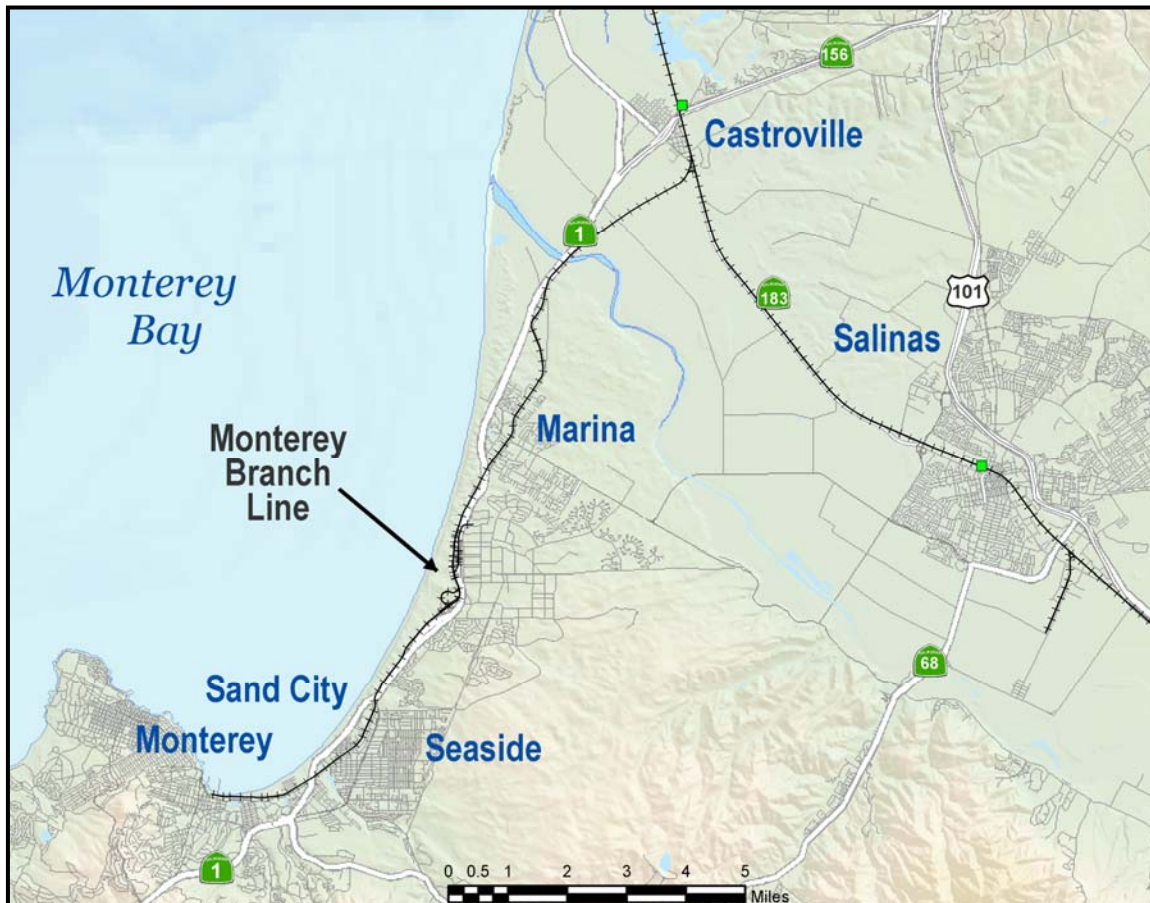


TRANSPORTATION AGENCY FOR MONTEREY COUNTY

GEOTECHNICAL EVALUATION REPORT FOR THE MONTEREY PENINSULA LIGHT RAIL PROJECT



November 16, 2010

**GEOTECHNICAL EVALUATION REPORT
FOR CONCEPTUAL ENGINEERING PHASE
MONTEREY PENINSULA LIGHT RAIL PROJECT
TRANSPORTATION AGENCY FOR MONTEREY
COUNTY (TAMC)
MONTEREY COUNTY, CALIFORNIA**

