GEOTECHNICAL DESIGN REPORT

State Route 58

For Widening and Realignment

08-SBd-58- PM 22.2/31.1

In Hinkley, San Bernardino County

08-043510

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Geotechnical Services

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1. INTRODUCTION

This report addresses the geotechnical issues involved in the realignment of a portion of State Route 58 (SR58) in San Bernardino County in the vicinity of the community of Hinkley (Figure 1C). The proposed SR58 will be constructed as a four-lane expressway south of the present two-lane conventional highway. The project limits of this new alignment are between PM 22.2 and PM 31.1, extending from approximately 1.5 miles west of Valley View Road and connecting to the current terminus of the existing 4-lane expressway Route 58 just east of Lenwood Road (See Site Map, Figure 1A and 1B). As proposed at the time of this report, the new alignment (Alignment 2, “Geotechnical Recommendations for Additional Alternatives”, Jan. 5, 2009) will have at-grade intersections with local roads crossing the alignment west of Lenwood Road, and a spread diamond interchange will be constructed at Lenwood Road.

Mr. Owen Spencer, Office Chief, District 8 Design B sent an August 30, 2000 memo to the Office of Geotechnical Design-South, requesting a Preliminary Geotechnical Report for the above mentioned improvement. The request specifically asked for the feasibility of using roadway cut material from the western part of the proposed alignment for fill material. An Addendum to the PGR, Nov. 26, 2003 was written by this office after our 2003 investigation.

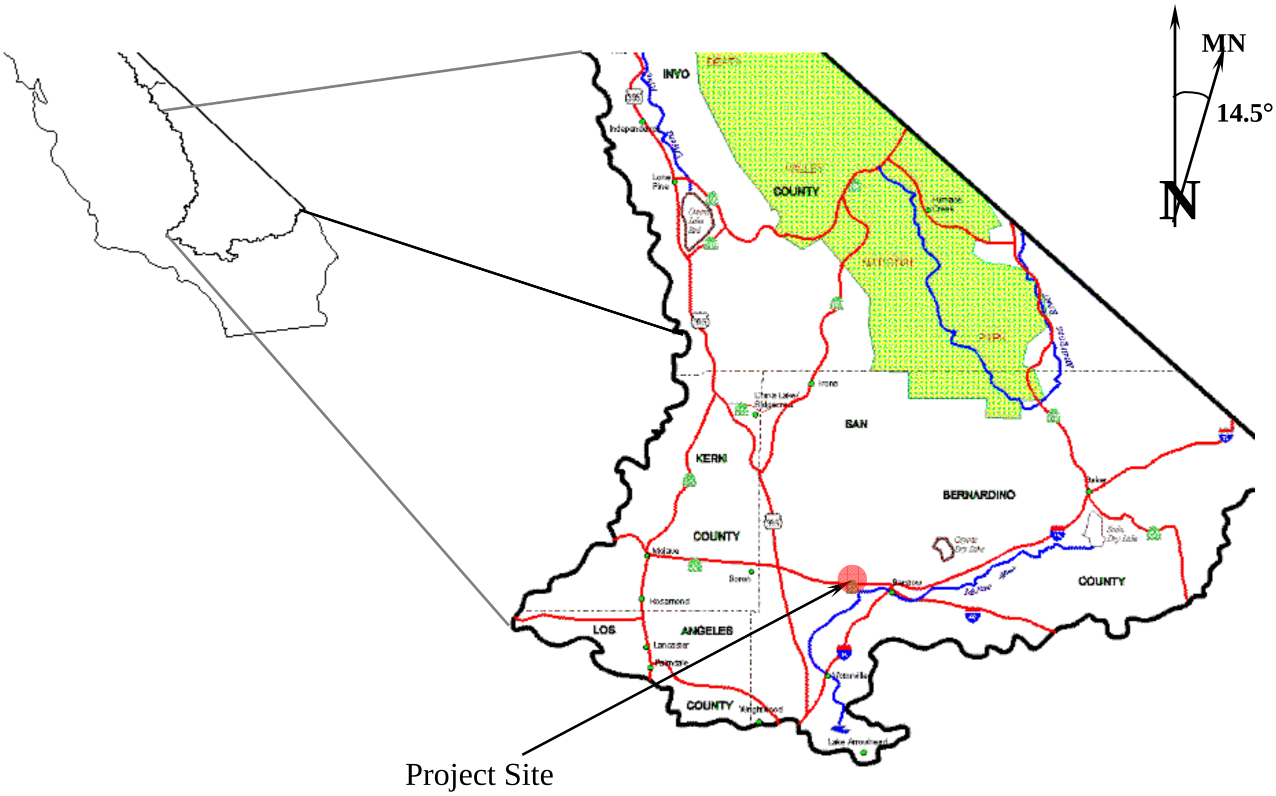
This Geotechnical Design Report (GDR) encompasses the preliminary geotechnical study for the main alignment (using year 2012 stationing), the approach embankments for the interchanges between Lenwood Road and both the main alignment, and the existing BNSF railroad, a noise barrier foundation, and an earth-retaining structure foundation. This report does not include geotechnical studies for bridge foundations or culverts.

The Office of Geotechnical Design South 2, revised this GDR based on the previously submitted “Preliminary Geotechnical Report”, prepared by the Office of Geotechnical Design – South, dated July 2002. This GDR presents our geotechnical design recommendations for use during the design phase of this project.

2. PERTINENT REPORTS AND INVESTIGATIONS

District 8 has provided us with a new US units route map for the proposed Route 58 re-alignment superimposed on a topographic map with two-foot contours. The District has also provided us with parcel maps, layout plans, profile plans, and agricultural use maps. Our literature search yielded several reports and maps, which were utilized in preparing this report and are cited in the attached list of references (Appendix I).

Figure 1C. Regional Location Map



3. DESCRIPTION OF EXISTING FACILITIES

Within the project limit (from PM 22.2 to Lenwood Road), the existing State Route 58 is an asphalt concrete (AC) paved, conventional two-lane highway with 12± ft wide lanes and unpaved shoulders ranging from 6 to 8 ft wide. A site plan with the location of existing and proposed roadway features is shown on Figures 1A and 1B.

From the western most point of the proposed improvement to 0.5 miles east of Summerset Road, the existing alignment follows the natural contour of the land. Being at grade with adjacent terrain, this part of SR 58 has no longitudinally directed AC dikes or ditches for water runoff control. No culverts cross below the pavement at drainage gullies (See dips, Figure 7F). Following a sheet flow drainage pattern, surface runoff from higher terrain south of the highway generally flows across the traveled way. However, runoff does concentrate to a degree and flows across the highway, through several existing dips at the west part of the alignment.

Approximately 0.5 miles west of Hinkley Road, the existing SR 58 deviates at a slight angle northward from the original east-to-west alignment, runs parallel to the present railroad, and becomes a two-lane, controlled access interim connector extending to the end of the project limits (PM 31.1) and the transition to the existing 4-lane expressway. Approximately 1.1 miles from the eastern terminus of the proposed alignment, the current Route 58 is built on embankment fill between PM 30.2 to 30.3. Embankments are more than 5 feet in height and appear to be performing well with no observed pavement settlement or heave. However, surficial erosion was found to occur on 2:1 (H:V) embankment slopes near the Route 58 intersection with West Main Street. Documentation of erosion features is found in Appendix IV, and erosion is discussed further in Section 6.5 of this report.

At the time of our site reconnaissance, the pavement of present Route 58 was in good condition, with no observed major distress.

4. PHYSICAL SETTING

4.1. Climatic Conditions

The climatic conditions in the Hinkley area can be characterized as high desert-like, having low humidity with average annual precipitation of 4 inches, large variation of daily temperature, and high frequency of strong winds. Maximum and minimum temperatures are 117ºF and 7ºF, respectively. Heaviest rainfall occurs in early winter and late summer where maximum rainfall for the area was recorded in 1965 when 7.5 inches fell in one year.

Average monthly weather conditions (temperature and precipitation) in the vicinity of project site are presented in Table 1 below. The recording station from which this data was acquired is located at Hinkley, CA 92347.

