Memorandum

Flex your power!
Be energy efficient!

To: JAMES PERANO
Senior Transportation Engineer
Office of Design II, Branch A

Date: March 4, 2014

File: 05-SB-101-2.2/3.3
Linden Casitas
EA 05-4482U1
Project ID 0500000543

Attn: Amir Saedi

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

Subject: Geotechnical Design Report

Introduction

A Geotechnical Design Report (GDR) is provided for the Linden Casitas project. The project is located on State Route 101 in the City of Carpinteria in southern Santa Barbara County. Improvements are proposed to increase the highway level of service and accessibility, and include replacement of existing bridges and construction of new bridges, retaining walls, sound walls, ramps and frontage roads. Review of published geologic data and previous geotechnical reports, field reconnaissance, and geotechnical analysis were performed as part of the geotechnical investigation.

The purpose of this report is to document geotechnical conditions and recommend design and construction criteria. It is intended for use by the project design engineer, construction personnel, bidders, and contractors. This report supercedes the Preliminary Geotechnical Report (July, 2010).

Existing Facilities and Proposed Improvements

Route 101 within the project limits is a four-lane divided urban freeway with (2) 12-foot portland cement concrete (PCC) lanes in each direction and asphalt concrete (AC) 5-foot inside shoulders and 8-foot outside shoulders. A landscaped median approximately 40 feet wide with thrie beam barriers facing each direction of travel provides separation between the northbound and southbound lanes.

The purpose of the Linden Casitas project is to improve access and operations on Route 101, reduce local trips, and provide pedestrian and bicycle connectivity. The project includes construction of (3) mainline bridges on Route 101 to replace existing structures at Linden Avenue, Casitas Pass Road, and Carpinteria Creek. Highway access will be modified and added at Linden Avenue and Casitas Pass Road. Extension of Via Real, a frontage road, will also require construction of a new bridge over Carpinteria Creek. Retaining walls and sound walls are also proposed as part of the project.

Pertinent Reports and Investigations

The following publications were used to assist in the assessment of site conditions:

1. *Preliminary Geotechnical Report.* Finegan, Michael. EA 05-448200. December 21, 2000.

2. *District Preliminary Geotechnical Report.* De Llamas, Zeke. EA 05-448200, July 30, 2008.

Physical Setting

Topography and Climate

The project is located in the Carpinteria Valley, a sunken area north of the elevated coastal fault block southeast of the City of Carpinteria. Carpinteria Valley is a low-lying, flat plain comprised of alluvial deposits. Some low-lying areas in the valley are partly submerged under brackish water of the Carpinteria Slough. The plain meets the Pacific Ocean to the southwest, gradually gains elevation to the east as it approaches the Rincon and La Granada Mountains, and abuts sharply against the foothills of the Santa Ynez Mountains at its the northern boundary.

A mild coastal climate prevails throughout the year, with marine fog often present in spring and summer months. Nearly all precipitation falls from Pacific storms between October and May, with the majority accumulating in the winter months. Average yearly rainfall is 17.5 inches. The average temperature is 72 degrees Fahrenheit with more than 275 days of sunny weather per year.

Regional Geology

The project area is located in the Transverse Ranges Geomorphic Province. State Route 101 is bordered to the north by the Santa Ynez Mountains and to the south by the Pacific Ocean as it traverses the Carpinteria Valley. Geologic maps and field observations indicate that soils in the valley were deposited in an alluvial environment and consist of deposits of clay, silt, sand, and gravel.

Exploration

Drilling and Sampling

Preliminary subsurface investigations for the project began in 2000 and consisted of mud rotary power borings and cone penetrometer test (CPT) soundings along the highway corridor and near existing bridge structures. Information from the preliminary investigations was used to develop viable foundation alternatives for the proposed bridge and wall structures. After choosing a preferred project alternative and identifying the proposed design features, a targeted subsurface investigation was initiated in October of 2010 and was completed in January of 2013. Seventeen mud rotary borings (RC-10-001 through RC-13-017) and nine CPT soundings (CPT-11-001 through CPT-11-009) were performed at proposed structure locations. Bulk and undisturbed samples were obtained and sent to materials laboratories for testing. Rotary drilling was performed using self-casing wireline equipment with continuous sampling and penetration testing at five-foot intervals using the Standard Penetration Test (SPT). Relatively undisturbed samples of fine-grained soils were also collected using a thin-walled sampler and sealed brass tubes to preserve the condition of soil samples. Log of Test Borings (LOTB) are included in the project plans for each structure.

Table 1. Subsurface Exploration Summary

Boring	*Completion Date*	*Equipment*	*Hammer Type*	*Hammer Efficiency (%)*	*Approximate Ground Elevation (ft)*	*Depth (ft)*
HA-12-001	4-12-2012	Hand	N/A	N/A	35.7	16.0
RC-10-001	11-3-2010	CS2000	Auto	93	23.8	151.5
RC-10-002	11-16-2010	CS2000	Auto	93	42.5	82.0
RC-10-003	11-17-2010	CS2000	Auto	93	42.3	72.0
RC-10-004	12-1-2010	CS2000	Auto	93	24.5	171.5
RC-10-005	12-8-2010	CS2000	Auto	93	26.4	152.0
RC-10-006	12-15-2010	Acker	Auto	74	44.5	121.0
RC-11-007	1-12-2011	Acker	Auto	74	42.5	122.0
RC-11-008	1-26-2011	CS2000	Auto	93	45.0	152.0
RC-11-009	1-28-2011	CS2000	Auto	93	51.0	170.5
RC-11-010	1-30-2011	CS2000	Auto	93	52.4	172.0
RC-11-011	2-1-2011	CS2000	Auto	93	50.1	171.5
RC-11-012	2-2-2011	CS2000	Auto	93	25.8	91.5
RC-11-013	3-2-2011	CS2000	Auto	93	31.7	135.5
RC-11-014	3-1-2011	Acker	Auto	68	20.8	152.5
RC-11-015	10-26-2011	CS2000	Auto	84	42.7	106.5
RC-13-016	1-30-2013	CME 85	Auto	68	19.2	153.5
RC-13-017	12-18-2013	CS2000	Auto	85	25.2	152.0
CPT-11-001	6-22-2011	CPT	N/A	N/A	25.2	100.0
CPT-11-002	6-22-2011	CPT	N/A	N/A	26.6	100.0
CPT-11-003	6-22-2011	CPT	N/A	N/A	27.5	20.0
CPT-11-004	6-22-2011	CPT	N/A	N/A	27.3	100.0
CPT-11-005	6-22-2011	CPT	N/A	N/A	45.3	69.1
CPT-11-006	6-22-2011	CPT	N/A	N/A	45.7	85.0
CPT-11-007	6-22-2011	CPT	N/A	N/A	52.6	86.0
CPT-11-008	6-22-2011	CPT	N/A	N/A	50.8	87.0
CPT-11-009	6-22-2011	CPT	N/A	N/A	51.1	100.0

