FOR CONTRACT NO.: 05-0N8901

INFORMATION HANDOUT

MATERIALS INFORMATION

FOUNDATION REVIEW

ROUTE: 05-SLO-101-R34.2/R34.5

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: **Structure Design**

1. Design
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services

1. GD - North ; South ; West
2. GS File Room

Date: 7/7/10

West Cuesta Grade Ret Wall
Structure Name

05 - SLO - 101 - R34.2/34.5
District County Route ~~km~~ Post M.

District Project Development ______ District Project Engineer

05-0N8901 E.A. Number ______ Structure Number

Foundation Report By: D. Appelbaum Dated: 4/6/10

Reviewed By: D. Murray (SD) R. Price (GS)

General Plan Dated: 5/25/10 Foundation Plan Dated: 5/25/10

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

Pile Types and Design Loads	Footing Elevations, Design Loads, and Locations	Effect of Fills on Abutments and Bents
Pile Lengths	Seismic Data	Fill Surcharge
Predrilling	Location of Adjacent Structures and Utilities	Approach Paving Slabs
Pile Load Test	Stability of Cuts or Fills	Scour
Substitution of H Piles For Concrete Piles ☐ Yes ☒ No	Fill Time Delay	Ground Water
		Tremie Seals/Type D Excavation

David P. Murray 18
Structure Design Bridge Design Branch No.

[signature]
Geotechnical Services

Rev. 06/02

State of California | Business, Transportation and Housing Agency

Department of Transportation

Memorandum

Flex your power!
Be energy efficient!

To: MICHAEL POPE
Branch Chief
Office of Structure Design Central
Bridge Design Branch 18-MS9-DES.18

Date: April 6, 2010

File: 05-SLO-101-R34.2/R34.5
EA 05-0N8901
West Cuesta Grade
Retaining Wall

Attn: David P. Murray
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

Subject: Revised Foundation Report

A revised Foundation Report (FR) is provided for the West Cuesta Grade Retaining Wall due to changes in design loads provided by Structure Design and revisions to tieback unbonded lengths proposed by Geotechnical Design. This report supercedes the Foundation Report dated February 18, 2010, and is intended to stand alone. Copies of the February 18, 2010 Foundation Report should be purged from the project files.

Located on Route 101 in San Luis Obispo County between the Cities of San Luis Obispo and Santa Margarita, the project proposes to construct a soldier pile retaining wall with timber lagging and tiebacks to stabilize a landslide on the roadway embankment. A Vicinity Map showing the project location is presented as Attachment 2.

The recommendations presented herein are based on reviews of published data, site reconnaissance, subsurface investigations, and laboratory testing. The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the structure portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of differing site conditions.

References

The following publications and references were used to assist in the evaluation of site conditions:

1. *Caltrans ARS Online.*

2. *District Preliminary Geotechnical Report, Michael S. Finegan, California Department of Transportation, March 29, 2007.*

3. *Route 101/Cuesta Grade Improvements Project Final Geotechnical Report, AGS, Inc., May 1993.*

Existing Facilities and Proposed Improvements

Route 101 between San Luis Obispo and Santa Margarita over the Cuesta Summit is a six-lane conventional highway traversing rolling to mountainous terrain. The highway profile grade averages nearly seven percent through most of the project area. The roadway cross section consists of 12-foot lanes, 8-foot outside shoulders, and 6-foot inside shoulders.

Construction of Route 101 at its present location over the Cuesta Grade was completed in 1937. Replacing the original two lane concrete highway, it was the first four-lane highway in the county. In 1968, a roadway realignment was constructed to straighten three curves on the Grade. One of those curves is within the current project's limits. The highway was moved up to 100 feet to the west to straighten the curve. Approximately 50 feet of fill was placed at the new centerline to accomplish the realignment. A stabilization trench with a perforated metal pipe (PMP) underdrain was constructed at the toe of the fill to improve the stability of the slope.

The latest major construction, a project to widen Route 101 over Cuesta Grade to six lanes, began in 2000 and was completed in 2003. The embankment at the current project location, designated Fill 7, was widened approximately 23 to 26 feet to provide a platform for the new lanes. The slope was flattened from 1:1.5 to 1:1.9 to improve global stability. The embankment in its current configuration is up to 130 feet thick at the southbound shoulder.

The roadway embankments along Route 101 over the Cuesta Grade have experienced varying degrees of instability since the roadway was constructed on its present alignment. Fill 7 has continually moved, and has generated numerous reports from the California Highway Patrol and Caltrans Maintenance regarding settlement of the roadway. The October 18, 1956 Materials Report for a 1959 project to reinforce areas of weak roadbed and correct fill settlements noted that this fill had shown marked movement since its construction in 1938, but had not experienced any sudden large displacements. The Geotechnical Design Report (GDR) for the 2000 six-lane project characterized the embankment in its pre-construction configuration as meta-stable with hummocky slopes and shallow slope failures, up to 10 feet in length.

Settlement and cracking of the roadway in its current configuration first occurred during construction of the 2000 widening project, beginning in the northbound No. 1 lane between approximately Station 227+00 and Station 227+70 and subsequently expanding to include the southbound lanes from the fill abutment at approximately Station 225+40 northward to nearly Station 229+70. It was initially presumed that the roadway distress was caused by piping in a sand layer that had been exposed while excavating a haul road adjacent to the southbound lanes. Following an inspection by personnel from Construction and the Office of Geotechnical Design-North, the distressed area was patched with hot mix asphalt. After subsequent cracking and settlement, five 650-foot horizontal drains were installed at the base of the fill between Station 227+00 and Station 227+30 to help stabilize the embankment. The three northernmost drains intercepted groundwater, and continue to flow to this day.

Another failure occurred at the base of Fill 7 in January 2005, after an extended period of heavy rain. The slide was located approximately 500 feet left of Station 228+00. The body of the landslide was approximately 45 feet wide, 100 feet long, and had a 10 to 15-foot high near-vertical headscarp. The landslide had the characteristics of an earth-flow failure, and was apparently caused by a spring bursting through the embankment. The failed portion of the slope was reconstructed with a rockery. Loose material was excavated from the body of the slide, filter fabric was installed, and one ton-sized rock was placed over facing and backing rock to fill the landslide to original lines and grades. A stability trench with an underdrain system was constructed at the toe of the slope in the area of the failure to improve global stability.