November 16, 2010

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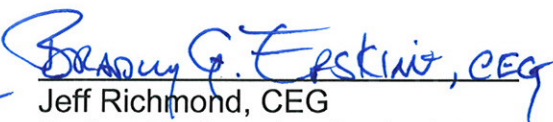
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TRANSPORTATION AGENCY FOR MONTEREY COUNTY (TAMC)
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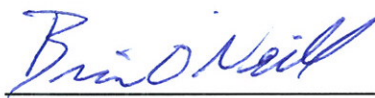
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November 16, 2010

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1 INTRODUCTION

This report presents the results of Kleinfelder's Geotechnical and Geologic evaluation in support of the conceptual engineering phase for the Monterey Peninsula Light Rail Project along the existing Monterey Branch Line (MBL) planned by the Transportation Agency for Monterey County (TAMC). Kleinfelder is a subconsultant to Parsons that is leading the project in preparation of conceptual engineering (approximate 10 to 20% level of design completion) studies and cost estimates to add 'definition' to the project for TAMC. This report was revised in November 2010 to include updated project description information and address review comments provided by Parsons' planning and design staff for the project.

The primary engineering objectives of the Parsons team for the conceptual design phase include (1) defining the major construction elements and to update the project cost estimates to more accurately reflect current market conditions, (2) establishing the vertical and horizontal alignment and approximate footprint for the facilities for use by other discipline design leads, and (3) review and update of the project design criteria and standards to current industry practices for use by the designers for the upcoming 'preliminary' and then 'final' engineering phases of the project. The work products developed by the Parsons team during this phase will be submitted to TAMC for review.

Geotechnical evaluations during the initial conceptual design phase have been accomplished with desktop studies (literature review and interpretation of available data), and will be followed in future preliminary and final design PS&E phases by site-specific subsurface exploration, field and laboratory testing (soil and rock samples), data interpretation and geotechnical analyses/calculations. Geotechnical studies for the 20% phase made use of available reports (geologic / seismic / geotechnical / hydrogeologic) from nearby projects, and existing technical information from various agencies, including TAMC. Geotechnical evaluations and preliminary recommendations were developed to provide guidance and support for civil and structural engineering conceptual design of infrastructure features for various structures and associated improvements to facilities. No subsurface exploration was performed for the current conceptual design phase.

The primary geotechnical issues of concern for project facilities include foundation type(s) for support of loads from bridges and other structures, presence of 'poor' soil (weak and compressible, and/or loose fill) in some reaches, trackbed rehabilitation, stability of slopes (natural/cut and fill) and at bridge abutment areas, drainage and sub-drainage of surface and groundwater, potentially expansive and/or corrosive soils, earthquake ground shaking and related secondary seismic hazard effects including liquefaction ground settlement/deformation.

2 PROJECT DESCRIPTION

2.1 GENERAL

The TAMC agency is proposing implementation of passenger rail service along a 15.2-mile rail corridor to be restored along the existing Monterey Branch Line (MBL) extending from Castroville to downtown Monterey. Plate 1 shows the project location, which includes the cities of Monterey, Seaside, Sand City, Marina and the unincorporated community of Castroville.

The rail corridor went into service about 100 years ago but currently has no operating train traffic. The railroad was originally used by the Southern Pacific Transportation Company, and most recently used and operated by the Union Pacific Railroad (UPRR). The MBL originally extended from Del Monte Junction at Castroville to Lake Majella in Pacific Grove, a distance of approximately 20 miles. The track that remains in place extends from Castroville to the vicinity of Fisherman's Wharf in Monterey. The track, where it remains, is generally in unusable condition. As shown on Plate 1, the rail corridor generally parallels Highway 1 in Monterey County located on the west coast of California. Elevations along the rail alignment range from about 8 feet above Mean Sea Level (MSL) at the far south end of the corridor to a maximum of approximately 165 feet MSL where the rail corridor traverses the large sand dune areas located in the southern half of the alignment.

The route of the proposed passenger rail transit line is comprised primarily of an existing single-track rail line with passing sidings and industrial spurs. There are several public road grade crossings along the corridor. Where the tracks cross roadways at grade, traffic would be controlled by bells, flashing beacons, gates, and at some locations with traffic signals.

The project includes restoration/reconstruction of at-grade trackwork as well as various structures including bridges and culverts, construction of new passenger stations, upgrading or replacement of street/grade crossings, reconstruction of a recreation trail (located in the southern reaches of the corridor) alongside the railway, and related improvements for minor utilities, drainage, and pavements. Additional description of key project features is provided in the following section.