Geotechnical Testing

In Situ Testing

Standard penetration tests (SPT) were performed at 5-foot intervals in all of the mud-rotary borings drilled for the project. Results of the SPT were correlated to relative density and soil strength parameters for cohesionless soils encountered in the borings. Data from the CPT soundings was also used to correlate to relative density and undrained shear strength of encountered soils.

Laboratory Testing

Consolidated undrained (CU) triaxial shear with pore pressure measurement testing was performed on relatively undisturbed clay samples obtained at depth in the borings. Results of the triaxial shear testing were used to estimate undrained and drained shear strengths of clay soils encountered in the borings.

Soil gradation, Atterberg Limits, moisture content, and corrosion testing were also performed to classify the encountered soils and identify potential material issues. Results of the laboratory testing are referenced in the project LOTB’s.

Geotechnical Conditions

Groundwater

Open observation wells were installed to observe fluctuations in groundwater levels and determine if groundwater will influence construction and foundation design. Results of the groundwater monitoring program are summarized in Table 2.

Table 2. Groundwater Elevations

Boring	Date	Depth to Groundwater (ft)	Groundwater Elevation(ft)
B-2-00	2-9-2011	5.1	19.6
B-2-00	2-24-2012	5.3	19.4
B-2-00	3-22-2012	5.7	19.0
B-2-00	4-19-2012	4.9	19.8
B-2-00	2-5-2013	5.9	18.8
B-2-00	4-5-2013	5.7	19.0
B-2-00	5-23-2013	6.3	18.4
HA-12-001	4-19-2012	12.5	23.2
RC-10-002	4-12-2012	16.6	25.9
RC-10-003	4-12-2012	16.0	26.3
RC-10-004	2-9-2011	3.9	20.6
RC-10-004	12-14-2011	5.0	19.5
RC-10-004	2-24-2012	4.8	19.3
RC-10-004	3-22-2012	5.0	19.5
RC-10-004	4-19-2012	4.3	20.2
RC-10-004	7-26-2012	5.4	19.1
RC-10-004	2-5-2013	5.3	19.2
RC-10-004	4-5-2013	5.7	18.8
RC-10-004	5-23-2013	6.0	18.4
RC-10-005	2-24-2012	7.3	19.1
RC-10-005	4-19-2012	6.7	19.7
RC-10-006	2-9-2011	15.4	29.1
RC-10-006	12-14-2011	14.3	30.2
RC-10-006	2-24-2012	14.6	29.9
RC-10-006	3-22-2012	14.9	29.6
RC-10-006	4-19-2012	14.2	30.3
RC-10-006	2-5-2013	16.0	28.5
RC-10-006	4-5-2013	13.4	31.1
RC-10-006	5-23-2013	17.5	27.0
RC-11-008	2-9-2011	15.3	29.7
RC-11-008	12-14-2011	14.0	31.0
RC-11-008	2-24-2012	14.5	30.5
RC-11-008	3-22-2012	15.0	30.0
RC-11-008	4-19-2012	14.2	30.8
RC-11-008	2-5-2013	16.5	28.5
RC-11-008	4-5-2013	16.9	28.1
RC-11-008	5-23-2013	14.4	30.6
RC-11-009	2-9-2011	25.6	25.4
RC-11-009	12-14-2011	25.6	25.4
RC-11-009	2-24-2012	25.4	25.6
RC-11-009	3-22-2012	25.0	26.0

RC-11-009	4-19-2012	24.9	26.1
RC-11-009	7-26-2012	25.1	25.9
RC-11-009	2-5-2013	25.6	25.4
RC-11-009	4-5-2013	26.4	24.6
RC-11-009	5-23-2013	26.7	24.3
RC-11-010	2-9-2011	27.9	24.5
RC-11-010	2-5-2013	29.9	22.5
RC-11-010	4-5-2013	35.8	16.6
RC-11-015	12-14-2011	14.0	28.7
RC-11-015	2-24-2012	12.3	30.4
RC-11-015	3-22-2012	12.4	30.3
RC-11-015	4-19-2012	12.0	30.7
RC-11-015	4-19-2012	12.0	30.7
RC-11-015	2-5-2013	12.1	30.7
RC-11-015	4-5-2013	12.5	30.2
RC-11-015	5-23-2013	12.7	30.1
RC-13-016	2-5-2013	7.7	11.5
RC-13-016	4-5-2013	7.5	11.7

Corrosion Evaluation

The department considers a site to be potentially corrosive to the foundation elements if the following conditions exist for the representative soil and/or water samples taken at the site: minimum resistivity of 1000 ohm-cm or less and/or PH of 5.5 or less. Samples found to be potentially corrosive based on this criteria are sent to the Headquarters Material Laboratory for additional corrosion testing based on chloride and sulphate content.

Soil and water samples were obtained during the subsurface investigation and tested for corrosion potential at the District Materials Laboratory. The results of the corrosion testing are presented in Table 3. Based upon the results of the testing, the site is not considered corrosive to foundation elements.

