W20 on four-inch centers.

    The Contractor's request to use the W20 on six-inch centers was approved. It was determined that the substitution would not adversely affect the wall performance.

7.0 PRE-CONSTRUCTION CONFERENCE

The Preconstruction Conference was held on November 1, 1990 with Kiewit-Marmolejo, the Prime Contractor, and Schnabel Foundation Co., the soil nailing specialty contractor. Following is a summary of the items discussed:

Due to inclement weather conditions, Schnabel requested and received approval to install and test all the Preproduction Test Nails at locations limited to the wall portion under the Oregon Slough Bridge. In return, Schnabel proposed to test four Preproduction Test Nails (Contract plans require three) and to subsequently load test the four nails to failure by pullout.

Schnabel Foundation Co. expressed concerns about the soil sloughing following the excavation of a lift and suggested they might use a flashcoat of temporary shotcrete to stabilize the open face immediately following the excavation and trimming of the open face.

The Contractor requested and received approval to delete the use of a stroke counter during the grouting of the boreholes. (It was later determined their grout pump had no stroke counter. In hindsight, this request should not have been approved.)

Schnabel requested and received approval to delete the placement of a mortar level pad behind the bearing plate if and when the structural shotcrete face was placed after soil nail installation. Schnabel proposed to place the bearing plate immediately following the placement of the shotcrete face and prior to shotcrete set; this was to insure a uniform bearing surface behind the plate. Schnabel was cautioned to allow the shotcrete to take initial set prior to placing and tightening the nut at the nail head.

Schnabel was required to include expansion joints through the shotcrete face at 30± feet spacing. These were not called for on the plans. The vertical expansion joint detail consisted of a 1/4-inch preformed expansion joint filler extending through the first shotcrete application (3.5-inch thick) and stopping at the interface of the first and second shotcrete applications. All reinforcing welded wire mesh was to be stopped two inches clear of the vertical joint. Extreme care was to be exercised to insure that the centerline of the vertical joint coincided with the alignment of the preformed drainage liner and the vertical scoring strip.

The Contractor stated that the excavation would be performed by Kiewit-Marmolejo under the direction of Schnabel's project superintendent.

8.0 CONSTRUCTION OF THE SOIL-NAILED WALL

8.1 Site Preparation:

The specialty contractor mobilized on November 3, 1990.

The existing concrete slope paving in front of the bridge abutment was removed and a 20-foot wide temporary drilling platform was erected using temporary fill.

Prior to any excavation and soil nailing activity, the contractor hand excavated a shored pit (nine feet long by four feet wide) under the bridge to instrument two existing pipe piles with two strain gauges each according to the contract specifications. The gauges were mounted five feet and ten feet below the bottom face of the existing pile cap. Following the installation of the strain gauges, the pit was backfilled with a lean mix concrete (one cement sack mix).

Next, the Contractor proceeded to install three tiltmeters to monitor the pile cap rotation, and drilled and installed an extensometer to monitor the pile cap horizontal deflection as excavation progressed.

Soil sloughing under the pile cap was a major problem during the hand excavation of the pit. The Contractor became concerned that when the soil nailing activity started following the excavation of a lift, the soil would not stand up long enough to allow the placement of the drainage liner, the steel welded wire fabric and finally the first shotcrete application. The Contractor decided to:

1. excavate the first lift,
2. trim the open face to the true lines and grades shown in the plans,
3. place the drainage liner at the required spacing; and,
4. stabilize the open face with a thin (±one-inch thick) flashcoat of shotcrete reinforced with a thin chicken wire mesh. The flashcoat was considered sacrificial and not part of the eight-inch structural shotcrete required by the contract plans.

8.2 Construction Sequence

The soil nailing installation construction sequence consisted of:

1. Preproduction shotcrete testing
2. Preproduction nail testing
3. stage excavation of the existing end berm from the top down to the layer limits shown in the plans
4. placing preformed permeable drainage fabric and welded wire fabric
5. applying air blown structural shotcrete
6. drilling nail holes at the inclination shown in the plans
7. placing and grouting the steel bars (soil nails), and load testing sacrificial nails at each excavation lift
8. securing each soil nail with a 5/8" x 6" x 6" epoxy-coated steel bearing plate
9. fastening each bearing plate to the nail with a nut and securing wrench tight with a minimum 100 foot-lbs torque.

The construction of the Soil-Nailed wall proceeded.

STEP I - PREPRODUCTION TESTING

Preproduction testing consisted of two tasks.

1. Preproduction shotcrete test panels: each shotcrete nozzleperson demonstrated their ability to apply shotcrete of the required quality on two test panels. One of the two panels was reinforced the same as in the proposed permanent wall; the Contractor was required to saw the completed panel into several pieces to allow a visual inspection of the shotcrete density, void structure and coverage of the reinforcement. The other test panel was constructed without reinforcing. Eight three-inch diameter cores were extracted from this panel for compressive strength testing. See Appendix B for compressive strength test results.

2. Preproduction Test Nails: Four preproduction tests were performed on sacrificial soil nails installed with the proposed production drilling and nail installation procedures. The purpose of the tests was to verify the Contractor's drilling procedure, proposed maximum hole diameter and the grouting method. The intent of the tests was to stress the bond between the grout and the surrounding soil to a total load equal to the minimum pullout resistance (kips) shown in the plans. The requirements of a no-load zone ungrouted test length were met by providing a grout bond breaker over the entire unbonded test length and simultaneously grouting the bond length and unbonded length. Three tests were successful. The fourth test nail was incrementally loaded to 80 percent of the required design capacity when excessive elongations were measured. See Appendix D for further information on test acceptance criteria. The Contractor was allowed to proceed based on the three successful tests.

STEP II - EXCAVATE SMALL CUT

The first lift was excavated to a maximum 1'-6" below the top row of nails for an excavation depth of 3.5 feet. Next, the open face was trimmed with a hand-held wood screed to true line and grade. The exposed face was moist cohesionless sandy soil which allowed the open face to stand open temporarily. No water seepage was noted at the open face. The Contractor placed the preformed drainage liner and a thin sacrificial 2" x 2" chicken wire mesh. There was localized soil sloughing at the open face when the Contractor installed one-foot long No. 4 anchor pins to secure the chicken wire mesh in close contact with the soil. The open cut was later stabilized with a thin (±one-inch thick) flashcoat of sacrificial shotcrete.

STEP III - ERECT THE STEEL WELDED WIRE MESH

The welded steel wire mesh was W20 on six-inch centers and consisted of individual panels 4.0 feet high by 10 feet long. The bottom 1'0" was temporarily covered with some of the excavated soil material to keep the wire mesh fabric free from any shotcrete rebound and to later provide the minimum required overlap of at least two mesh dimensions (1'-0") at all seams. The Contractor was directed to use concrete block spacers to insure that the minimum cover requirement was met.

STEP IV - PLACE THE FIRST SHOTCRETE APPLICATION

Prior to placing the first shotcrete application, the Contractor formed blockouts at the nail locations. The blockouts consisted of six inch by six inch pieces of styrofoam tightly secured to the six-inch by six-inch steel welded wire mesh fabric. The Contractor was directed to provide a blockout with adequate thickness to insure that the structural shotcrete was placed around the blockout. The contractor stretched horizontal piano wires to guide the nozzleman in placing the initial structural shotcrete facing to the minimum required thickness of 3.5 inches.

The structural shotcrete was placed by a prequalified nozzleman. A helper with an air blowpipe cleaned the overspray and rebound in advance of the nozzleman applying the structural shotcrete.

Application of the shotcrete began at the bottom of the excavated lift and proceeded upward.

Following the application of the structural shotcrete facing, the nozzleman used a wood screed to trim and scrape excess material to true line and grade. Structural shotcrete was placed during cold weather. Insulation blankets were used to cover the newly placed structural shotcrete and to maintain a curing temperature above 40°F. The following day, the one-day old structural shotcrete facing was covered with wet burlap and water was applied periodically to prevent the burlap from drying. This curing method, coupled with covering the face with the heavy insulation blankets to retain moisture, held the curing temperature well above 40°F during the night hours.

The slump and the air content of the shotcrete mix were checked for each shotcrete application to insure conformity with the specification requirements of a slump in the range of 1.5 inches to 3 inches and air content of 7.5 percent ±1 percent. Six concrete test cylinders were prepared for compressive strength testing. Three cylinders were lab cured and the remaining three cylinders were cured on site to approximate actual field curing conditions. See Appendix B for compressive strength results.

STEP V - DRILLING HOLES AT NAIL LOCATIONS

The Contractor used a Krupp crawler drill to drill the nail holes at predetermined locations. The "cased hole drilling method" was used. It consisted of an outer casing and an inner drill steel advancing simultaneously. The outer casing functioned as a temporary casing to prevent the nailhole from collapsing following the completion of drilling. The temporary casing consisted of six-foot segments threaded at both ends. The Contractor could drill and temporarily protect 20 nailholes prior to pressure grout.

The Krupp drill was equipped with an impact hammer that helped advance the outer casing through dense material and large pieces of concrete. Soil cuttings were removed with air. The drilling was performed at nail locations spaced three feet vertically and 4.5 feet horizontally. The boreholes were drilled at approximately 15° to the horizontal. The Contractor averaged 15 drilled holes per day which were subsequently first-stage pressure grouted on the same day. A total of 281 permanent nails were drilled and installed. An additional 28 sacrificial Production Test Nails were drilled, installed and subsequently load tested.

The use of blockouts at the nail locations allowed the Contractor to drill the nailholes the day following the placement of the structural shotcrete face.

STEP VI - PLACE NAILS AND PRESSURE GROUT THE HOLES

After the temporary casing and the inner drill steel reached the required depth, the inner drill steel was removed and the soil nail was inserted in the nailhole. PVC centralizers were used to center the nail inside the borehole and to ensure adequate grout cover around the nail.

The grouting of the nailhole was performed by pumping a neat cement grout into the temporary casing. The grouting method used by the Contractor was:

1. The Contractor lowered the drill head on the guiding skids and engaged the threaded casing used to form the drilled hole.

2. The grout was pumped with a positive displacement grout pump equipped with a pressure gauge at the pump. The reading varied from 100 psi to 200 psi during grouting. The grout was pumped through a one-inch hose attached to the drill head, and was injected at the top of casing through the water swivel. The grout pressure was maintained when the grout column could no longer be advanced inside the casing.

3. Grouting was continued as the casing was pulled out.

4. The grouting was interrupted to allow the removal of the six-foot temporary casing segments. Grout backflow was observed when the Contractor completely stopped grouting to "break casing".

5. The Contractor next lowered the drill head, engaged the threaded casing anew and repeated the same process described above.

6. When the last casing segment was removed, the Contractor proceeded to pressure grout another drilled nail hole leaving the top two to five feet of each drilled nail hole ungrouted.

7. Subsequently, second-stage grouting was performed to fill the top ungrouted two to five-foot segment of each drilled hole. The second-stage grouting was performed with a one-inch grout tube inserted at the bottom end of the ungrouted segment and was used to tremie grout the drilled hole.

STEP VII - PLACE THE 6-INCH BY 6-INCH STEEL BEARING PLATE

Following the second-stage grouting of the drilled borehole, the Contractor dry packed around the nail head and placed the six-inch by six-inch bearing plate. Each plate was fastened to the nail with a nut and secured wrench tight with a minimum 100-foot lbs torque.

STEP VIII - REPEAT THE PROCESS TO BOTTOM GRADE

Steps II through VII were repeated with some alterations for all subsequent excavation lifts until bottom of the wall elevation was reached.

The sequence of construction was altered such that the drilling of the boreholes at the nail locations, the placing of the nails and pressure grouting the boreholes (Steps V and VI) were performed prior to erecting the welded wire mesh and the placement of the first shotcrete application. The wall portion of the first lift extending west of the bridge abutment and the westerly half of the third lift were nailed prior to placing of the structural shotcrete facing. When the structural shotcrete facing was applied after soil nail installation, the Contractor placed the bearing plate immediately following the placement of the shotcrete face and prior to shotcrete set; this ensured a uniform bearing surface behind the plate. The shotcrete was allowed to take initial set prior to tightening the nut at the nail head.

The curing of the shotcrete facing, described in Step IV, was later enhanced by adding a protective tent and by using heat blowers during the night hours. This modification was required to prevent the newly placed shotcrete facing from freezing during cold weather curing.

STEP IX - PLACE THE SECOND SHOTCRETE APPLICATION FULL HEIGHT FROM THE BOTTOM UP

After the wall excavation was completed to the specified grade, the Contractor cleaned the exposed shotcrete face with compressed air and water blasts to remove laitance and to ensure good bond to the second shotcrete application.

The Contractor next erected the welded wire mesh fabric corresponding to final shotcrete application. Number 5 reinforcing bars at nine-inch center to center were used in place of the wire mesh at several wall locations for ease of construction. The Contractor drilled into the existing shotcrete facing and placed No. 4 anchor pins at six-foot spacing to support the reinforcing bars. Full-depth anchor penetration into the existing shotcrete facing was not permitted, and a 2.5-inch maximum anchor depth was strictly enforced. Extreme care was exercised to insure that the uncoated reinforcing steel was not in direct contact with the epoxy coated nail head and the nail appurtenances.

Following the erection of the reinforcing bars, the Contractor placed the scoring strips. The scoring strips spacing was modified to ensure that the centerline of the expansion joints coincided with the alignment of the vertical strips. (See Appendix E for scoring strip details.) Threaded anchors were located on the existing shotcrete facing in predrilled holes to hold the scoring strips in place at the proper alignment.

The Contractor next erected a protective tent in preparation for the placement of the shotcrete face.

The second shotcrete application was placed full height from the bottom up in one and one-half days. The Contractor used a screed to scrape excess rebound material and to trim the face to true line and grade flush with the front face of the wood scoring strips. The Contractor used hand-held steel trowels to smooth finish the wall. Hand-held rubber floats were used to texture the wall. A clear liquid curing compound was later applied to cure the shotcrete face. The Contractor used propane heat blowers to maintain a curing temperature above 40°F for six days.

Following the placement of the final shotcrete facing, the Contractor removed the scoring strips. The threaded anchors used to position the scoring strips were burned one-inch behind the shotcrete face and the surface refinished.

9.0 CONSTRUCTION PROBLEMS

The following is a summary of the construction issues and problems encountered on the Soil-Nailed wall at the Swift - Delta Park Interchange Project and how they were resolved:

1. When the first Preproduction Test nail was load tested, the nail elongated excessively and the pressure gauge, attached to the hydraulic pump, failed to register any load. The Contractor later discovered that the pressure gauge was mounted improperly and subsequently retested the nail. The problem could have been avoided had the Contractor used an electronic load cell to monitor the loads on the nail as the contract specifications required.

2. The testing of the second Preproduction Test nail initially was not considered successful; the nail failed to meet the third criterion in Subsection 614.41(a-3.a) of the Supplemental Standard specifications, "Total movement measured at the maximum test load does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length." The Contractor stated that the third criterion in question was too conservative. He requested the State delete the third criterion of the load testing acceptance criteria.

   The Contractor's request was investigated. Subsequently, the load testing acceptance criteria was revised by deleting the third criterion. Our investigation indicated that this criterion was applicable to pullout testing in rock but was not applicable any longer in soil deposits, because it assumed that the skin friction along the nail was uniform. In reality, the skin friction distribution along the nail in the assumed bond zone would not be uniform if the nail was installed in a non-uniform soil deposit. Also, the total elastic movement may exceed the maximum recommended in the third criterion if the upper portion of the bond length was in weak soil.

3. As the Contractor proceeded to pressure grout the first row of nails, it became apparent that the fluid grout was being "gravity injected" into the drilled hole, and no pressure grouting was taking place. Grout backflow through the flushing head ports, as the grout was pumped through the one-inch grout hose into the 4.5 inch ID casing, indicated that the pressure was not maintained inside the casing.

   Our observation was shared with the Contractor and he agreed to use a steel collar to seal the flushing head ports at the drill head. This modification was judged successful since:

   a. no further grout backflow was observed during the grouting operation except when the Contractor completely stopped grouting and "broke casing", and

   b. following the injection of the grout into the casing, the pressure appeared to be maintained when the grout column could no longer be advanced inside the casing.

The suspicion that no pressure grouting was taking place could have been immediately verified if the Contractor had been asked to measure the grout pressure with gauges located at both the grout pump and the drill head. Locating pressure gauges at both the grout pump and the drill head would have verified that the pressure was being maintained inside the casing.

4. Two problems were identified during first-stage grouting of the first row of nails and following the extraction of the temporary casings:

a. The two-inch minimum cover around the nails was not being maintained. It was discovered that the Contractor was using three-inch diameter centralizers which resulted in a three-inch clear cover above the nail and only a one-inch clear cover below the nail.