Table 1. Average Monthly Weather Conditions

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Ave. High (°F)	60.0	66.0	70.0	78.0	87.0	97.0	103	101	93.0	82.0	69.0	60.0
Ave. Low (°F)	36.0	41.0	46.0	51.0	59.0	67.0	74.0	72.0	65.0	55.0	44.0	36.0
Mean (°F)	49.0	54.0	58.0	65.0	74.0	83.0	89.0	87.0	80.0	69.0	57.0	49.0
Rainfall (in)	0.50	0.40	0.40	0.20	0.10	0.10	0.40	0.50	0.40	0.20	0.30	0.50

Although sub-freezing temperatures do occur, freeze-thaw conditions are not anticipated.

4.2. Topography and Drainage

Located in a rural area of relatively flat to gently rolling terrain, the proposed alignment traverses a series of coalescing alluvial fans, sloping down to the northeast. The relative elevation for the area between project's western limit (PM 22.2, STA 1197+98) and Valley View Road (PM 24.3, STA 1314+00) is high compared to the proposed alignment east of Valley View Road. The absolute elevation ranges from 2356 to 2251 feet above the sea level, with rock outcrops dotted between PM 23.4 and 24.3, where deep cuts will occur. Continuing to the east until the eastern limit of the project, topography is generally flat with a gradient of 15 feet per mile (descending to the northeast). The surface elevations of the future expressway will change from 2355 feet (STA 1282+00) at the west part of alignment to 2175 feet (STA 1671+64) at the eastern end of alignment.

The local topography features a general drainage pattern of superficial flow from southwest to northeast. Surface water flows from Iron Mountain (elev. 2782 ft) near the west portion of the proposed facility, crosses over the area of the proposed alignment and drains northeasterly to the north part of Hinkley Valley, which is between Mountain Lynx Cat (elev. 2566 ft) and Mountain General (elev. 2927 ft).

No major creeks or tributaries crossing the proposed alignment have been identified, but four unnamed washes transect the proposed Route 58 at STA 1229+72, 1241+00, 1297+10, 1321+65 in the western part of project. The largest two of the four drainage courses emanate from the northern side of Iron Mountain and drain northeasterly, crossing the proposed alignment at STA 1297+10 and 1321+65. The drainage crossing at STA 1297+10 is incised into soil and is approximately 6.7 feet wide and 3 feet deep where it crosses the proposed main alignment. The drainage crossing at STA 1321+65 is incised into soil and bedrock and is less than 3 feet wide and 0.65 feet deep where it crosses the main alignment. However, these drainages are dry year-around unless long-term moderate-to-heavy rainfall occurs. Those drainage courses may contain loose alluvial deposits, and will be buried by the embankment of the proposed alignment.

The east portion of the proposed alignment lies within the Mojave River Flood Plain, where the landscape is flat with a slight slope down towards the Mojave River to the south and east. Levees shown on the site map (Figure 1B) are approximately 1.25 miles away from the proposed alignment, and have been constructed to contain the Mojave River at flood stage.

No man-made drainage exists within the vicinity of present Route 58 for flood control. The lack of positive drainage is listed as an important reason for the proposed improvement.

4.3 Man-made and Natural Features of Engineering and Construction Significance

The proposed alignment is located in a rural area, on relatively flat to gently rolling terrain. The land use in and around the project is predominantly open space and agricultural land, with some residential development scattered throughout.

From its western limit to Hinkley Road, the proposed alignment will cross a vast stretch of uncultivated land, which is also identified as the natural habitat of desert tortoise and Mojave ground squirrel. The vegetation throughout this area is relatively sparse and generally consists of desert grass, creosote, and sagebrush.

Running parallel to each other and south of the proposed right of way (R/W), three buried utility (gas) lines (Mojave South Pipelines, El Paso Natural Gas, and PG&E) reach their closest (approximately 330 feet) approach to the proposed southerly right of way line between STA 1280+61 and 1283+89. Within the R/W opposite these pipelines, a cut slope 16.5 to 26 feet high is proposed. Given the distance between the future hinge point of the cut slope and the gas lines, no conflict is anticipated.

At the western end of the project, possible loose drainage areas exist between STA 1206+00 to 1255+00, 1297+00 to 1306+00, and 1311+40 to 1500+00 (Fairview Rd.). At the eastern part of the project to Lenwood Road, the proposed alignment traverses stretches of agricultural land where plowed and loosened surface soil will be encountered, and heavy irrigation likely occurs. The future alignment will also transect several residential complexes in the east. The affected area, shown on the layout plan provided by the District, will be from STA 1411+00 to STA 1430+00, from STA 1449+00 to STA 1453+00, and from STA 1470+00 to 1472+50. Agricultural areas are between STA 1500+00 to 1552+50, and from STA 1580+00 to STA 1617+00, and on the south side of Lenwood Rd. STA 1633+00 to 1655+00, respectively. The cultivated land and residential areas will impact the future embankment foundations. Buried utilities (e.g. water, gas, and on-site sewage disposal systems) may also be expected throughout this area. Depending on locations and embedment, plans for their removal or protection are required, as discussed in Recommendation Section.

4.4 Subsurface Investigation

Subsurface investigation was conducted to obtain preliminary geotechnical information for this report. A total of 37 boreholes have been drilled, including ten rock corings for cut sections in the west. We also conducted cone penetration soundings at ten locations and two geophysical lines in the western cut section. Our subsurface investigations focused on areas of geotechnical and geologic interest such as where deep cuts or high embankments will be graded, or where future structures such as the soundwall or retaining wall will be built. The soil exploration program conducted through October 2013 is summarized in Table 2 (Appendix III, converted). Locations of borings and cone penetrometer soundings are shown on the attached Boring Location Map sheets (Figures 2-1 through 2-20).

See Appendix III for a description of the drilling, sampling, CPT and testing program. See Appendix VI for the logs of the borings completed. Some early sampling was done in metric units and will not be converted.

Additional borings were done in 2003 for the cut sections to the west. In two of those borings, B9 and B13, we conducted mechanical caliper and acoustic televiewer geophysical measurements to determine fracture patterns. In 2013, we drilled 3 borings for Retaining Wall 1645 at Lenwood Road (included in numbers above).

5. REGIONAL GEOLOGY

The subject site lies within the Mojave Block geomorphic province. This province is characterized by isolated mountain ranges with broad coalescing alluvial fans terminating at dry lakebeds (playas). There are two topographic trends within this province, a northwest-southeast trend controlled by the San Andreas Fault on the southwest border of the province, and a secondary east-west trend controlled by the Garlock Fault, which is the northern boundary of the province.

6. SITE GEOLOGY

Between Stations 1198+00 and 1251+10 the proposed alignment passes through undifferentiated older Quaternary Alluvium (Qo) and younger Quarternary Alluvium (Q).

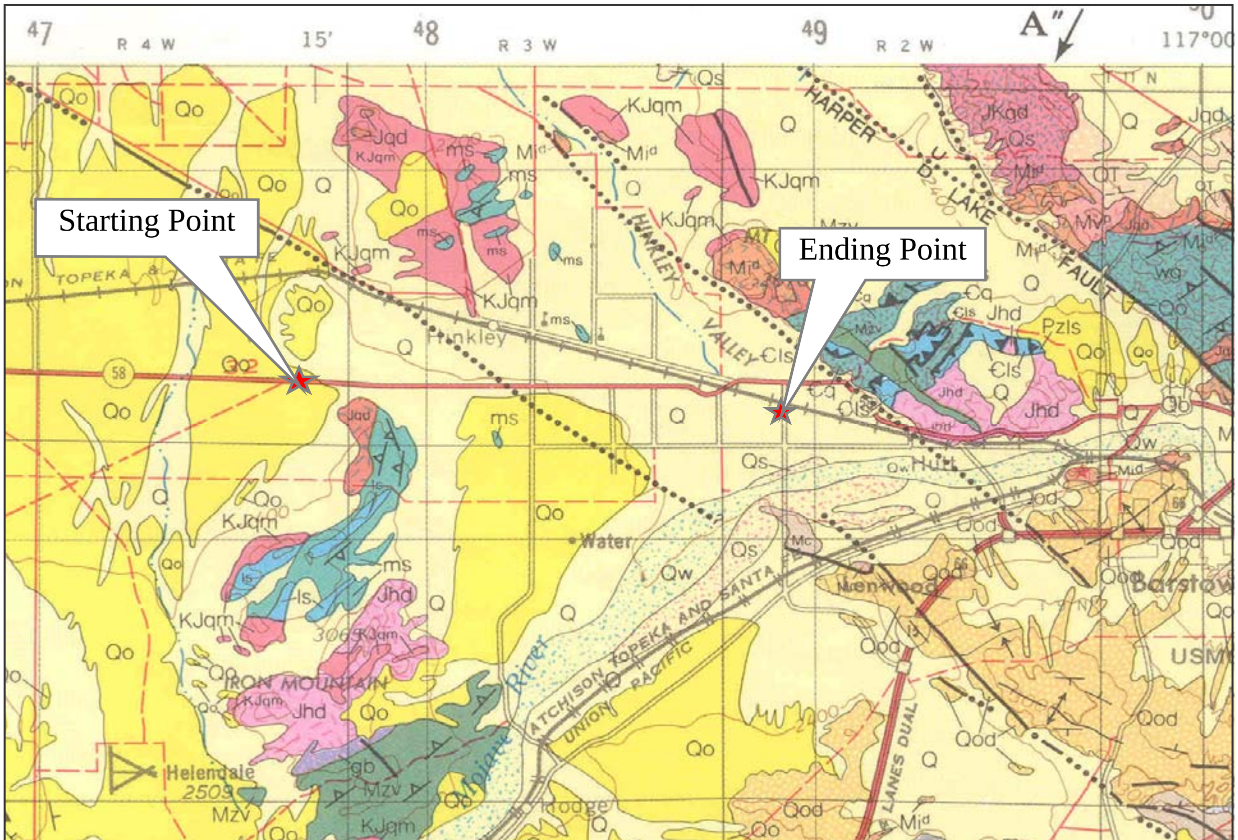
This material is composed of various sand, silt, gravel and clay combinations and is shown on the geologic map of the San Bernardino Quadrangle, Figure 3.