Table 3. Corrosion Testing Summary

Boring	*Depth (ft)*	*pH*	*Resistivity ohm-cm*	*Chloride ppm*	*Sulphate ppm*	*Corrosive*
RC-10-001	6.5-10.0	8.1	1940	-	-	NO
RC-10-001	22.0-25.0	8.2	5600	-	-	NO
RC-10-001	48.0-50.0	8.0	2450	-	-	NO
RC-10-001	67.0-70.0	7.8	2150	-	-	NO
RC-10-002	1.0-3.0	8.2	3150	-	-	NO
RC-10-002	23.0-25.0	8.2	5620	-	-	NO
RC-10-003	1.0-4.0	8.0	4410	-	-	NO
RC-10-004	0.0-3.0	8.1	660	140	870	NO
RC-10-004	25.0-26.5	8.0	2650	-	-	NO
RC-10-004	57.0-59.0	8.2	6330	-	-	NO
RC-10-005	2.0-4.0	8.4	1260	-	-	NO
RC-10-005	47.0-49.0	8.4	2350	-	-	NO
RC-10-005	87-0-89.0	8.6	3560	-	-	NO
RC-10-006	8.0-10.0	7.9	1260	-	-	NO
RC-10-006	87.0-89.0	8.4	2100	-	-	NO
RC-11-008	10.0-11.5	8.25	2826	-	-	NO
RC-11-008	160.0-161.5	7.76	2070	-	-	NO
RC-11-011	10.0-11.5	7.55	3297	-	-	NO
RC-11-011	55.0-56.5	7.56	2596	-	-	NO
RC-11-011	70.0-71.5	7.42	1763	-	-	NO
RC-11-014	6.0-7.5	8.04	3658	-	-	NO
RC-11-014	16.0-17.5	7.54	2499	-	-	NO
RC-11-014	60.0-61.5	7.79	2602	-	-	NO
RC-11-014	86.0-87.5	7.80	2591	-	-	NO

Embankments and Fills

From south of Carpinteria Creek Bridge to near the Casitas Pass Road OC, from approximately 101 Station 116+00 to 140+00, Design proposes elevating the highway profile approximately 10 feet to accommodate the 100-year flood. Raising the profile by addition of fill material will increase the vertical stress on the in-situ soils and cause settlement of the existing soils beneath of the fill. Consolidation settlement is expected due to the presence of fine-grained soils. Estimated total settlements of 3-8 inches were calculated using Hough's method for cohesionless soils, and correlations to index properties for cohesive soils. Similar settlement magnitudes are anticipated at the Carpinteria Creek Bridge approach embankments. Due to the traffic staging at Carpinteria Creek Bridge, the embankment fills will be placed in stages. Placement of a 5-foot high surcharge above the embankment finished grade for stage 2 is recommended to pre-consolidate soils in the vicinity of the bridge abutment foundations. Refer to the stage construction plans for details. Surcharge soils should extend for the width of the stage embankment, and 150 feet along the Highway 101 alignment from the face of the bridge abutments at Carpinteria Creek. Where required, support of vertical side slopes of the embankment surcharges should be designed and proposed by the contractor. The surcharge load will reduce consolidation time and reduce pile down-drag forces following placement of the embankment fills next to the installed pile foundations from previous stage. A 30-day fill delay period for the surcharges is recommended to allow for settlement and consolidation to occur prior to construction of the bridge structure during the first stage. A 30-day fill delay is also recommended for the embankments placed in later stages within 150 feet of the bridge. It is recommended to monitor the magnitude and rate of settlement of the embankments by periodically reading the elevation of surface monuments during the fill delay periods.

The proposed Via Real Bridge will require construction of approach embankments approximately 11 feet above existing grade to match the raised highway profile. Estimated settlements of approximately 4 inches were calculated using Hough's method for cohesionless soils, and correlations to index properties for cohesive soils. Placement of a 5-foot high surcharge above the embankment finished grade for stage 2 is recommended to pre-consolidate soils in the vicinity of the bridge abutment foundation at Abutment 1. The surcharge will include the area of surcharge for Carpinteria Creek Bridge in stage 2 and extend to the hinge point of the finished grade slope at the Via Real Bridge. Refer to the stage construction plans for details. A 30-day fill delay period for the surcharge is recommended to allow for consolidation of fine-grained soils to occur and to prevent differential settlement of the embankment during construction of the bridge structure. A 30-day fill delay period of is also recommended at the Abutment 2 embankment. The fill delay periods will minimize down-drag on the pile foundations due to continuing consolidation of the foundation soils after installation of the piles. It is recommended to monitor the magnitude and rate of settlement of the fill during the surcharge by periodically reading the elevation of surface monuments during the fill delay period.

The new Casitas Pass Road overcrossing will approximately follow the same alignment as the existing structure. The embankment will be widened to accommodate the proposed wider structure, and the proposed bridge deck will be approximately 5 feet higher than the existing bridge deck. Addition of relatively low height of fill, approximately 7 feet at the highest, over the existing approach embankment will have relatively low influence on the vertical stresses in the in-situ soils beneath of the existing embankment. Predicted settlements of 2" are anticipated, with most of the settlement occurring in the existing embankment soils. A 30-day fill delay period for each stage of embankment construction is recommended to allow for differential settlement of the embankment under different vertical stress increases to occur prior to construction of the bridge abutments. It is recommended to monitor the magnitude and rate of settlement of the fills by periodically reading the elevation of surface monuments during the fill delay period.

The new Linden Avenue Overcrossing is proposed on a new alignment to the east of the existing structure, and will require increasing the height and footprint of the existing approach embankments at each end of the structure. The western portions of the new embankments will be constructed over the existing embankment fills, while the eastern portions will be constructed over native soils to the east of the existing embankment fills. The maximum height of the new embankment is proposed to be approximately 21 feet. Estimated settlements ranging from 7 to 13 inches were calculated using Hough's method for cohesionless soils, and correlations to index properties for cohesive soils. Evidence of embankment settlement was observed on both sides of the existing bridge. A 30-day fill delay period for each stage of embankment construction is recommended to allow for consolidation to occur and to minimize differential settlement of the embankment during construction of the bridge abutments. The fill delay period will also minimize down-drag on the pile foundations due to consolidation of the foundation soils after installation of the piles. It is recommended to monitor the magnitude and rate of settlement of the fill by periodically reading the elevation of surface monuments during the fill delay period.