The Contractor claimed that pressure grouting should center the nails within the boreholes. This claim was disputed since a fluid grout (cement-water mix) was being used to grout the boreholes.

Additional clear cover was achieved when the Contractor squeezed the ends of the centralizers inward in an effort to increase their diameter. Extreme care was exercised to insure that the compressed centralizers did not impede the advancement of the Dywidag bars inside the boreholes. In a few instances, the compressed centralizers broke when the Contractor rotated the casing to facilitate its extraction during grouting.

b. It appeared that the Contractor was not drilling the boreholes to the required hole depth as evidenced by the nails projecting 9 to 18 inches outside the drilled holes.

It was later discovered that the Contractor ordered a total bar length equivalent to the design length which was defined in the Contract plans as the distance from the bottom of the drilled hole to the fill side of the structural shotcrete facing. The Contractor made no allowance for the thickness of the shotcrete facing, the thickness of the bearing plate, the thread thickness to engage the washer and the nut at anchorage head nor for overdrilling. By the time this problem was noticed, the top row of nails were already installed and grouted. This omission resulted in an embedment length 9 to 18 inches less than the required nail embedment length specified in the Contract Plans.

All five Production Test nails in the first row of nails met the production test nail criteria. And all five tests showed a higher bond stress at the soil-grout interface than the assumed bond stress used in the design. Because of this, the Contractor was permitted to use the shorter nail length at subsequent excavation lifts, provided the drilling for the remaining nail holes achieved an embedment depth closely comparable to the Contract Plans. The Contractor also agreed to load test all remaining production test nails to a minimum overload of 120 percent of the design load.

5. The requirement to overlap the steel welded wire mesh fabric at all seams while maintaining a typical two-inch clear cover at the fill face of the wall was not practical with the 3.5-inch thick initial shotcrete layer. Irregularities in the cut face stabilized with the flashcoat of sacrificial shotcrete compounded the problem and prevented the Contractor from maintaining a typical two-inch clear cover at the fill face of the wall.

   The Contractor was directed to decrease the minimum clear cover where the steel wire fabric overlaps. It was suggested he gradually bend the wire fabric away from the fill face to re-establish the minimum clear cover of two inches at the fill face of the wall.

   The Contractor was later directed to stop overlapping the wire mesh and to splice adjacent horizontal panels with No. 4 rebar spliced to the horizontal wires; the same splicing procedure was to apply to adjacent vertical panel segments.

   The Contractor agreed not to overlap adjacent horizontal panels and to splice with No. 4 rebar. However, he refused to splice adjacent vertical runs with No. 4 rebar. He argued the splicing of vertical runs would require cutting all the prefabricated welded wire panels. The overlapping of vertical panels at all seams proved not to be a problem due to shotcrete overruns when the first structural shotcrete application was placed.

6. The drilling and installation of the top row of nails was performed through six-inch by six-inch blockouts formed with styrofoam cut to fit tight between the horizontal and vertical runs of the six-inch by six-inch steel welded wire mesh. The Contractor later provided 6-inch by 18-inch blockouts at the soil nail locations and placed two No. 4 waler bars to reinforce the dry packing at the nail head immediately behind the 6-inch by 6-inch bearing plate. This modification was made to decrease the possibility of the bearing plate punching through the shotcrete facing. The Contractor was compensated for the cost of providing the waler bars.

7. The top row of nails corresponding to Zone E, located westerly from the bridge, were drilled, installed and first-stage grouted prior to placing the structural shotcrete facing. The Contractor proposed to shotcrete the remaining unfilled top three to five-foot segment of each hole when the structural shotcrete facing was being placed. The proposal to shotcrete the unfilled portion was not approved, and the Contractor was directed to second-stage grout or to dry pack the unfilled portion of the nailholes. The Contractor dry packed the top ungrouted segment prior to placing the first structural shotcrete application.

   Dry packing the unfilled three to five-foot portion of the drilled holes was judged to be substandard for the following reasons:

   a. the installed nail and the nearest centralizer were impeded the proper dry packing of the drilled hole.

b. the dry packing was performed following the extraction of the temporary casing, leaving the sides of the nailhole unprotected and susceptible to caving. Sloughing of the sides of the borehole presented a threat of cement contamination resulting in poor quality protection at the nail head.

The Contractor was directed to second-stage tremie grout the unfilled portion of the nailholes on subsequent excavation lifts.

8. The Contractor excavated the westerly half of the third lift, stabilized the open face with a flashcoat of shotcrete, drilled and first-stage grouted the corresponding third row of nails during the week of December 17. Extreme cold weather prevented any shotcrete delivery and forced the Contractor to shut down his soil nailing activity.

When work was resumed on January 2, it was discovered that several nail holes on the westerly half of the wall had caved material in the unfilled top portion of the hole. The drilled nail holes filled with the first stage grout during the week of December 17 and left without a temporary casing at the top ungrouted three to five feet did not remain open; drying and the resulting reduction of apparent cohesion in the sand caused the sides of the unfilled portion of the nailholes to collapse.

This problem could have been avoided had the Contractor second-stage grouted the boreholes immediately following first-stage grouting and prior to shutting down his operation.

9. The Contractor proposed to shotcrete the unfilled top portion of the boreholes corresponding to the third row of nails at the westerly half of the wall during the placement of the structural shotcrete facing. This request was approved.

The process of shotcreting the unfilled top three to five feet of the drilled holes was judged to be unsuccessful and poor practice. It was suspected that the blast of shotcrete and air from the shotcrete nozzle have caused the top ungrouted segment of the nail holes to collapse. This opinion was based on the following observations:

a. The installed nail required the nozzleman to shotcrete the unfilled portion of the nail holes with the nozzle placed at the top of the five-inch diameter drilled holes.

b. The Contractor did not have a temporary casing to prevent the sides of the nailholes from collapsing under the blast of the shotcrete and air.

The Contractor was later directed not to shotcrete the unfilled top portion of the nailholes on subsequent excavation lifts. Second stage grouting the unfilled portion was made mandatory.

Our suspicions that the process of shotcreting the unfilled portion of the hole may have caused the sides of the hole to collapse were proven right as discussed later in (10).

10. On January 8, 1991 the Contractor excavated the fourth lift at Zones C through F and corresponding to the fourth row of nails. The open face consisted of cohesionless sandy soil with some moisture which allowed the open face to stand open temporarily. Soil sloughing at the open face was minimum, and no water seepage was recorded.

    The Contractor proceeded to place the preformed drainage liner and a thin sacrifical chicken wire mesh against the open face. Prior to stabilizing the face with a thin flashcoat of sacrificial shotcrete, the open face collapsed due to excessive ground vibration from the Prime Contractor's bulldozer driving nearby. The collapsed section started 20 feet from the west end of the bridge abutment and extended westerly about 15 feet along the open face.

    As the open face collapsed, it undermined the soil nailed portion above; the soil material behind the structural shotcrete facing immediately above the collapsed open face was brought down creating a void behind the structural shotcrete face. The void extended vertically about nine inches above the level of the installed third row of nails. The collapsed soil in the upper soil nailed portion uncovered one production nail, and it was discovered that the nail did not have any grout cover. The ungrouted nail length behind the shotcrete face measured approximately 1.5 feet. No attempt was made to further investigate the exact ungrouted length for fear of causing more soil collapse.

    The structural shotcrete facing in the nailed portion above was surveyed. No signs of distress or cracks were observed.

    The Contractor immediately formed the collapsed face with plywood and placed a temporary flashcoat of shotcrete against the forms. Additional shotcrete was placed behind the forms to replace the collapsed material and was brought to the top of the formed face. The Contractor was directed to place reinforcing rebar through the forms and anchor it into the shotcrete mass behind the forms; the rebar was to act as shear anchors to transfer the dead weight of the structural shotcrete facing to the shotcrete mass that replaced the collapsed material at the formed face.

    A three-inch diameter hole was drilled through the existing structural shotcrete face, nine inches above the third row of nails. A fluid cement grout was injected through the hole to fill the void behind the existing shotcrete face. The grouting was done in stages to allow the grout to set and to prevent the buildup of excessive fluid pressure behind the existing structural shotcrete face.

    Our suspicions that the process of shotcreting the unfilled top portion of the nail holes, as discussed in (9) above, may have caused the sides of the holes to collapse were proven right.

11. On January 8, 1991, the Contractor excavated the fourth lift corresponding to the fourth row of nails. It was discovered during the excavation that the westerly most two nails corresponding to the third row of nails were not drilled, installed and pressure grouted. The Contractor claimed that the inspector directed him to delete the two nails in question.

The Contractor was directed to proceed with the excavation of the fourth lift provided the ground be lowered no more than 1.5 feet below the third row of nails at the location where the two nails were omitted pending our investigation into their deletion.

The general contractor overexcavated the open face at the westerly end of the wall where the ground was lowered 5.5 feet below the missing third row of nails. The Contractor later stabilized the open face with a flashcoat of temporary shotcrete reinforced with chicken wire mesh.

The westerly most 12-foot soil face, excavated and stabilized with a flashcoat of temporary shotcrete, collapsed overnight. A four-foot wide wedge of soil behind the excavated face slid into the excavation. The adjacent soil nailed portion of the wall, including the shotcrete facing, showed no sign of distress.

The collapse of the open face was attributed to (1) over excavation, and (2) the failure of the prime contractor who performed the excavation to grade the sloping ground at the top of the wall to the required 2:1 slope and remove the excess material prior to excavating the lift.

The Contractor drilled and pressure grouted two temporary sacrificial nails to stabilize and anchor the failure zone. He later formed the wall at the collapsed section with plywood sheathing, drilled through the forms and pressure grouted the permanent nails corresponding to both third and fourth row of nails. The steel welded wire mesh was erected followed by the placement of the structural shotcrete facing. Finally, a lean concrete mix was placed behind the forms to replace the collapsed soil material.

12. The general contractor severed a telephone line running perpendicular to the wall alignment while excavating the final lift at the east end of the wall. The estimated repair cost was $10,000 and was paid by the Contractor. A blockout was formed around the telephone line and the shotcrete facing placed.

13. The drilling at nail locations corresponding to the fifth row of nails at Zone C intercepted the ground water table. Surging sand formed a plug at the low end of the drilled holes which prevented the contractor from lowering the production nails past the sand plug. Pressured grout was used to blow the casing free, followed by the installation of the nails to full depth.

14. Following the excavation of the fifth lift, the strain gauge data readout at the instrumented section under the bridge abutment indicated bending at the nail head. The pair of strain gauges located ± 2.5 feet from the nail head at each of the top three rows of nails showed an increase in tensile

strains at the top gauge (located at the 12 o'clock position) and a decrease in tensile strains at the bottom gauge (located at the six o'clock position). Bending was most pronounced at the second and the third row of nails where the bottom gauges at the six o'clock position showed a strain reversal from tension to compression.

Typically, in Soil-Nailed walls, much of the dead weight of the shotcrete facing, should be transferred to the soil by friction at the soil-shotcrete interface. To that effect, following the placement of the shotcrete facing, the bearing plate was fastened to the soil nail with a minimum 100 lbs./foot torque to insure close contact between the soil and the shotcrete facing. Bending at the nail head indicated the soil nails were supporting a portion of the shotcrete facing dead weight.

During construction it was observed that the chicken wire mesh, used to reinforce the flashcoat of shotcrete, was preventing the sacrificial flashcoat from bearing directly against the open soil face at locations where the mesh in turn was not in contact with the open soil face. This problem was most pronounced at locations where the open face sloughed. The Contractor first placed rolled up segments of chicken wire mesh in the sloughed zone; he then placed the flashcoat against the chicken wire mesh leaving a temporary void behind the flashcoating, and filled the voids with shotcrete when the structural shotcrete facing was later placed.

It was suspected that the Contractor was not successful in filling the voids completely which reduced the available shotcrete to soil contact surface. That in turn may have caused the soil nails to carry a larger portion of the weight of the shotcrete facing thus introducing unnecessary bending at the nail head.

15. The Contractor encountered a 12-foot section of an abandoned pipe pile running parallel to the wall alignment, large pieces of concrete, and a thick abandoned concrete slab at the open soil face during various stages of excavation. The contractor saw cut all protrusions flush with the soil face and incorporated into the wall the embedded sections to avoid disturbing the open face.

16. The Preproduction shotcrete core samples did not meet specifications. A total of 16 three-inch diameter cores were extracted by the Contractor from two preproduction shotcrete test panels. The test panels were 24" x 24" x 8" thick. Four sets of four cores each were tested for the 4, 7, 14, and 28-day compressive strength. See Appendix B for test results.

    The test results were erratic; some cores tested higher at the 14-day break than at the 28-day break.

    It was suspected that the lower strength at the 28-day break than the 14-day break was attributed to minute cracks in the cores, being the result of: (1) poor workmanship in coring the test panel, and (2) core drilling the test panels only three days following shotcreting.

17. Compressive tests of field and lab cured shotcrete cores samples were not consistent. Production testing of the structural shotcrete facing for compressive strength was performed to insure that the newly placed shotcrete had reached 25 percent of its required 28-day minimum compressive strength (4000 psi) prior to further excavation.

Six concrete test cylinders were prepared for compressive testing at each excavation lift. Three cylinders were lab cured and the remaining three cylinders were placed under heavy insulation blankets and were cured on site to approximate actual field curing conditions. Test results are reported in Appendix B.

The lab cured cylinders tested substantially higher than the field cured cylinders at the first and second excavation lifts, where wet burlap and heavy insulation blankets were used to cure the shotcrete face; 35 percent higher strength at the second excavation lift.

The average five-day strength of the lab cured cylinders, corresponding to the third excavation lift, was 5140 psi, 14 percent higher than the field cured cylinders. This may be attributed to the Contractor having modified his curing process by adding a protective tent and using heat blowers during the night hours. The curing process was interrupted, and the tent uncovered to perform drilling and nail installation during the day time hours.

The average five-day strength of the lab cured cylinders, corresponding to the fourth excavation lift, was 4147 psi, 6.5 percent lower than the field cured cylinders. It is believed that the field cured cylinders on the fourth lift out performed the lab cured ones because: (1) the Contractor continued using the protective tent and the heat blowers during the night hours; (2) the protective tent was partially uncovered the day following the shotcrete placement to perform drilling and nail installation, followed by two consecutive days of uninterrupted curing due to the weekend holiday; (3) lab curing started in the afternoon on the day following the shotcrete placement.

18. Some 28-day compressive tests of the production shotcrete failed to meet the required compressive strength of 4000 psi. Twenty-eight days after the final shotcrete layer was applied, the Contractor extracted six cores from the completed wall (per specification requirement). The cores were three-inches in diameter and the full thickness of the wall. The cores were tested for compressive strength in accordance with ASTM C 42. See Appendix A for complete test results.

Two cores, Nos. 2 and 5B, tested at a 28-day strength of 2950 psi and 3330 psi, respectively. The remaining four cores tested well above the minimum required 28-day strength of 4000 psi.

Both cores, Nos. 2 and 5B, had no visible physical defects including reinforcing steel, that could have resulted in their premature failure. All six cores were three inches in diameter and six inches long and were carefully trim sawed on both ends. Approximately three inches of each carefully trim sawed on both ends. Approximately three inches of each six-inch core was from the first layer of shotcrete placed during the excavation process while the remaining core length was from the second layer of shotcrete.

It was suspected that the core length portion corresponding to the first layer of shotcrete may have resulted in the premature failure of both cores. This was based on the following observations:

a. Higher quality control and better curing was achieved after the second layer of shotcrete was placed. A curing compound was applied to seal the shotcrete face after the second layer was applied. The curing process of blowing warm air under the protective tent was uninterrupted for several days.

b. The curing process of the first layer of shotcrete was regularly interrupted to allow the drilling and the installation of the soil nails. This action may have led to the lower compressive strength.

A review of the construction process also revealed the following:

a. The two failed cores, Nos. 2 and 5B, and Core No. 6A (which tested at 6280 psi) correspond to the shotcrete facing placed at the third excavation lift.

b. Core No. 6A is located at the westerly half of the wall, where drilling and soil nail installation preceeded the placement of the structural shotcrete facing.

c. The curing process was interrupted on the first day following the shotcrete placement. The protective tent and the heavy insulation blankets were removed at Core No. 2 wall location to perform drilling and nail installation. The protective cover was kept in place at cores No. 6A and 5B wall locations on that day, maintaining a better curing temperature than at core No. 2 location.

d. The heating process was continued overnight but was interrupted the second day to perform additional drilling and soil nail installation.

e. On the third day the protective cover was removed at core No. 5B wall location to drill and install the soil nails. On that day, a second-stage grout was performed at boreholes corresponding to core No. 2 wall location. The protective cover was kept in place at core No. 6A wall location.