2.2 SUMMARY OF KEY PROJECT FEATURES

The proposed rail corridor improvements are described and illustrated on the updated Conceptual Plans for Track Restoration set of drawings prepared by Parsons dated October 2010. Primary project features for the rail transit project include:

- Restoration of track, grade crossing protection, signal systems as needed, turnouts, bridge structures, etc. as necessary to operate passenger rail between the proposed Castroville commuter rail station and a station located in the City of Monterey.
- Construction of 12 rail passenger Stations in Castroville, Marina, Seaside, Sand City and Monterey.
- Construction of a multi-modal transit center at a site in Marina, California on lands formerly occupied by Fort Ord.
- Repair of the major bridge over the Salinas River.
- Construction of a light rail layover and joint Maintenance Facility and Operations Center at the former Fort Ord, adjacent to the old Quartermaster warehouses, located east of Highway 1. Alternatively, this facility may be constructed on TAMC-owned lands located west of Highway 1 and adjacent to the balloon-spur track.

Additional descriptions for the key project features along the rail transit line are summarized below.

2.2.1 Track Restoration or Replacement, and Street/Grade Crossings

The track would be restored or reconstructed (where necessary) generally within the current configuration/alignment in a single-track line with new concrete or timber ties, ballast sections and grade crossing protection for the 15.2 mile MBL segment. Passing sidings would be constructed where needed to allow for two-way train operations. For typical track sections, refer to the illustrations in Parsons Conceptual Plans for Track Restoration, Figures 8-8, 8-9 and 8-10. Existing track along an approximate 2-mile

reach within the former Fort Ord area would be reused because this segment was reconstructed in the 1960s.

No grade separations are proposed as part of this project, so all points where the proposed track alignment will intersect local roadways will be at-grade. Each street/grade crossing would be constructed with a high durability pre-cast concrete crossing surface.

2.2.2 Bridges / Track Structures

There are six existing bridges/trestles within the MBL rail segment corridor, including the ‘major’ bridge over the Salinas River (with spans of over 140 feet in length). Most of the bridges are either timber open deck trestles or timber ballast deck trestles constructed around 100 years ago. Original construction of most of the bridges appears to be based on standard drawings common to the railroads, with timber piles supporting the abutments and pier bents. Bridge inspection reports, alternatives analysis information, and recommendations for rehabilitation or replacement are presented in the updated Bridge Strategy Report prepared by Parsons, dated May 2010.

The bridge report revealed that most of the structures inspected were generally in poor to fair condition. According to the strategy report, for the Salinas River Bridge the new/replacement track on the existing structure would be “open deck” (timber bridge ties fastened directly to the steel cross members, as they are now). For track on any new bridges, ballasted track is assumed. Ballasted track would also be constructed on any remaining timber trestles. The updated Conceptual Plans for Track Restoration by Parsons includes a list (Table 4-2) of the bridge structure attributes, including location by milepost number, bridge name, and description of structure type.

The northernmost bridge structure along the MBL rail line is an existing 150-foot long 10-span timber trestle located at the Tembladero Slough (MP 111.05). This structure is to be replaced with a pre-stressed concrete girder bridge supported on cast-in-place pile caps founded on driven piles. Another existing trestle bridge is located at the Alisal Slough (MP 111.93) crossing; however this relatively short, 45-foot-long bridge is to be replaced by an earth embankment and culvert undercrossing to address mosquito abatement concerns. The existing structure would remain.

Just south of Nashua Road, there are three existing ballast deck timber trestle bridge structures supported on timber piles. These structures consist of a 120-foot, 8-span drainage channel bridge (MP 112.54), a 225-foot, 15-span floodplain equalizer bridge (MP 112.80), and a 90-foot, 6-span floodplain equalizer bridge (MP 113.04). These structures and adjacent trackwork are in generally poor condition. Agricultural uses have encroached very near the base of rail track sections and the bridges. All three structures have been recommended for replacement by culvert structures and earth embankments as part of the planning process for the project.