Later in 2005, cracking developed in the southbound shoulder between approximately Station 225+55 and Station 229+90, with a noticeable dip appearing in the number 3 lane between approximately Station 227+20 and Station 227+50. The dip became more and more pronounced, until it was repaired with a hot mix asphalt overlay in August 2006. No additional roadway distress has been noted to date, but precipitation has been below average for the past four years.

Physical Setting

Topography and Drainage

The project corridor crosses the relatively rugged terrain of the Santa Lucia Mountain Range. Topographic relief is great. Elevations range from about 550 feet at the base of the Cuesta Grade to 1521 feet at Cuesta Pass. Nearby ridge tops exceed 2000 feet in elevation.

The major drainage in the project area is San Luis Obispo Creek, which flows in a southerly direction from Cuesta Pass to San Luis Obispo Bay near Avila Beach. Route 101 on the Cuesta Grade is located to the east of the creek.

Climate

The project area has the Mediterranean climate that is typical of much of California: cool, wet winters and warm, dry summers. Average annual precipitation ranges from about 22 inches in San Luis Obispo to approximately 41 inches at the summit of Cuesta Pass. Most of the precipitation occurs between November and March. Nearly all of the precipitation falls as rain, but dustings of snow are not uncommon at the higher elevations during the winter.

The mean daily maximum temperature in San Luis Obispo during the summer ranges from the mid to upper 70's (degrees Fahrenheit). Wintertime daily minimums average in the low to mid 40's. Temperatures north of the Cuesta Pass tend to be more extreme than in San Luis Obispo: warmer summertime temperatures and cooler wintertime temperatures.

Regional Geology

The project area is in the southern portion of the Coast Ranges geomorphic province, which extends from the Oregon border south to the Santa Ynez River. The Coast Ranges province is comprised of northwest-southeast trending mountain ranges and valleys, and is characterized by similarly trending faults and fold axes. Lithologies are complex, and range in age from Mesozioc to Holocene.

The project corridor crosses the Santa Lucia Range, which separates the coastal margin from the Salinas Valley in the project area. The Santa Lucia Range has a core of Mesozoic rocks: Franciscan complex in fault contact with Jurassic metavolcanics and serpentinite, Jura-Cretaceous interbedded shale/claystone with minor sandstone, and Cretaceous interbedded sandstone and shale. Mid to late Tertiary marine sedimentary and volcanic rocks unconformably overlie the Mesozoic rocks.

Field Investigation and Laboratory Testing

Several geotechnical borings were conducted to locate the slope failure plane and to characterize subsurface conditions in the project area. In-situ soil strength parameters were determined using the Standard Penetration Test (SPT) for cohesionless soils and pocket penetrometer measurements of unconfined compressive strength for some of the cohesive soils. Installation and monitoring of survey points was undertaken to determine the direction and magnitude of surficial slope movement.

Laboratory tests were performed on some of the soils samples collected during a 1997 subsurface investigation to estimate the shear strength of the embankment soils. Representative soil samples obtained at depth during more recent subsurface investigations were tested to determine corrosion potential.

Geotechnical Engineering Considerations

Site Geology

Geologic mapping of the project area prepared by AGS, Inc. for the 2000 widening project indicates that the project area is underlain by Toro Formation, Monterey Formation, and artificial fills and embankments. Fill 7, the predominant feature of the project area, was constructed across a drainage that contains variable depths of alluvium, generally consisting of sandy lean clay with gravel. The alluvium can be distinguished from the man-made fill by the rounded shape of its gravels.

The cut slope on the easterly side of the highway to the north of the drainage is in Monterey Formation. The Miocene age Monterey Formation consists of cherty shale and interbedded laminated shale.

The cut slope on the easterly side of the highway, south of the drainage is in Toro Formation. The Toro Formation consists of interbedded shale/claystone and sandstone. Fossils indicate an age ranging from late Jurrasic to early Cretaceous. Toro shale is prevalent in the project area. Deep clayey soil develops on the shale, which slakes rapidly upon exposure or from alternating wetting and drying. Many of the artificial fills in the project area are constructed of material derived from Toro Formation.

Seismicity

The project area is located within a seismically active region of California. Based on the *2009 Caltrans Seismic Design Procedure*, the controlling fault at this site is the Oceanic-fault zone, a reverse fault with a maximum magnitude 7.4. Using the Caltrans ARS Online measuring tool, the fault was determined to lie approximately 0.7 kilometers southwest of the project site. According to the Caltrans-adopted Chiou & Youngs and Campbell & Bozorgnia ground motion prediction equation (CY-CB GMPE) and the 2009 USGS Probabilistic Seismic Hazard Analysis Interactive Deaggregation Tool, the peak ground acceleration in the project area due to an earthquake along the Oceanic fault zone is estimated to be 0.6 g (gravity). An average shear wave velocity of 238 meters per second for the upper 100 feet of soil was plugged into the ground motion models to calculate peak ground acceleration. The average shear wave velocity was estimated using correlations to soil undrained shear strength.

Liquefaction potential under existing soil and ground water conditions is considered low. Loose to medium dense cohesionless soils below the water table are most susceptible to liquefaction during seismic shaking. Soils encountered in the subsurface borings were generally gravelly and sandy clays in embankment areas and sandy lean clays in the alluvial valleys. These cohesive soils are generally not susceptible to liquefaction.

Scour Evaluation

There is no potential for scour at this site.

Subsurface Conditions

Two geotechnical borings were conducted at the base of Fill 7 in 1965 for the design of the 1968 realignment project. Boring PH-8, located approximately 345 feet left of Station 226+21, was advanced to a depth of 35 feet. The material encountered was logged as silty clay and rock. Boring PH-9, located approximately 345 feet left of Station 226+38, was advanced to a depth of 60 feet. The soil extracted from that boring was logged as silty clay.

Three rotary wash borings were completed at roadway level in January 1997 to characterize subsurface soil conditions for the 2000 highway-widening project. The borings show that the existing fill consists of silty clay and medium plasticity clayey sand with gravel. Soil consistency ranges from firm to hard. Unified Soil Classification System (USCS) classifications determined from laboratory testing were CL, sandy lean clay and gravelly lean clay; and CH, sandy fat clay. Boring R-97-045, located 29 feet left of Station 226+05, encountered approximately 85 feet of fill and alluvium overlying bedrock. Boring R-97-046, located 31 feet left of Station 227+82, encountered approximately 60 feet of fill and alluvium overlying bedrock. Boring R-97-047, located 34 feet left of Station 231+37, encountered 50 feet of fill overlying bedrock.