At the beginning of the cut section at STA 1251+00, the material is composed of Jurassic quartz diorite (Jqd) bedrock which quickly becomes metasedimentary rock (ms) of marble and limestone (ls). Between Stations 1251+00 and 1297+00 bedrock is at or near the surface. The quartz diorite in some places, exhibits mild foliation and is schistose where mica percentages are high. Both the quartz diorite and limestone, as revealed in core samples, are soft to hard, moderately to intensely weathered, and contain fractures which are closely spaced, intensely jointed and sheared, and which have been healed. All 2003 borings showed most of these bedrock fractures contained slickensided, clay gouge-filled shears within fractured gneiss and limestone. This bedrock zone may have an impact on the project design as described later in Section 12.8. Fracture spacing was measured at 2 to 14 inches, and fracture dips ranged from 40 to 70 degrees (measured from horizontal). On the flanks of the hill between the above stationing, bedrock is covered by a thin veneer of Alluvium (Q) and Colluvium (Undifferentiated) of Quaternary age, expanding from 6.5 to 16 feet thick as you get closer to the valley floor. The hilltop has none. Alluvium and colluvium are composed of weathered fragments of bedrock ranging in size from sand to cobbles.

At STA 1297+00, the material changes back to younger, then older alluvium, and back to younger alluvium to the end of the project limits in the eastern side of the project.

At Lenwood Road adjacent to the Railroad, the sandy sediments have very stiff to hard, silt and clay lenses that contains minor caliche.

Cultural and agricultural areas may contain imported materials that were not investigated for the preparation of this report.



Holocene

- Q — Alluvium *(Undifferentiated)*
- Qw — Wash deposits *(Alluvial deposits of modern washes)*
- Qow — Older wash deposits *(Alluvial deposits of abandoned washes)*
- Qls — Landslide deposits
- Qs — Wind-blown sand
- Qyf — Younger fan deposits
- Qf — Fan deposits
- Qya — Younger alluvium
- Ql — Lake deposits
- Qof — Older fan deposits
- Qo — Older alluvium *(Undifferentiated)*
- Kqd — Cretaceous quartz diorite
- ms / ls — Metasedimentary rocks of uncertain age *(Quartzite, phyllite, and schist)* ls - limestone and marble

MN
14.5°
N

0 5 10 km

0 5 10 Miles

Most conspicuous foliation strike and dip

Qv — Basalt
Cinder cone

Fault

Solid where well located; dashed where approximately located or inferred; queried where continuation or existence is uncertain; dotted where concealed by younger rocks. Arrows show relative or apparent direction of movement. U, upthrown side and D, downthrown side (relative or apparent).

CALTRANS Division of Engineering Services Office of Geotechnical Design – South 2	EA: 08-043510	Geologic Map	
	Date: May 2013		
	08-SBd-58-PM 22.2-31.1		FIGURE 3

6.1 Mineral Resources

Other than soil (alluvium) and excavated rock that may be processed to generate fill or aggregates, no known mineral resource was identified within the project limits.

6.2 Aggregates/Construction Material Source

Borrow Source: The earth materials excavated (ripped or blasted) in the cut area between STA 1251+10 and STA 1316+69 can be utilized as fill material for embankment construction in the east part of the alignment. Given the geologic conditions in the proposed cut area in the west, oversized rocky materials, which will not readily be broken by passage of normal earthwork equipment, will likely be produced.

A soil stockpile located 0.5 miles east of Lenwood Road within Route 58 right of way (from STA 1654+94 to STA 1669+70, 50 feet south of edge of pavement) might be also available for use as borrow. From surface observations the stockpile appears to consist of silty fine sand with gravel (sized up to 2.5 inches). This trapezoidal stockpile has a base/top width of 72.5/11.5 feet with 2:1 (H:V) slope on each side. The above earth materials were transported from a cut/borrow area along Route 58 between Main Street and Interstate 15 when this section of the highway was under construction around 1997. No obvious signs of surficial erosion were observed in the stockpile. Bulk samples from a few shallow, hand-dug pits have been collected from the stockpile and are being tested for compaction, corrosion, gradation, sand equivalent and remolded direct shear.

The prospective fill materials we encountered during our site investigation and office review may not quantitatively satisfy the future earthwork balance.

Aggregate Subbase/Base: Blasted and ripped material from the cut sections of the proposed alignment might be usable as aggregate subbase or base, however rock fragment sizes anticipated from excavation are greater than 4 inches and therefore would require further processing to conform to Sections 25 or 26 respectively of the Standard Specifications.

Aggregate for Concrete: Excavated rock derived from blasting and cutting might be useable as aggregate for Asphalt Concrete (AC) and Portland Cement Concrete (PCC), however, testing would need to be performed to assess the ability of the rock to conform to Sections 39 and 90 of the Standard Specifications.

Note: Core samples of rock from boring B8 (Run length 30 to 45 ft.) were given to District 8 Materials Engineering for testing regarding the above materials issues.

6.3 Excavation Characteristics

We expect that conventional earth moving equipment can make excavations within alluvium.

A "Seismic Refraction Survey for Rippability Evaluation, State Route 58" was performed for the hilly area on the western end of the proposed alignment. In that study, the earth materials were separated into three layers. The first layer, colluvium, is considered easily rippable and extends to less than 6.6 feet below ground surface (bgs). This layer is interpreted to be composed of loose soil over bedrock. The second layer is confirmed to be composed of hard, fractured rock that, between stations 1247+82 to 1310+13, will require difficult ripping and/or light blasting to a depth of 46 feet. Cuts extending below 46 feet will extend into a third layer consisting of very hard quartz diorite/gneiss that, according to the rippability report, will require blasting. The March 14, 2002 rippability study is located in Appendix V.

6.4 Grading factor

Earthwork factors for the cut sections on the west side of the alignment are discussed here in terms of alluvial materials (erosional products of underlying rock) and rock-like materials.

Alluvial Materials: Grading factors for alluvium overlying rock in the proposed cut area need to be confirmed through further field testing and sampling when additional permits allow further exploration. Nevertheless, since the subsurface materials appear to be uniform on either side of the cut area, we have estimated earthwork factor for alluvial deposits based on our current testing for the adjacent parts of the project. In those areas, the natural soils were considered to be dense to very dense sand or silty sand. The estimated grading factor is presented in Section 15.2 of this report.

Rock-Like Materials: The earthwork factor for the rock and rock like materials in the cut section of the project is approximately 1.3, which is the ratio of embankment to excavation volume. This earthwork factor was found by correlating seismic velocities to site specific rock types, as explained in the May 23, 2002 memo, “Earthwork Factors, State Route 58,” submitted by the Office of Geotechnical Support, Geology and Geophysics Branch. The above memo is located in Appendix V.