The 715-foot long through-truss steel bridge over the Salinas River (MP 113.50) is the most significant structural feature along the MBL corridor. As discussed in the updated strategy report, the 5-span bridge will be repaired. The structure has experienced damage from earthquakes (including ground displacements) over the past century, and significant corrosion of some of the steel structure members of the bridge.

Further to the south, the MBL corridor forms the south edge of Roberts Lake and crosses the estuary connecting Roberts Lake and Laguna Grande Lake with a 45-foot long 2-span prestressed concrete railroad trestle bridge (MP 123.80). This bridge was reportedly constructed when the Southern Pacific railroad still owned the line around the 1970s/1980s, and was most likely designed for a minimum E72 live load. According to the Parsons report, this bridge would likely require no repairs and limited or no seismic retrofitting prior to re-establishing train service, but the existing bike path over the old rail bridge would be removed and the track would be reconstructed. The existing recreational trail would be relocated closer toward the Del Monte roadway on a new pedestrian bridge constructed over the estuary.

There are also some minor culvert structures within the project limits that would need to be evaluated for repair, replacement, or elimination.

2.2.3 Passenger Stations / Stops

The proposed project includes the construction of a total of 12 passenger Stations. The conceptual Station layouts and planned features to be constructed at the Station sites are shown on the conceptual drawings by Parsons. Each station would consist of a low-level platform for boarding and off-boarding the light rail trains, and various passenger amenities. Park-and-ride lots with paved surfaces are proposed for

construction at the Station sites identified in Table 4-6 of conceptual drawings set by Parsons.

Five Stations are proposed to serve Marina at Marina Green Drive, Beach Road, Reservation Road, Palm Avenue, and at Eighth Street. Two Stations are proposed to serve Seaside and Sand City at Playa Avenue and Contra Costa Street. In Monterey, four Stations are proposed: Casa Verde Way, U.S. Naval Postgraduate School, El Estero Park, and either Figueroa Street or west of Washington Street at the Maritime History Museum gateway to Portola Plaza. Light rail transit service would also serve one Castroville Station at Blackie Road.

2.2.4 Maintenance Facility and Operations Center

The rail transit project includes the construction of a centralized Maintenance Facility and Operations Center on the former Fort Ord lands. The proposed location for the joint facility is east of Highway 1, adjacent to former Fort Ord Quartermaster warehouse buildings on three to five acres of land now owned by TAMC and Monterey-Salinas Transit (MST) agency. Alternatively, this facility may be constructed on TAMC-owned lands located west of Highway 1 and adjacent to the balloon-spur track. The potential layout and orientations for the joint facility (if located on the TAMC/MST lands) are illustrated on Figure 6-3 of Parsons Conceptual Plans for Track Restoration. One layout option aligns the tracks from north to south, parallel to Highway 1, while the second orientation aligns the tracks from west to east. The site of the Maintenance Facility will be surrounded by a proposed transit oriented development upon full build out.

The maintenance facility and light rail vehicle layover yard are likely to be largely enclosed to minimize visual impacts; however the layover tracks could be constructed with an “open air” layout according to Parsons conceptual plans. The maintenance building would be set back approximately 100 feet or more from the Highway 1 right-of-way. The proposed facility includes the following major features:

- Two to three outdoor storage/layover tracks
- Two indoor service and inspection tracks and shop
- Stores and support shop area
- Office and employee welfare area

- Ancillary support areas
- Yard lead tracks and turnouts
- Parking and internal roadways

As conceptually designed, the shop and service building would occupy a footprint of approximately 340 feet by 90 to 150 feet. The total building height will be approximately 45 feet or less. A pre-engineered steel frame (Butler-type) building is assumed.

Just north of the First Street passenger Station, the railroad spur track connection extending toward the Fort Ord quartermaster warehouses area at Fifth Street will be restored. The ground surface terrain is generally rolling low hills and sloping ground through this entire section.

2.2.5 Recreation Trail Restoration

A recreational trail meanders within the corridor from Canyon Del Rey Boulevard in Seaside to the end of the MBL rail line in Monterey and extends onward to the Fisherman's Wharf area. The existing recreation trail southwest of Canyon Del Rey Boulevard would be used as much as possible in conjunction with and adjacent to the restored rail line. The recreation trail would be reconstructed at various locations where its current location conflicts with the proposed railroad track alignment. The original railroad tracks, which were left in place when the trail was constructed, are generally covered over by the trail and adjacent landscaping. The portion of the right-of-way between Canyon Del Rey Boulevard and Contra Costa Street in Seaside, about 1,400 feet long, is undeveloped and still contains remnants of the original railroad track. Pieces of the original Southern Pacific railroad track are occasionally visible along the corridor from Canyon Del Rey Boulevard to Fisherman's Wharf.