In response to the roadway cracking during construction of the 2000 six-lane project, a slope inclinometer was installed in the southbound shoulder of the roadway during October 2000 to monitor subsurface slope movement. Boring R-00-703, located 33 feet left of Station 230+33, encountered 60 feet of fill and alluvium overlying bedrock.

Additional slope inclinometers were installed in October 2002 because of continued slope movement. Boring R-02-201, located 48 feet left of Station 226+38, encountered 99 feet of fill and alluvium over bedrock. Boring R-02-203 was drilled adjacent to boring R-00-703 to replace the slope inclinometer in that hole that was destroyed during the highway construction. The new boring was not logged. Boring R-02-204, located 86 feet right of Station 226+58, encountered approximately 88 feet of fill and alluvium overlying bedrock.

In December 2003 Crux Drilling, a specialty drilling contractor, was hired to conduct three geotechnical borings and install slope inclinometers on the embankment slope of Fill 7. Boring R-03-001, located 213 feet left of Station 226+38, encountered approximately 118 feet of fill and alluvium overlying Toro Formation. Boring R-04-002, located 394 feet left of Station 226+54, encountered 40 feet of fill and alluvium overlying Toro Formation. Boring R-04-003, located 213 feet left of Station 229+50, encountered approximately 28 feet of fill overlying Toro Formation.

Two additional slope inclinometers were installed in the southbound shoulder of the highway, evenly spaced between the existing inclinometers in borings R-02-201 and R-02-203, in February 2006. Boring R-06-205, located 47 feet left of Station 227+46, encountered approximately 105 feet of fill and alluvium overlying bedrock. Boring R-06-206, located 48 feet left of Station 228+78, encountered approximately 62 feet of fill overlying bedrock.

Six additional geotechnical borings were conducted during the months of August and September 2009 to better characterize the subsurface conditions at the northerly end of the proposed retaining wall and in the tieback zone. Boring R-09-001, located 47 feet left of Station 234+28, encountered 10 feet of fill over lying shale. Boring R-09-002, located 47 feet left of Station 232+94 encountered 32 feet of fill overlying shale. Boring R-09-003, located 46 feet right of Station 233+67, encountered 8 feet of fill overlying shale and sandstone. Boring R-09-004, located 57 feet right of Station 225+26, encountered approximately 22 feet of fill and alluvium overlying shale and siltstone. Boring R-09-005, located 65 feet right of Station 229+38, encountered approximately 11 feet of fill overlying shale and siltstone. Boring R-09-006, located 64 feet right of Station 227+57, encountered approximately 71 feet of fill and alluvium overlying sandstone.

Ground Water

Ground water was monitored in open-standpipe observation wells installed in borings PH-8, PH-9, R-97-045, R-97-046, and R-97-047. No water was encountered in borings R-97-046 and R-97-047. Water was measured 97.8 feet below the road surface at elevation 1115.1 feet, within the Toro Formation, in boring R-97-045. In boring PH-8, ground water was encountered 25.9 feet below the ground surface, at elevation 997 feet. Ground water was measured at 11.8 feet below the ground surface at elevation 1010.1 feet in boring PH-9.

The ground water regime in the project area can best be described as chaotic. Ground water is encountered sporadically where it flows through continuous fractures in the bedrock.

Corrosion

Representative soil samples taken during the subsurface investigation were tested for corrosion potential. The Department considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Chloride concentration is greater than or equal to 500 ppm
- Sulfate concentration is greater than or equal to 2000 ppm
- The pH is 5.5 or less

Since resistivity serves as an indicator parameter for the possible presence of soluble salts, tests for sulfate and chloride are usually not performed unless the resistivity of the soil is 1,000 ohm-cm or less.

Corrosion Test Summary

Boring	Sample Depth	PH	Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
R-09-002	4.8'-6.3'	7.0	3230	N/A	N/A
	17.5'-17.0'	7.4	1440	N/A	N/A
	19.8'-21.3'	8.1	1850	N/A	N/A
R-09-005	4.8'-6.3'	7.9	2580	N/A	N/A
	9.8'-11.3'	8.0	1650	N/A	N/A
R-09-006	9.8'-11.3'	7.8	2160	N/A	N/A
	14.8'-16.3'	7.8	2270	N/A	N/A
	19.8'21.3'	7.9	2080	N/A	N/A
	24.8'-26.3'	7.8	1960	N/A	N/A
	34.8'-36.3'	8.0	1990	N/A	N/A
	39.8'-41.3'	8.1	1850	N/A	N/A
Corrosive if		<5.5	<1000	>2000	>500

Based on corrosion test results, and because the project area is not within 1000 feet of salt or brackish water, the site is considered non-corrosive.

Geotechnical Analysis

Slope inclinometer (SI) readings indicate slope movement near the contact between the embankment constructed as part of the 2000 widening project and the underlying pre-existing embankment. Movement was also recorded deeper, within bedrock, in the SI installed in boring R-02-203, located 48 feet left of Station 230+32. The magnitude of the latter movement, however, is very small: approximately ¼-inch over the course of seven years. Furthermore, no surface expression of the movement has been observed to date. In geotechnical boring R-00-703, located approximately 15 feet right of boring R-02-203, a loss of drilling fluid was noted at approximately the same elevation as the slope inclinometer movement in boring R-02-203. This loss of fluid likely indicates a continuous fracture zone in the bedrock. Displacements along fractures that dip towards a slope face are likely, particularly if ground water is present and the discontinuities dip steeply towards the slope face.

Monthly monitoring of survey points that were installed in November 2005 has not yet revealed a pattern of displacement, but tape measurements between PK nails that were placed on both sides of cracks in the southbound outside shoulder in June 2005 indicate that some of the cracks expanded more than an inch between June 2005 and June 2006.

some of the cracks expanded more than an inch between June 2005 and June 2006. Additional cracks have appeared in the pavement and in the dirt shoulder since monitoring of the roadway began. It appears that the entire embankment between Station 225+40 and Station 233+60 that was widened as part of the 2000 six-lane project is moving.

