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LANDSLIDE MITIGATION ON THE SONOMA COAST IN NORTHERN CALIFORNIA

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ABSTRACT

Heavy rains in late December 2001 and January 2002 caused approximately 100 meters of roadway to settle approximately 0.5 meters within a week along Highway 1 on the Sonoma County Coast (Post Mile 30.3). This area has a complex landslide history involving two active landslides. These landslides coalesce on a narrow section of Highway 1 approximately 130 meters above the Pacific Ocean. A tieback wall, sheet piles and a lightweight fill embankment had previously been constructed at this location to try to stabilize and maintain the roadway. The subsurface material at the site is composed of a matrix of very weak and extremely fractured shale and mudstone with the inclusion of sandstone blocks and fragments. The landslides are mainly driven by erosion at their base caused by storm related flows in Timber Gulch Creek and wave action undermining the slopes below the roadway.

The Office of Geotechnical Design West was requested to provide Geotechnical expertise for mitigating the landslide in an expeditious manner. Several mitigation measures were considered to stabilize the landslide. The selected repair strategy was to move the roadway approximately 30 meters inland behind the failure plane of the landslide. This required the construction of a 21-meter high soil nail wall and the excavation of approximately 100,000 cubic meters of rock material. In addition, a new tieback wall needed to be constructed on the outside shoulder of the new realigned highway to prevent the current landslide scarp from encroaching into the new roadway.

Design of the mitigation system was completed by March 2002 and construction started early April 2002 and completed by June 2003. This paper describes the geology and landslide history of the site and the observations, design details, soil nail pull out testing data, and wall-monitoring data obtained during the construction of the soil nail and the soldier beam tieback walls. This project demonstrated the efficiency and flexibility of soil nail and post tensioned tieback anchors for mitigating large landslides in extremely unfavorable geologic and topographic conditions. California Department of Transportation sponsored the project.

BACKGROUND AND SLIDE HISTORY

This project is located in Sonoma County on Route 1 (See Fig. 1). This location has a very complicated landslide history. The slope has failed repeatedly at several locations within the limits of this project over the last 20 years, most recently during heavy rains in December 2001 and January 2002 (See Photo 1). This site is located at the midpoint of a ridge that has been mapped as a landslide complex. It is further complicated by the fact that the San Andreas fault lies approximately 0.3 kilometers west of this location. At the roadway elevation, there are two basic directions of recent (within the last 35 years) landslide movement (See Photo 2). The larger landslide is moving southeast towards Timber Gulch Creek. Besides rainfall, this slide appears to be driven by erosion at its base caused by large storm related flows in Timber Gulch Creek. The other smaller coalescing landslides are moving southwest towards the ocean. These slides are the consequence of mass wasting at their base by the Pacific Ocean. In addition, both slides have been adversely affected by groundwater as evidenced by springs that can be seen below both slide locations and most likely indicate the toe of the landslide movement.

In 1980 the road was undercut by slipouts. In 1986, the road was moved about 3 meters (9 feet) inland at several locations. After heavy rainfall in January 1995, slides developed at 2 sites within the current project limits. At site 1 (PM 30.1), a slide formed a head scarp that extended to the roadway centerline stripe. The scarp was 18.2m long within the roadway and continued another 9m along the hinge point at each end. Existing embankment material at this location was replaced with lightweight fill material to a depth of 4.5m (15 feet) in 1995. In 1998, sheet-pile was driven to approximately 50-foot depths adjacent to the outside shoulder, at this location (approximately from station 10+10 to 10+65 on plan shown in Figure 2), because the scarp was reflecting through the lightweight fill repair at the same location. At site 2 (PM 30.16) a slide developed adjacent to the outside shoulder of the road for approximately 90 meters (300 feet). The scarp adjacent to the southbound roadway shoulder was approximately 4.6m to 6.1m (15'-20') in height. A tieback wall was constructed in 1995 to temporarily stabilize the slide (See lower left of Photo 1).

In 1998, a portion of the roadway was lost just north of the tieback wall. Due to environmental related issues for disposing excavated materials, it was decided that an interim project would be constructed that would open the roadway temporarily until a long-term solution could be constructed. The interim project consisted of driving sheet piles, grading the cut slope east of the roadway, and relocating the roadway inland approximately 15m (50 feet).

The heavy rains in December 2001 and January 2002 caused the landslide to reactivate causing approximately 100m of roadway to settle approximately 0.5m within a week. If the rains had continued the roadway would most likely have continued to settle and eventually forced the roadway to be closed. Because the rains slowed the roadway remained open to the public with the help of some maintenance work. However, the slide remained an imminent threat to the roadway.