The trail would be generally at the same level as the tracks. At two locations, one just west of the Highway 1 overpass and the other just west of Palo Verde Avenue, the trail would be elevated to avoid excavation into the adjacent sand dunes on the north side of the right-of-way. The trail crosses over the estuary to Roberts Lake (Lake George) on a concrete bridge that is located along the former railroad track alignment. A new pedestrian bridge would be constructed over the estuary south of the railroad, and the existing bridge will be used for the new railroad track.

2.3 ENGINEERING DESIGN STANDARDS

There are several sources of standards and criteria that will be applicable to the restoration of passenger rail service to the Monterey Peninsula. Various design standards that have been identified to date that are applicable to the project include the following:

- American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering. The manual is a compendium of criteria and standards that are generally followed in the railroad industry.
- California Building Code (CBC), 2007 version.
- American Society for Testing of Materials (ASTM) Standards.
- Other project-specific criteria may need to be established by the design team at the time when preliminary engineering begins.

3 COMPILATION AND REVIEW OF AVAILABLE GEOLOGIC AND GEOTECHNICAL DATA SOURCES

The following project-specific reports, plans and documents were compiled during our data review:

- Parsons (2010) Transportation Agency for Monterey County, Conceptual Plans for Track Restoration, in Monterey County, California, on Monterey Branch Line from MP EE-110.4 in Castroville to MP EE-125.85 in Monterey
- Parsons (2010) Transportation Agency for Monterey County, Bridge Strategy Report for Monterey Peninsula Fixed Guideway Corridor Study
- Kleinfelder, Inc. (2003) Phase II Environmental Site Assessment 13-Mile Segment Monterey Branch Line, Monterey County, California
- Shannon & Wilson, Inc. (2001) Preliminary Geotechnical Recommendations for Monterey Branch Line, Castroville to Seaside, Transit Authority of Monterey County, California
- STV Incorporated (2002) Hazardous Materials Assessment Report Monterey Intercity Rail for Transportation Agency for Monterey County, Monterey, California

In addition to the documents, plans and reports produced for the MBL project, previous Kleinfelder Geotechnical Investigation Reports for other projects were compiled, based on site proximity to the alignment and proposed improvements. A complete list of these projects is presented in Section 10 – References.

At Kleinfelder's request, the California Department of Transportation (Caltrans), Division of Maintenance released Log of Test Borings, through the Bridge Inspection Records Information System. A complete list of the Caltrans subsurface data is presented in Section 10 - References.

Published geologic maps, reports and databases were also compiled to facilitate evaluation of the alignment geology. In addition, selected stereoscopic pairs of

historical aerial photographs were also reviewed and incorporated into pertinent sections of this report. A complete list of these maps, reports and air photos is presented in Section 10 - References.

The compiled reports, maps, plans and databases were reviewed, and pertinent information applied to the following geologic hazard assessment, geotechnical evaluation, recommendations for conceptual design, and to the production of the attached plates.

Review of compiled data included the identification of insufficiencies or 'gaps' in the overall coverage of available reports, publications, maps and databases. Our review indicates incomplete coverage of the MBL alignment by various geotechnical reports, and geologic map references at varied scales and extent of detail. The updated project plan and profile sheets provide a limited swath (left and right of centerline) of topographic contours which is insufficient for assessment of potential hazards located outside of the project right of way. The Caltrans Logs of Test Borings and Kleinfelder geotechnical data compiled during the process provide initial insight into the subsurface conditions which can be anticipated along the alignment. However, this data is typically not of sufficient depth or within sufficient proximity to proposed structures or improvements to substantiate their sole utilization. As such, site-specific subsurface investigations will be required to obtain data to provide geotechnical conclusions and recommendations for design and construction of the proposed structures and reconstruction of railroad track sections.