Residual shear strength of the sliding mass was back calculated by modeling the slope in SLOPE/W, a slope stability computer program. The slope model was evaluated using the Morgenstern-Price method, a limit equilibrium type of analysis for assessing slope stability that satisfies both force equilibrium and moment equilibrium equations of statics. A specified failure surface, estimated from SI readings, was used in the analysis. The embankment soil was assumed to have a friction angle of 30 degrees and 200 psf of cohesion, strength values determined from laboratory tests conducted on samples obtained during the design of the 2000 widening project. The soil from the sliding mass was assumed to have no cohesion, and the friction angle was determined in an iterative process until the slope stability factor of safety was calculated to be slightly lower than unity. The moist unit weight of both the original embankment soil and the new embankment soil was assumed to be 120 pounds per cubic foot. The influence of ground water on slope stability was modeled using a pore water pressure ratio, R_u, of 0.04 in the sliding soil mass. A residual effective friction angle of 28 degrees with 0 psf of cohesion was calculated for the sliding soil mass.

Recommendations

It is recommended that a soldier pile retaining wall with timber lagging and ground anchors be constructed approximately 8 feet left of the southbound Route 101 edge of pavement between Station 224+91.84 and Station 233+57.25 to mitigate ongoing slope movement. The bottom of lagging should extend approximately 8 feet below the interface between the pre 2000 embankment and the post 2000 embankment. Soil excavated from in front of the wall to facilitate installation of the lagging should be replaced to an elevation that will result in a 10-foot wide bench between the face of the lagging and the sloped face of the embankment. The bench should be sloped a minimum of 2% away from the face of lagging to provide for positive drainage away from the retaining wall. The slope in front of the wall should be considered to be meta-stable, and should not be relied upon for passive resistance above the bottom of lagging elevation.

Gaps should be provided between lagging members to allow ground water to drain from behind the wall. Filter fabric must be provided between the lagging and the retained soil to prevent the migration of soil through the gaps between lagging members.

Earth Pressure

Coulomb Theory was used to calculate active lateral earth pressure coefficients for the soil. Passive lateral earth pressure coefficients were calculated using the logarithmic spiral method. The following table presents the soil strength parameters and lateral earth pressure coefficients that are recommended for the anchored soldier pile retaining system design:

Recommended Soil Strength Parameters

Location	Friction Angle (degrees)	Cohesion (psf)	Unit Weight (pcf)	Active Earth Pressure Coefficient (K_a)	Passive Earth Pressure Coefficient (K_p)
Lagged Section	28	0	120	0.36	2.80
Below Lagging	30	200	120	0.33	3.04

Tieback Anchors

The soils beneath and behind the sliding mass are clayey, or will likely degrade to clay during the design life of the retaining wall. Ground anchors in cohesive soils are susceptible to long-term creep and consequently are not recommended for permanent structures. Therefore, it is recommended that the unbonded portion of the ground anchors extend beyond the cohesive soils, into bedrock. The bedrock in the project area is predominantly shale. Past experience constructing ground anchors in the project vicinity suggests that the unit ultimate grout to ground bond stress in the shale is approximately 35 psi. That value equates to an ultimate ground anchor transfer load of approximately 8 kips per foot when the anchor is grouted into a six-inch diameter hole. Testing has shown that bonded lengths greater than 40 feet will not significantly increase anchor capacity unless specialized methods of installation are used to transfer load from the top of the anchor bond zone towards the end of the anchor. Therefore, the maximum bonded length of the ground anchors should be 40 feet, resulting in an ultimate transfer load of approximately 320 kips per ground anchor for anchors placed in six-inch diameter holes.

Soldier Piles

The ultimate geotechnical axial capacity of the soldier piles was calculated using the Thomlinson method for axial capacity of drilled shafts in cohesive soil. It was assumed that the foundation soils could degrade to clay during the design life of the structure. A conservative value of 1400 psf for soil undrained shear strength was used in the analysis. A factor of safety of two was used to determine the allowable geotechnical axial capacity of the soldier piles. The piles are assumed to obtain their axial capacity from skin friction only.

The attached Pile and Tieback Data table summarizes design loads, minimum pile tip elevations, tieback locations, and tieback unbonded lengths for the proposed retaining wall. The table was prepared using data provided to this office by Structure Design in an e-mail dated March 16, 2010. Table values were calculated using the following data and assumptions:

- Soldier pile spacing will be 8 feet beginning at Retaining Wall Station 10+00 and ending at Station 18+80.
- Bottom of lagging elevations were estimated from the minimum elevations shown on the December 2009 retaining wall profile prepared by District Design. The bottom of lagging was stepped down or up in increments of one foot, assuming that the nominal height of the individual lagging members would be 12 inches.
- Wall height is the difference between top of wall elevation and bottom of lagging elevation.
- Vertical loads were calculated as the vertical component of the tieback load (provided by Structure Design) inclined 15 degrees down from horizontal.
- Pile embedment was calculated assuming that the diameter of the drilled holes would be 36 inches. The pile embedment is the difference between pile tip elevation and bottom of lagging elevation.
- Tieback elevations shown are at the retaining wall layout line, and are at the given depths below top of wall elevation. Top of wall elevations were determined from the December 2009 retaining wall profile prepared by District Design.
- Tieback unbonded length is the distance between the face of the retaining wall and the interface between soil and bedrock, sloping down 15 degrees from horizontal. Top of rock profiles were generated from a digital terrain model (DTM) of the top of rock surface. The top of rock surface was estimated from subsurface investigations and inspection of surface topography of rock slopes on the easterly side of the highway.

Construction Considerations

Difficult drilling conditions can be expected for the soldier pile holes. The holes at the ends of the retaining wall will extend through bedrock below the bottom of lagging elevation. Some of the rock encountered during the geotechnical borings was logged as very hard.

Furthermore, caving conditions are likely while drilling through the alluvium and man-made fill due to the high gravel content of the soils. Temporary casing may be necessary to maintain an open hole.

Due to the size of the soldier pile holes in relation to the pile spacing the contractor may have to sequence his operations to avoid drilling one hole adjacent to another, open, hole. Caving is likely if soldier pile holes are drilled consecutively and left open.