Evidence of high groundwater was observed in the existing highway pavement near the proposed alignment of the new Linden Avenue Overcrossing on the southbound side of Highway 101. Groundwater is rising up through cracks in the pavement and accumulating in the southbound outside shoulder against the dike. The spring was observed year-round, even at the end of the dry summer and fall months. Monitoring wells near the toe of the existing approach embankment also indicate that groundwater is at or near the elevation of the pavement. Pumping subgrade and/or groundwater may be encountered at the base of the proposed bridge embankment. Construction of a permeable mattress is recommended at the southern approach embankment if pumping subgrade is encountered. The mattress will bridge potentially wet and pumping soils and provide a firm and dry base to begin embankment construction. It is recommended to sub-excavate 18 inches below the bottom of embankment elevation, place Class B3 subgrade enhancement geotextile on the bottom and sides of the excavation, place 18 inches of Class III permeable material over the geotextile, and then wrap the geotextile over the top of the aggregate base to achieve a minimum 3-foot overlap on all seams to fully encapsulate the aggregate base.

Cut slopes of 2:1 maximum (Horizontal:Vertical) and embankment slopes of 2:1 maximum are recommended.

Earthwork

A large volume of imported borrow will be required to raise the profile and construct the new bridge embankments. The contractor will be required to identify a borrow location and ensure that imported borrow meets the project specifications. Once a borrow location is identified, soil testing should be performed to ensure the suitability of the material for use in roadway embankments. Refer to Standard Specifications Section 19-6 for embankment construction requirements and Section 19-7 for imported borrow material requirements.

Drainage Systems

Culvert headwalls and wingwalls are proposed for Drainage Systems 1, 3, and 7, as identified in the project plans. Evaluation of bearing capacity and potential construction considerations was performed at the proposed location of each headwall and wingwall. Design recommendations are provided in Table 4. Bearing capacity evaluation was performed using Vesic’s Bearing Capacity Equation and allowable bearing resistances were calculated using a factor of safety of 3.0. Headwall/wingwalls for Drainage System 3 are very near the observed groundwater elevation in the vicinity, groundwater may be encountered during the excavation to construct the headwall/wingwalls. Refer to 2010 Standard Specifications Section 19-3.03D for water control and foundation treatment requirements.

Drainage System 7 and the headwall/wingwalls for Drainage System 7 are proposed below the observed groundwater table. Construction of a dewatered excavation such as a cofferdam will be required to construct the culvert and headwall/wingwalls. Buoyancy of the culvert should be considered and concrete backfill may be required to prevent the pipe from floating. Type D excavation should be specified for the headwall and wingwalls at Drainage System 7 and the excavation required to install the pipe. The very loose and saturated silt encountered at the proposed bottom of footing elevation for the Drainage System 7 headwall/wingwalls will not provide adequate bearing capacity. It is recommended to over-excavate to elevation 5.6 feet and 8 inches beyond the sides of the excavation and place Class B3 subgrade enhancement geotextile on the bottom and sides of the excavation, place Class III permeable material over the geotextile to the design bottom of footing elevation, and then wrap the geotextile over the top of the aggregate base to achieve a minimum 3-foot overlap on all seams to fully encapsulate the aggregate base. Pumping of the base layer will be required during foundation construction. Contact Geotechnical Design North during excavation of the footing to observe the over-excavation and ensure that the foundation soils will provide adequate bearing capacity.

Table 4. Culvert Wingwall Recommendations

Drainage System and Wingwall Type	*Max Design Height (ft)*	*Approximate Bottom of Footing Elevation (ft)*	*Footing Width (ft)*	*Factored Bearing Resistance (psf) SF=3*	*Case I Equivalent Uniform Bearing Stress (psf)*	*Approximate groundwater elevation at nearest monitoring well (ft)*
1-A	10.0	49.50	8.25	5500	3690	30 ft (RC-10-004)
1-A	11.0	34.42	8.67	8000	3770	30 ft (RC-10-004)
3-B	10.0	30.77	8.25	4700	3690	30 ft (RC-11-015)
7-B	10.0	7.18 *	8.25	4000	3690	12 ft (RC-13-016)
7-B	11.0	7.18 *	8.67	5200	3770	12 ft (RC-13-016)

NOTE: * Over-excavate to elevation 5.6 feet and replace with geotextile encapsulated permeable material per above recommendations

Construction Considerations

Groundwater is shallow throughout the region and particularly high in the vicinity of the proposed Linden Avenue Overcrossing. Excavations may intercept groundwater and measures to control groundwater infiltration and facilitate operations may be required. Refer to 2010 Standard Specifications Section 19-3.03D for water control and foundation treatment requirements.

Contact Geotechnical Design North during construction of Drainage System 7 to observe the bottom of the excavations for the culvert and headwall/wingwalls. If very loose and saturated soils are encountered below the proposed elevation at the bottom of the geotextile encapsulated permeable material, increasing the depth of permeable mattress may be required. The base layer shall be pumped during forming, placement of concrete, and concrete curing, as required to keep the groundwater elevation below the bottom of footing, providing a dry uniform surface for construction and preventing segregation of the concrete by flowing water prior to curing. Alternatively, if groundwater infiltration into foundation excavations cannot be controlled by pumping, or at the option of the contractor instead of pumping, the contractor may propose a dewatering plan to be reviewed and approved by the Engineer.

If you have any questions or comments, please contact Ryan Turner at (805) 549-3750, or Michael Finegan at (805) 549-3194.