It was suspected that the lower compressive strength of core No. 2 was attributed to the interruption in curing during the day time hours and uncovering the shotcrete face immediately one day following the shotcrete placement. Maintaining the protective cover for a minimum of two days followed by uncovering the face on the third day allowed the shotcrete to develop a higher strength at core No. 5B wall location. A much higher strength was achieved at core No. 6A wall location where the protective cover was kept in place for several days and heat blowers were used during the night hours.

19. The excavation was performed by the Prime Contractor. The construction procedure submitted by the specialty contractor called for the excavation to be performed by "others" under his direction.

    Prior to any soil nailing activity, the Prime Contractor partially dressed the side slopes of the bridge embankment; the slope was overexcavated. This overexcavation resulted in the deletion of the top row of nails at the wall locations immediately to the east and west of the bridge. This deletion included three nails on the east end and four nails on the west end for a total of seven nails.

    During each excavation lift, the excavator consistently overexcavated the soil. At one excavation lift, the inspector directed the excavator to replace 18 inches of overexcavated material.

20. Following placement of the final shotcrete layer, the Contractor removed the scoring strips. Extensive rock pockets were visible behind the scoring strip locations. The Contractor removed the rock pockets and refinished the face at those locations.

    As expected, vertical shrinkage cracks were reported within the vertical scorings at ±32 foot spacing at the location of the expansion joints. The vertical cracks began at the top of the wall, crossed the horizontal scoring and extended downward approximately the full height of the wall.

21. Following completion of the wall, it was discovered that the Dywidag bars used on the job had an epoxy coating thickness of 8 mils; the Specification minimum coating thickness requirement was 14 mils.

10.0 CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions:

Soil nailing is a viable lateral earth support system used to retain an existing bridge fill embankment, and to allow for a roadway widening under the bridge.

The adoption of the soil nailing technique to support the excavated embankment face under the Oregon Slough Bridge enabled the project to proceed without disrupting bridge traffic. The soil nailing technique offered the following advantages over other types of wall:

a. Soil nailing is better suited than Tiedback walls for a roadway widening under an existing bridge. Soil nailing required no soldier pile installation; therefore, holes did not have to be cut through the existing bridge deck.

b. Ease of construction and reduction in construction time; no soldier pile installation was required, construction equipment was small scale and mobile allowing the contractor to work in low overhead clearance conditions.

c. Soil nailing is a flexible form of construction. The sequence of construction was altered, and the modification easily adapted during construction to fit soil site conditions.

The permanent exposed shotcrete facing, placed full height from the bottom up, was aesthetically pleasing. The shotcrete subcontractor did an excellent job of: (1) finishing the face with hand-held steel trowels, (2) texturing the face with hand-held rubber floats, and (3) heat curing the shotcrete face for several days.

The architectural treatment consisted of vertical and horizontal scoring. This detailing matched the architectural treatment specified on several conventional cast-in-place walls being built in the vicinity. The placement of the scoring strips, prior to shotcreting the final application, proved to be a challenging task to the contractor. It took nine working days to place and adjust the strips to the required grade.

Locating the vertical scoring at the expansion joints to control shrinkage cracks proved successful. The vertical cracks were confined at the location of the expansion joints and extended vertically within the scoring which made them less visually obtrusive.

Key areas identified to have significant impact on the constructability and long-term performance of Soil-Nailed walls are:

1. Excavators and General Contractors generally do not have experience with soil nailing, and they do not appreciate how sensitive Soil-Nailed walls are to overexcavation.

2. The excavation process must be carefully coordinated with the soil nailing Contractor.

3. The excavation work adjacent to the Soil-Nailed wall should be directed by the soil nailing Contractor when it stands to affect the constructability of the wall.

4. Relatively competent ground can lose significant strength with time. Unsupported cohesionless ground should not be left exposed overnight. Special treatment of the exposed open face may be necessary to keep it from sloughing following excavation.

5. Vibrations from nearby construction equipment can cause the open soil face to collapse and to potentially undermine to soil nailed portion above.

6. Where abandoned utility pipes, large pieces of concrete and other debris are encountered at the cut face, sloughing of the open face can occur due to the looseness of the soil if the excavator attempts to dislodge the protruding objects.

7. Variations in soil conditions can impact the constructability of a Soil-Nailed wall. A thorough geotechnical investigation should be performed and all existing information made available to the specialty contractor to allow proper selection of: drilling method, grouting method and final drilled hole diameter.

8. Soil nailing is a in-situ soil improvement concept; the existing soil is the primary element of the structural system, while the nails and the shotcrete improve the strength of the structural system. Therefore, the contractor's method of installation, sequence of construction, and the excavation process have an important effect on the wall strength during construction.

9. Proper curing of the shotcrete facing is critical for both the first shotcrete application and following additional shotcrete applications.

   Qualified soil nailing specialty contractors are best at identifying field conditions which affect the constructability of the wall and modifying their construction procedures accordingly, but are not necessarily well versed on the curing methods of shotcrete. To that effect, the Contractor should be asked to discuss in writing prior to any soil nailing activity: (1) the curing method to be used immediately after the placement of the shotcrete facing, and (2) his procedure to maintain a curing temperature over 40°F and hence allow the shotcrete to properly cure and fully develop its required compressive strength and durability.

10. Field cured shotcrete cylinders are not truly representative of the actual field curing of the shotcrete face. Thin shotcrete applications will exhibit lower strength than the six-inch diameter test cylinders due to their large exposed surface area which provides for rapid loss of heat of hydration. Therefore, proper curing of the shotcrete face and maintaining a curing temperature above 40°F is critical.

11. Perhaps most important, recognizing the limited U. S. experience to date with permanent Soil-Nailed walls, it is in the State's best interest to contract the work to qualified specialty contractors.

    The contractor should be prequalified in order to assure a reliable installation and adequate long-term performance with prequalification requirements based on experience in <u>permanent</u> soil nailing. These contractors are best able to identify field conditions which affect the constructability of the wall and to quickly modify their construction procedures accordingly.

<u>Recommendations:</u>

The use of the soil nailing technique as a lateral earth support system is recommended as an alternative to conventional Tiedback walls.

The following are recommendations to update the Standard Specifications and to aid in preventing the construction problems listed earlier in this report:

1. The load testing acceptance criteria in Subsection 614.41 (a-3.a) of the Specifications should be revised by deleting the third criterion, "Total movement measured at the maximum test load does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length".

2. The use of an electronic load cell to monitor the loads on the nails during the tests should be emphasized as specified in Subsection 614.41 (a-1) of the Specifications.

3. When pressure grouting is specified and the grout is injected through the temporary casing, the pressure of the grout should be measured with a pressure gauge mounted on the drill head. In addition, the withdrawal of the casing should not begin until the grout pressure inside the casing rises to the required level. The grout pressure should be maintained during casing pullout.

4. Soil nailing materials should be carefully inspected to ensure compliance with the specifications.

5. Plastic centralizers with adequate out-to-out diameter should be used to keep the nails centered in the hole. The practice of compressing the centralizers to increase their diameter should be discouraged.

6. The requirement to overlap the steel welded wire mesh fabric at all seams should be modified. Adjacent wire mesh panels should be spliced in both directions using regular rebar having adequate splice length and spacing.

7. The use of shotcrete or dry packing to fill the ungrouted upper portion of the nail holes should not be allowed following first-stage grouting. The Contractor should perform a second-stage grout, and if necessary a third stage grout to insure that the nail hole is completely filled with grout.

Second- and third-stage grouting should be performed with a grout tube inserted at the bottom end of the ungrouted segment and used to tremie grout the nail hole. The grout should be allowed to backflow to ensure that contaminants trapped at the bottom end are completely flushed out.

8. Drying and the resulting reduction of apparent cohesion in moist cohesionless soil material can cause the sides and top of the unfilled portion of the nail holes to collapse if left uncased for an extended period of time.

   Second-stage and, if necessary, third-stage grouting should be performed as soon as possible following first-stage grouting.

9. Corrosion studies for Tiedback walls indicate that the upper portion of the tendon, near the excavated face, is the most susceptible for corrosion due to exposure to oxygen and water.

   The use of a "Heat Shrink Sleeve" similar to Schnabel's corrosion protection system should be further investigated and potentially specified provided it does not act as a grout bond breaker. An alternate would be to require nail bars fully encapsulated along their entire length.

10. A stroke counter should be used to measure the quantity of grout injected into each borehole.

11. After wall excavation is completed to the specified grade, the exposed face of the first shotcrete application should be cleaned by pressure washing or by sandblasting to remove debris prior to the placement of the final shotcrete application.

12. When a flashcoat of shotcrete is used to stabilize the open face, consider the use of epoxy-coated rebar to positively anchor the shotcrete facing at each excavation lift to the ground behind the facing. The anchor bars should be installed following the placement of the flashcoat and should have adequate embedment depth and spacing to ensure that the dead weight of the facing is transferred to the soil behind.

    The Contractor should be required to replace sloughed material at the open soil face with sacrificial shotcrete during the flashcoating process.

13. During cold weather shotcreting, the curing process should not be interrupted during the day hours. The soil nailing activity should be performed under a protective tent to ensure that a curing temperature above 40°F is being maintained at all times.

14. Consider extending the vertical scoring to the top of wall at the expansion joint locations. This modification will allow the vertical scoring to intercept the vertical shrinkage crack starting at the top of wall, making it less visually obtrusive.

15. Casting-in-place the exposed permanent facing should be specified as an option to shotcreting the final application.

The minimum cast-in-place wall thickness should be six inches if a standard concrete mix is used. This minimum thickness is a must to attain good concrete consolidation by mechanical vibration. A thinner reinforced, cast-in-place wall face can be provided if a pea-gravel mix designed with super plasticizers is used. This will make the mix fluid to be self consolidating.

The connection between the nail head assembly and the concrete facing can be made by extending the nail and bolting an additional plate to provide the anchorage in the concrete. Another option is to drill into the first shotcrete layer and install short resin-bonded anchors to act as shear connectors.

APPENDIX A: CONSTRUCTION PHOTOGRAPHS

South end abutment of the Oregon Slough Bridge prior to any soil nailing activity.

Lift excavation for first row of nails.

Placing prefabricated drainage liner and chicken wire mesh prior to stabilizing the open face with a temporary flashcoat of shotcrete.

Steel welded wire mesh reinforcement corresponding to first shotcrete application. Notice (1) blockouts at nail locations;(2) preformed expansion joint filler at the expansion joint location; and (3) stretched piano wires to control face thickness during shotcrete placement.

Nozzleman placing first structural shotcrete application; helper with an air blow pipe keeping work area free of rebound material.

Drilling and placing temporary casings at nail locations; Dywidag bars were later inserted and the boreholes pressure grouted prior to placing the shotcrete application.

Inserting a dywidag bar in a cased borehole.

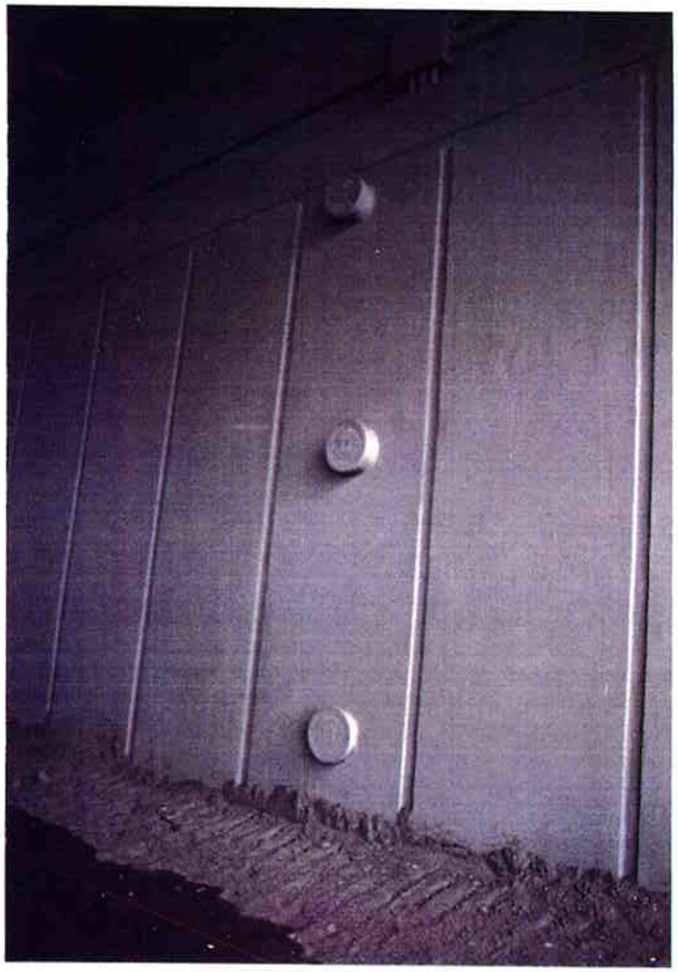
Installed nail following first-stage pressure grouting.

Contractor "braking casing" during the grouting process of the borehole.

Erecting steel reinforcement for the second shotcrete application.

Placing the second shotcrete application (4.5 inches thick) under protective cover.

Texturing the wall face with a hand-held floating rubber trowel.



South end abutment of the Oregon Slough Bridge following the painting of the Soil-Nailed wall face. Notice blockouts at instru ted nail location .
men

The westerly most 12-foot wall segment collapsed overnight due to over- excavation.

The Contractor formed the wall at the collapsed section, drilled through the forms to install temporary casings, pressure grouted the permanent nails, erected the welded wire mesh and placed the structural shotcrete facing. A lean concrete mix (one bag cement) was placed behind the forms to replace the collapsed soil.

Soil sloughing at the open face during the placement of the chicken wire mesh.

Collapsed face was formed with plywood and shotcrete was placed behind the forms to replace the sloughed material.

Abandoned steel pipe encountered at the open face during lift excavation.

APPENDIX B: SHOTCRETE COMPRESSIVE STRENGTH RESULTS

B-1 PREPRODUCTION TEST PANELS
SHOTCRETE COMPRESSIVE STRENGTH

	PANEL A		PANEL B	
AGE IN DAYS	CORE A 1	CORE A 2	CORE B 1	CORE B 2
4-Day Strength	1430 psi	1440 psi	1750 psi	1470 psi
7-Day Strength	2280 psi	2650 psi	1720 psi	2300 psi
14-Day Strength	2290 psi	3420 psi	3760 psi	3740 psi
28-Day Strength	3560 psi	3570 psi	3680 psi	3780 psi

B-2 PRODUCTION TESTING

SHOTCRETE COMPRESSIVE STRENGTH

EXCAVATION LIFT	FIELD CURED CYLINDERS		LAB CURED CYLINDERS	
	Age In Days	Compressive Strength (psi)	Age In Days	Compressive Strength (psi)
1	6	4030	5	4890
	6	4100	7	5620
	6	4260	7	5500
2	5	4050	5	5410
	5	4030	5	5390
	5	4080	5	5520
3	3	3570	5	5290
	5	4580	5	5030
	5	4420	5	5100
4	5	4420	5	4210
	5	4400	5	4110
	5	4420	5	4120

APPENDIX C: SHOTCRETE AIR CONTENT RESULTS

I. Background :

The main purpose of this effort is to provide Federal Highway Administration - Region 10 and Oregon DOT information on the air content distribution as determined by the Image Analysis Procedure for the Shotcrete project. Two 3-inch cores were extracted from the I-5 swift-delta soil nail wall project in Portland Oregan. The total shotcrete wall thickness was between 8 and 9 inches. This thickness was applied in two layers, each layer comprising 4 to 4.5 inches in thickness. Because of this application, two slices from each core were prepared and tested, one at about 2 to 3 inches and the other at about 6 to 8 inches from the face of the wall. The slices were polished and prepared for air content and parameter determination in accordance with a modified ASTM C457. The modification included an additional step to prepare the sample for Image Analysis Examination; mainly : to darken the test area and apply special white powder to fill the air voids. The test was conducted on all 4 samples twice, once at the Turner - fairbanks (TF) Research and development laboratory in Washington D.C, and the second time by the Omnicon 3600 Image analyzer available at the DP-75 mobile laboratory. However, since both results were lower than the expected values judged by visual examination with catalogue comparison charts, samples # 5a&b were retested using a more elaborate petrographic microscope to verify the above results.