SITE GEOLOGY

The project is located within the California Coast Range Geomorphic Province, on the northern California coast within the North American continental plate, 0.3-km east of the boundary of the Pacific Plate. The San Andreas fault defines this boundary. The bedrock at the site is mapped as Coastal Belt Franciscan Assemblage, a chaotic mixture of several rocks known as tectonic melange. The coastal Franciscan rocks are characterized by a lack of internal continuity of strata and by the inclusion of fragments and blocks of all types and sizes, both native and exotic, embedded in a sheared, fragmental matrix of finer-grained material. Graywacke sandstone, mudstone and shale predominate. At this location landslide deposits overlie the bedrock.

Seismicity

The site is located in an extremely active (seismically) region of northern California. It lies 0.34 km east of the San Andreas Fault. Healdsberg and Rodgers Creek, faults, part of San Andreas Fault system, are located at 31.25 and 33 km in east of the project site. The San Andreas Fault dominates the seismic conditions of the project area.

The Maximum Credible rock acceleration within the project location is estimated between 0.73g for San Andrea Fault to 0.15g for Rodger Creek Fault (Maulchine 1996).

FIELD INVESTIGATION & FINDINGS

A total of 13 borings have been drilled within the project limits from 1995 to present. Three Slope Indicators (SI-1, SI-2, & SI-3) were placed in March 2000 and one Slope Indicator was placed in early January 2002 (SI-2a), See Figures 2 and 3. GPS Field mapping and photo interpretation, of flights from 1965 to present, was also done at this location. It can be seen from the attached aerial photo 2 that the slope above and below the project site has numerous recent and historic landslides predominantly contributed mainly by the San Andreas fault zone. Fault zones often contain weak, crushed and broken sheared rock in a clayey matrix, which was verified by our investigation. Our most recent boring (SI-2a) revealed very intensely fractured fault breccia and gouge throughout the entire depth (30.8m) of boring.

Previous boring logs show soft to hard mudstones and weathered shales alternating with fractured sandstone. SPT blow counts "N" varied from 10 to 83, predominately between 30 and 60 blows per foot. The unconfined compressive strength recorded by pocket penetrometer in silty clay layers ranged from 0.1 MPa to 0.25 MPa. No laboratory tests such as unconfined or triaxial compression or direct shear tests were conducted on the representative rock samples due to very tight project schedule. However, based on available information from adjacent sites and the recorded SPT blow counts at this site, the unconfined compression of the rock is estimated to range from 0.4 to 3.0 MPa and locally higher in harder layers of shale/mudstone and sandstone.

Groundwater elevations vary from 4.0 to 22.2 m below original ground. Groundwater rose by 6.8 meters in SI-3 from October 11, 2001 to January 14, 2002 in response to heavy rains in late December 2001. The most recent failure occurred during this time, early January 2002. This shows the importance of drainage at this site.

The mean, high, and low precipitation recorded at Fort Ross monitoring station closest to this site between 1948 and 2003 are 965 mm, 1803 mm, 460 mm, respectively. The mean, high, and low precipitation for Spring season are 222 mm, 660 mm, and 79 mm. The amount of rainfall for 2002 and Spring of 2003 significantly exceeded the above mentioned mean levels. In the year 2003, the rain season continued to the end of the Spring.

The following is a summation of the SI data that has been collected at the site:

SI #	Failure Depth
SI-1	30m
SI-2	11.6m
SI-2a	6.7m
SI-3	17m

These values were used to determine how far the roadway had to be moved in order to get behind the slide plane (See Figures 2 and 3). A soldier beam tieback wall was designed at this location to withstand a 6 m cantilever.

Directional movement vectors were calculated from the SI data. These movement vectors indicate that there are two separate and distinct landslide movements at roadway level (See slide mapping on Photo 2). The northern slide is shallower and moving in a South-Southwesterly direction towards the Ocean. The southern slide is larger and appears to be moving in a southeasterly direction towards Mill Gulch. Field observations of spring seeps verify the apparent depth of these landslides.

REPAIR STRATEGY

Comparing aerial photos from 1965 to present indicated that the headscarp of the major landslide that most affects the roadway has remained at basically the same location. This would indicate that there has been little or no headward migration of the slide. Based on this evaluation of the landslide characteristics, our proposed repair strategy included relocating the existing roadway behind the identified slide plane as shown in Figures 2 and 3. The roadway relocation required construction of a soil nailed wall (Sections #1 and #2, See Figure 2) into the adjacent hillside on the north. In addition, in order to intercept the future progress of the down slope movement into the new roadway and prevent toe failure of the soil nailed wall, a soldier beam tieback wall was also proposed along and near the outside edge of the new roadway within the unstable limits of the road as shown on Figures 2 and 3.