Ground water may be expected to enter the borings for the soldier piles through the fractures in the bedrock. Depending of the contractor's equipment and methodologies, a significant amount of water may enter the hole before the contractor is able to place concrete. Temporary casing and/or pumping may be necessary to ensure a dry hole in which to place piles and pour concrete. The appropriate specification language should be included in the contract special provisions to address the possibility of accumulated water in the soldier pile holes.

Horizontal borings for the ground anchors may encounter caving conditions and ground water. Temporary casing may be required. Borings for ground anchors on retaining walls that were part of the 2000-widening project had to be cased for much of their length to maintain open holes. The geologic materials present at the sites of those walls were very similar to the materials that will be encountered at the present project location.

Loss of drilling fluid circulation was noted during exploratory drilling operations. "Grout socks" may be necessary during ground anchor installation to prevent excessive grout loss into the fractured rock.

Closure

Standard Special Provision S5-280, "Project Information", discloses to bidders and contractors a list of pertinent information available for their inspection prior to bid opening. The Department makes the following supplemental project information available:

Supplemental Project Information

Means	Description
Included in the Information Handout	Foundation Report for the West Cuesta Grade Retaining Wall dated April 6, 2010.
Available for inspection at the District Office	Borehole Core Samples.
Available for inspection at the Transportation Laboratory	
Available for inspection at _____; telephone (___) - ____	
Available as specified in the Standard Specifications	
Available at: http://www.dot.ca.gov/hq/esc/oe/weekly_ads/index.php	

The District Office is located at 50 Higuera Street, San Luis Obispo, California, 93401.

Project Log of Test Borings have been finalized by this office and are being drafted by the Engineering Graphics Unit. Your office will be notified once they have been completed. For information regarding the status and delivery of the LOTB's, contact Irma Gamarra-Remmen at (916) 227-7203.

An exception to the Department's policy regarding procedures and standards for reporting subsurface conditions presented in the Soil & Rock Logging, Classification and Presentation Manual, dated June 2007 has been approved. Many of the geotechnical borings referenced in the Log of Test Borings were conducted before the manual was issued. Some of the field descriptions of the subsurface materials deviate from the standards as follows:

- Soil and rock colors described in the boring records do not correspond to the color names from the *Munsell Color System*.
- Consistencies reported for some of the cohesive soils were not based on measurements using a pocket penetrometer or torvane.
- Percent or proportion of soil constituent sizes were not explicitly reported, rather they were assumed from the group names.

If you have any questions or comments, please contact Dan Appelbaum at (805) 549-3745 or Mike Finegan at (805) 549-3194.

Supervised by,

REGISTERED PROFESSIONAL ENGINEER
D.L. Appelbaum
No. C 50001
Exp. 6-30-11
CIVIL
STATE OF CALIFORNIA

DANIEL L. APPELBAUM, PE
Transportation Engineer
Geotechnical Design – North
Branch D

MICHAEL S. FINEGAN, PE, Chief
Geotechnical Design - North
Branch D

c: Roy Bibbens / GDN File
GS File Room
Job File / Branch D Records
Kelly Holden (E-copy)
Andrew Tan (E-copy)
Douglas Lambert (E-copy)
R.E. Pending

LIST OF ATTACHMENTS

ATTACHMENT 1	**PILE AND TIEBACK DATA**
ATTACHMENT 2	**VICINITY MAP**
ATTACHMENT 3	**TYPICAL CROSS SECTION**

Pile and Tieback Data

Pile No.	Station	Top of Wall Elevation	Bottom of Lagging Elevation	Wall Height	Tieback Load	Vertical Load	Pile Embedment	Pile Tip Elevation	Tiebacks		
									Depth	Elevation	Unbonded Length
		(ft)	(ft)	(ft)	(Kips)	(Kips)	(ft)	(ft)	(ft)	(ft)	(ft)
1	10+00.00	1204.37	1190.25	14	35	9	11	1179.25	4.0	1200.37	95
2	10+08.00	1204.93	1190.25	15	35	9	11	1179.25	4.0	1200.93	95
3	10+16.00	1205.49	1190.25	15	43	11	12	1178.25	4.0	1201.49	95
4	10+24.00	1206.05	1190.25	16	43	11	12	1178.25	4.0	1202.05	95
5	10+32.00	1206.62	1190.25	16	43	11	12	1178.25	4.0	1202.62	95
6	10+40.00	1207.19	1190.25	17	73	19	14	1176.25	5.0	1202.19	105
7	10+48.00	1207.77	1190.25	18	73	19	14	1176.25	5.0	1202.77	120
8	10+56.00	1208.34	1190.25	18	73	19	14	1176.25	5.0	1203.34	120
9	10+64.00	1208.94	1190.25	19	73	19	14	1176.25	5.0	1203.94	120
10	10+72.00	1209.55	1190.25	19	91	24	15	1175.25	6.0	1203.55	130
11	10+80.00	1210.17	1190.25	20	91	24	15	1175.25	6.0	1204.17	130
12	10+88.00	1210.78	1190.25	21	91	24	15	1175.25	6.0	1204.78	140
13	10+96.00	1211.40	1190.25	21	106	27	16	1174.25	6.0	1205.40	150
14	11+04.00	1212.01	1190.25	22	106	27	16	1174.25	6.0	1206.01	160
15	11+12.00	1212.61	1191.25	21	106	27	16	1175.25	6.0	1206.61	170
16	11+20.00	1213.22	1191.25	22	106	27	16	1175.25	6.0	1207.22	180
17	11+28.00	1213.83	1191.25	23	106	27	16	1175.25	6.0	1207.83	195
18	11+36.00	1214.45	1191.25	23	132	34	18	1173.25	8.0	1206.45	195
19	11+44.00	1215.07	1191.25	24	132	34	18	1173.25	8.0	1207.07	200
20	11+52.00	1215.71	1192.25	23	132	34	18	1174.25	8.0	1207.71	210
21	11+60.00	1216.34	1192.25	24	132	34	18	1174.25	8.0	1208.34	220
22	11+68.00	1217.00	1192.25	25	132	34	18	1174.25	8.0	1209.00	220
23	11+76.00	1217.66	1192.25	25	132	34	18	1174.25	8.0	1209.66	220
24	11+84.00	1218.32	1194.25	24	132	34	18	1176.25	8.0	1210.32	220
25	11+92.00	1218.99	1194.25	25	132	34	18	1176.25	8.0	1210.99	210
26	12+00.00	1219.66	1194.25	25	132	34	18	1176.25	8.0	1211.66	210
27	12+08.00	1220.32	1194.25	26	132	34	18	1176.25	8.0	1212.32	210
28	12+16.00	1220.99	1196.25	25	132	34	18	1178.25	8.0	1212.99	200