6.5 Erosion

The roadbed for existing Route 58 parallel to and north of the project limits is either at natural grade or of limited embankment height (maximum height of 5 feet from Dixie Road intersection to the Lenwood Road intersection). No significant erosion was observed west of Lenwood Road. However, some erosion does exist off site for the embankment of existing Route 58 from east of the Lenwood Rd/Rte 58 junction to the Main Street/Route 58 interchange.

The present embankment east of Lenwood Road varies from 3.3 to 43 feet high, has slope ratios varying from 2:1 (H:V) to 1.7:1, and was built in approximately 1997.

According to our field observations, the embankment soils consist primarily of poorly graded, non-plastic silty fine sand with gravel, which can be highly susceptible to erosion. For example, the northern slope of the embankment approximately 330 feet east of Lenwood Road has a height of 5 feet and slope inclination of 4:1 (H:V). In the absence of AC dikes the water sheet flows off the pavement and over the top of the fill. Flow has concentrated and has eroded rills up to 4in/4in (width/depth) into the surface of the slope. In addition to the erosion-susceptibility of the on-site fill materials, we attribute the erosion to both the lack of slope surface stabilization treatments and the lack of runoff control.

Surface erosion was also observed at the Route 58 approach embankments to the BNSF railroad overhead and to the West Main Street undercrossing, both constructed around 1997.

At the West Main Street undercrossing (Br. No. 54-1113L, PM R33.5), the height of the embankment at the abutment is approximately 23 feet with a slope ratio of 1.7:1 (H:V). Although straw mats had been applied, rills sized up to 8in/8in (width/depth) were observed both close to the hinge point and at the end slope/abutment interface, with appreciable debris collected at the toe of the slope. There are no AC dikes to prevent runoff from flowing over the slopes in this location.

Erosion rills were also observed at the west slope of the approach embankment to the railroad overhead (Br. 54-1112, PM R33.8). Numerous rills up to 6in/6in have been formed commencing one-third of the way down from the top of the embankments. The embankment height in this area is 43 feet with slope inclination of 2:1 (H:V). The slope is practically devoid of vegetation with no obvious erosion protection mat (Figures 7A through 7C). No AC dikes were employed to divert the flow in this area.

Surface erosion on the rock slope cut face is expected to be minimal, as existing 1:1 slopes in similar rock in the region exhibit only minor erosion features if any (See Figures 7G and 7H in Appendix IV).

6.6 Scour

No perennially flowing creek or stream was observed within the limits of the proposed facilities during our site reconnaissance. Drainage washes winding through the west part of the proposed alignment are dry year-around, except for during moderate to heavy rainfall. The climatic conditions within the region are arid and normally precipitation is negligible, however, flash floods do occur and are unpredictable in their intensity. Therefore, scour may be an issue with regards to culverts.

During heavy precipitation, washes are expected to fill with water and suspended sediments. Our field review of these dry washes revealed deposited sediments ranging from silt to gravel, with the largest fraction of the deposits being coarse sand.

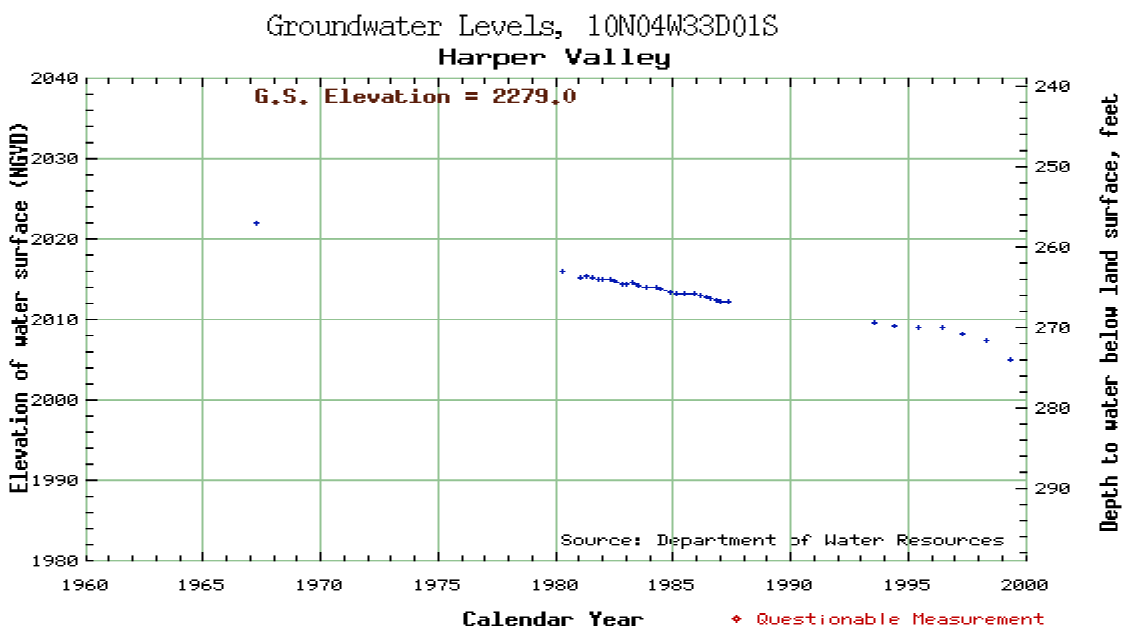
7. GROUNDWATER

Groundwater was not encountered in our preliminary site exploration. Supplemental groundwater information obtained through the Department of Water Resources, Division of Planning and Local Assistance (DPLA) reveals that the shallowest groundwater measurement in their database was 36 feet below ground surface (bgs) near the eastern end of the proposed alignment. Based on readings from two observation wells adjacent to our project limit, groundwater levels have fluctuated over time, but exhibit a general decrease in elevation since the mid 1990's (Figure 4). Groundwater is not expected to affect the proposed alignment. However, groundwater can occur as perched water, where water collects on impermeable layers in the subsurface strata. These perched water conditions vary seasonally, depending on rainfall and local recharge conditions. The locations of observation wells are shown on

Figure 5 with historical records of groundwater measurements for the selected wells summarized in Table 3.

Within the cut sections of the alignment groundwater may be perched, or may become perched, on the contact between rock and alluvium. It is possible that, upon completion of the cuts in this area, water flowing along the bedrock/soil contact may seep out along the line of intersection between the cut face and the aforementioned geologic contact. In this case water may seep out and flow down slope toward the proposed roadway. Seepage out of the cut face is not expected to be a permanent condition, as there is not enough rainfall to create year-round flow. However, this condition is expected to occur after periods of heavy rainfall.