4 GEOLOGIC AND GEOTECHNICAL RECONNAISSANCE

A preliminary Geologic/Geotechnical reconnaissance of the rail alignment was conducted by 4 members of our Geotechnical staff on February 24, 2010. Transport during the reconnaissance was by passenger vehicles along adjoining roadways which allowed observation of the alignment in its entirety. The reconnaissance included viewing of slopes and erosion on, and in the vicinity of, the existing track rail bed and ballast section. In addition, the locations of proposed replacement structures, bridge and passenger station sites, and proposed maintenance facility location were assessed for equipment accessibility to and from the sites during a future subsurface investigation program.

From the MBL rail line's junction with the mainline right-of-way in Castroville, the track generally runs (oriented) in a southwesterly direction to Monterey. The right-of-way is generally 100 feet wide. The track is generally in poor condition, with the exception of a reach about 2-miles long that was reconstructed in the 1960s in the Fort Ord area. For most reaches of the track, the ballast is fouled with fines that have been blown from the adjacent fields or sand dunes. The rails are much worn and pre-date the invention of controlled-cooling, which reduced the propagation of internal rail flaws upon casting. The timber cross ties are also in poor condition.

Alignment locations subject to slope instability and erosion were identified along the MBL alignment during our reconnaissance. The features and processes affecting the structures, ballast section and track rail bed at the locations include, but are not limited to, the following:

- Over-steepened cut slopes adjacent to the rail line
- Scour, undermining of trackway crossing structures
- Distress, failure of ballast section and retaining structures due to proximal erosion, undermining
- Insufficient and/or compromised site drainage
- Standing water on the track bed and adjacent to the ballast section

- Fines contamination, vegetation of the track bed, resulting in insufficient surface drainage, tie decomposition

Alignment segments affected by selected geologic hazards are identified on Plates 2 through 7.

Assessment of site accessibility indicates subsurface investigation of a select number of structures, passenger station sites, trackways, and rail sidings can be achieved with standard truck or track mounted drill rigs or Cone Penetration Test (CPT) equipment. However, the remaining locations have access issues which preclude, complicate or restrict the use of standard equipment. These locations will likely require utilization of a low-rail or equivalent transport vehicle with integral drilling and CPT equipment, to allow mobilization to the sites along the existing rail line.

5 GENERAL GEOLOGIC CONDITIONS

5.1 REGIONAL GEOLOGIC SETTING

The MBL alignment parallels the southern coastline of Monterey Bay, between Castroville and Monterey, California. Monterey Bay lies within the Coast Ranges Geomorphic Province, which is comprised of a discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. Geologic structures within the Coast Ranges Province are generally controlled by the San Andreas fault system, which is a major tectonic transform plate boundary. This right-lateral strike-slip fault system extends from the Gulf of California in Mexico, to Cape Mendocino in northern California and forms a portion of the boundary between two global tectonic plates. In this portion of the Coast Ranges Province, the Pacific plate moves north relative to the North American plate, which is located east of the transform boundary. Deformation along this plate boundary occurs across a wide zone that is referred to as the San Andreas fault system.

The subject rail line lies within the Salinian block, which is one of the major geologic features of the central Coast Ranges. This large wedge of basement rock is composed of Cretaceous Age (about 140 to 65 million years old) granitic and high-grade metamorphic rocks. The Salinian Block is bounded by the San Andreas fault on the east and the Sur-Nacimiento fault zone on the west (Page, 1966). Major physiographic features within the Salinian Block in the Monterey Bay area include the Gabilan Range on the northeast and the Santa Lucia Range on the southwest, which are separated by the Salinas Valley.

Overlying the granitic basement rocks of the Salinian block are Cretaceous and Tertiary (about 65 to 1.6 million years old), marine and continental sedimentary rocks and occasional Tertiary volcanic rocks. These Cretaceous and Tertiary age rocks are typically folded and faulted into a series of generally northwest-southeast trending blocks, largely as a result of stresses related to movement along the San Andreas fault system. The inland valleys, including Salinas Valley, are filled with unconsolidated to semi-consolidated alluvium (stream channel and over-bank deposits) of Quaternary age (1.6 million years old to current).