Pile and Tieback Data

Pile No.	Station	Top of Wall Elevation	Bottom of Lagging Elevation	Wall Height	Tieback Load	Vertical Load	Pile Embedment	Pile Tip Elevation	Tiebacks		
									Depth	Elevation	Unbonded Length
		(ft)	(ft)	(ft)	(Kips)	(Kips)	(ft)	(ft)	(ft)	(ft)	(ft)
29	12+24.00	1221.65	1196.25	25	150	39	20	1176.25	8.0	1213.65	200
30	12+32.00	1222.31	1196.25	26	150	39	20	1176.25	8.0	1214.31	200
31	12+40.00	1222.97	1196.25	27	150	39	20	1176.25	8.0	1214.97	200
32	12+48.00	1223.65	1199.25	24	150	39	20	1179.25	8.0	1215.65	190
33	12+56.00	1224.33	1199.25	25	150	39	20	1179.25	8.0	1216.33	185
34	12+64.00	1225.01	1199.25	26	150	39	20	1179.25	8.0	1217.01	185
35	12+72.00	1225.68	1199.25	26	150	39	20	1179.25	8.0	1217.68	180
36	12+80.00	1226.35	1199.25	27	150	39	20	1179.25	8.0	1218.35	180
37	12+88.00	1227.00	1202.25	25	150	39	20	1182.25	8.0	1219.00	180
38	12+96.00	1227.66	1202.25	25	150	39	20	1182.25	8.0	1219.66	170
39	13+04.00	1228.32	1202.25	26	150	39	20	1182.25	8.0	1220.32	170
40	13+12.00	1228.99	1202.25	27	150	39	20	1182.25	8.0	1220.99	170
41	13+20.00	1229.65	1205.25	24	150	39	20	1185.25	8.0	1221.65	160
42	13+28.00	1230.32	1205.25	25	150	39	20	1185.25	8.0	1222.32	145
43	13+36.00	1230.99	1205.25	26	150	39	20	1185.25	8.0	1222.99	130
44	13+44.00	1231.66	1205.25	26	150	39	20	1185.25	8.0	1223.66	130
45	13+52.00	1232.33	1208.25	24	150	39	20	1188.25	8.0	1224.33	125
46	13+60.00	1233.00	1208.25	25	150	39	20	1188.25	8.0	1225.00	125
47	13+68.00	1233.62	1208.25	25	150	39	20	1188.25	8.0	1225.62	125
48	13+76.00	1234.24	1208.25	26	150	39	20	1188.25	8.0	1226.24	125
49	13+84.00	1234.85	1208.25	27	150	39	20	1188.25	8.0	1226.85	120
50	13+92.00	1235.46	1211.25	24	150	39	20	1191.25	8.0	1227.46	120
51	14+00.00	1236.07	1211.25	25	150	39	20	1191.25	8.0	1228.07	120
52	14+08.00	1236.67	1211.25	25	150	39	20	1191.25	8.0	1228.67	110
53	14+16.00	1237.30	1211.25	26	150	39	20	1191.25	8.0	1229.30	110
54	14+24.00	1237.95	1213.25	25	150	39	20	1193.25	8.0	1229.95	105
55	14+32.00	1238.61	1213.25	25	150	39	20	1193.25	8.0	1230.61	105
56	14+40.00	1239.28	1213.25	26	150	39	20	1193.25	8.0	1231.28	105

Pile and Tieback Data

Pile No.	Station	Top of Wall Elevation	Bottom of Lagging Elevation	Wall Height	Tieback Load	Vertical Load	Pile Embedment	Pile Tip Elevation	Tiebacks		
									Depth	Elevation	Unbonded Length
		(ft)	(ft)	(ft)	(Kips)	(Kips)	(ft)	(ft)	(ft)	(ft)	(ft)
57	14+48.00	1239.95	1213.25	27	150	39	20	1193.25	8.0	1231.95	105
58	14+56.00	1240.61	1216.25	24	150	39	20	1196.25	8.0	1232.61	105
59	14+64.00	1241.28	1216.25	25	150	39	20	1196.25	8.0	1233.28	105
60	14+72.00	1241.93	1216.25	26	150	39	20	1196.25	8.0	1233.93	105
61	14+80.00	1242.59	1216.25	26	150	39	20	1196.25	8.0	1234.59	105
62	14+88.00	1243.24	1219.25	24	150	39	20	1199.25	8.0	1235.24	105
63	14+96.00	1243.88	1219.25	25	150	39	20	1199.25	8.0	1235.88	105
64	15+04.00	1244.53	1219.25	25	150	39	20	1199.25	8.0	1236.53	105
65	15+12.00	1245.16	1219.25	26	150	39	20	1199.25	8.0	1237.16	105
66	15+20.00	1245.80	1221.25	25	150	39	20	1201.25	8.0	1237.80	105
67	15+28.00	1246.46	1221.25	25	150	39	20	1201.25	8.0	1238.46	105
68	15+36.00	1247.11	1221.25	26	150	39	20	1201.25	8.0	1239.11	105
69	15+44.00	1247.74	1221.25	26	150	39	20	1201.25	8.0	1239.74	105
70	15+52.00	1248.35	1223.25	25	150	39	20	1203.25	8.0	1240.35	105
71	15+60.00	1248.96	1223.25	26	150	39	20	1203.25	8.0	1240.96	105
72	15+68.00	1249.54	1223.25	26	150	39	20	1203.25	8.0	1241.54	100
73	15+76.00	1250.13	1223.25	27	150	39	20	1203.25	8.0	1242.13	100
74	15+84.00	1250.72	1227.25	23	132	34	18	1209.25	8.0	1242.72	100
75	15+92.00	1251.30	1227.25	24	132	34	18	1209.25	8.0	1243.30	100
76	16+00.00	1251.88	1227.25	25	132	34	18	1209.25	8.0	1243.88	100
77	16+08.00	1252.45	1227.25	25	132	34	18	1209.25	8.0	1244.45	100
78	16+16.00	1253.02	1230.25	23	132	34	18	1212.25	8.0	1245.02	95
79	16+24.00	1253.59	1230.25	23	132	34	18	1212.25	8.0	1245.59	95
80	16+32.00	1254.16	1230.25	24	132	34	18	1212.25	8.0	1246.16	95
81	16+40.00	1254.73	1230.25	24	132	34	18	1212.25	8.0	1246.73	95
82	16+48.00	1255.31	1233.25	22	132	34	18	1215.25	8.0	1247.31	90
83	16+56.00	1255.88	1233.25	23	132	34	18	1215.25	8.0	1247.88	90
84	16+64.00	1256.46	1233.25	23	132	34	18	1215.25	8.0	1248.46	90