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Geotechnical Design – North
Branch D

MICHAEL S. FINEGAN, P.E.
Branch Chief
Geotechnical Design – North
Branch D

c: Job File / Branch D Records
Structure Construction RE Pending File (email RE_pending_file@dot.ca.gov)
Craig Whitten / DES Office Engineer
Andrew Tan / PCE
Eric Karlson/ DME

LIST OF ATTACHMENTS

INDEX OF SHEETS

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
05	SB	101	3.5/5.5		

STATE OF CALIFORNIA

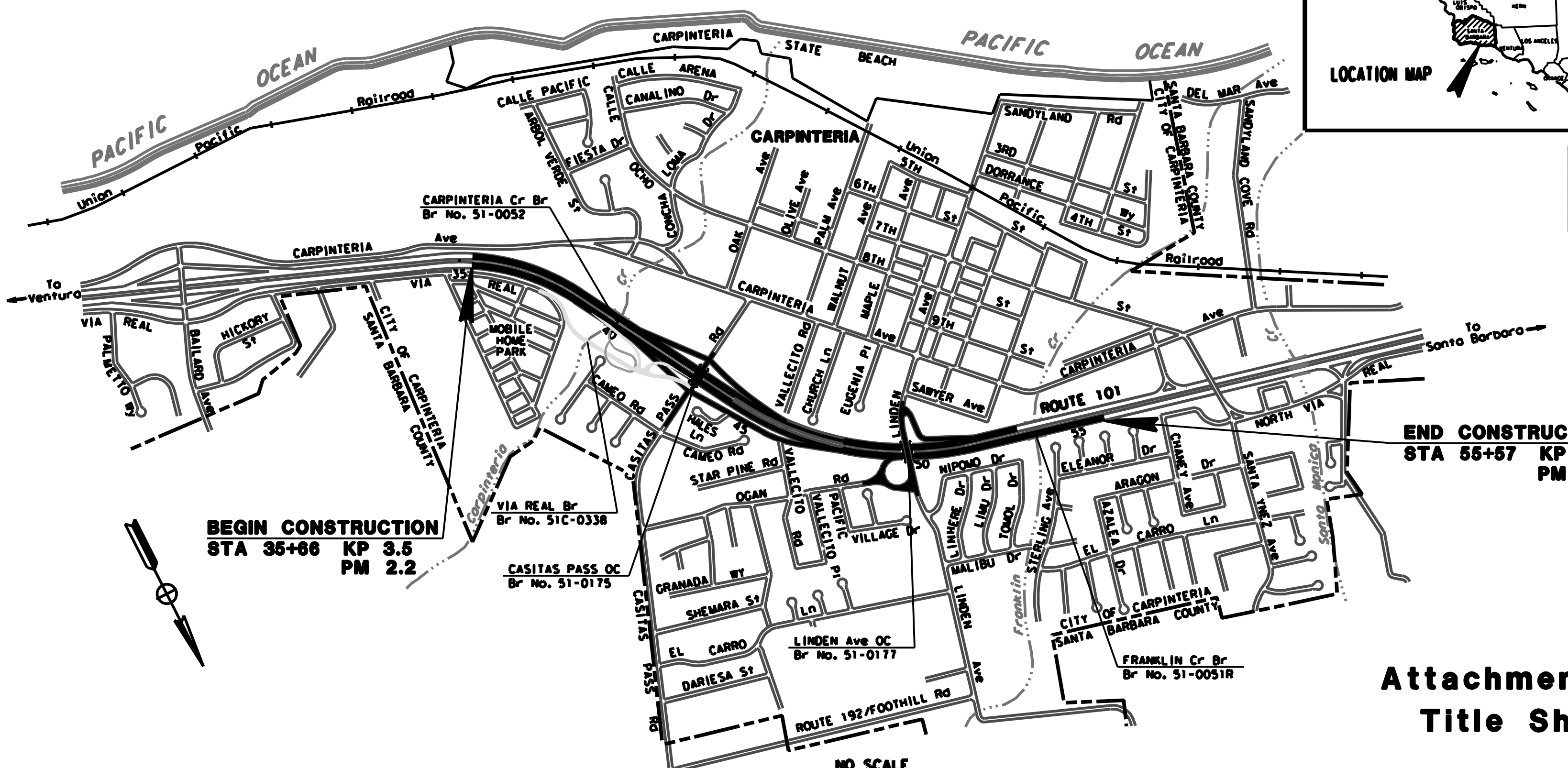
DEPARTMENT OF TRANSPORTATION

PROJECT PLANS FOR CONSTRUCTION ON

STATE HIGHWAY

IN SANTA BARBARA COUNTY
IN CARPINTERIA
FROM 0.4 KM SOUTH OF CARPINTERIA CREEK BRIDGE
TO 0.5 KM NORTH OF LINDEN AVENUE OVERCROSSING