Figure 4. Groundwater Levels



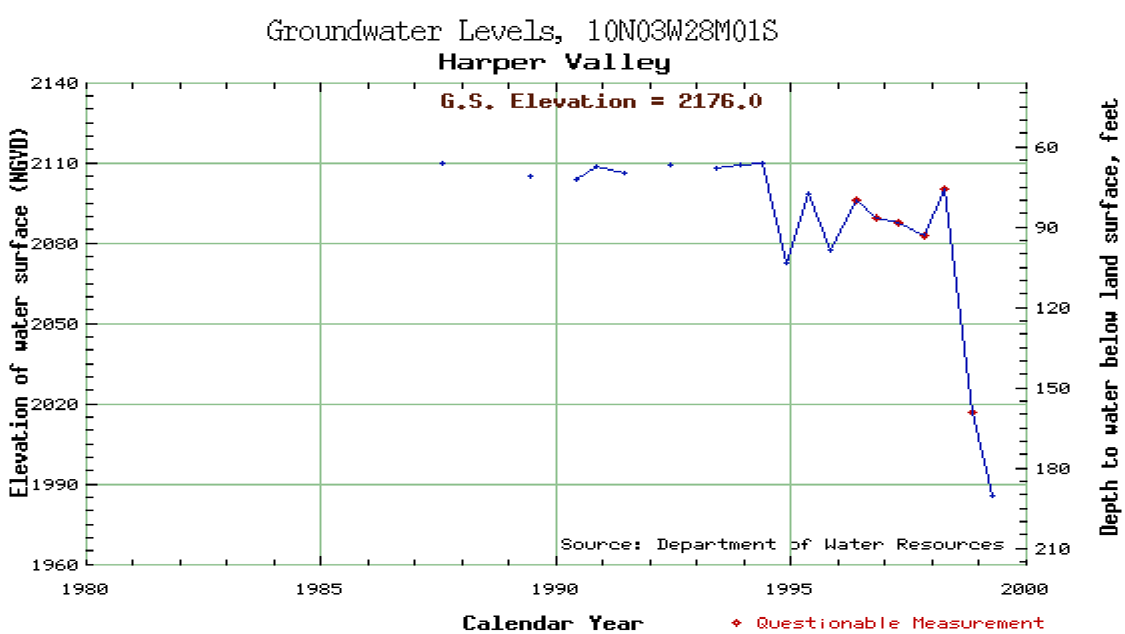


Figure 5. Well Locations

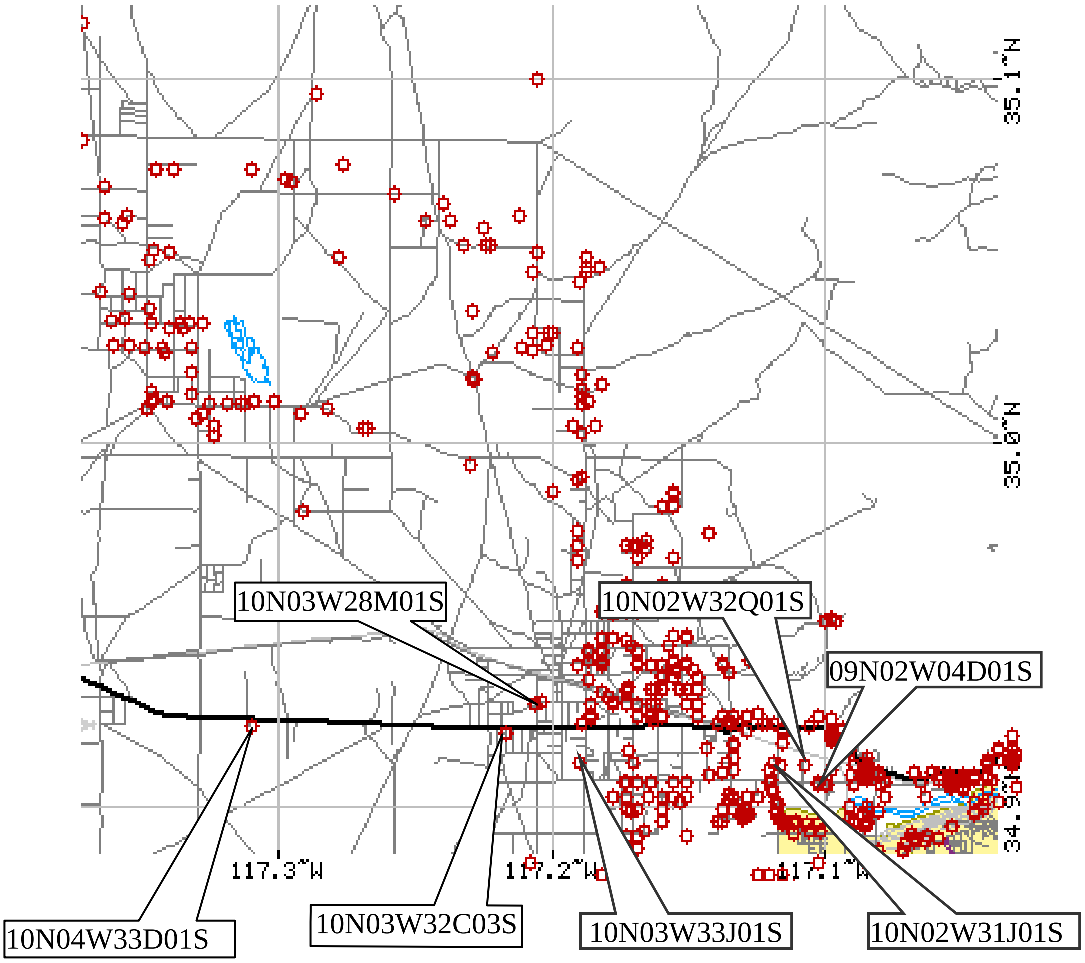


Table 3. Groundwater Database

Well Number	Latest Measuring Date	Approx. Ground Elevation (ft/m)	Approx. Ground Water Elevation (ft/m)	Depth to Ground Water Table (ft/m)
10N04W33D01S	4/8/99	2279/695	2004.8/611	274.2/84
10N03W32C03S	1/19/93	2215/675	2138.3/652	76.7/23
10N03W28M01S	4/8/99	2176/663	1985.7/605	190.4/58
10N03W33J01S	4/19/71	2230/731	2142.3/653	87.7/27
10N02W31J01S	3/31/98	2175/663	2101.2/641	73.8/23
09N02W04D01S	3/27/96	2170/662	2116.4/645	53.7/17
10N02W32Q01S	3/26/58	2172/662	2135.8/651	36.3/11

8. SEISMICITY

This project is located in a seismically active area. The geologic processes that have caused earthquakes in the past can be expected to continue. Seismic events that are likely to produce the greatest bedrock accelerations could be a moderate or large event on the Active Lockhart (LHT) fault zone or a large event on another, more distant fault. A fault is considered by the State of California to be active if geologic evidence indicates that movement on the fault has occurred in the last 15,000 years (used for fault rupture source), and active if movement is demonstrated to have occurred in the last 700,000 years (not used for fault rupture source).

The closest active fault is the strike-slip Lockhart Fault (LHT), which crosses the proposed alignment near the intersection of Hinkley Road. An Alquist Priolo Earthquake Fault Zoning Act (APEFZA) map for this area has not yet been completed by the California Geological Survey (CGS), however, referenced material describes the southeastern portion of the fault as being active.

Ground shaking is the primary cause of structural damage during an earthquake and is considered to be one of the most likely damage producing phenomena for this project. The magnitude, duration and vibration frequency characteristics will vary greatly, depending on the particular causative fault and its distance from the project. From the 1996 Caltrans California Seismic Hazard Map, the Maximum Credible Earthquake (MCE) would be a M7.25 magnitude earthquake on the Lockhart (LHT) fault zone. The project site falls within the 0.6g peak bedrock acceleration contour on this map and, utilizing the curve by Maulchin, 1992, for estimating the acceleration factor, the peak site acceleration would be estimated to be in excess of 0.6g. A map showing the project location and the nearest faults is presented on Figure 6.

9. LANDSLIDES AND ROCKFALL

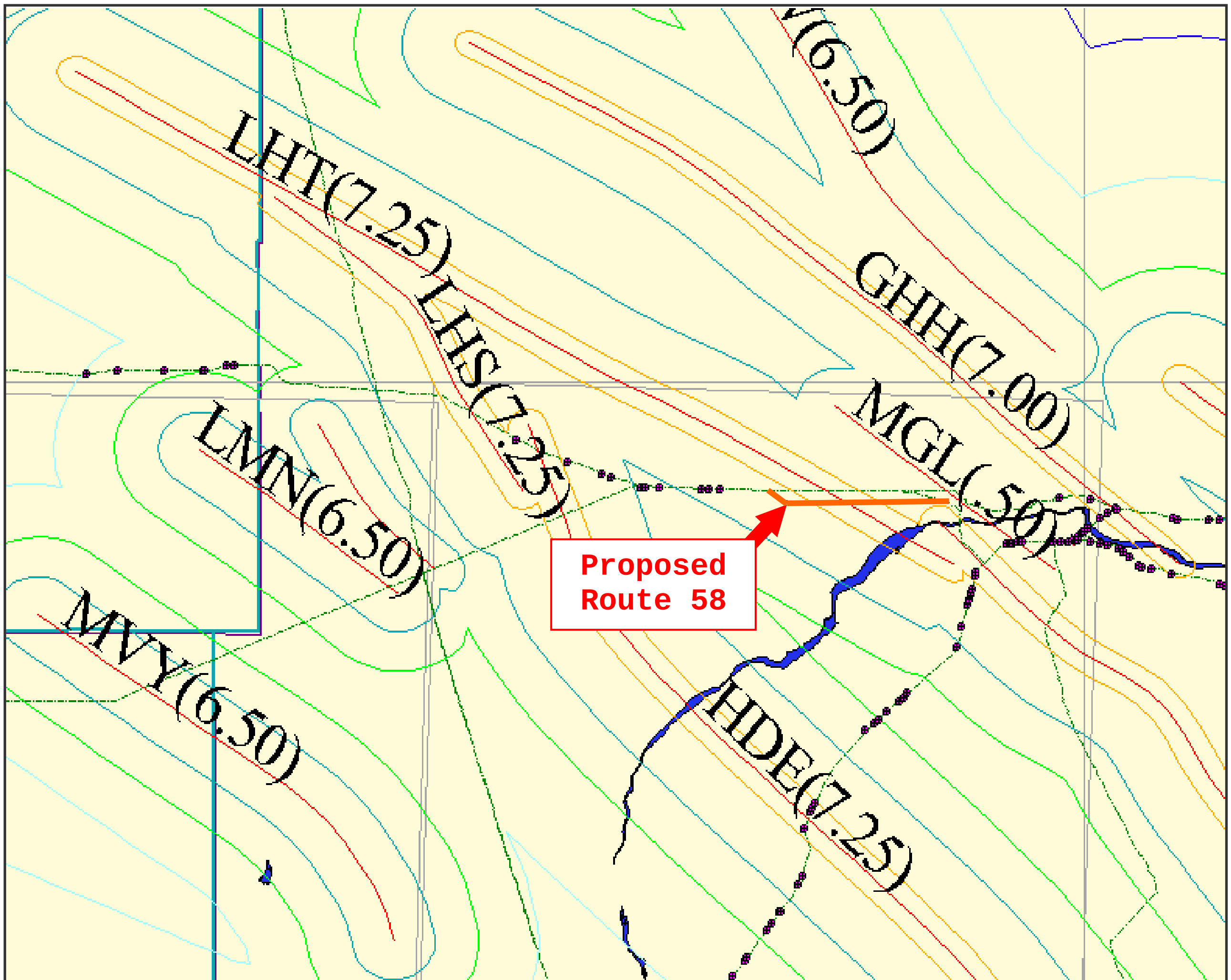
Landslide and rockfall are described in Section 12.6, “Slope Stability.”