The limits of the soil nailed and the soldier beam tiebacks walls are shown in Figure 2. The soil nail columns were labeled from 1 to 109 northerly. The soil nail column numbers at various locations along the soil nail wall are shown in Figure 2 for reference. The wall is at its maximum height between C44 and C62.

The existing slope above the soil nail wall was excavated to a slope not steeper than 1(V): 2(H) to reduce the earth pressure on the wall as well as minimizing the long term instability of the sloping area immediately above the soil nail walls.

GEOTEHNICAL ANALYSES & RECMMENDATION

Soldier Beam Tieback Wall

The following is a summary of the design parameters that have been recommended and incorporated in the design by Structures Design (SD).

Soil/Rock Strength Parameters

Friction Angle (ϕ) = 22 degree above the existing slide plane because remolding weakens the soil

Friction Angle (ϕ) = 35 degree below the existing slide plane

Unit Weight (γ) = 19.64 KN/ m^3 (dry) 22.0 KN/ m^3 (saturated)

Cohesion (C) = ignored

Design Wall Height

It was assumed that future undermining would cause the ground surface against the tieback wall to subside and thus, a wall design height of 8 m was used in the design for the reason described below

Piles and Tiebacks

Piles designed are 2W360x101 profiles (soldier beams) in 900-mm diameter concrete cast drilled holes. Spacing is 2.75 m center to center. Pile length varies between 14 m and 18 m long.

Tiebacks are used at every pile in the wall. This may be conservative for the current situation but reasonable as the slide continues to move creating an oversteepened slope below the new roadway alignment The surplus of tiebacks are considered as insurance against future slides. The second or lower row of “tiebacks" will not be installed in this contract. With T1 alone, the design height of the wall is H=6 m. Once the existing slide exposes 6 m of the soldier piles, it is time to drill level T2 at the appropriate locations. With levels T1 and T2 together, the design height is H=8 m.

The tiebacks used were 5-1862 MPa low relaxation strands. The length of tieback was about 30 m to have their bonded zone fall below the soil nail walls preventing tension on the roadway. All tiebacks were protected with corrugated HDPE sheathing with corrosion inhibiting grease in smooth sheeting along the unbounded length.

All tieback were proof tested. 5% of the tiebacks were performance tested at 1.5 times design load in accordance with FHWA procedure (FHWA, 1999). Design load was 650 KN.

Soil Nailed Wall

The soil nail wall design is based on the following requirements:

- The ground slope behind the wall is not steeper than 1V:2H
- No disturbance of native soil/rock material behind the wall
- The height of wall is limited to 21 m and the wall is battered at 10V: 1H.

Below is a summary of our geotechnical input:

Strength Parameters and the Wall Stability Analyses

The design parameters, friction angles and cohesion of the rock material were determined based on the available soil/rock material strength data and stability condition of the existing stable slope and bench north side of the slide zone. The soil nail wall is entirely located in the stable material behind the current slide plane. Therefore, a series of slope stability analyses were performed for the existing slope/bench to back calculate a reasonable friction angle and cohesion values which result a factor of safety of 1.5 for a critical slide plane comparable to the critical failure plane determined for soil nail wall. The following average soil/rock design parameters were determined based on the above mentioned back-analysis and the available laboratory testing results:

Friction Angle (ϕ) = 35 degrees

Cohesion (C) = 25 kPa

Unit Weight (γ) = 19.6 KN/ m^3

For seismic load condition, the Cohesion was increased to 50 kPa. These values are comparable to those commonly used for the rocks in the general site area in absence of weak bedding planes and extreme weathering. The slope above the bench was about 1V: 1H and stable since their development in 1998 which justified the use of the above strength parameters.

The design of the soil nailed retaining wall was based on rock/soil parameters developed in this study and anticipated forces from a local major seismic event. Rock/soil properties were determined based on in situ tests and results of laboratory tests from previous projects with similar subsurface conditions.

The design for the Soil Nailed Retaining Walls was performed using Caltrans' computer program (SNAILZwin, 2002). The following limiting criteria were used in the design of the Soil Nailed Retaining Walls:

1. The minimum static factor of safety: $FOS_{construction}$ = 1.5 with diameter of the grout hole (D)= 152 mm and the inclination angle (θ) of the nails to the horizontal = 15 degree. The inclination angle (θ) alternates between 10 and 20 degrees in order to avoid nails intercepting each other.

2. The minimum factor of safety with seismic loading (pseudo-static): $FOS_{dynamic}$ = 1.1; a horizontal pseudo-static coefficient of 0.25g was used to simulate seismic loading conditions. The wall movement in an event of MCE was estimated to range between 150 to 200 mm assuming that the entire soil nailed block would act like a massive gravity wall.