TO BE SUPPLEMENTED BY STANDARD PLANS DATED JULY 2004



Attachment 1
Title Sheet

PROJECT MANAGER DAVID BEARD

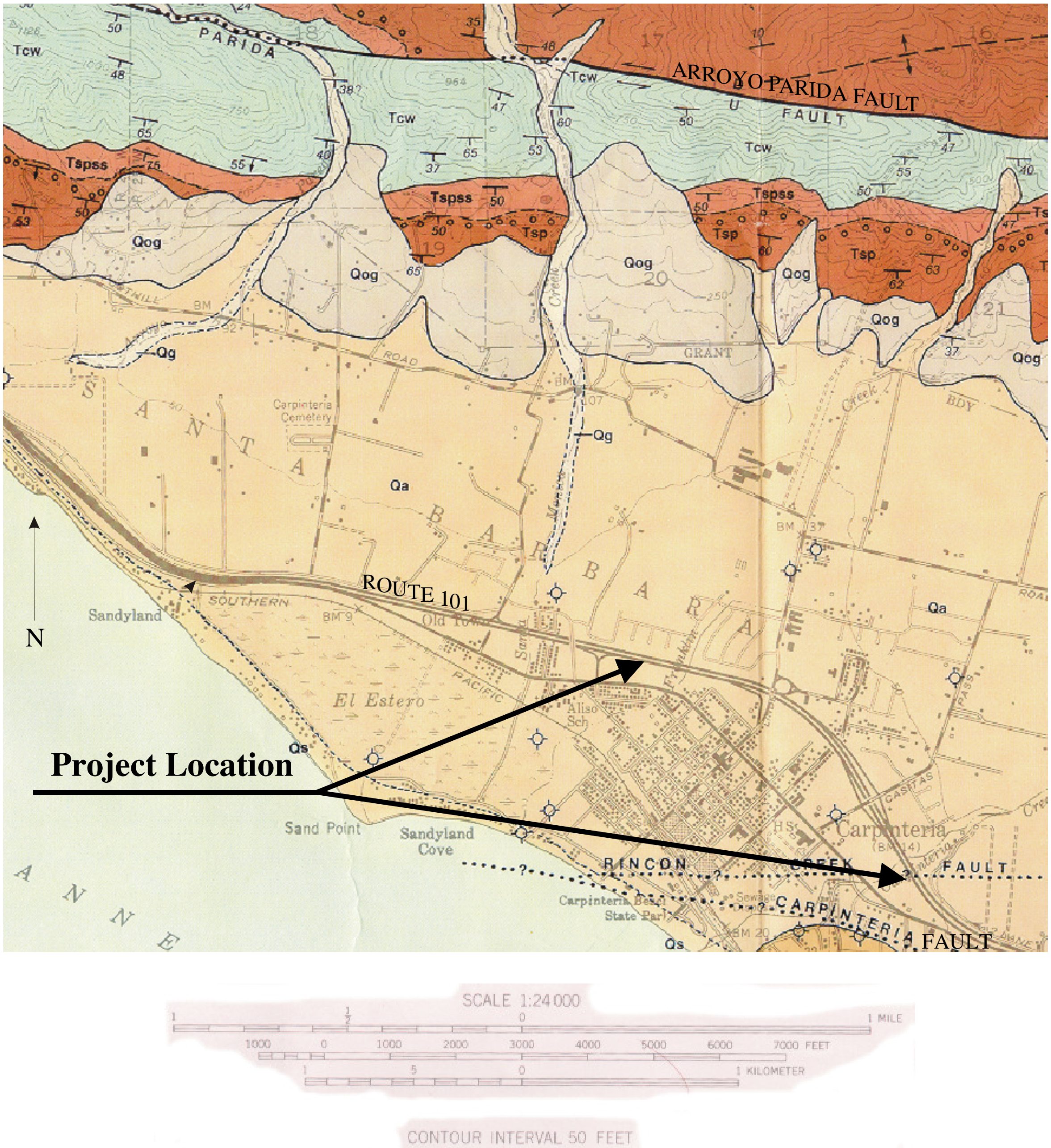
DESIGN ENGINEER KARI BHANA

THE CONTRACTOR SHALL POSSESS THE CLASS (OR CLASSES) OF LICENSE AS SPECIFIED IN THE "NOTICE TO CONTRACTORS."

CONTRACT No. 05-4482U0

CU 06233 | EA 4482U0

01-09-08



PART OF:

GEOLOGIC MAP OF THE CARPINTERIA QUADRANGLE
BY THOMAS W. DIBBLEE, JR., 1986

<u>Geologic Map</u>

05-SB-101-2.1/3.4
05-448201
Attachment 2

LEGEND

GEOLOGY - CARPINTERIA QUADRANGLE

QUATERNARY — Holocene — Pleistocene

Qs Qg Qa

SURFICIAL SEDIMENTS

Qs *beach sand deposits*
Qg *stream channel deposits, mostly gravel and sand*
Qa *alluvium: unconsolidated flood-plain deposits of silt, sand and gravel*

Qls

LANDSLIDE DEBRIS

Qoa Qog

OLDER DISSECTED SURFICIAL SEDIMENTS

Qoa *former alluvial deposits of silt, sand and gravel, in places weakly consolidated; local unconformities at base*
Qog *cobble-boulder fan gravel and fanglomerate deposits composed largely of sandstone detritus*

U N C O N F O R M I T Y

Qca

CASITAS FORMATION
nonmarine; early to middle(?) Pleistocene age

Qca *weakly consolidated alluvial deposits: gray to tan cobble-boulder gravel and gray to reddish sand and clay*

U N C O N F O R M I T Y

Miocene

Tm Tml

MONTEREY FORMATION
marine; early to late Miocene age

Tm *upper shale unit: white-weathering, thin bedded, hard, platy to brittle siliceous shale; Mohnian Stage*
Tml *lower shale unit: white-weathering, soft, fissile to punky clay shale with interbeds of hard siliceous shale and thin limestone strata; lower Mohnian to uppermost Saucesian Stages*

Tmss

UNNAMED SANDSTONE
(TEMBLOR SANDSTONE OF DIBBLEE, 1966)
shallow marine; early Miocene age

Tmss *light gray to tan fossiliferous arkosic sandstone and pebble conglomerate; local unconformity(?) at base; exposed north of Santa Ynez fault*

Tr

RINCON SHALE
marine; early Miocene age

Tr *poorly bedded gray clay shale and siltstone, locally concretionary; Saucesian and upper Zemorrian Stages*

Tvq

PART OF:

GEOLOGIC MAP OF THE CARPINTERIA QUADRANGLE
BY THOMAS W. DIBBLEE, JR., 1986

Geologic Map – Legend

05-SB-101-2.1/3.4
05-448201
Attachment 2

TERTIARY — Oligocene — Eocene

Tsp Tspss

SESPE FORMATION
nonmarine; predominantly Oligocene age

Tsp *maroon, red and green silty shale or claystone with interbedded red, tan and gray sandstone; red arkosic sandstone and conglomerate at base*
Tspss *red to pink sandstone and red claystone*

Tcw Tcwsh

COLDWATER SANDSTONE
marine; late Eocene age

Tcw *hard, tan, bedded arkosic sandstone with minor interbeds of greenish-gray siltstone and shale, local oyster shell beds common in upper part; Narizian Stage*
Tcwsh *greenish-gray siltstone and shale with occasional interbeds of tan sandstone*

Tcd Tcdss

COZY DELL SHALE
marine; late Eocene age

Tcd *dark gray, argillaceous to silty micaceous shale with minor light gray to tan arkosic sandstone; Narizian Stage*
Tcdss *light gray to tan arkosic sandstone with minor interbeds of gray micaceous shale*