10. SNOW AVALANCHES

Snow avalanche is unlikely, due to the generally flat terrain along with infrequent and relatively light seasonal snowfall within the project limits.

11. GEOTHERMAL ACTIVITY

No known geothermal activity was identified within the project limits



Legend:

- 0.6g Peak Acceleration Contour
- 0.5g Peak Acceleration Contour
- 0.4g Peak Acceleration Contour
- 0.3g Peak Acceleration Contour
- 0.2g Peak Acceleration Contour
- Faults with Fault Codes (MCE)
- State Highways
- State Bridges
- County Boundary

M
14.5
N

0 25 kilometers

0 5 10Miles

LHT: Lockhart Fault
LHS: Lockhart South Fault
HDE: Helendale Fault
MGD: Mount General Fault

GHH: Gravel Hills
Harper-Harper Lake Fault
MVY: Mirage Valley Fault

CALTRANS Division of Engineering Services Geotechnical Design-South 2	EA: 08-043510 Date: May 2013	Seismic Map
	08-SBd-58-PM 22.2/31.1 Realignment at Hinkley, San Bernardino, CA	FIGURE 6

12. GEOTECHNICAL CONSIDERATIONS

12.1 General

The geotechnical considerations presented here are based on the site investigation, literature review of reports for previous projects adjacent to this project, soil exploration and laboratory testing for the soil samples collected from the borings.

12.2 Liquefaction, Ground Shaking, and Surface Rupture

The potential for liquefaction is relatively low based on the reported groundwater depths and generally dense nature of the subsurface granular soils as defined by SPT blowcounts. Ground shaking is expected to occur at the site, considering the predicted magnitude of peak bedrock accelerations for earthquakes along nearby faults. Although ground shaking can cause densification of loose soil layers, inducing settlement of roadways, the generally dense soils at the site will minimize or preclude seismically induced settlements.

Surface rupture has been documented as having occurred on the southeast portion of the Lockhart (LHT) Fault during the Quaternary (geologically recent) period. However, surface rupture has not been studied in detail where the trend of the Lockhart Fault intersects the proposed alignment between stations 1336+38 and 1500+41.

Neither ground shaking nor fault rupture can be avoided in the design of highways crossing active faults. However, placing the proposed highway either at natural grade or in low cuts or on low embankments limits the potential for, and consequences of, failure in the cuts and fills. That allows the highway to be restored to service with comparative minimum of maintenance or re-construction effort following a seismic event. Accordingly, the currently proposed design is favorable for accommodating ground shaking or rupture.

12.3 Embankment Foundations

Embankments will be constructed in three areas of the project. The first section, from STA 1205+15 to STA 1251+10, extends from the project's west limit at existing Route 58 to the beginning of the cut through the hilly terrain west of Valley View Road. The second section of embankment will be from STA 1316+69 to STA 1671+01, in general extending from the east side of the cut area all the way to the east limit of the proposed project. The third section of embankment will be structure approach fills along Lenwood Road, running perpendicular to the main alignment at STA 1633+28. The maximum embankment height for the first and second sections (along the main alignment) will be 10 feet. The height of approach embankment for the proposed Lenwood Road overcrossing will be 40 feet.

Visual inspection and limited soil exploration along the above embankment sites indicate that the surficial soils are alluvial fan deposits that can be categorized as sand, silty sand, and sandy silt scattered with gravel and interceded with occasional lean clay layers. No cobble or boulder size particles have been detected upon both surficial inspection and subsurface exploration.

According to our soil exploration, subsurface materials for the first section of embankment are expected to consist of dense to very dense sand, silty sand and sandy silt. About 10 feet of hard silty clay (with sand) was found at STA 1221+56 about 10 feet below ground surface. These soils are expected to provide a firm foundation for the proposed embankment without remedial grading measures.

Twelve boreholes were drilled during the foundation exploration of the second section of embankment including B23A and B23B for the future sound wall (approximately 260 feet left of main alignment). The soil from STA 1323+25± to STA 1500+41± consists of old alluvial fan deposits of dense to very dense sand, silty sand, and sandy silt with occasional angular or subangular gravel sizes of up to 0.4 inches. The soils observed at STA 1526+65 (B39) and STA 1579+15 (B40) consist of medium dense to dense sand/gravel mixtures with dense to very dense sandy silt at about 23 ft below the original grade. For planning purposes, the embankment foundation conditions from STA 1500+41 to the east project limits may be presumed to be reflected by boring B39 and B40.

For the 40 feet high approach embankment (Section 3) at the Lenwood Road overcrossing and the BNSF underpass, four boreholes (B28, B29, B30 and B32) were drilled to depths ranging from 51.5 to 61.5 feet below ground surface. Close to the edge of the Mojave Flood Plain, the foundation materials underlying the proposed embankment in this area are generally comprised of medium dense to very dense, stiff-to-hard sand, silty sand, and silt with gravel. These alluvial deposits are interbedded with silty and sandy clay layers of varying thickness.

12.4 Structure Foundations

Soundwall. According to District 8, no soundwall will be built north and east of Valley View Rd. Nevertheless, the foundation soils in the vicinity are relatively uniform and consist primarily of dense to very dense silty sand/sandy silt with gravel. The correlated internal friction angles (ϕ) from the adjusted SPT blow counts at different depths (from the original grade) range from 39^o to 44^o. Neither soil caving nor groundwater was observed in the borings.

Retaining Wall. A retaining wall will be required to retain the embankment of the westbound exit ramp to Lenwood Road along an area where the embankment would otherwise encroach on the BNSF railroad. Wall height is 23 feet, and the wall will be founded on original ground and retain the embankment. Our Office conducted a site sub-surface investigation in January, 2013, which consisted of three vertical auger borings to a maximum depth of 60 feet.

The Lenwood/Rt. 58 interchange area is an older, consolidated Mojave River depositional terrace. The sediments are interbedded silt, sand, and clay. The predominant material recovered from the borings consists of Sandy SILT in a dry, very stiff condition from the surface. This material’s lower surface dips to the east from 12 feet in boring A-13-001 to 15 ft. at boring A-13-003. Below this is SAND to Clayey SAND in a dry, dense to very dense condition at 25 to 33 ft. Below the sand is a Silty CLAY in a moist, stiff to very stiff condition. The clay has a flat bottom from 45 to 46 ft. Below the clay is a Silty SAND to Clayey SAND in a moist to dry, dense to very dense condition to the bottom of the boring.