Grout/Rock Bond Stress

The following design parameters were used:

- Ultimate bond stress between grout in drill hole and the rock: 124 kPa (18 psi)
- Drill Hole Diameter: 152 mm
- Bond stress at the maximum test loads of 186 Kpa (1.5 times of Ultimate bond stress)

The design bond stress was based on the existing rock material conditions at the site, the available in-situ strength data, results of the previously performed proof testing for the existing tieback wall where a minimum design bond stress of 280 Kpa was achieved, and the effects of the overburden pressure.

It was specified that contractor should use a drill rig type, drill method, drill hole size, and grout placement method and material to achieve the maximum test loads specified in test nails program.

Soil Nails Spacing Requirements

- S_v is the horizontal spacing of the nails, $S_{v,\ MAX}$ = 1.675 m (5.5 ft.)
- S_h is the vertical spacing of the nails, $S_{h,\ MAX}$ = 1.525 m (5.0 ft)
- Minimum and maximum spacing, both horizontal and vertical, of soil nail assembly = 0.46 m and 1.7 m, respectively.

Soil Nail Bars Grade, and Corrosion Resistance

Grade 1040 kPa (150 ksi) bars conforming to ASTM Designation: A 722/AASHTO M275 were used for wall height above 9 m. Grade 520 kPa (75 ksi) bars were used for wall height below 9 m in height. The use of higher strength steel was used for two reasons. One to achieve a high factor of safety against yielding of the soil nails for seismic load and high hydrostatic loading conditions. Secondly, to achieve an appreciable shear/bending capacity of the soil nailed block perpendicular to the soil nails.

The soil nail bars used varied in diameter from 25 and 36 mm, which included sacrificial thickness required for a low to moderate corrosion potential conditions at the site. All bars were also coated with 0.3 mm of epoxy for additional corrosion protection.

Ultimate punching capacity of 245 KN (55 Kips) was used for up to 9 m high wall and 334 KN (75 Kips) for up to 21 m high wall. The calculated critical overall static factor of safety was 1.45. Generally, For the calculated critical factor of safety, pull out mode of failure governed for the upper and punching mode of failure for the lower rows of the nails. The factor of safety calculated for steel bar yield condition was significantly higher than the critical factor of safety, as required.

The primary and permanent wall thicknesses were 200 mm and 175 mm thick. Sculpted shotcrete finish facing was used.

Soil Nail Lengths

The designed lengths (embedment depth) of the soil nails varied between 100% to 115% of wall height at the most top row and between 40 and 50% of the wall height at the most bottom row of nails. The design nail lengths were mostly based on the seismic stability requirements, effects of sloping ground above the wall, and in order to minimize the wall excessive movement at the top and overstressing of the lower nails in an extreme loading condition. Also, the nails were made long enough to resist the seismic loading by a reinforced (soil nailed) block that is locally stable in pull out, face punching and steel yield modes of failure. The use of high strength steel was also for making the nailed block strong for shear/bending mode of failure. Unfortunately, due to lack of time, the actual capacity of the nailed block for shear/bending was not evaluated.

Because of the variation of wall heights and the steepness of the ground above it, the Soil Nailed Retaining Wall was divided into several schedules (A through E) that include variable soil nails lengths (embedment depths).

Soil Nail Wall Drainage

To prevent the build up of hydrostatic pore pressure behind the wall and facing, the drainage system included the following:

- A drainage concrete gutter immediately above the wall.
- 300 mm wide prefabricated geocomposite drain strips placed vertically on 1.5 m centers, prior to applying shotcrete. These drains near bottom of wall through specially design weep holes to reduce hydrostatic pressure on the wall facing.
- PVC pipe weep holes through the shotcrete face at the center and base of the prefabricated drainage strip.
- Underdrain (UD) below the base of the soil nail wall.
- Horizontal drains (HD) 60 m long were installed at height of about 1.2 to 1.5 m above the base of the wall at an interval of 15 m. Both UD and HDs discharges into several DIs.
- Large diameter culverts to drain Mulch Creek and DIs across the roadway on the downhill slope with proper rock slope protection at its outlet.
- 200 mm perforated pipe wrapped in permeable concrete and geotextile fabric installed along the base of the tieback wall with several outlets on the downstream slope.

Soil Nail Pull out Tests

Pullout Tests on sacrificial Test Soil Nail assemblies were performed in accordance with procedure in (FHWA, 1996) Manual at the designated locations shown in Figure 4. The testing schedule was in accordance to the following table.