II. Results and Analysis :

The air content determination using TF facilities are summarized in table 1. below. It should be noted that only the total air content was determined as the equipment was not set to determine the entrained/entrapped air void content

TABLE 1. Percent total air content by the Image Analysis Method

Sample #	Slice depth*	Test #1	Test #2	Average
3a	2" - 3"	1.9	1.0	1.45
3b	6" - 7"	2.1	1.6	1.85
5a	2" - 3"	1.9	1.4	1.65
5b	7" - 8"	2.2	1.1	1.65

* From the core face matching the wall face side

The results obtained using the Omnicon 3600 was similar to the above found in table 1. Although these results appear to be consistent and indicate minimum variations in air voids content between the two concrete thickness application, however, the total air content seems to be extremely low for the shotcrete process. Based on the above cited observation and to eliminate doubts about the results, slices # 5a&b were retested in an independent laboratory under the supervision of the DP - 75 staff using a petrographic microscope. test results were as follows:

	Sample #	
	5a	5b
% entrained air	3	2.6
% entrapped air	1.2	2.4
% total air	4.2	5.0

III. Conclusion :

Based on the above results, the following could be concluded:

1. The total air content in the submitted samples is between 4% to 5% . This result is satisfactory for hardened concrete properties.

NOTE: Total air content test results on the fresh shotcrete were as follows:

Deep application : 5.0 %
Shallow application: 7.5 %

APPENDIX D: CONTRACT SPECIFICATIONS

SECTION 614 - NAILED SOIL RETAINING WALLS

614.01 Scope - This work shall consist of constructing soil nail walls in accordance with the Standard Specifications, these special provisions and in reasonably close conformity with the lines, grades and dimensions shown on the plans or established by the Engineer.

(a) Soil nailing - Soil nailing shall consist of staged excavation of the existing south end slope from the top down to the layer limits shown in the plans, placing preformed permeable drainage fabric and welded wire fabric, applying air blown structural shotcrete, drilling holes at the inclination shown in the plans, and placing and grouting steel bars (soil nails).

The Contractor shall select the nail installation method, the maximum hole diameter, and the grouting method. The Contractor shall install nails that meet the design requirements shown on the plans and the testing requirements specified herein. Once the Contractor selects a nail installation method, the Contractor shall not change the system without written approval of the Engineer.

(b) Shotcreting - This work shall consist of constructing a pneumatically applied shotcrete blanket on soil surfaces at locations shown in the plans or as directed by the Engineer. These specifications refer to premixed cement and aggregate pneumatically applied by suitable equipment and competent operators.

614.02 Prequalification of Contractor and Contractor's Personnel - The Contractor performing the work described in this special provision shall have at least 5 projects successfully completed in the last 3 years involving construction of earth reinforced walls using permanent soil nails. The Contractor's personnel shall meet the following requirements:

(a) Supervising engineer - The Contractor shall assign an engineer to supervise the work with at least 3 years of experience in the design and construction of permanently nailed structures.

(b) "On-site" supervisors - "On-site" supervisors shall have a minimum of 1 year of experience installing permanent soil nails with the approved Contractor.

(c) Drill operators - Drill operators shall have a minimum of 1 year of experience installing permanent soil nails with the approved Contractor.

(d) Foremen - The foremen shall have performed satisfactory work in similar capacities elsewhere for a sufficient length of time, as determined by the Engineer, to be fully qualified to perform their duties. Foremen shall have at least 2 years of experience as a structural shotcrete nozzleman.

(e) Nozzlemen - Nozzlemen shall have served at least 1 year of apprenticeship on similar applications as determined by the Engineer and with the same type of equipment. Prior to the start of shotcreting on this project, the nozzlemen shall in the presence of the Engineer, demonstrate their ability to apply shotcrete of the required quality on 2 test panels. Two satisfactory test panels, described under 614.33, shot in a vertical position for each mix used during the course of the work shall be the minimum qualification test for nozzlemen before they will be permitted to place shotcrete in permanent construction.

(f) Delivery equipment operators - The delivery equipment operators shall have performed satisfactory work in similar capacities elsewhere for a sufficient length of time, as determined by the Engineer, to be fully qualified to perform their duties.

The Contractor shall not use consultants or manufacturer's representatives in order to meet the requirements of this subsection.

614.03 Submittals - No later than the preconstruction conference, the Contractor shall submit, in writing, resumes documenting that the Contractor performing the work described in this Section and the Contractor's personnel, have the required experience as set forth in 614.02. For the Contractor performing the work, a brief description of each project and a reference shall be included for each project listed. As a minimum, the reference shall include an individual's name and current phone number. For the Contractor's personnel, the list shall contain a summary of each individual's experience and it shall be complete enough for the Engineer to determine whether or not each individual has satisfied the qualifications of 614.02.

The Engineer will approve or reject the Contractor's qualifications and staff within 15 working days after receipt of the submission. Work shall not be started on the nailed soil wall nor materials ordered until approval of the Contractor's qualifications are given. The Engineer may suspend the soil nailing work if the Contractor substitutes unqualified personnel for approved personnel during construction. If work is suspended due to the substitution of unqualified personnel, the Contractor shall be fully liable for additional costs resulting from the suspension of work and no adjustment in contract time resulting from the suspension of work will be allowed.

The Contractor shall submit, in writing, to the Engineer not less than 15 working days prior to start of wall excavation, the proposed schedule and detailed construction sequence; proposed method of excavation; proposed drilling methods and equipment; proposed hole diameter; grout and shotcrete mix designs; and nail steel corrosion protection details.

The shotcrete mix design shall be prepared, tested, and submitted for approval by the Engineer. The results of compatibility testing done in accordance with ACI 506.2 shall also accompany this submission to verify that any proposed admixtures to accelerate set are compatible with the cement to be used.

The Contractor shall submit certified mill test results and typical stress-strain curves along with samples from each heat, properly marked, for the nail steel to the Engineer for approval. The typical stress-strain curve shall be obtained by approved standard practices. The guaranteed ultimate strength, yield strength, elongation and composition shall be certified.

The Contractor shall submit the procedures for placing the grout to the Engineer for approval.

The Contractor shall submit detailed plans, as specified in 614.41, for the method proposed to be followed for the permanent soil nail testing to the Engineer for approval prior to the tests. This shall include all necessary drawings and details to clearly describe the methods proposed.

The Contractor shall submit to the Engineer for review and approval calibration data for each load cell, test jack, pressure gage, stroke counter on the grout pump, and master gage to be used. The calibration tests shall have been performed within 60 calendar days of the data submitted. Testing or work shall not start until the Engineer has approved the load cell, jack and pressure gage calibrations.

Materials

614.11 Materials:

(a) Reinforcement soil nails - Soil nails shall be epoxy coated for corrosion protection. Epoxy coating shall conform to AASHTO M 284 in accordance with 709.05(d) of these special provisions, found under Section 505. The coating thickness shall not be less than 14 mils or greater than 18 mils. Epoxy coat only nonthreaded portion of the nail. The exposed threaded portion of the nail shall be epoxy painted after the installation and the tightening of the nut according to 709.05 of these special provisions.

Soil nails shall be clean and free of oil, grease and other foreign substances that would destroy or reduce bond.

Soil nails shall be installed using plastic centralizers to keep the nails centered in the hole. Wood shall not be used. Centralizers shall be spaced no further than 10 feet apart. Any other method selected by the Contractor shall be approved, in writing, by the Engineer.

The soil nails shall not be spliced. Soil nails shall be threaded on one end a minimum of 6 inches. Soil nails shall be coarse threaded with a diameter 1/8-inch less than the nominal diameter of the bar.

If the resisting soil nails fail to develop the pullout resistance (kips) specified on the plans and, in the opinion of the Engineer, all work conformed with the best general practices, lengthening of the nails by approved mechanical splicers will be allowed. The splicing shall conform to the provisions in 505.35 "Splicing" of these special provisions. Nail splicers shall develop the ultimate tensile strength of the bars without evidence of failure. All work associated with furnishing, placing, grouting and joining of the spliced length of the soil nail shall be done at the Contractor's expense.

The bearing plate shall be as shown in the plans, conforming to ASTM A 36.

The nuts shall conform to ASTM A 563, Grade B Hexagonal. The nut shall be fitted with a special washer such that the nut will bear uniformly on the plate.

(b) Welded steel wire fabric - Unless shown otherwise in the plans, welded steel wire fabric shall be galvanized meeting the requirements of ASTM A 185. The welded wire fabric shall be clean and free from loose mill scale, rust, oil, or other coatings interfering with bond.

Welded deformed steel wire fabric of equal or greater diameter and yield strength may be substituted for the welded steel wire fabric. Welded deformed steel wire fabric shall conform to the specifications of ASTM A 497.

Fabric shall be overlapped at least 2 mesh dimension at all seams. Tie wires shall be bent flat in the plane of the fabric and shall not form large knots.

(c) Grout - The grout to be used for soil nailing shall consist of a pumpable mixture of Types I, II, or III portland cement, sand and water. Chemical additives shall not be allowed.

Cement should be fresh and should not contain any lumps or other indications of hydration. Water for mixing grout should be potable.

The grout shall be capable of reaching a cube strength of 3500 psi in 7 days as per AASHTO T 106.

(d) Shotcrete - Shotcrete shall be composed of portland cement, fine and coarse aggregate, and water. Wet-mix shotcrete shall be used. The shotcrete shall be reinforced with welded wire fabric.

Shotcrete shall comply with the current requirements of the American Concrete Institute's ACI 506R, "Guide to Shotcrete", and ACI 506.2, "Specifications for Materials, Proportioning, and

Application of Shotcrete". The ACI specifications and recommendations are hereby made a part of this Section as specification requirements except as modified herein.

Materials in the shotcrete shall conform to the following requirements of the Standard Specifications modified and/or supplemented as follows:

Portland Cement (Type I, II or III)	701.01
Air-Entraining and other Chemical Admixtures	701.03
Curing Materials	701.05
Water	701.02
Fine Aggregate	703.01(d)
Coarse Aggregate	703.02(d)

(d-1) Prepackaged product - Premixed and prepackaged concrete product specifically manufactured as a shotcrete product may be provided for "on-site" mixed shotcrete if approved by the Engineer. The packages shall contain cement and aggregate conforming to the materials portion of this specification.

(d-2) Admixtures - Admixtures shall not be used without permission of the Engineer. If admixtures are used to entrain air, reduce water-cement ratio, retard or accelerate setting time or accelerate the development of strength, they shall be used at the rate specified by the manufacturer and must be compatible with the cement used. Use of calcium chloride accelerating agent will not be permitted. When used, admixtures shall be dissolved in water before introduction into the mixture. Wet-mix shotcrete shall have 7.5 (plus or minus 1) percent air meeting the requirements of 701.03.

(d-3) Water - In addition to the requirements set forth in 701.02, the water used in the shotcrete mix shall be free of elements which cause staining.

The Contractor shall be responsible for the design of shotcrete mixes and for the quality of shotcrete placed in the work.

(e) Aggregate - Aggregate used in shotcrete shall have a combined gradation of fine and coarse aggregates meeting the following gradation requirements:

Sieve Size	Percent Passing by Weight
1/2"	100
3/8"	90-100
No. 4	70-85
No. 8	50-70
No. 16	35-55
No. 30	20-35
No. 50	8-20
No. 100	2-10

(f) Preformed permeable drainage liner - The preformed permeable liner shall consist of 12-inch wide MIRADRAIN 6000, AMERDRAIN 200, or approved equal, fully wrapped with filter fabric.

Should the fabric on the preformed liner be torn or punctured, the damaged section shall be replaced completely or repaired by placing a piece of fabric that is large enough to cover the damaged area and is at least 6 inches on each side of the damaged area.

Construction

614.31 Construction Sequence:

(a) General - As-built plans for the existing pier 10 are available in the Project Manager's office. The Contractor shall verify the actual location of the pipe piles in the field prior to any drilling operations. Any damage to the existing piles shall be remedied to the Engineer's satisfaction at the Contractor's expense.

(b) Excavation - Excavation for the nailed soil wall shall conform to the provisions in Section 251 and this subsection.

The excavation shall proceed from the top down in a horizontal lift sequence with the ground level excavated no more than 18 inches below the level of the next uninstalled row of nails. Only the amount of excavation that can be covered with shotcrete and nailed during a work shift shall be performed.

Each stage of excavation shall have all preformed permeable drainage liners, all soil nails and appurtenances installed, the required number of production nails tested and a 6.5-inch shotcrete cover placed over the excavation before excavation of the next lift is to begin. After a lift is excavated, the cut surface shall be trimmed to line and grade to provide adequate support and to assure the design thickness of the shotcrete. The cut surface shall be cleaned of all loose material, mud, rebound and other foreign matter that could prevent or reduce shotcrete bond.

The tolerance on the soil cut shall be such that overexcavation does not damage overlying shotcrete sections by undermining or other means. Costs associated with additional thickness of shotcrete due to overexcavation or irregularities in the cut face shall be borne by the Contractor.

(c) Drains - After each excavation lift, and before any shotcrete is placed, place the preformed permeable drainage liner against the exposed face at the required spacing shown in the plans. The drainage liner installed after each excavation lift shall be hydraulically connected with the drain installed in the previous lift.

Four-inch weep holes shall be installed at locations and at the spacing shown in the plans. Weep holes shall be protected during shotcrete application to prevent formation of a plug. A continuous drain pipe wrapped with gravel drain material shall be provided as shown in the plans.

(d) Inner welded steel wire fabric layer - After each excavation lift, place the inner welded wire fabric layer providing cutouts and markers at nail locations shown in the plans. The wire fabric shall be attached firmly in proper position to prevent vibration while the shotcrete is being applied. The fabric shall be positioned in such a manner that the fabric is not in physical contact with the nail once the nail is installed.

(e) Initial shotcrete layer - The sequence of wall construction is based on short duration of soil standup. After each excavation lift and the placement of the preformed permeable liner and steel welded wire fabric, place the initial shotcrete lift to the lines and grades shown in the plans. The Engineer may allow an alternate sequence of construction if the Contractor can demonstrate sufficient duration of soil standup is achievable with the construction methods, soil/groundwater and weather conditions to allow nail installation prior to placing initial shotcrete layer.

(f) Nail installation - After placement of the initial shotcrete layer, holes shall be drilled through the initial shotcrete layer. The method used for drilling the holes shall be chosen by the Contractor and approved, in writing, by the Engineer. Subject to the Engineer's approval, the Contractor may place blockouts at nail locations prior to placing the initial shotcrete layer. The location, length and minimum diameter of the holes shall be as shown in the plans. Holes shall be cleaned to remove all material resulting from the drilling operation or any other material that would impair the strength of the nails. Water or other liquids shall not be used to flush cuttings, but air may be used. Subsidence, damage to the shotcrete face or any other detrimental impact from drilling shall be cause for immediate cessation of drilling and repair of all damages at the Engineer's direction and the Contractor's expense.

After drilling, the nail shall be installed in the hole, any casings used to stabilize the hole shall be removed during the grouting operation. Each soil nail shall be secured with a steel plate as shown in the plans conforming to ASTM A 36. Each plate shall be fastened to the soil nail with a nut and shall be secured wrench tight with a minimum 100 ft.-lbs. torque after the initial shotcrete layer has set sufficiently to provide bearing for the plate.

(g) Intermediate shotcrete layer - After installing the nails, placing the plates and tightening the nuts, place the outer welded steel wire fabric layer. The fabric shall be positioned in such a manner that the fabric is not in physical contact with the nail. The wire fabric should be held firmly in proper position while shotcrete is applied. Apply the intermediate layer of shotcrete to the lines and grades shown in the plans.

(h) Subsequent excavation lifts - Further excavation shall not start until the shotcrete on the preceding lift has reached 25 percent of its required 28-day minimum compressive strength and the production nails, in the preceding lifts, tested as specified in 614.41. Each excavation lift shall be completed using the sequence outlined in steps (a) through (g) above.

(i) Final shotcrete layer - The final shotcrete layer shall be placed full height after the wall excavation is completed to grade using the sequence outlined in steps (a) through (h) above. Place the final shotcrete layer a fraction beyond the guide pins and wires. Excess material shall be trimmed to the true lines and grades shown in the plans, the guide pins and wires removed and their impressions covered.

(j) Architectural finish - The exposed shotcrete face shall be given a Class I finish and an architectural treatment as shown on Drawing 45817.

614.32 Nailing Application:

(a) Preproduction testing - The preproduction testing is performed on nails installed with the proposed production drilling and nail installation system.

The purpose of the test is to verify the Contractor's procedures, hole diameter, grouting method and design assumptions.

Preproduction test nails shall be sacrificial and shall not be incorporated in the production nail scheme. Drilling and installation of production nails shall not be permitted unless preproduction testing has been completed and approved by the Engineer, using the same equipment, hole diameter and installation methods proposed for the production nails.