The correlated ϕ values from the adjusted SPT blow counts generally range from 32^o to 37^o below the toe area. However, 1 to 2 feet of the surficial materials are relatively loose at the eastern part of retaining wall alignment. The loose soil zone creates a potential for distress due to differential settlement. It is recommended that the wall foundation soils be sub-excavated and restored to grade with compacted fill. This will also improve the external stability of retaining wall/embankment system. With the foundation area subexcavated and restored as compacted fill, a Type 1 Standard Plan retaining wall can be used.

The potential for liquefaction is not anticipated based on groundwater depth and generally dense nature of the subsurface soils. No groundwater was encountered during our investigation at this location. Water levels will fluctuate with rainfall events but should not affect construction.

12.5 Settlement

12.5.1 Immediate Settlement

Immediate settlement (including elastic settlement) due to the self-weight of embankment fill and compression of foundation soil will occur during placement of the embankment and will be completed by the end of construction.

The estimations for immediate settlement of embankment foundations were based on *Soils and Foundations Workshop* (FHWA NHI-00-045, August 2000). The settlement was estimated according to the bearing capacity index (C’) correlated from the corrected SPT blow counts. The anticipated settlement for all three sections of embankment is presented in Section 15.6 (Recommendations) of this report.

Other than immediate settlement, subsidence of embankment foundations is expected to occur during construction operations. Subsidence is estimated to be 1.2 inches.

12.5.2 Primary Consolidation (Embankment Sections 1 and 2)

Primary consolidation of embankment foundations is unlikely beneath the first and second sections of embankment due to the absence of high groundwater, the primarily granular nature of foundation soils, and the limited height of embankment. Although fluctuations in the ground water table may occur in the future, they should not induce objectionable settlement as excess pore pressures generated by placement of embankment should dissipate during or shortly after grading, considering the granular soils. Consolidation testing of the silty clay encountered 10 to 20 feet below present grade in Boring B1 in the first embankment foundation area shows that the pre-consolidation pressure is higher than anticipated stress increase due to the added surcharge of the proposed embankment. Therefore, primary consolidation will not be a major issue for these sections of embankment.

12.5.3 Primary Consolidation (Embankment Section 3)

Based on the subsurface investigation, several sandy/silty lean clay layers with consistency varying from stiff to hard were found in boreholes B28, B29, B30, and B32. These clay layers were not uniformly distributed, and were located at depths varying from 10 to 50 feet with an estimated total thickness of less than 10 feet. Since groundwater was not encountered and the clayey material was relatively dry, primary consolidation for this section of embankment should not be an issue for the proposed improvements.

12.5.4 Secondary Settlement.

Soluble minerals such as caliche which could cause secondary settlement upon saturation, are abundant within on-site soils, especially for the soils close to the east and west limits of the project. A review of the collapse potential (CP) test data for the soil samples collected to date shows the maximum CP value as being 2% for STA 1420+03 (B35, approximately 10 feet below ground surface). Southeast of our project, in the area between the intersections of Route 58 with Community Blvd. and Main Street, Moore & Taber, Inc. (see ref. #1) found the upper 5 to 10 feet of soil to have a moderate (2.1% to 4.1%) collapse potential. However, while soil samples from our field exploration along Lenwood Road did show well cemented, chalk white caliche veins within some silty soils, our laboratory testing indicated only a slight collapse potential with CP values less than 1%.

As seen to date, both in the lab and in the field, caliche ($CaCO_3$) exists only within less permeable materials such as very stiff silt. Moisture infiltration and consequent dissolving of the soluble soil (caliche) particles will be hard to achieve, especially considering the arid climatic condition, and the low groundwater table. Therefore, secondary settlement resulting from soil collapse under future embankment loading is considered to be extremely low.

12.6 Slope Stability

Within the planned cut sections of the proposed alignment, the roadway will be constructed into rock, with a maximum of 56 feet of cut, although the upper reaches of the cuts may be in alluvial or colluvial soil, not rock. As stated in the site geology section, the medium-to-very hard gneiss/quartz diorite/marble rock contains healed fractures that are closely spaced (2-14 inches) and that dip 50 to 75 degrees down from horizontal. Based on field exploration, a cut slope angle of 1.5:1 (H:V) should be used, which is flatter than the shallowest dipping fracture orientation. Nevertheless, for the purpose of this report, cut slopes in rock may be planned a 1.5:1 (H:V) or flatter. The soil profile resting on the rock at the top of cut slopes is erodible and should be slope-rounded or flattened to 2:1 or flatter.

Embankments constructed of soil native to the project limits and graded at a 2:1 or flatter slope ratio are considered stable. Taller or steeper embankment slopes should be brought to our attention for possible stability analysis.

12.7 Erosion

Given the close proximity of the proposed project to the Main St. U.C. and the BNSF overhead approach embankment of the existing Route 58 expressway, and given likely similarities in the engineering properties of embankment borrow used for the project and for those two sites, the erosion performance of the existing approach embankments could be an indicator of erosion performance of the proposed project's embankments. The observed slope performance infers that measures should be taken to mitigate the potential for erosion on future slopes. While we offer the following discussion on erosion mitigation, actual mitigation measures should be selected through interaction between Design, the District Landscape Architecture Section and the Office of Geotechnical Design-South.

1) Vegetate and mulch the slope surface. Depending on the vegetation selected, irrigation might be required. Water availability could therefore preclude this alternative. Besides vegetation and mulch, erosion protection coverings like rice straw or geotextiles (erosion control mats) could also be used, although these measures tend to degrade with time and exposure to the sun and weather. The erosion control mats should be placed on the completed embankment slopes as soon as practicable after grading. Specifications could require the embankment construction to be done in phases, with completed slopes covered following each phase of grading. We defer to the District Landscaping Section for techniques, specifications and materials in vegetating slopes.

2) Time the embankment construction to minimize soil exposure. Precipitation is a key factor in slope erosion. If possible, it would be best not to perform embankment construction during the relatively wet season. Embankment could be constructed during late spring to early summer months and vegetated/mulched prior to the rainy season.

3) Divert runoff away from the slope surface. Use a combination of pavement cross-slope and asphalt concrete (AC) dikes to prevent flow over the top of slope.

4) Roughen the slope surface by applying salvaged topsoil (with vegetation) from the clearing and grubbing operation. This would reduce the runoff velocity and enhance the growth of native vegetation.

5) Armor the slope using rock fragments derived from blasting/cutting the cut slope section on the west side of the proposed alignment.

6) Build "zoned" embankments such that the sides of the embankments are equipment width "shells" of rock fill derived from cutting the hard rock segment of the project.

Flattening embankment slopes reduces the velocity of runoff flowing down the slope, thereby reducing the ability of the flow to erode. However, flatter slopes have longer surfaces exposed to rain and therefore generate large flow quantities. Nevertheless, in areas such as this where the ability to vegetate slopes is questionable, we believe that flattening the fill slopes is one of the best erosion resisting techniques. However, as noted above, even low height 4:1 (H:V) slopes in fine grained, granular soils can erode if runoff is allowed to flow over the slope face.

12.8 Gouge Filled Shears

For the cut section between STA 1255+00 and 1297+00, clay gouge was encountered below existing grade that have implications for project design. Depending on the strike and dip of the gouge/fault, the gouge could allow a wedge-shaped landslide in the cut. Also, if the feature dips up toward profile grade such that it would be exposed in the road subgrade, there is a potential for the clay to expand, heaving the pavement. We recommend over-excavation of 1 foot of the profile sub-grade material to be recompacted with structural backfill (no clay size material) to mitigate expansion potential.

13. CORROSION

Sampling for corrosion testing to date was limited to the areas where future reinforced concrete structures may be built. Bulk soil samples were recovered using a drill rig or hand auger, and we requested pH value, minimum electrical resistivity, chloride and sulfate concentration testing. The tests for sulfate and chloride are usually not conducted unless the resistivity of the sample soil is 1000 Ohm-cm or less. Where the resistivity is greater than 1000 Ohm-cm, the soil is considered non-corrosive. Corrosion sampling has been requested for samples taken at the following locations, listed in Table 4.