Tma Tmash

MATILIJA SANDSTONE
marine; middle to late Eocene age

Tma *hard, thick bedded, tan to mottled light greenish gray arkosic sandstone with thin partings to thick interbeds of gray micaceous shale; lower Narizian and upper Ulatisian Stages*
Tmash *gray micaceous shale with minor tan sandstone interbeds; exposed north of Santa Ynez fault*

Tjsh Tjss

JUNCAL FORMATION
marine; early to middle Eocene age

Tjsh *dark gray micaceous shale with minor thin interbeds of hard, gray-white to tan arkosic sandstone; lower Ulatisian(?) to upper Penutian(?) Stages*
Tjss *hard, gray-white to tan arkosic sandstone with minor interbeds of dark gray micaceous shale*

Tsb Tsbs

SIERRA BLANCA LIMESTONE
shallow marine; early Eocene age
(exposed north of Santa Ynez fault)

Tsb *hard, white-weathering sandy algal limestone*
Tsbs *hard, light gray calcareous sandstone, locally fossiliferous*

U N C O N F O R M I T Y

CRETACEOUS — Upper — Lower

Ksh Kuss Kucg Kush

UNNAMED MARINE STRATA
(Kush, Kuss, Kucg - Jalama ? Formation)
late Cretaceous age

Ksh *gray micaceous clay shale with minor interbeds of hard, gray-white to tan arkosic sandstone; exposed north of Santa Ynez fault*
Kush *dark gray to black micaceous clay shale with minor interbeds of hard tan arkosic sandstone*
Kucg *gray to brown cobble conglomerate of granitic, porphyritic-andesitic, and quartzitic detritus in arkosic sandstone matrix*
Kuss *hard, light gray to tan arkosic sandstone with minor interbeds of micaceous shale*

Ke

ESPADA FORMATION
marine; late Jurassic (?) to early

Ke *dark greenish-gray carbonaceous shale with thin interbeds of hard, olive-gray arkosic sandstone and minor dark gray impure limestone*

JURASSIC & CRETACEOUS

sp

SERPENTINITE
metamorphosed intrusive rock

sp *severely sheared, bluish-green to black serpentinite and serpentinized peridotite in Santa Ynez Fault zone*

fg fc fs gl

FRANCISCAN ASSEMBLAGE
slightly metamorphosed, pervasively sheared assemblage of marine sedimentary, igneous and metamorphic rocks

fg *greenish-brown to black, massive to crudely bedded, fine-grained greenstone, metamorphosed from basalt*
fc *thin bedded, hard and brittle, varicolored green to red chert*
fs *greenish-brown, hard graywacke sandstone and dark gray micaceous siltstone and shale, moderately to severely sheared, in part to mélange*
gl *hard, deep blue glaucophane blueschist*

SYMBOLS

FORMATION CONTACT
dashed where inferred or indefinite

MEMBER CONTACT

CONTACT BETWEEN SURFICIAL SEDIMENTS
located approximately in places

U D 70 ?

FAULT

dashed where indefinite or inferred, dotted where concealed, queried where existence doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement shown by U/D (U = upthrown side D = downthrown side). Short arrow indicates dip of fault plane.

ANTICLINE **SYNCLINE**

arrow on axis indicates direction of plunge, dotted where concealed

horizontal — 25 *inclined* — 20 *inclined (approximate)* — *vertical* — 25 *overturned* — 25 *inclined metamorphic foliation*

STRIKE AND DIP OF BEDDED ROCKS

Qls

DIRECTION OF LANDSLIDE MOVEMENT — **SANDSTONE BED** — **CONGLOMERATE BED**

ABANDONED EXPLORATORY OIL WELL

GEOLOGY - CARPINTERIA QUADRANGLE

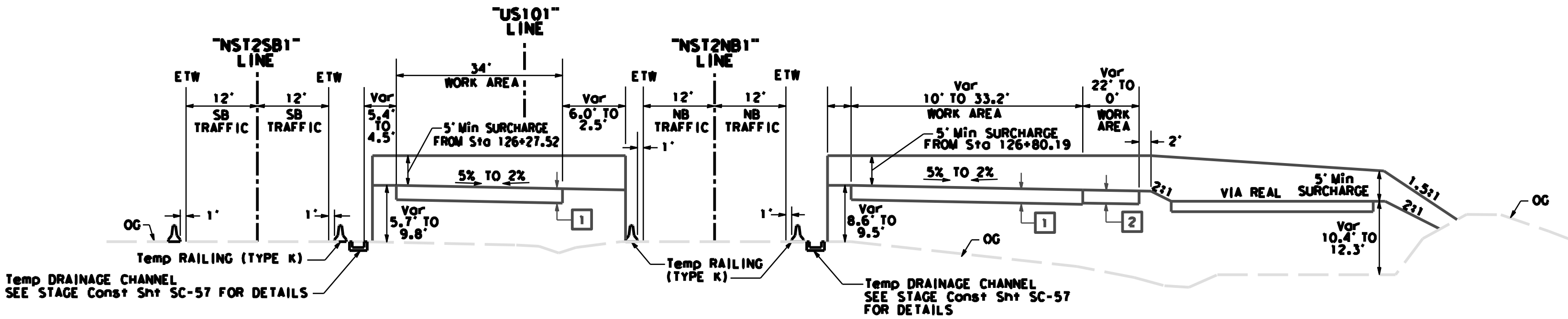
Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
05	SB	101	2.2/3.3		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