TABLE 1- TEST NAILS PROGRAM

Adjacent Production Nail Length, m	Test Bond Length(L) m	Ultimate Bond Stress, kPa	Drill Hole Diameter mm	Max. Test Load (M) KN
Up to 12	3	124	152	265
>12	6	124	152	530

TOTAL TEST NAIL LENGTH (m)=(2/3)*Adjacent production nail rounded up to the nearest 0.5 m. (M) = MAX. TEST LOAD (kN) = $0.0047L\sigma_bD$ Where L= bar's bond length in the test, σ_b=Ultimate bond stress between grout and drilled hole as shown on the plans, in kPa; and D= actual drilled hole diameter, in millimeters. (AL=ALIGNMENT LOAD = 0.1M)

The test soil nail assembly was considered acceptable if the following two criteria are met.

1. The measured movement of the soil nail head is larger than 80% of the theoretical elastic elongation of the unbonded length at the maximum test load, and
2. The movement measured between one minute and 10 minutes is less than 1 mm.

If the maximum load cannot be maintained for 10 minutes with 1mm or less movement, the 1.00M load was maintained for an additional 50 minutes. The test soil nail assembly was considered acceptable if the movement is less than 2 mm for log cycle of time between the 6 and 60 minutes readings observed during the creep testing.

Typical load-deformation results are plotted in Figure 4.

CONSTRUCTION SPECIAL PROVISIONS

The excavation and installation of the soil nailed and tieback soldier beam walls were conducted in accordance with detailed special provisions, which were modified and expanded for this project. This provisions cover all aspects of earthworks/excavation requirements, drilling holes for nails and soldier beams, use of casing and slurry for caving and unstable hole conditions during the drilling, dewatering, test nail program and acceptance criteria, soldier beam and tieback anchors installation and their proof and performance testing criteria, structural criteria, and safety and environmental issues, etc.

CONSTRUCTION

Photos 3 through 13 demonstrate the representative construction activities. The soil/rock condition toward the north side of site consisted of silty clayey gravel of low shear strength.
Most of the soil nail drilled holes in that area were cased (Photo 7).

Groundwater seepage was observed during the drilling of many soil nail holes and some of drilled holes for piles. The seepage was observed even at holes drilled near the top of soil nail wall during the summer. The groundwater seepage at and near the north end of the site was significant requiring continuous pumping. The upstream creek water flow was diverted to the down stream slope of the roadway to allow the excavation and walls construction, and rip rap placement on the creek bed.

The excavation of the slide material in front of the soil nailed wall and construction of the wall were initially slow due to insufficient space for working equipment requiring night work for soil nail drilling, installation and shotcreting.
The construction of soldier beam piles was subsequent to near completion of the soil nail walls. Some of the holes for the piles were drilled with use of slurry due to higher groundwater and sloughing conditions.

The construction of return walls on Mulch Creek and rip rap placement at its streambed were carried out during the soldier beam piles installation.

In order to develop the design load on tiebacks, the existing lightweight fill placed previously for the slide mitigation had to be grouted. This work also added to the congestion of construction equipment traffic. However, the road was kept open during the entire construction with planned traffic control, which was relatively costly. The installation of tieback at sections where the new roadway crosses the older one was performed after the traffic was shifted on the new road.

SOIL NAIL TEST RESULTS

The results of some of the pull out soil nail tests are shown in Figure 4. Each test defined by TN (top of the nail) followed by the nail column and the row level close to the nail. Two tests TN40-41, LE1-2, and TN 33-34/LE3 were failed near the design load at about 15 mm and 40 mm elongation, respectively. However, an additional test performed adjacent to those test locations met the approval criteria. The grout intake was mostly higher than the theoretical value and based on the contractor, grout topping was required for most holes, which were due to the fracturing of the rock and groundwater seepage. All of tieback proof and performance tests met the approval criteria

MONITORING

The regular survey of the wall was initiated on September 18, 2003 when the wall reached 10^{th} row of the soil nails at the maximum height. Based on occasional wall survey conducted before this date, the wall horizontal movement ranged between 10 to 60 mm with higher movements occurred toward the north end of the wall. Figure 5 shows the horizontal wall movements surveyed at the several locations along the top and the wall face between September 18, 2002 to February 27, 2003. As shown, the wall movement was higher toward the north end of the wall due to poor soil conditions and steeper backfill slope (1V: 1.5H or steeper). During November, some additional and longer nails were installed from Column 90 to the north end of the wall to help reducing the wall movement at that area. As shown the rate of all recorded movements with time have been very small to negligible since January 2003. In May, the survey points were abandoned and Teflon reflectors were installed on the wall facing and the survey of the wall is scheduled to continue using Total Stationing survey method. The differential wall movement at the reflectors near the top of wall between 2/27/03 and 5/27/03 varied between 2 to 6 mm.

The horizontal wall movements have exceeded the 0.3% times wall height criteria considered on normal projects. However, the recorded movements appear to be acceptable since the wall movement has stabilized and no sign of distress has been observed on the wall facing or the sloping ground above the wall.