Any changes in the installation or drilling method may require additional preproduction testing as determined by the Engineer and will be done at the Contractor's expense.

The Contractor shall submit detailed plans describing his proposed preproduction nail testing method to the Engineer for approval prior to the tests.

Three successful preproduction tests are required. Test nail locations shall be selected by the Engineer.

The intent is to stress the bond between the grout and the surrounding soil. The soil shall be loaded to a total load equal to the pullout resistance (kips) shown in the plans. This test requires a no load zone (ungrouted test length) and a bond zone (grouted length). The bonded length of the preproduction nail should be equivalent to the effective length of the closest production nail within the control area for that level of wall.

After the effective length is grouted and the grout has gained sufficient strength to withstand the test load, the test nail shall be loaded in increments of 25 percent of the pullout resistance (kips) to a total load equal to the pullout resistance (kips) shown in the plans.

Each load increment shall be held for at least 1 minute except for the final load. The load-hold period shall start as soon as the test load is applied.

The final load shall be held for 10 minutes. Measurement of nail movement with respect to a fixed reference point shall be obtained and recorded at 1 minute, 2, 3, 5, 6, and 10. The preproduction test will be considered successful and concluded if the test nail meets the criteria for a preproduction tested nail in subsection 614.41(a-3a) of these special provisions.

If a final load equal to the pullout resistance (kips) cannot be maintained for 10 minutes with less than 0.04-inch of movement between 1 minute and 10 minutes, the load shall be maintained for an additional 50 minutes. The preproduction test will be considered successful and concluded if the test nail meets the criteria for a preproduction test of nail in 614.41(a-3.b) of these special provisions.

The Engineer will evaluate the results of each preproduction test, make a determination of the suitability of the test and the Contractor's proposed production nail design and installation system. Tests which fail to meet the design criteria will require retesting or an approved revision in the Contractor's proposed production nail design and installation system. The soil nail shall be unloaded and completely grouted, only after completion of the test.

(b) Location and length - The location and length of the nails shall be as shown in the plans or as directed by the Engineer. The Contractor shall locate the holes within 3 inches of the predetermined location and in such a manner that the nail is not in physical contact with the welded wire fabric.

(c) Drilled hole diameter and length - The Contractor shall determine the maximum diameter of the hole. Minimum hole diameter shall provide a 3-inch clearance around the outer surface of the soil nail. Holes shall be drilled to a depth sufficient to provide the minimum embedment length (L) shown in the plans.

(d) Nail capacity - The nail capacity shall equal or exceed the pullout resistance (kips) shown on the plan. Embedment lengths for nails shall in no case be less than the minimum shown in the plans.

(e) Nail handling - Nails shall be handled and stored in such a manner as to avoid damage or corrosion. Damage to the nail steel as a result of abrasions, cuts, nicks, welds, and weld splatter will be cause for rejection by the Engineer. The nail steel shall be protected if welding is to be performed in the vicinity. Grounding of welding leads to the nail steel will not be allowed. Nail steel shall be protected from dirt, rust, and foreign substances. A light coating of rust on the steel is acceptable. If heavy corrosion or pitting is noted, the Engineer shall reject the affected nails.

(f) Nail installation - The nail shall be inserted in the hole to the required depth without difficulty. If the bar cannot be completely inserted, the Contractor shall remove the bar and clean or redrill the hole to permit insertion. Partially inserted nails will be rejected.

(g) Grouting - The grout shall be injected at the lowest point of each drilled hole and the hole filled in a continuous operation. The grout may be pumped through grout tubes, casing, or drill rods. The grout shall be placed after insertion of the nail. The quantity and pressure of the grout shall be carefully controlled and recorded. The grout equipment shall produce a uniformly mixed grout free of lumps. A positive displacement grout pump shall be used. The pump shall be equipped with a pressure gage which can measure at least twice the intended grout pressure and a stroke counter. The grouting equipment shall be sized to enable the grout to be pumped in one continuous operation. The mixer shall be capable of continuously agitating the grout.

(h) Installation of plate and nut - After the first layer of shotcrete and the grout have had time to gain the specified strength, the plate shall be placed as shown in the plans and the nut secured wrench tight with a minimum 100 ft.lbs. of torque.

614.33 Shotcrete Construction:

(a) General - Shotcrete test panels shall be prepared by each crew on vertically supported molds. The material used to form the back and sides of the molds shall be rigid, nonabsorbent and be nonreactive with cement. The shotcrete placement in vertical molds shall be accomplished utilizing the same equipment, shotcrete mix, air and water pressure, and nozzle tip as used for the actual placement of shotcrete on production surfaces. The panels shall be constructed at the project site in the presence of the Engineer. The panels shall be left undisturbed and protected at the point of placement for at least 24 hours or until the final set has taken place.

(b) Preproduction testing - Each crew shall prepare at least two test panels for each mix design for testing. The test panels shall be a minimum of 24 inches by 24 inches and shall be fabricated to the same thickness as in the proposed application. Material to form the sides shall be 3/8-inch hardware cloth.

One of the two panels shall be reinforced with the same welded wire fabric as in the proposed application. The Contractor shall saw the completed panel into at least 6 pieces to allow a visual inspection of the shotcrete density, void structure and coverage of the reinforcement. This panel shall have 3 applications of shotcrete as shown in the plans. The other test panel shall be constructed without reinforcing and the Contractor shall extract at least six 3-inch diameter cores from this panel in the presence of the Engineer for compressive strength testing by the Engineer in accordance with ASTM C 42.

The test panels shall be cured using the proper curing compound in a manner similar to the anticipated field conditions. The Contractor shall provide the Engineer with a copy of the mix design at least 5 working days prior to starting any production work. Production shotcrete work shall not begin until satisfactory test panel results are obtained.

(c) Deficient shotcrete - If any shotcrete produced by the Contractor fails to meet the requirements of these special provisions, the Contractor shall immediately modify procedures, equipment or system, as necessary and as approved by the Engineer, to produce specified material. All substandard shotcrete already placed shall be repaired to the satisfaction of the Engineer at the Contractor's expense. Such repairs may include removal and replacement of all effected materials, or placement of additional thickness, as determined by the Engineer.

(d) Equipment - The pump system utilized to convey premixed shotcrete ingredients shall deliver a uniform and uninterrupted flow of material, without segregation or loss of the ingredients.

The air compressor shall be capable of maintaining a supply of clean air adequate for maintaining sufficient nozzle velocity for all parts of the work and for the simultaneous operation of a blow pipe for clearing away rebound.

Batching and mixing shall be done according to ASTM C 94. Aggregate and cement shall be batched by weight. Mixing and placing equipment shall be capable of continuous operation and shall deliver a uniform and uninterrupted flow of material without segregation or loss of any ingredients. Ready-mixed shotcrete may be delivered in transit mixers which comply with AASHTO M 157.

The delivery equipment shall be capable of discharging the premixed materials into the delivery hose and delivering a continuous stream of uniformly mixed material to the discharge nozzle. Recommendations of the equipment manufacturer shall be followed on the type and size of nozzle air hoses and supplies to be used, and on cleaning, inspection and maintenance of the equipment.

(e) Application - Immediately prior to shotcrete application, soil surfaces shall be cleaned of loosened material. Areas where raveling develops shall be immediately shotcreted. Shotcrete should not be placed on any surface which is frozen, spongy, or where there is free water. The surfaces to be shot shall be damp but have no free-standing water. Thickness, method of support, air pressure and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete.

The thickness of the shotcrete blanket shall be controlled by installing noncorrosive guide pins, nails or other gaging devices normal to the face, such that they protrude the required shotcrete thickness outside the face. These pins shall be placed on a maximum 5-foot square pattern. A minimum cover of shotcrete shall be placed over the welded wire fabric as shown on the plans.

The shotcrete shall be applied from the lower portion of the area upwards so that rebound does not accumulate on the portion of the surface that still has to be covered. The nozzle shall be held at a distance and at an angle approximately perpendicular to the working face so that rebound material will be minimal and compaction will be maximized. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. When, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until steady flow resumes. A helper equipped with an air blowout jet shall attend the nozzleman at all times during the placement of shotcrete, to keep the working area free from rebound.

Rebound material shall not be worked into the finished product. Rebound is defined as the shotcrete constituents which fail to adhere to the surface to which shotcrete is being applied. It shall not be salvaged and included in later batches.

Shooting shall be suspended if:

1. High wind prevents the nozzleman from proper application of the material.

2. The temperature is below 40°F.

3. External factors, such as rain, wash cement out of the freshly placed material or cause sloughs in the work.

Construction joints shall be tapered over a minimum distance of 12 inches to a thin edge, and the surface of such joints shall be thoroughly wetted before any adjacent section of mortar is placed. Square construction joints shall not be permitted.

Surface defects shall be repaired as soon as possible after initial placement of the shotcrete. All shotcrete which lacks uniformity, which exhibits segregation, honey combing, lamination, or which contains any dry patches, slugs, voids or sand pockets shall be removed and replaced with fresh shotcrete at the Contractor's expense and to the satisfaction of the Engineer. If the wire fabric reinforcement is damaged or destroyed by such repairs, the damaged area shall be replaced by properly lapped and tied additional wire fabric.

Where a layer of shotcrete is to be covered by succeeding layers, it shall first be allowed to take its initial set. The initial layer shall be cleaned of all loosened material prior to placing succeeding layers.

(f) Curing - Air placed shotcrete shall be cured by applying a clear pigmented, liquid membrane-forming compound as specified in 701.05 of the Standard Specifications. The curing compound shall be applied immediately after gunning. The air in contact with shotcrete surfaces shall be maintained at tempertures above freezing for a minimum of 7 days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cement finishing materials are to be bonded unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the application of such additional materials. All hot and cold weather shotcreting procedures shall conform to ACI 506.2 except as modified herein.

Acceptance Testing

614.41 Acceptance Testing - Acceptance testing shall comply with the following:

(a) Nail load testing - Preproduction sacrificial nails and a percentage of the production nails shall be load tested to check the capacity of the proposed system to sustain the minimum pullout resistance (kips) shown on the plans for the service life of the wall. The Contractor shall supply all material, equipment, and labor to perform the tests. The Engineer will record all required test data. The cost of all nail testing is considered incidental and shall be borne by the Contractor.

Load testing of the preproduction nails shall be performed against a temporary bearing yoke which bears directly against the existing soil. Temporary bearing pads shall be kept a minimum of 12 inches from the edges of the drilled holes.

(a-1) Testing equipment - A hydraulic jack and pump are used in testing to apply the load. The ram travel of the jack shall not be less than the theoretical elastic elongation of the total nail length at the maximum test load plus 1 inch. The jack shall be independently supported and centered over the nail so that the nail does not carry the weight of the jack.

The elongation of the nail shall be measured with a dial gage or vernier scale fixed to a tripod or some other support device independent of the structure. The dial gage should permit reading to a maximum accuracy of 0.001 inch. The ram travel of the dial gage shall not be less than the theoretical elastic elongation of the total nail length plus 1 inch. The axis of the dial gage ram shall be aligned to within 5 degrees from the axis of the nail. A pressure gage attached to the hydraulic pump shall be used to measure the applied load. The pressure gage dial face shall be graduated in 100 psi increments or less and the full scale range shall not be greater than twice the pressure required for the maximum load to be applied.

The hydraulic jack and the pressure gage shall be calibrated as a set by an independent testing laboratory. Proof of calibration must be submitted before use. The loads on the nails during the tests shall be monitored with an electronic load cell. The Contractor shall provide the electronic load cell and a readout device. Care should be taken that the axis of the nail and the load cell are parallel to prevent eccentric loading. The stressing equipment shall be placed over the nail in such a manner that the jack, bearing plates, load cell and stressing anchorage are in alignment.

(a-2) <u>Production testing</u> - Ten percent of the nails in each shotcrete lift shall be tested to demonstrate that the minimum required pullout resistances (kips) shown in the plans are being developed. The location of the production nails to be tested shall be determined by the Engineer.

Production test nails can be either sacrificial or used as production nails. This test requires a no-load zone (ungrouted test length) and a bond zone (grouted length). The ungrouted length of the production nail shall be equivalent to the ungrouted test length shown on the plans. After the effective length is grouted, and the grout has gained sufficient strength to withstand the test load, the test nail shall be loaded to a total load equal to the pullout resistance (kips) shown on the plans. The ungrouted test length shall be grouted after testing if the nail is to be used as a production nail.

Applied test loads shall be measured with the pressure gage or load cell. Movement of the end of the nail, relative to a fixed reference, shall be measured and recorded to the nearest 0.001 inch.

Production testing shall be performed by loading the tested nail in increments of 25 percent of the pullout resistance (kips) to a total load equal to the pullout resistance (kips) shown in the plans.

The load shall be held for 1 minute between increments except for the final load which shall be held for 10 minutes. The load-hold period shall start as soon as the maximum load is applied.

Nail movements with respect to a fixed reference point shall be measured and recorded at one minute, 2, 3, 4, 5, 6 and 10 minutes.

If the change in movement between 1 and 10 minutes exceeds 0.04 inch, then the maximum test load shall be held for an additional 50 minutes. If the observation period is extended to 60 minutes, then the nail movements shall be recorded at 15 minutes, 20, 25, 30, 45, 50 and 60 minutes. If the nail fails in creep, retesting will not be allowed.

(a-3) Load testing acceptance criteria - Production testing of nails shall comply with the following requirements:

(a-3.a) A preproduction or production tested nail with a 10-minute load-hold is acceptable if:

- Nail carries the maximum test load with less than 0.04 inch of movement between 1 minute and 10 minutes; and

- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded test length.

- Total movement measured at the maximum test load does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length.

(a-3.b) A preproduction or production tested nail with a 60-minute load-hold is acceptable if:

- Nail carries the maximum test load with a creep rate that does not exceed 0.08 inch during the final log cycle of time and is a linear or decreasing creep rate; and

- Total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded test length.

- Total movement measured at the maximum test does not exceed the theoretical elongation of a tendon length measured from the jack to the center of the bond length.

(a-4) Replacement nails - If a production test fails, the Engineer may direct the Contractor to replace some or all of the installed production nails between the failed test and the adjacent production test nail that met the test criteria. Alternatively, nail length on the succeeding row may be lengthened to make up the additional capacity needed, additional design analysis by the Engineer would be required to determine the additional lengths required. The Engineer may also require additional testing. Costs associated with additional tests or installation of additional and/or longer nails and Engineer's redesign costs shall be at the Contractor's expense.

(b) Shotcrete - Acceptance testing of shotcrete shall conform to the following:

(b-1) Production testing - A minimum of 28 days after the initial and intermediate layers of shotcrete have been placed, the Contractor shall core 3 test specimens from each 2,000 square feet of shotcrete placed in the field at locations designated by the Engineer. These cores shall be 3 inches in diameter and the full thickness of the wall. This coring shall be done in the presence of the Engineer and the Contractor shall individually seal the cores in plastic bags and tag them for identification. The cores will be tested by the Engineer for compression strength in accordance with ASTM C 42. The holes resulting from the cores shall be sealed in a manner satisfactory to the Engineer.

The shotcrete shall be capable of attaining the following minimum compressive strength (f'c) as determined by ASTM C 42 testing of cores drilled for compressive strength determinations:

Age-days	Compressive strength-psi
28	4000

Shotcrete work will be accepted based on 28-day strengths. The Contractor may propose a method of expediting the work. The Contractor's proposal shall detail methods to assure that the 28-day strength is attained.

Measurement

614.81 General - The method of measurement for soil nailing payment shall be per square foot of shotcrete complete in place and accepted by the Engineer. It shall include furnishing soil nails, plates, washers, welded wire fabric, wire holding devices, centralizers, preformed permeable liner, weep holes, anchor grout and structural shotcrete as shown in the plans. Measurement shall include only those areas where the full thickness called for in the plans is in place.

Structure excavation will be measured in accordance with Section 251.

Drain pipe and drain backfill material associated with retaining walls will be measured for payment in accordance with Section 605 under "Roadwork".

Payment

614.91 General - The accepted quantity measured as provided above will be paid for at the contract unit price per square yard for the item "Nailed Soil Retaining Wall" which payment will be full compensation for furnishing all materials, labor, equipment, tools and incidentals necessary to complete the work as specified in this Section and detailed on the plans, with the exception of structure excavation which will be measured and paid for in accordance with Section 251.

Full compensation for cutting and removing the existing end slope shall be considered as included in the contract price paid per cubic yard for structural excavation and no additional compensation will be allowed.

Drain pipe and drain backfill material associated with retaining walls will be paid for under the "Roadwork" portion of the job, as set forth in Section 605.