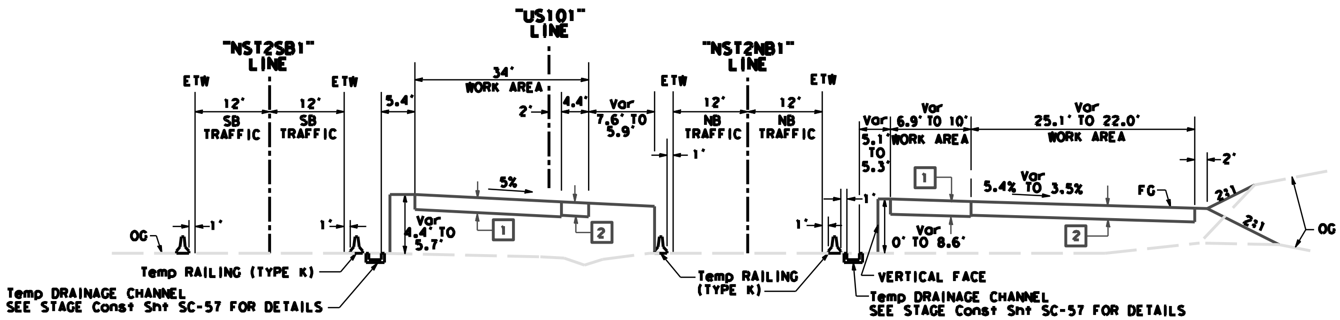
THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER
AMIR SAEDI
No. 56083
Exp 12-31-14
CIVIL
STATE OF CALIFORNIA



Sta "NST2SB1" 122+75.00 TO 128+77.52

Sta "NST2SB1" 125+25.00 TO 128+30.19



Sta "NST2SB1" 121+50.00 TO 122+75.00

Sta "NST2NB1" 119+75.00 TO 125+25.00

TYPICAL PAVEMENT STRUCTURE SECTIONS

1 — 0.90' CRCP, 0.20' HMA-SP (TYPE A), 1.35' CLASS 1 AS

2 — TEMPORARY ROADWAY, 0.55' HMA, 1.65' CLASS 2 AB

3 — "CA4" SEE SHEET X-3

STAGE 2

ROUTE 101

STAGE CONSTRUCTION
TYPICAL CROSS SECTIONS (STAGE 2)

NO SCALE

SC-43

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans
DESIGN

FUNCTIONAL SUPERVISOR: JAMES R. PERANO
CALCULATED-DESIGNED BY: AARON WOLFRAM
CHECKED BY: AMIR SAEDI
REVISED BY
DATE REVISED

LAST REVISION 02-13-14
DATE PLOTTED => $DATE
TIME PLOTTED => $TIME

BORDER LAST REVISED 7/2/2010
USERNAME => $USER
DGN FILE => $REQUEST
RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3
UNIT 1447
PROJECT NUMBER & PHASE 05000005431

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
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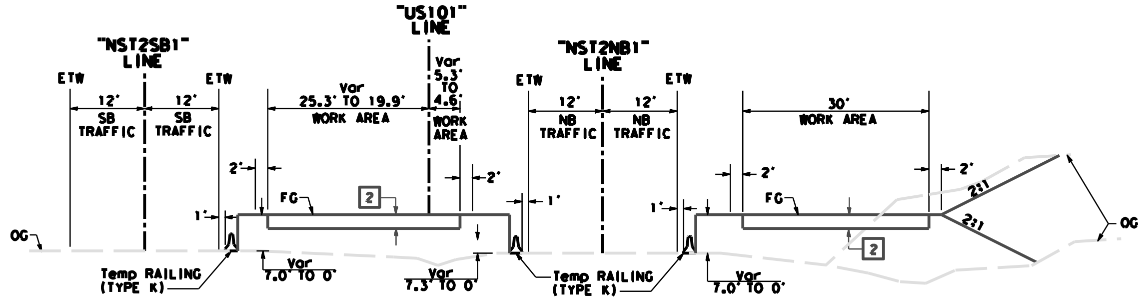
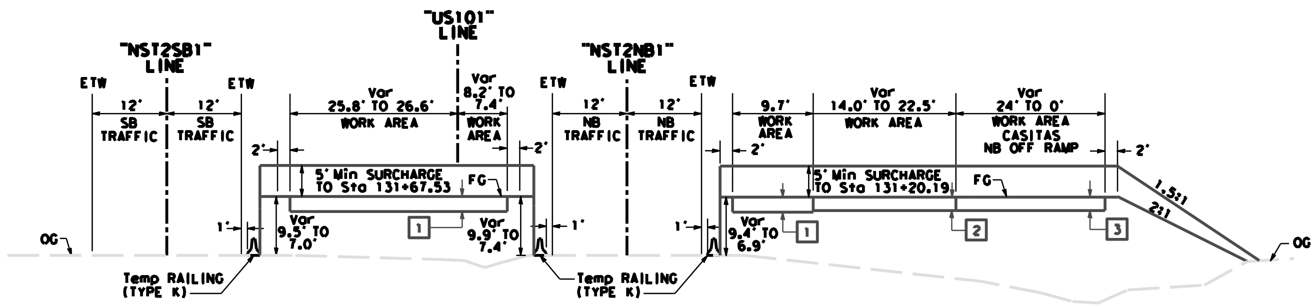


Sta "NST2SB1" 133+30.52 TO 136+30.56

Sta "NST2NB1" 132+83.19 TO 135+83.19



Sta "NST2SB1" 130+17.53 TO 133+30.52

Sta "NST2NB1" 129+87.00 TO 132+83.19

STAGE 2

ROUTE 101

TYPICAL PAVEMENT STRUCTURE SECTIONS

1 — 0.90' CRCP; 0.20' HMA-SP (TYPE A); 1.35' CLASS 1 AS

2 — TEMPORARY ROADWAY; 0.55' HMA; 1.65' CLASS 2 AB

3 — "CA4" SEE SHEET X-3

STAGE CONSTRUCTION
TYPICAL CROSS SECTIONS
(STAGE 2)

NO SCALE

SC-44

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

DESIGN

FUNCTIONAL SUPERVISOR: JAMES R. PERANO

CALCULATED-DESIGNED BY: AARON WOLFRAM

CHECKED BY: AMIR SAEDI

REVISED BY

DATE REVISED

LAST REVISION 02-12-14

DATE PLOTTED => $DATE

TIME PLOTTED => $TIME

BORDER LAST REVISED 7/2/2010

USERNAME => $USER

DGN FILE => $REQUEST

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

UNIT 1447

PROJECT NUMBER & PHASE 05000005431