Two tiltmeters have been installed on the Soil Nailed wall and two were installed along top of the tieback wall. Also, two slope inclinometers are going to be installed to depth of about 30 m on the roadway. The results of this additional monitoring are not available at the time of this paper.

CONCLUSIONS

1) This project demonstrated that a high soil nailed wall with sloping backfill could be built in a very weak rock and low shear strength residual soils under groundwater seepage condition. The cost of the mitigation system constructed was about 35 percent cheaper than a high tieback wall option.

2) Design of soil nailed wall relies on understanding the rock/soil characteristics at the site and using suitable soil/rock strength parameters and adhesion between grout and rock/soil. These parameters should be assessed initially based on in-situ data, laboratory testing, slope failure back calculations, and finalized based on engineering judgments. The construction success highly relies on a well-prepared specifications and plans, use of a suitable construction method, equipment, and contractor experience as well as stringent quality control including performance testing.

3) The horizontal wall movement appeared to be more affected by the backfill rock/soil strength, the sloping backfill, and the wall end restraint conditions than the wall height. The wall movements stabilized at a relatively short period after its completion though the total wall movements exceeded 0.3% of wall height criteria normally used by Caltrans.

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Figure 1. Site Map

Photo 1. Slope Failure on January 2002

Photo. 2. Interpreted Mapping of Recent Slides

Photo 3. Graded Slope above Soil Nail wall and 1st Exc. Lift

Photo 4. Soil nailed Wall Construction at 4th Lift