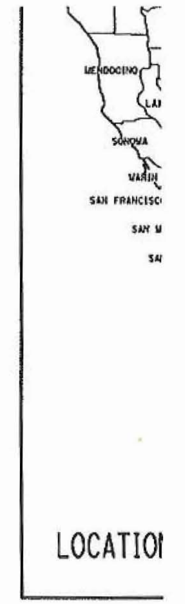
Pile and Tieback Data

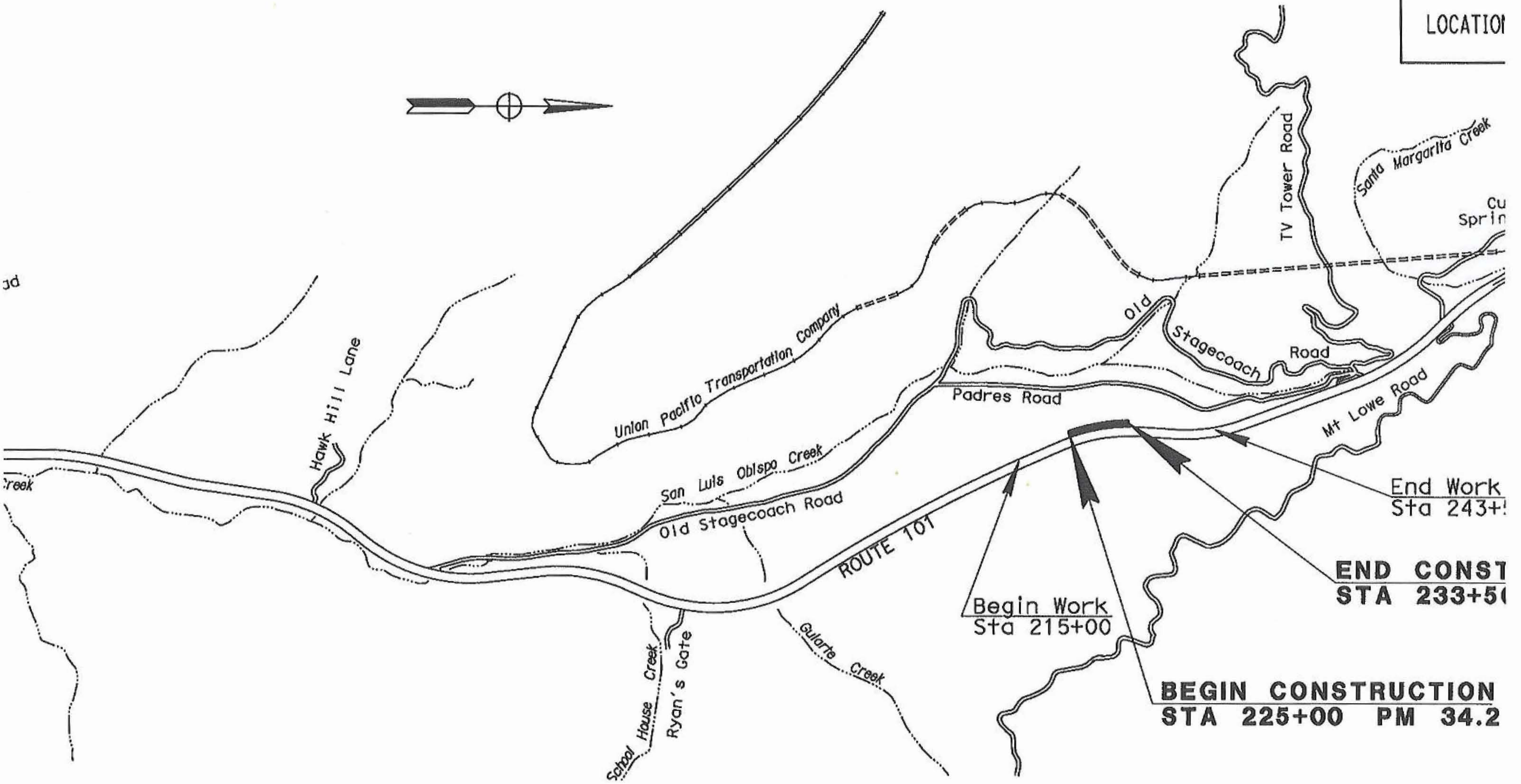
Pile No.	Station	Top of Wall Elevation	Bottom of Lagging Elevation	Wall Height	Tieback Load	Vertical Load	Pile Embedment	Pile Tip Elevation	Tiebacks		
									Depth	Elevation	Unbonded Length
		(ft)	(ft)	(ft)	(Kips)	(Kips)	(ft)	(ft)	(ft)	(ft)	(ft)
85	16+72.00	1257.04	1233.25	24	132	34	18	1215.25	8.0	1249.04	90
86	16+80.00	1257.62	1237.25	20	106	27	16	1221.25	6.0	1251.62	90
87	16+88.00	1258.20	1237.25	21	106	27	16	1221.25	6.0	1252.20	85
88	16+96.00	1258.79	1237.25	22	106	27	16	1221.25	6.0	1252.79	85
89	17+04.00	1259.37	1237.25	22	106	27	16	1221.25	6.0	1253.37	85
90	17+12.00	1259.95	1239.25	21	106	27	16	1223.25	6.0	1253.95	80
91	17+20.00	1260.53	1239.25	21	106	27	16	1223.25	6.0	1254.53	80
92	17+28.00	1261.09	1239.25	22	106	27	16	1223.25	6.0	1255.09	80
93	17+36.00	1261.65	1239.25	22	106	27	16	1223.25	6.0	1255.65	80
94	17+44.00	1262.21	1241.25	21	106	27	16	1225.25	6.0	1256.21	80
95	17+52.00	1262.78	1241.25	22	106	27	16	1225.25	6.0	1256.78	80
96	17+60.00	1263.34	1241.25	22	106	27	16	1225.25	6.0	1257.34	80
97	17+68.00	1263.91	1241.25	23	106	27	16	1225.25	6.0	1257.91	80
98	17+76.00	1264.48	1246.25	18	91	24	15	1231.25	6.0	1258.48	80
99	17+84.00	1265.04	1246.25	19	91	24	15	1231.25	6.0	1259.04	80
100	17+92.00	1265.61	1246.25	19	91	24	15	1231.25	6.0	1259.61	80
101	18+00.00	1266.17	1246.25	20	91	24	15	1231.25	6.0	1260.17	80
102	18+08.00	1266.74	1251.25	15	43	11	12	1239.25	4.0	1262.74	80
103	18+16.00	1267.32	1251.25	16	43	11	12	1239.25	4.0	1263.32	80
104	18+24.00	1267.89	1251.25	17	43	11	12	1239.25	4.0	1263.89	80
105	18+32.00	1268.46	1251.25	17	43	11	12	1239.25	4.0	1264.46	80
106	18+40.00	1269.03	1257.25	12	35	9	11	1246.25	4.0	1265.03	80
107	18+48.00	1269.60	1257.25	12	35	9	11	1246.25	4.0	1265.60	80
108	18+56.00	1270.17	1257.25	13	35	9	11	1246.25	4.0	1266.17	80
109	18+64.00	1270.74	1262.25	8	35	9	11	1251.25	4.0	1266.74	80
110	18+72.00	1271.31	1262.25	9	35	9	11	1251.25	4.0	1267.31	80
111	18+80.00	1271.88	1262.25	10	35	9	11	1251.25	4.0	1267.88	80