Table 4. Corrosion Testing

Boring	Sample Depth	Sample Date	PH	Minimum Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
A-13-001	3-5 feet	1/8/2013	7.90	1153	N/A	N/A
A-13-002	5-10 feet	1/9/2013	7.69	752	264	480
A-13-003	5-10 feet	1/9/2013	8.10	2441	N/A	N/A
B-2	0-1 foot	1/29/2002	8.71	5600	N/A	N/A
B-20	8-9 feet	10/2/2003	9.29	1800	N/A	N/A
B-20	29-30 feet	10/2/2003	9.33	950	230	210
B-22	8-9 feet	10/2/2003	8.33	950	320	200
B-22	22-33 feet	10/2/2003	9.5	1500	N/A	N/A
B-23A	1-5 feet	2/5/2002	8.76	470	750	520
B-33, B-34	0-1 foot	2/26/2002	8.01	620	250	240
Soil pile-W	Surface	N/A	8.08	470	710	340
Soil pile-E	Surface	N/A	7.55	1100	N/A	N/A

Soils tested were found to be non-corrosive except at the location of Boring B-23A, STA 1323+25 (chlorides in excess of 500 ppm). Material from this area should not be used for structural backfill. If a structure is planned for this area, our office should be contacted for additional recommendations.

14. HAZARDOUS WASTE IMPACT

As of the writing this report, subsurface investigation has been conducted where permits allowed. Groundwater was not encountered to explored depth of 90 feet.

According to a hazardous waste questionnaire received from the district:

- Hexavalent Chromium (Chrom VI) is known to occur in area groundwater.

- Hexavalent Chromium (Chrom VI) may possibly occur in area soils.

Excavations for the proposed roadway do not exceed 56 feet bgs and therefore the risk of encountering groundwater during construction is low. However, water required for construction purposes should not be taken from existing or constructed groundwater wells within the project limits without the consent of the District Materials Section and Hazard Waste Units, nor without the involvement of other water use regulatory agencies..

Additionally, on-site septic disposal systems for residences located along the proposed alignment need to be removed prior to construction. We understand that the District will assess the numbers and locations of such systems and provide for their removal as part of the R/W clearing process. We further understand that excavations created during that process will be refilled with fill compacted under the District's inspection.

15. RECOMMENDATIONS

15.1 Cut Slopes

Cut slopes for this project lie between stations 1257+60 and 1314+00. For planning purposes, assume the cut slope ratios will be 1.5:1 (H:V) and provide 10 foot wide catchment areas at the slope toe. The catchment area should be sloped down at 5% from the horizontal from the edge of pavement towards the toe of the cut.

The top 7 ft or less of soil on top of the cut should be sloped 2:1(H:V). If a cut slope angle of less that 2:1 is used in the rock cuts then remedial measures such as rocknet fences or mesh/cable drapery may need to be used to stop rockfall or wedge failures. Where the natural slope above the cut dips down toward the cut, brow ditches are recommended.

15.2 Grading factor

We recommend a value of 1.3 for earthwork factor in the rock cuts and a value of 1.05 for cuts in alluvium.

15.3 Embankment

Embankment slopes may be graded at 2:1 (H:V) or flatter. Measures such as AC dikes are recommended to prevent flow from the roadway over the slope face; however, dikes may be omitted where erosion control measures are used.

Wherever the future embankment will be constructed across natural drainage courses, 1.5 feet of alluvium shall be subexcavated from the embankment culvert foundation area and replaced as compacted fill.

Embankment foundations shall be prepared in accordance with Section 19 of the Standard Specifications. For PS&E cost estimating purposes, where embankment crosses existing cultivated land, the embankment foundation shall be sub-excavated 2.5 feet and restored to grade with compacted fill, as discussed in Section 4.3. Embankment foundation areas disturbed by building demolition or basement backfilling operations should be overexcavated 2.5 feet and restored with compacted fill, as listed in Section 4.3. The exposed overexcavated surface should be scarified, moisture conditioned to near optimum moisture, and uniformly compacted to 90% of relative compaction.

Roadway excavation from the rock area on the western part of the alignment is expected to contain potentially significant volumes of rocky materials. Size of the generated borrow particles will depend largely on the contractor's excavation techniques and, if used, blasting program. Rocky embankment fill should be covered with a layer of geotextile prior to placing subsequent layers of soil fill or structural section - subbase or base. The placement and compaction of the above materials should be in accordance with Section 19.5 and 19.6 of Standard Specifications (2010).

15.4 Erosion Control Measures

Embankment erosion can be mitigated by the implementation of the following:

- AC dikes
- Erosion control mats draped over and affixed to the completed slope face.
- Reinforcing the embankment slopes with geotextiles
- Light slope armoring using a 12-inch thick blanket of angular, cobble size rock derived from excavation of the cut section for the proposed alignment.
- Vegetating the slopes as prescribed by the District Landscape Architects.
- Creating "zoned embankments" using an exterior equipment width "shell" of rocky material for the outside portions of the embankment and common borrow for the interior portion.

15.5 Excavation Techniques

Excavations can be accomplished by conventional techniques for this project, except for the cut sections between stations 1257+60 and 1314+00. This crystalline rock mass contains a weathered horizon that appears rippable to a depth of approximately 6.5 feet below the top of the rock. At depths between 6.5 and 46.5 feet, the rock will require difficult ripping and/or light blasting. Rock excavated below a depth of 46.5 feet will likely require blasting. For estimation purposes, 15% to 20% of the total excavation volume may be assumed to be by blasting. If blasting is not viable then realignment may be considered.

15.6 Settlement

The estimated settlement for embankment Section 1 and Section 2 will be less than 1 inch. Settlement (including subsidence) on the order of 6 inches is anticipated for the highest portions of the approach embankment on Lenwood Road, with differential settlement between the toe and centerline of the embankment being on the order of 3 inches.

15.7 Retaining Wall

At the location where loose materials were disclosed for Retaining Wall 1645 (see Section 12.4 "Structure Foundation"), the wall foundation soil should be subexcavated to a depth of at least 2 feet below the retaining wall footing. The lateral limits of the subexcavation should be in conformance with Section 19-5.03 of the Standard Specifications (2010). The exposed ground surface should be scarified, moisture conditioned to near optimum moisture, and uniformly compacted to 90% of relative compaction. The overexcavated area should be backfilled with recompacted excavated soil. The placement and compaction of the soils placed back into the subexcavated area should be in conformance with Section 19 of the Standard Specifications (2010).

APPENDIX I

References

References

1) Materials Study, Route 58 and Interstate 15 Highway Improvements, Barstow, California (December 1990, by MOORE & TABER Inc.)
2) Foundation Recommendation for Mojave River Bridge (Br. No. 54-1110R) (Memo from Joseph S. Pratt to OSD, March 18,1998)
3) Draft Project Report (for CA58 realignment and widening to 4-lane expressway, Andy DaSilva, District 8)
4) Project Study Report (for CA58 realignment and widening to 4-lane expressway, District 8, July 1991)
5) Jennings, Charles W. (1994). Fault Activity Map of California and Adjacent Areas with Location and Ages of Recent Volcanic Eruptions. California Geologic Data Map Series, Map No. 6. California Division of Mines and Geology.
6) Bortugno, E. J., and Spittler T., Geologic Map of the San Bernardino Quadrangle, California Division of Mines and Geology 1986.
7) California Seismic Hazard Map, Maulchin 1996.
8) Soil Exploration Plan for Right of Entry Permit (Austin Perez, Oct. 2000)
9) Principles of Geotechnical Engineering, Braja M. Das, 1985, PWS-KENT Publishing Company
10) Soils and Foundations, Workshop Manual, FHWA, 1993
11) Geotechnical and Foundation Engineering – Design and Construction, Robert W. Day, 1999, McGraw-Hill Companies, Inc.
12) Foundation Engineering Handbook, H. F. Winterkorn & H. Fang, 1975, Van Nostrand Reinhold Company
13) State Regional Water Quality Control Board, Lahontan District Regional Map URL: http://www.swrcb.ca.gov/rwqcb6/images/maps/reg6lah.gif, June 2002.
14) California Department of Water Resources, Water Data Library URL: http://wdl.water.ca.gov/gw/gw_data/hyd/gwater/clickmap.plx?type=move&rgpr=1170,345&qdpr=1172,348&ckpr=crumple&mvpr=0,1 June 2002.

APPENDIX II

Site Location Map

APPENDIX III

Boring Information

APPENDIX IV

Site Photos

APPENDIX V

Geophysical Reports

APPENDIX VI

Boring Logs

APPENDIX VII

Laboratory Test Data