Photo 5. Primary Wall Section

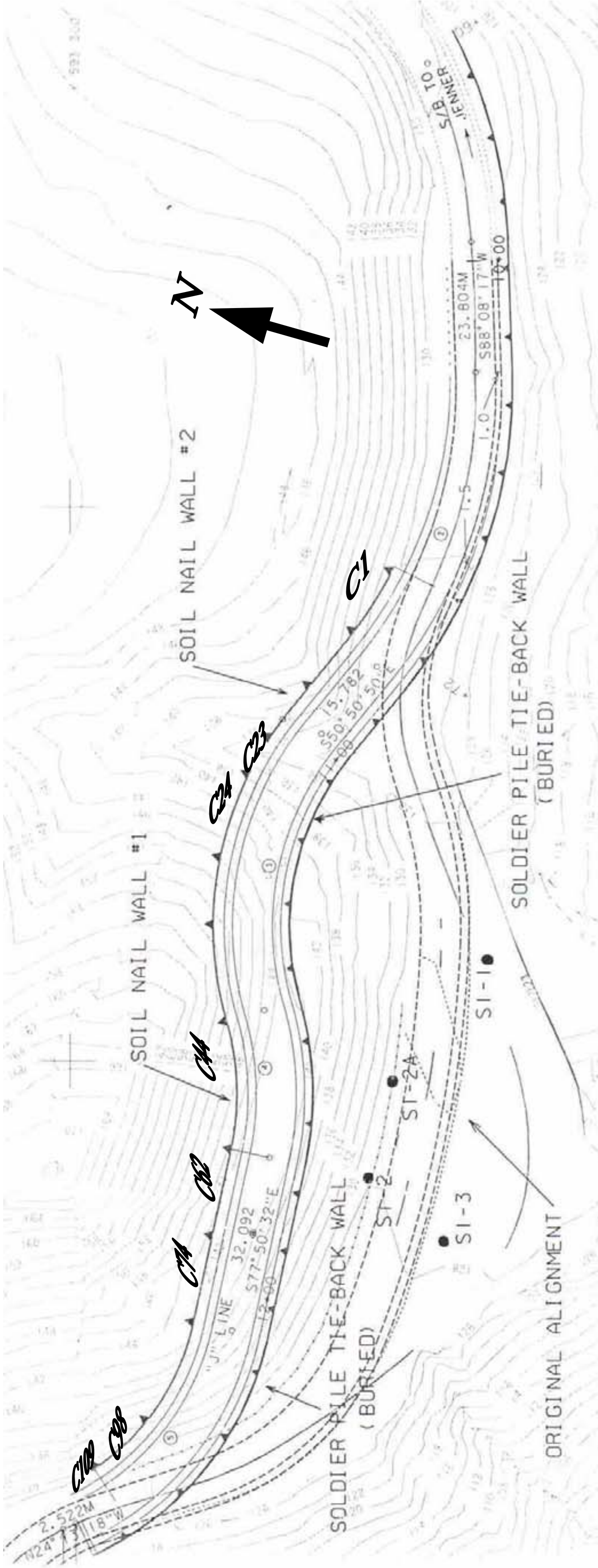

Figure. 2. Site Plan Showing Both Wall Alignments

Photo 6. Setting up a Soil nail Test

Photo 7. Drilling on north side of Soil Nailed Wall Using Casing

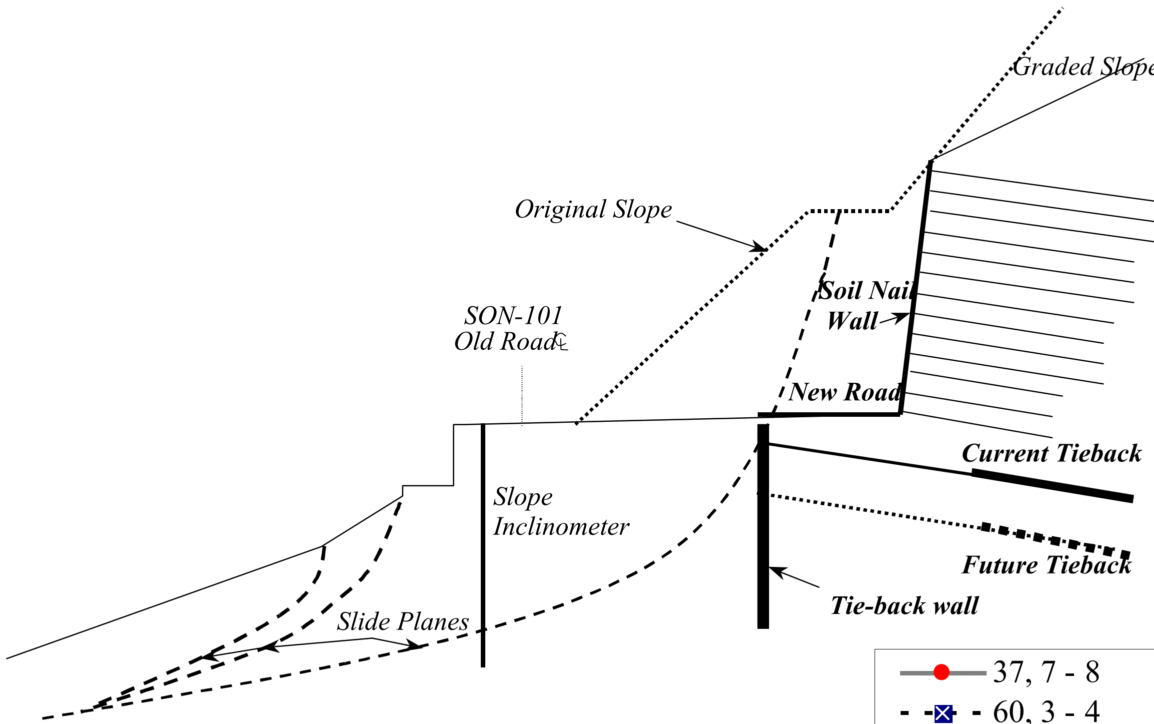

Figure 3. Slope Cross Section

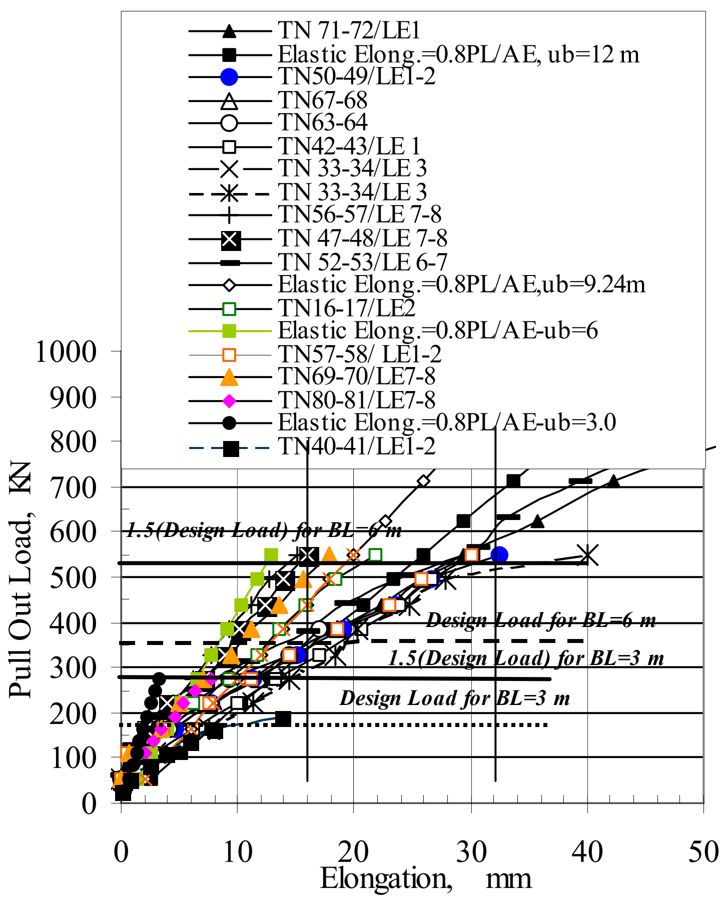

Figure 4. Soil Nail Pull Out Test Results

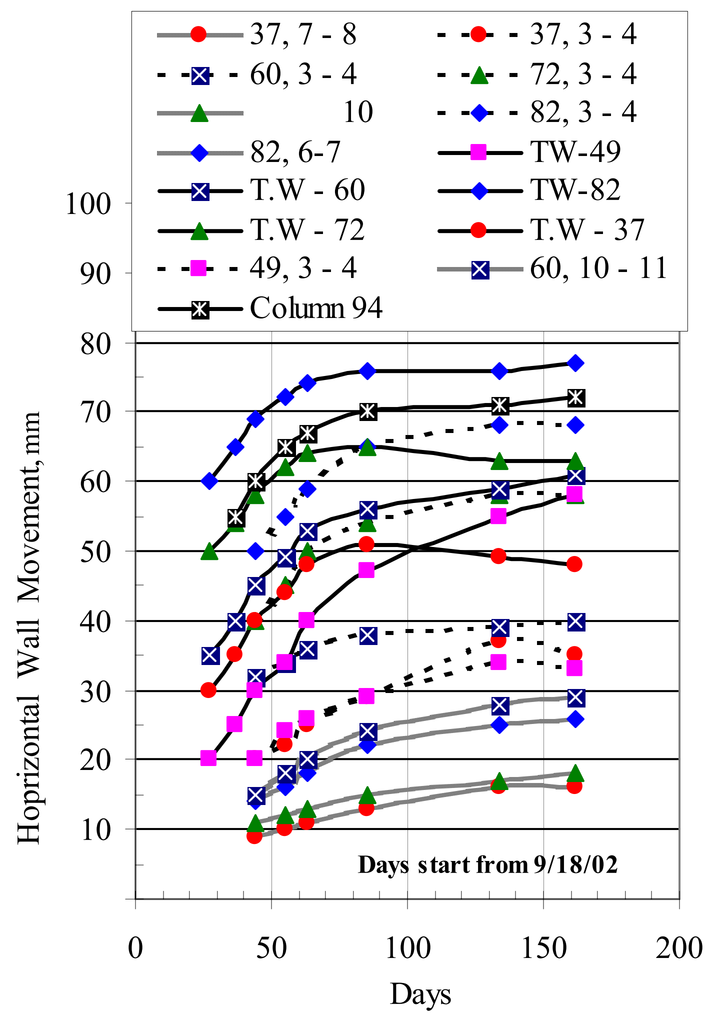

Figure. 5. Recorded. Horizontal Movements of Soil Nailed Wall

Photo 8. Completed Soil Nailed Wall with Primary Facing

Photo 9. Drilling between double beams for tieback Wall

Photo 10. Installing a Sheathed Tieback Anchor in a Drilled Hole

Photo 11. Drilling for Return Walls on Side Slopes

Photo 12. Creek Return Walls and Rip Raps Slope Protection

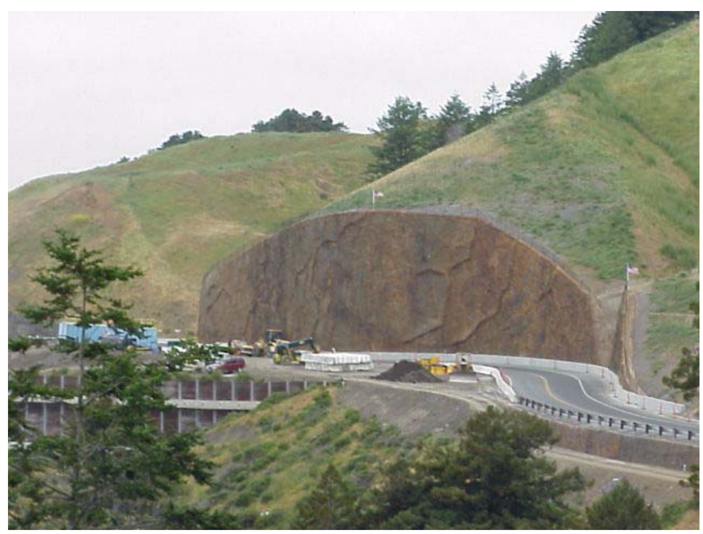

Photo 13. Stained Soil Nailed and Soldier Tieback Wall