NAILED SOIL WALL INSTRUMENTATION

Scope - This work shall consist of furnishing all instruments, tools, materials and labor and performing all tests necessary to install instruments in accordance with the plans and these special provisions. Station "UV" 130+40 shall hereafter be referred to as Section 1 and Station "UV" 131+12 as Section 2.

Each instrumented section shall have all instruments installed under this work and wired to a central control panel. Wiring to the control panel will be completed after installation of each instrument and is test proven by the Contractor to the satisfaction of the Engineer that the system is working in accordance to the manufacturer's specifications.

The Contractor shall install the instruments under the supervision of a qualified geotechnical instrumentation specialist having a minimum 3 years of experience in the design and installation of similar instrumentations.

Inclinometers will be furnished and installed by the Engineer. The Contractor shall cooperate in the installation of the inclinometers.

All instrumentation shall be protected by the Contractor during the term of the contract and shall be replaced or restored at the Contractor's expense if damaged by reason of his operations, to the satisfaction of the Engineer.

The Engineer will conduct research activities within the limits of the nailed soil wall structure. Visual observations and instrumentation readings will be made by the Engineer. Preconstruction readings will be taken immediately after the installation of the top strain gages to two existing concrete filled steel pipe piles at Section 1. Readings will be taken from the instrumentation after each instrumented nail is installed. Postconstruction readings will be taken monthly for the next 24 months.

Submittals - At least 5 weeks prior to start of nailed wall excavation, the Contractor shall submit in writing, 5 copies of a list of the instruments including instrument specifications, installation procedures and a wiring diagram detailing the wiring of the instruments to the central control panels. Also, at this time, the Contractor shall submit resumes' of those individuals responsible for instrument installation and testing. The list shall include references, including current telephone number, that can verify the experience requirements. Nailed soil wall construction shall not begin until the Engineer has approved instruments, installation procedures, personnel, and two adjacent piles at Section 1 instrumented with strain gages.

Materials

Instruments:

Strain Gages - Nails shall be instrumented to assess the load on the nails over the long term. Existing piles shall be instrumented to monitor any lateral load transfer and subsequently flexural stress build-up during the construction phase. Instrumentation to monitor loads shall consist of vibrating wire strain gages. The strain gages shall be weldable vibrating wire gage such as slope indicator part Nos. 52602100 and 52602200 or an approved equal. Strain gage accessories include Model Nos. 52602600, 52602300, 06700180, 06700019, 52606956, 52604100 and 52604110 or approved equal. The strain gages will be read using the Engineer's strain gage readout box Model No. 52669 manufactured by Slope Indicator Company.

Nail Load Cells - The nail load cells shall have an ultimate capacity not to exceed 50 tons. The load cells shall be center hole load cells with minimum hole diameter of 1.5 inches. Slope indicator parts Nos. 51301050, 56400800 and 51300960 or approved equal shall be used. The load cells will be read using a load cell indicator Model No. 51300900 manufactured by Slope Indicator Company or approved equal.

Earth Pressure Cells - The earth pressure cells shall exhibit an ultimate capacity not to exceed 50 tons. Slope indicator parts Nos. 51408200, 51417800, 51416900, 51421115, 51401510, 51400095 and 51407302 or approved equal shall be used. The earth pressure cells will be read using a readout box No. 51421100, 211 model 0.1 percent, manufactured by Slope Indicator Company or approved equal.

Tiltmeters - Tiltmeters shall be used to monitor the existing Bridge Pier 10 pile cap rotation. Ceramic tiltplates shall be slope indicator Model No. 50323 or Terra Technology Corp. Model No. TP-C or approved equal. The plates shall be mounted on the exposed face of the pile cap using Devcon UW No. 11800 bonding compound. The portable tiltmeter censor (english version) shall be slope indicator Model No. 50304400 or Terra-Technology Model No. TT-2 or approved equal. The censor will be read using the State's readout box Model No. 50309 manufactured by Slope Indicator Company.

Extensometer - One extensometer shall be used to monitor the pile cap deflection as excavation progresses. Slope indicator parts Nos. 51815800, 51815835, 51815855, 51815860, 51809600, 51703900, 51702701 and 517046FM or approved equal shall be used.

Central Control Panel - The central control panels shall be of sufficient size and capacity to handle the specified number of instruments outlined in these special provisions for each instrumented section. There shall be a channel for each individual instrument.

Installation

Strain Gages - The nails to be instrumented are located at Section 1 and Section 2. Three nails in Row Nos. 1, 3 and 5 in each respective section will be instrumented. Five pairs of strain gages shall be welded to each nail in Row Nos. 1, 3 and 5 at Section 1, and four pairs of strain gages shall be welded to each nail in Row Nos. 1, 3 and 5 at Section 2. The strain gages which are mounted opposite each other shall be micro-welded to the nail and the complete gage, sensor and wire assembly protected from moisture. Two pairs of gages shall be mounted 3 feet from the nail ends with the remaining three pairs at Section 1 and two pairs at Section 2 mounted and evenly spaced in between. The instrumented nails shall be installed in the drill holes with the strain gages aligned vertically. All wire connections shall be of an approved waterproof type. Installation and protection of the strain gage and connections shall be in accordance with the manufacturer's specifications.

The concrete filled steel pipe piles to be instrumented are located at Section 1. Two adjacent piles shall be instrumented with two strain gages each. The strain gages shall be micro-welded to the piles and the complete gage, sensor and wire assembly protected from moisture. The strain gages shall be mounted 5 feet and 10 feet below the bottom face of the pile cap.

Nail Load Cells - A total of six load cells (three in each instrumented section) shall be installed. The electric load cells shall be located in nail Row Nos. 1, 3 and 5. A 12"x12" blockout shall be provided in the shotcrete facing at the instrumented nail Row Nos. 1, 3 and 5 after the first application of shotcrete. The load cell shall be mounted on the nail between the bearing plate and the nut. The Contractor shall attach the cells and protect the connections according to the manufacturer's specifications. All wire connections shall be of an approved waterproof type. When the instrumentation program is completed, the Contractor shall remove the load cells and the blockouts, retighten the nuts, apply the second and final shotcrete layers to the true lines and grades shown in the plans, and apply a Class I finish as shown on Drawing 45817.

Earth Pressure Cells - A total of 12 earth pressure cells (six in each instrumented section) shall be installed. The earth pressure cells will be aligned vertically. Two earth pressure cells will be located in nail Row Nos. 1, 3 and 5. Locate one cell adjacent to the instrumented nail and the second cell midway between the instrumented section and an adjacent nail. The earth pressure cells will be installed at the interface between the soil and the first layer of shotcrete. The cells shall be positioned such that the lateral earth pressures bearing against the shotcrete wall will be monitored. The installation and protection of the earth pressure cells and their connections shall be in accordance with the manufacturer's specifications. All wire connections shall be of an approved waterproof type.

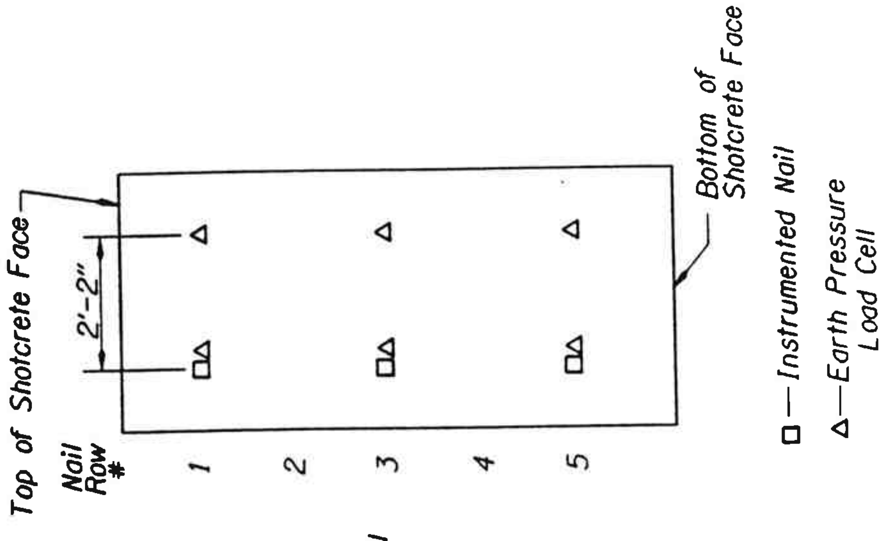
Central Control Panel - Two central control panels shall be installed at each instrumented section. The control panel at Section 2 will be attached to a steel or treated wooden post which is firmly secured in the soil. This control panel will be located 3 feet behind (south) the nailed soil wall. The control panel at Section 1 shall be installed at a location to be selected by the Engineer. The mounting details shall be submitted for the Engineer's approval. All instrumentation wiring to the control panels will be done in accordance with the manufacturer's specifications. The control panels must be sealed and completely waterproof. All above-ground wiring shall be enclosed in a steel conduit which is firmly attached to the control panels. All instrument-control panel wiring shall be done during instrument installation.

Measurement and Payment

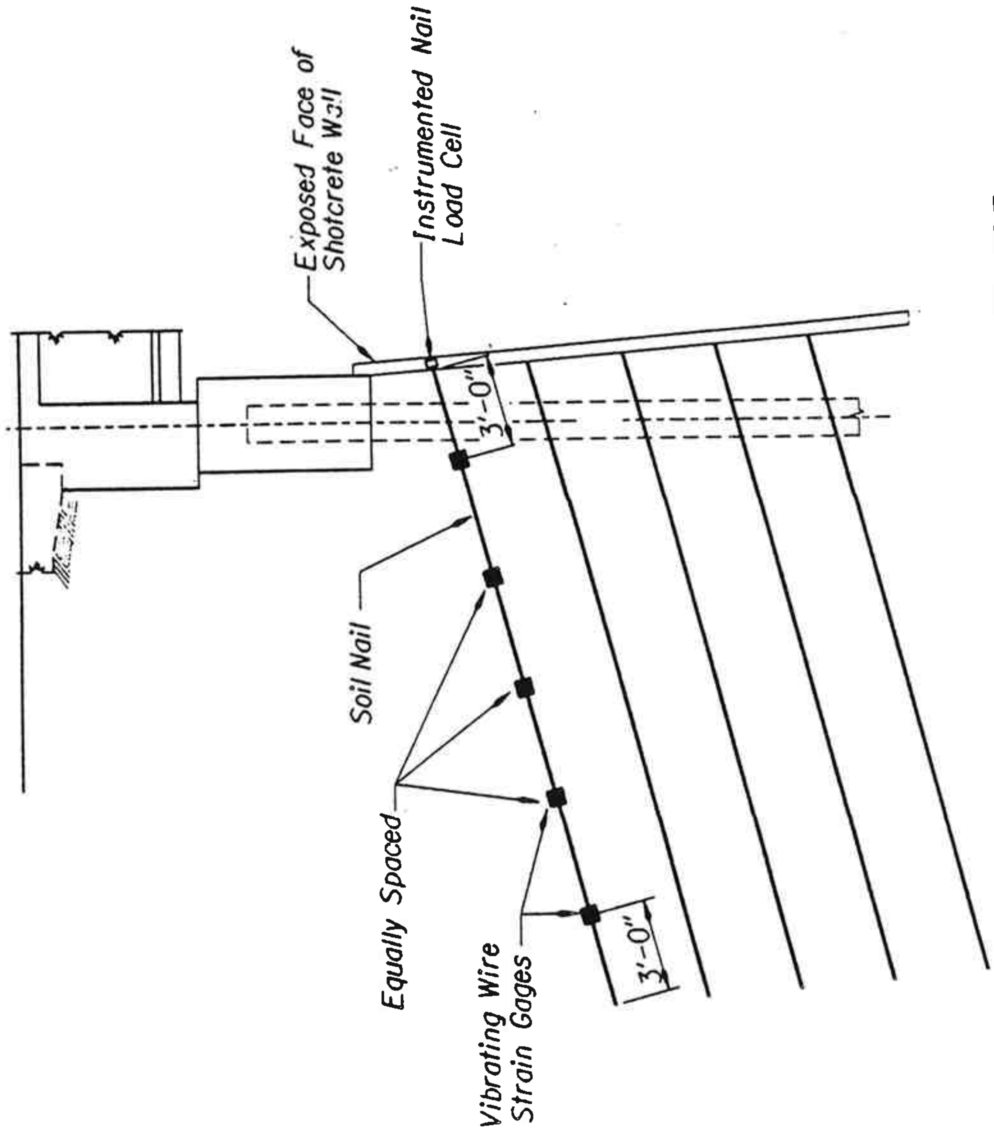
Measurement - No separate measurement will be made for the materials and work specified in this Section.

Payment - Soil nailing instrumentation will be paid for at the contract lump sum amount for the item "Soil Nailing Instrumentation" which payment will be full compensation for furnishing all materials, labor, equipment, tools and incidentals necessary to complete the work as specified in this Section.

All instruments furnished and installed under this Section shall become the property of the Division.



FRONT VIEW
(Sta. "UV" 130+40.00)
No Scale



SECTION @ OREGON SLOUGH BRIDGE
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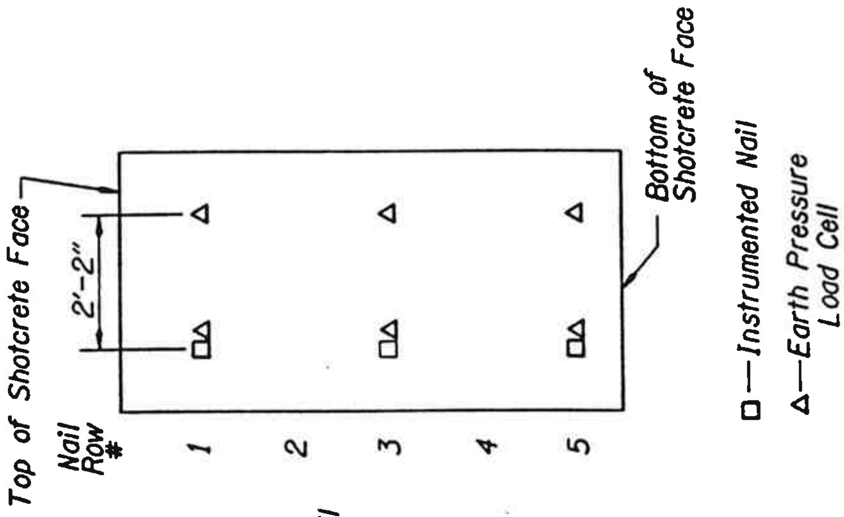

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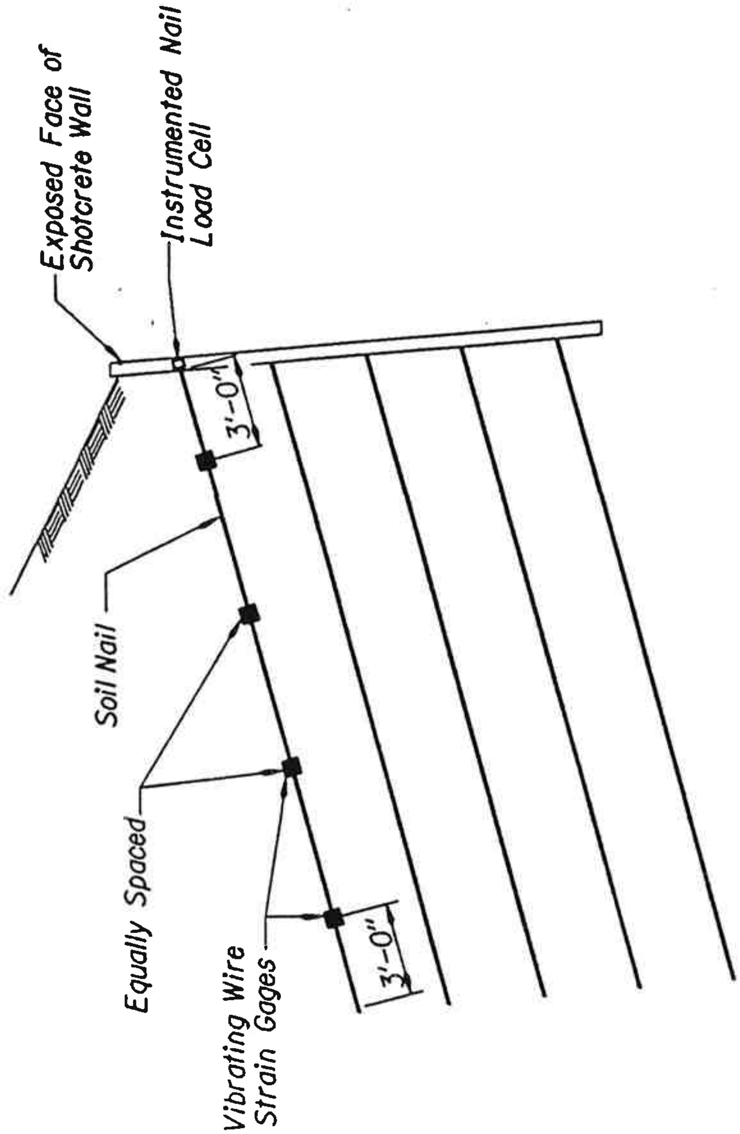