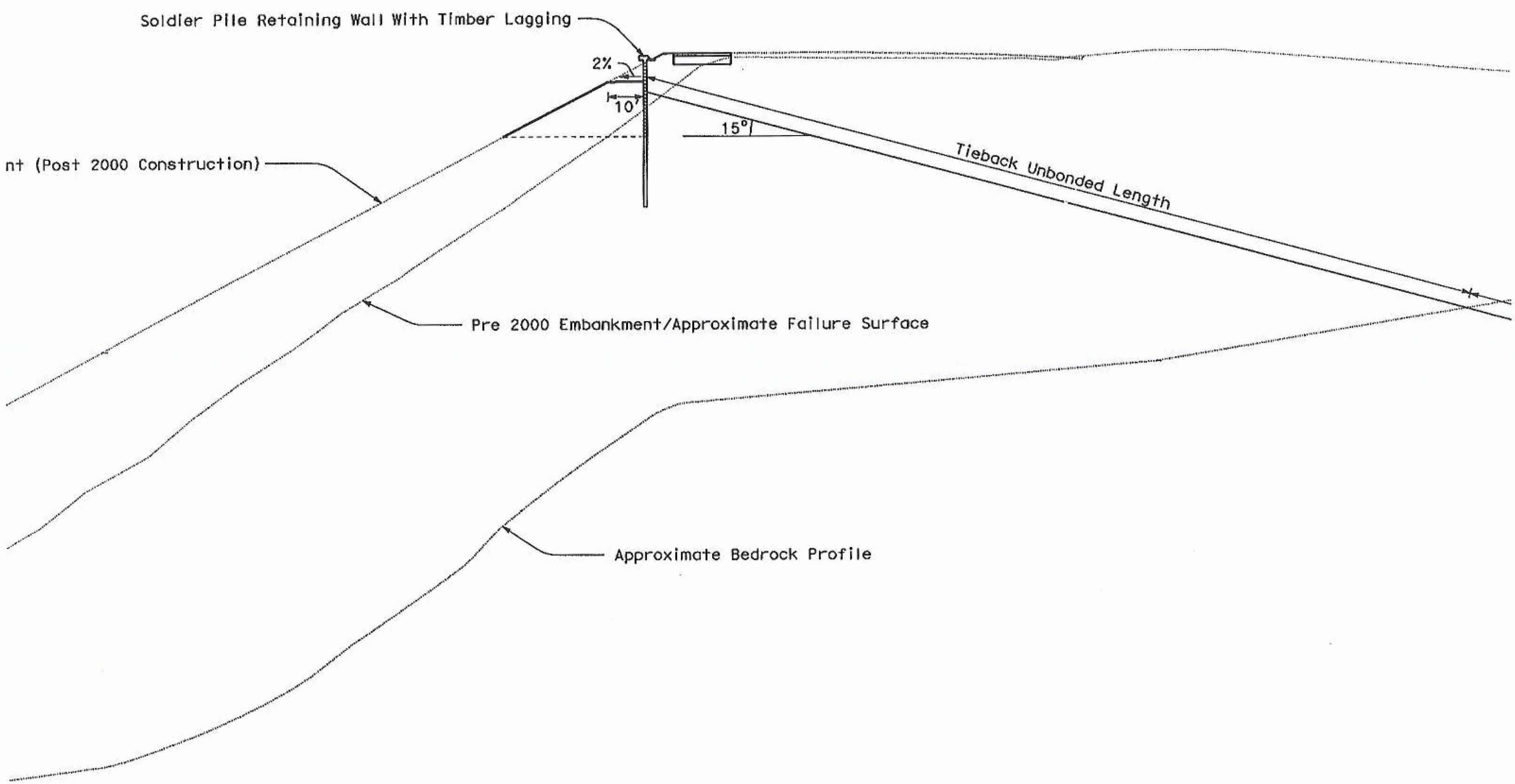
STATE HIGHWAY

IN SAN LUIS OBISPO COUNTY
ON ROUTE 101
BETWEEN SAN LUIS OBISPO AND SANTA MARGARITA

TO BE SUPPLEMENTED BY STANDARD PLANS DATED MAY 2006







TYPICAL CROSS SECTION