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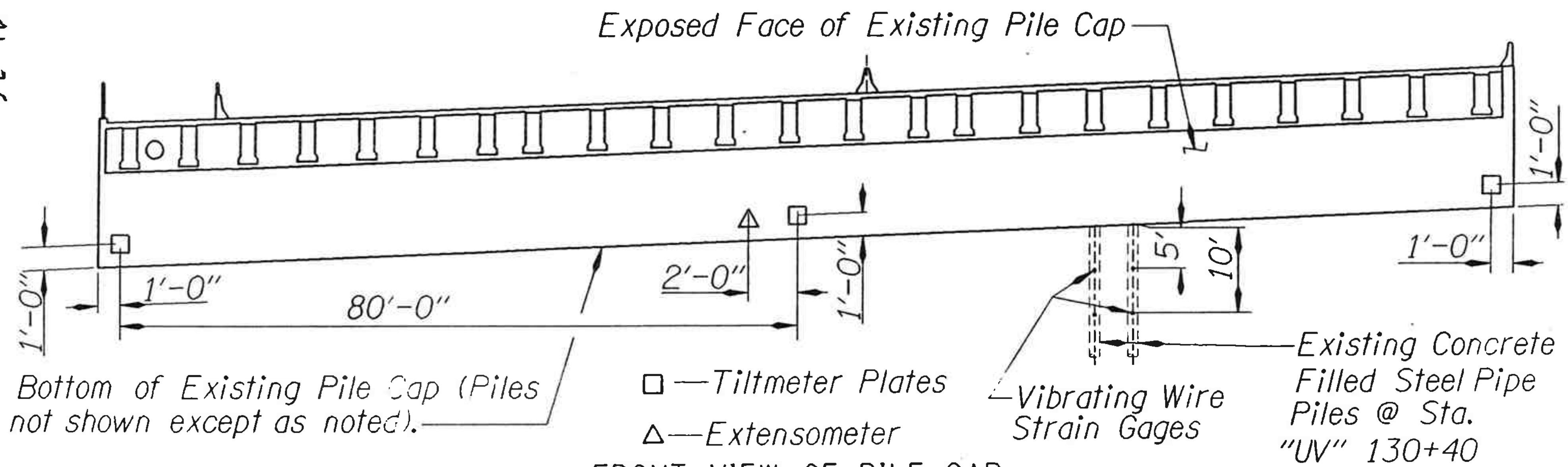
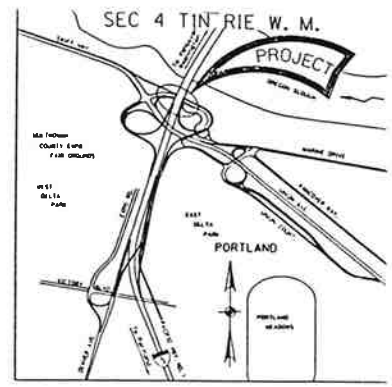


FRONT VIEW OF PILE CAP

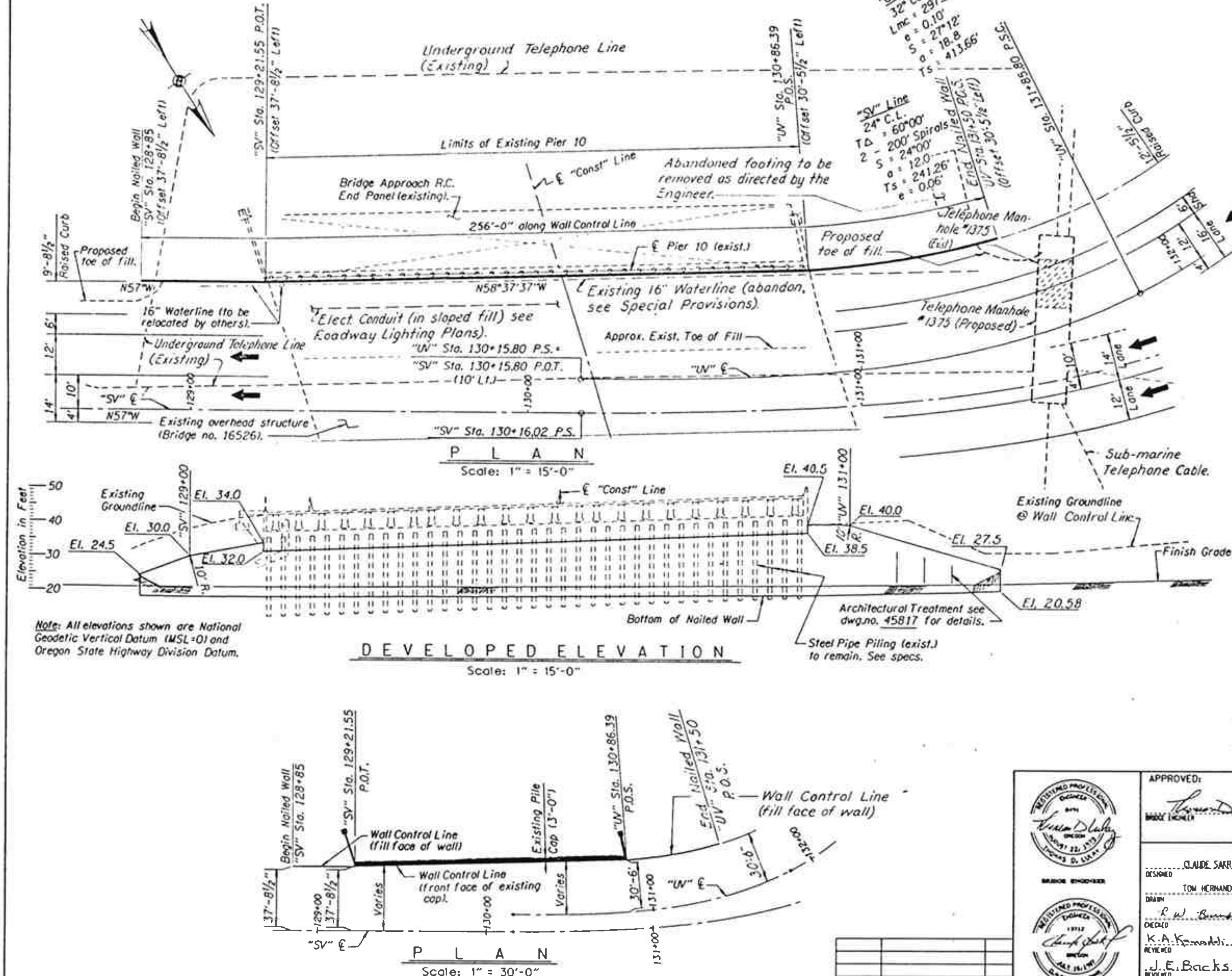
No Scale

SOIL NAILING INSTRUMENTATION

APPENDIX E: CONTRACT PLANS



VICINITY MAP
NO SCALE



GENERAL NOTES:

All materials and workmanship shall conform to be the Standard Specifications for Highway Construction of the Oregon State Department of Transportation, Highway Division.

Concrete facing designed by Load Factor Design Method.

Concrete for Shotcrete Facing shall be Class 4000 - ½" (see Special Provisions).

Anchor grout shall be capable of reaching a strength of 3500 p.s.i. in seven days (see Special Provisions).

All steel material shall conform to the following Specifications:
Epoxy Coated Soil Nails: ASTM A615, Grade 60
(Coat only non-threaded portion of the nail) the Exposed threaded portion of the nail, after tightening, shall be epoxy painted, as per Special Provisions.

Welded Wire Fabric: ASTM A497 or A185
Bearing Plates: ASTM A36
Nuts: ASTM A563 Grade B, Hexagonal
Washers: ASTM F436

All welded wire fabric, bearing plates, nuts, washers and threaded portion of the nail shall be hot-dip galvanized per specification after fabrication.

Soil Nails shall be installed to the minimum length shown on dwg. no. 45816.

DATE	REVISION	BY

APPROVED: [signature] BRIDGE ENGINEER

CLAUDE SAKR — DESIGNED
TOM HERNANDEZ — DRAWN
R. W. Burns — CHECKED
K.A. Kennedy 7-3-89 — REVIEWED
J.E. Backstrand — REVIEWED

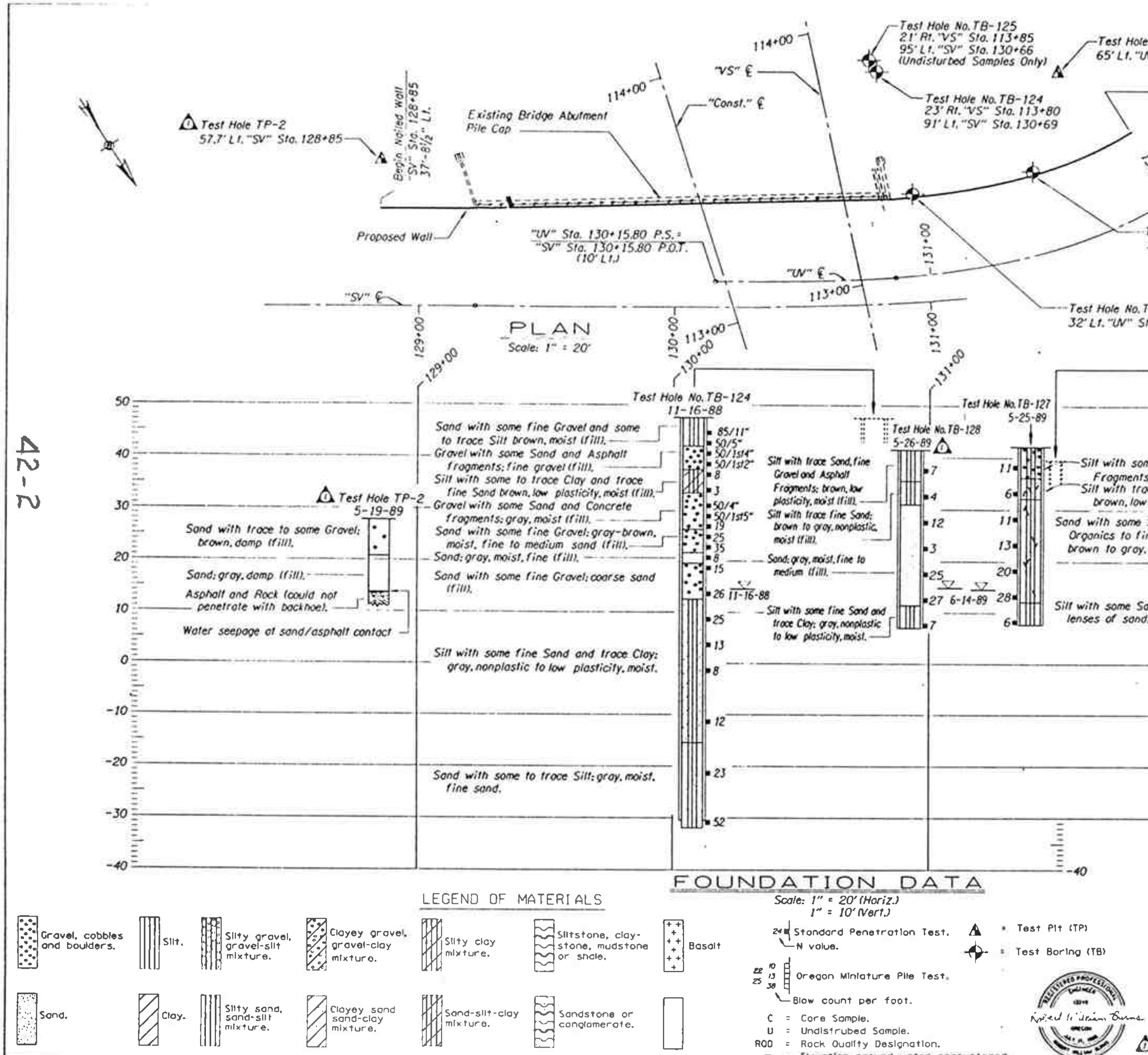
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL
Swift Intchge-Delta Park Intchge Sec.
Pacific Hwy.(MP 307.46) Multnomath Co.
PLAN AND ELEVATION

ACCOMPANIED BY DWGS. 45813 thru 45817
FOR BRIDGE ONLY

DATE April 1989 CALC.BOOK 2876 SHEET 1 OF 6
BRIDGE NO. 16526A DRAWING NO. 45812

42-1



42-2

Foundation data shown on this drawing is a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Geological Exploration Logs used in compiling this drawing are available upon request.

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL
SWIFT INTCH. - DELTA PARK INTCH. SEC.
PACIFIC HIGHWAY (M.P. 307.46)
MULTNOMAH COUNTY

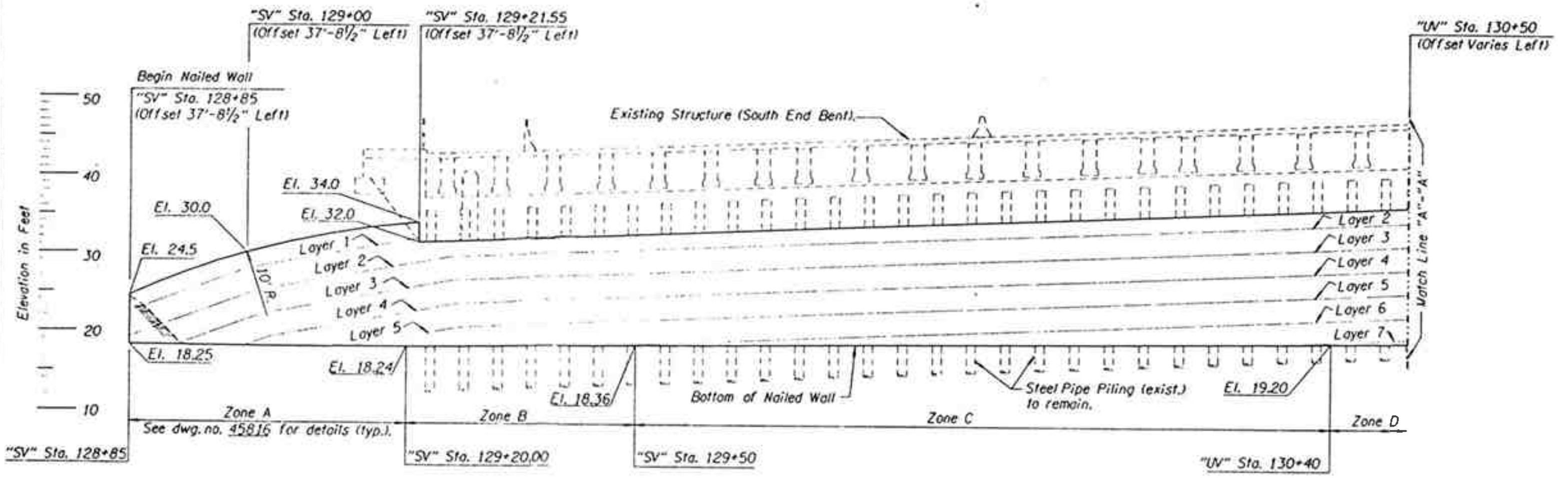
FOUNDATION DATA

BRIDGE ENGINEER	PE NO.
DESIGNED R. W. Burns	PE NO. 13249
DRAWN D. Wright	
CHECKED Jan Six	
REVIEWED Glen R. Thommen	

DATE	REVISION	BY
11/06/89	Test Holes added	DW

DATE January 1989 CALC. BOOK ________ SHEET 2 OF 6
BRIDGE NO. 16526A DRAWING NO. 45813

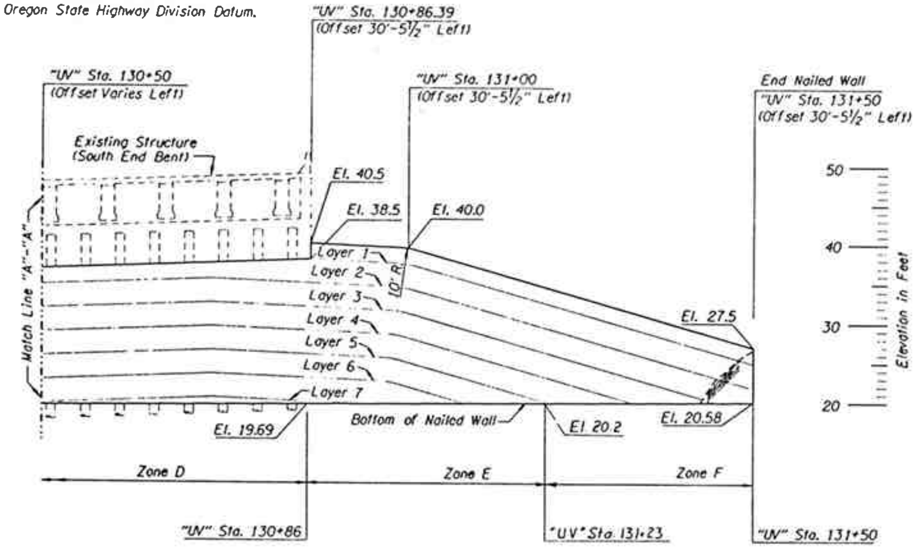
2-NOV-1989 (PGRID:01)



DEVELOPED ELEVATION

Scale: 1/8" = 1'-0"

Note: All elevations shown are National Geodetic Vertical Datum (MSL=0) and Oregon State Highway Division Datum.



DEVELOPED ELEVATION

Scale: 1/8" = 1'-0"

See dwg. no. 45817 for wall control line.

DATE	REVISION	BY

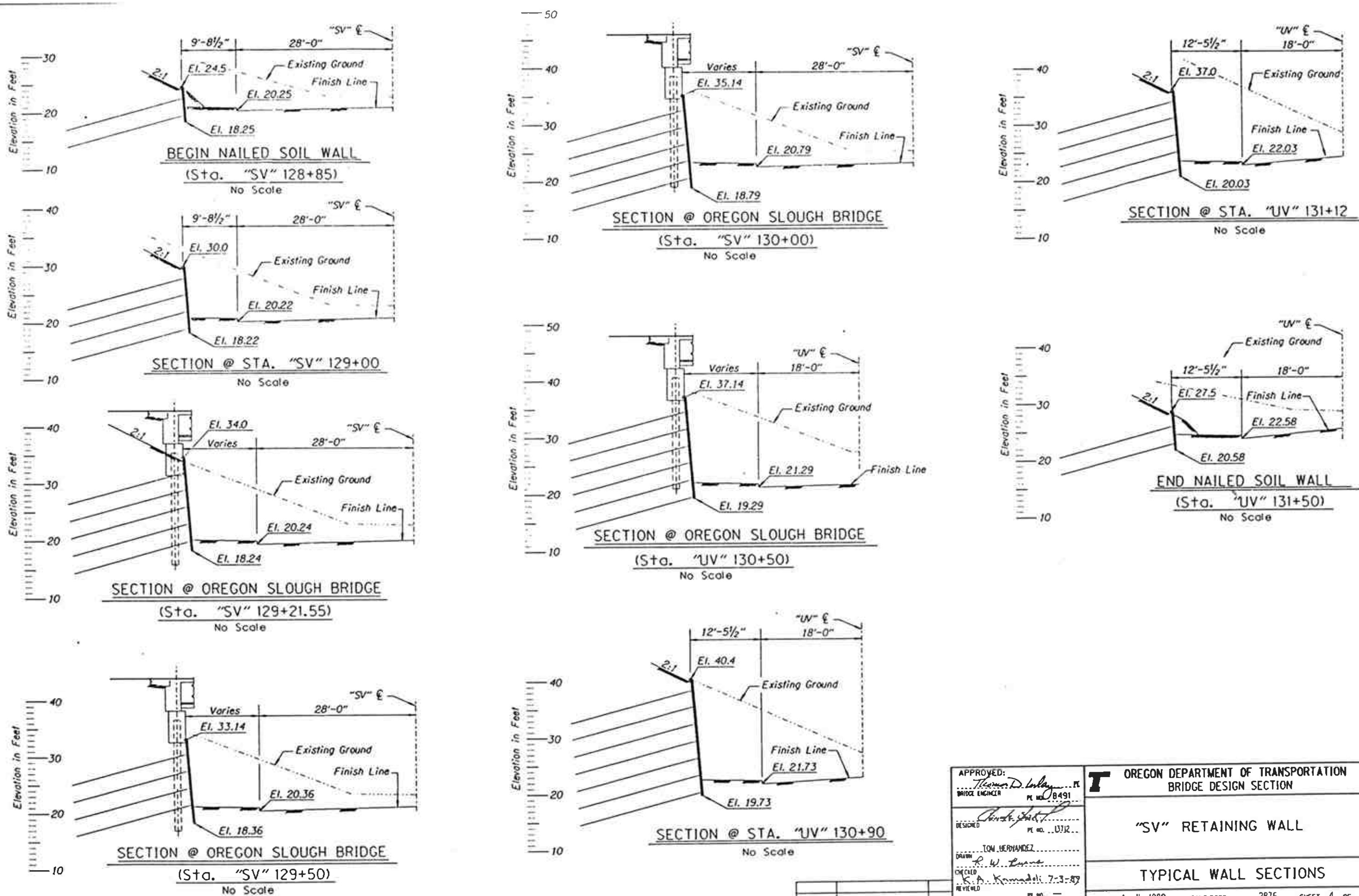
APPROVED: Bridge Engineer, PE No. 8891
Designed, PE No. 13112 — Tom Hernandez
Drawn
Checked: K.A. Kennedy 7-3-89
Reviewed, PE No. —
Reviewed: J.E. Brockstrand

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

NAILED WALL DETAILS

DATE April 1989 CALC.BOOK 2876 SHEET 3 OF 6
BRIDGE NO. 16526A DRAWING NO. 45814



42-4

APPROVED: Thomas D. Lulay, BRIDGE ENGINEER, PE NO. 8491	OREGON DEPARTMENT OF TRANSPORTATION BRIDGE DESIGN SECTION	
DESIGNED [signature] PE NO. U12	"SV" RETAINING WALL	
DRAWN TOM HERNANDEZ / R. W. [signature]		
CHECKED K.A. Komadel 7-3-89 REVIEWED PE NO. —	TYPICAL WALL SECTIONS	
J.E. Buckstrand REVIEWED	DATE April 1989 CALC.BOOK 2876 SHEET 4 OF 6	BRIDGE NO. 16526A DRAWING NO. 45815

DATE	REVISION	BY

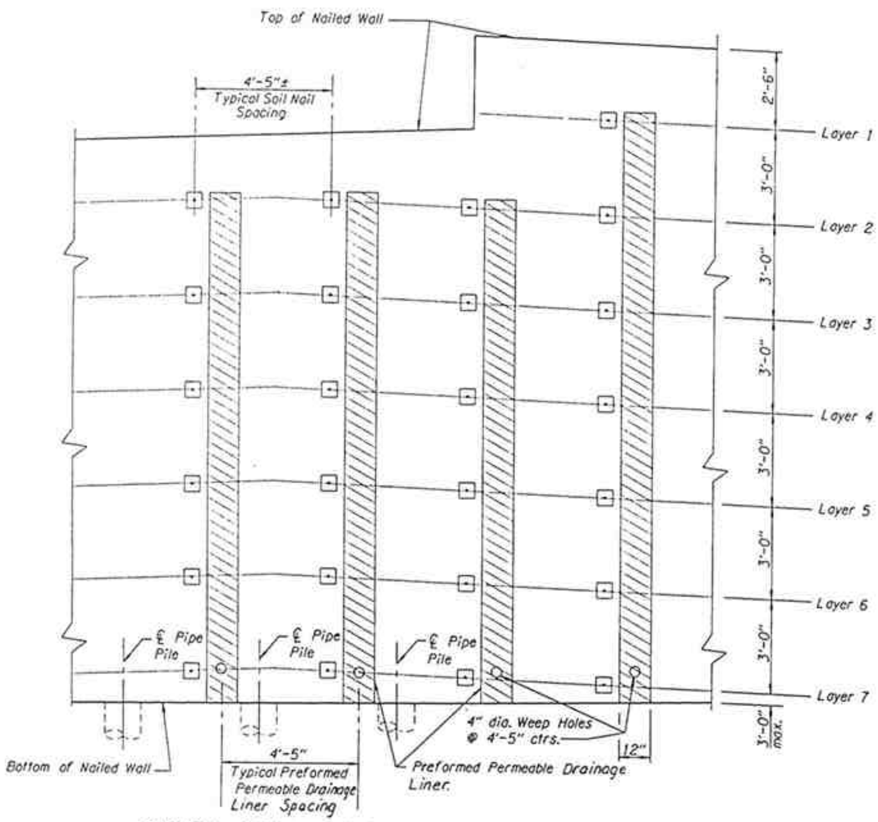
ZCS:1200.2011H16526A.DGN:1 (VIEW=3) (PGRID=G2)

PULL-OUT RESISTANCE AND UNGROUTED TEST LENGTH												
LAYER NO.	PULL-OUT RESISTANCE (KIPS)						UNGROUTED TEST LENGTH (FT.)					
	ZONE											
	A	B	C	D	E	F	A	B	C	D	E	F
1	3	-	-	-	2.5	2	12	-	-	-	16.5	11.5
2	6	20	15	10	5	5	10	11.5	15	16	15	9.5
3	9	23.5	18.5	13	8	8.5	8.5	9.5	13	14	13	7
4	12	27	22	16.5	11	12.5	6	7.5	11	12	11	4
5	17	32	27	21.5	14.5	—	2	4.5	8	9	8.5	—
6	-	-	33.5	26.5	20	-	-	-	4	6	5	-
7	-	-	-	34.5	23.5	-	-	-	-	1	2.5	-

Note:
Pullout resistance to be developed within effective length.

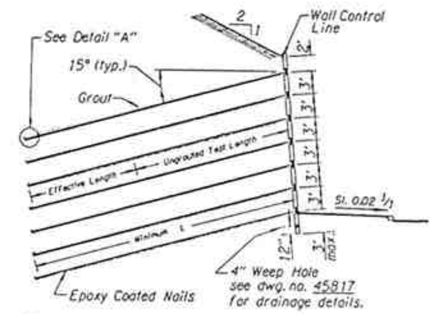
BAR SIZE (BY ZONE)					
A	B	C	D	E	F
#8	#9	#9	#9	#8	#8

MINIMUM "L" LENGTH (BY ZONE)					
A	B	C	D	E	F
14'	23'	24'	22'	18'	13'



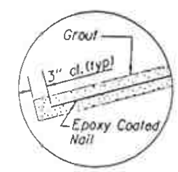
TYPICAL SOIL NAILING AND DRAINAGE ARRANGEMENT

Scale: 1/2" = 1'-0"



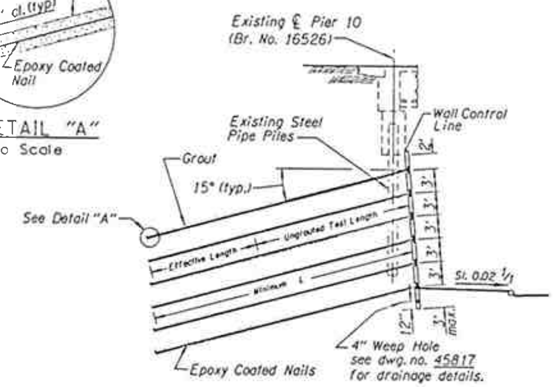
TYPICAL SECTION AWAY FROM BRIDGE

Scale: 1/8" = 1'-0"



DETAIL "A"

No Scale



TYPICAL SECTION @ OREGON SLOUGH BRIDGE

Scale: 1/8" = 1'-0"

APPROVED: [signature] BRIDGE ENGINEER PE NO. 8491
DESIGNED [signature] PE NO. 13712
TOM HERNANDEZ
DRAWN R. W. [signature]
CHECKED K. H. [signature] 7-3-89
PE NO.
REVIEWED J. F. [signature]

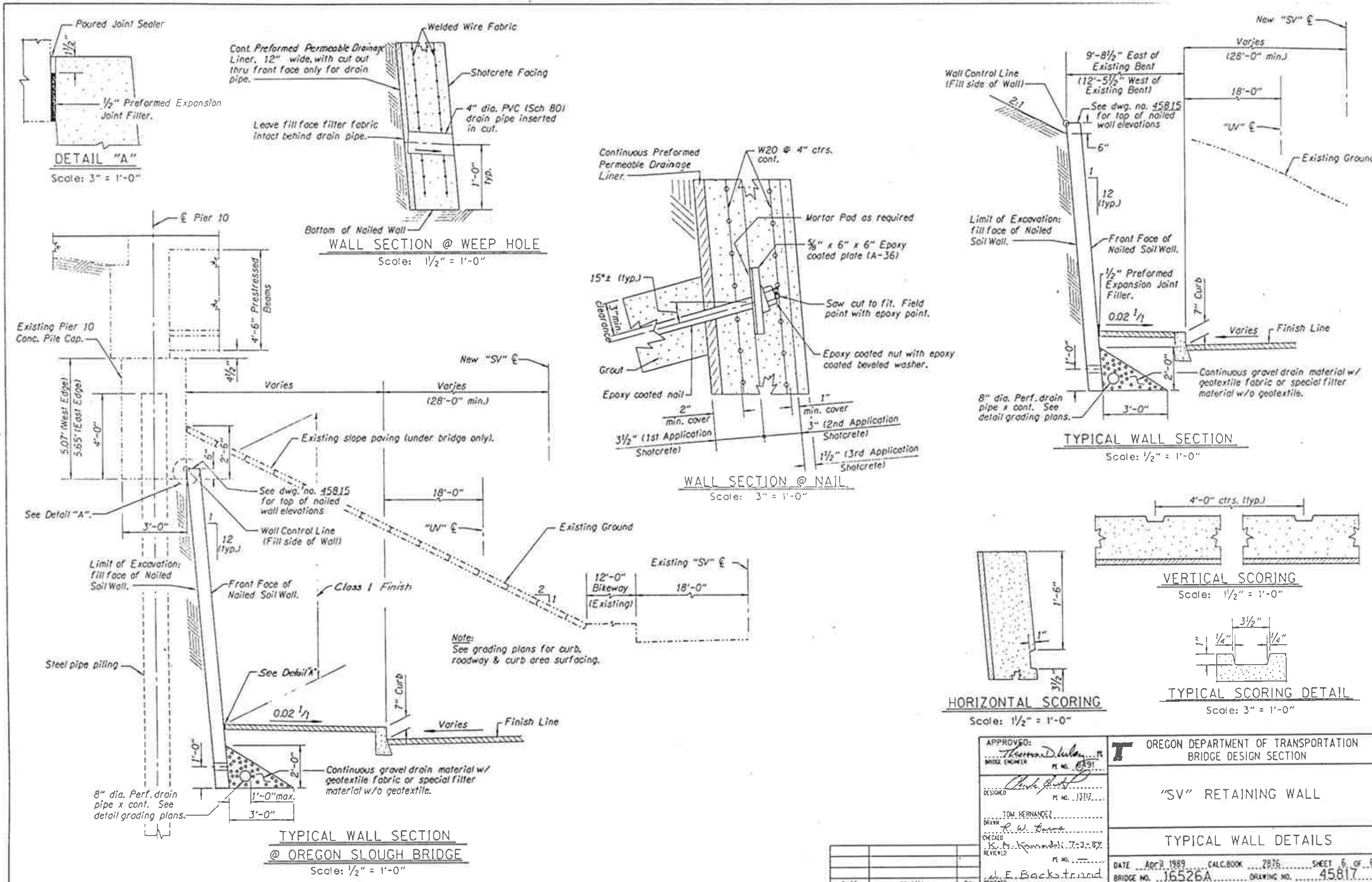
DATE	REVISION	BY

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

SOIL NAIL DETAILS

DATE April 1989 CALC.BOOK 2876 SHEET 5 OF 6
BRIDGE NO. 16526A DRAWING NO. 45816



APPROVED: Thomas D. Lulay, BRIDGE ENGINEER, P.E. NO. 6391
DESIGNED: P.E. NO. 13717
DRAWN: TOM HERNANDEZ / R. W. Evans
CHECKED: K. A. Kannadi 7-3-89
REVIEWED: P.E. NO. —
REVIEWED: J. E. Backstrand

DATE	REVISION	BY

OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

"SV" RETAINING WALL

TYPICAL WALL DETAILS

DATE April 1989 CALC. BOOK 2876 SHEET 6 OF 6
BRIDGE NO. 16526A DRAWING NO. 45817