Geologic formations found along the MBL alignment are all of Quaternary age and vary from recent and older sand dune deposits to fine-grained alluvial deposits associated with the Salinas Valley. Man-made fills are also present along the MBL alignment, most notably associated with Monterey Harbor at the south end and those materials placed during construction of the rail line. The approximate limits of these geologic units along the alignment are depicted on the *Geologic Map*, Plate 2 (Wagner and others, 2002). General lithologic descriptions of these formations are provided in the Alignment Geologic Conditions Section (Section 5.5) of this report.

5.2 SEISMIC SETTING AND HISTORICAL SEISMICITY

The Monterey Bay Region and the San Francisco Bay Area are considered to be two of the most seismically active regions in the United States. Much of the seismicity is associated with the San Andreas fault system. Moderate and large magnitude earthquakes, with moment magnitudes ranging from 6 to 8, have occurred periodically throughout the Monterey and San Francisco Bay regions in historic time. Two of the largest and most destructive earthquakes were the 1868 earthquake, which was centered on the Hayward fault, and the 1906 San Francisco earthquake that occurred on the San Andreas fault. Considerable damage also occurred in Monterey County during the 1989 Loma Prieta earthquake that was centered on the San Andreas fault system in the nearby Santa Cruz Mountains.

Several faults capable of generating moderate to large magnitude earthquakes are located within 100 km of the MBL rail alignment. Some of these faults include the San Andreas, Zayante-Vergeles, Rinconada, Monterey Bay-Tularcitos, and San Gregorio-Palo Colorado faults. Table 1 provides a selected list of these faults and related parameters, which are based on data derived from the USGS/CGS for the State of California (Cao et al., 2003) and by the Working Group on California Earthquake Probabilities (2007) for the greater San Francisco Bay Area. The locations of the faults presented in Table 1 and other active and potentially active faults in the area with respect to the MBL alignment are shown on Plate 3, *Regional Fault Map*, which is from Jennings (1994).

Table 1
Significant Faults

Fault Name	Fault Length (km)	Closest Distance to North End (km)	Closest Distance to Center (km)	Closest Distance to South End (km)	Magnitude of Maximum Earthquake ***
Monterey Bay – Tularcitos*	84	18	9	1.3	7.3
Rinconada**	190	9.5	4.7	14	7.5
Zayante-Vergeles	58	13	23	34	7.0
San Gregorio (SGS + SGN)	176	32	23	12	7.4
San Andreas (SAS + SAP + SAN + SAO)	473	19.6	30	41	7.9
Calaveras (CS + CC + CN)	123	32	40	51	6.9
Monte Vista-Shannon	45	47	56	64	6.7
Quien Sabe	23	38	48	59	6.4

* The Monterey-Bay-Tularcitos fault crosses the alignment west of MP 125

** The Reliz Segment of the Rinconada Fault crosses the alignment near MP 117

*** Moment magnitude: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture). Maximum earthquake magnitude shown may assume multiple rupture scenarios on combined segments of the fault indicated.

In addition to strong ground motion, another seismic related hazard is the potential for faults to rupture the ground surface during an earthquake. Regulations for zoning faults with ground rupture potential in California are provided in the Alquist-Priolo Earthquake Fault Zoning Act of 1972 (Bryant and Hart, 2007). According to the AP EFZ Act, an active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time, approximately the last 11,000 years (Bryant and Hart, 2007). As defined by the California Geological Survey (CGS) in accordance with the AP EFZ act, the MBL alignment is not located within an Earthquake Fault Zone. Recent evaluations of the AP Act provided in Cato (2010) might change the current terminology used to define active faulting in the future.

There are several faults that are considered capable of generating earthquakes in the vicinity of the MBL alignment. Their potential for rupturing the ground surface, however, is not well established. There are five such faults that transect the MBL alignment which fall within this category. These faults include, from north to south, the Reliz-Rinconada, Ord Terrace, Seaside, Chupines, and Monterey Bay-Tularcitos. Regional geologic maps of the area depict these faults as lying buried beneath young geologic deposits and in some cases extending into Monterey Bay. The approximate locations of these faults are depicted on Plates 2 and 3. Following are brief summaries regarding these faults obtained from the U.S. Geological Survey Quaternary Fault and Fold Database (<http://earthquake.usgs.gov/regional/qfaults/>) and other sources.

Reliz Fault – The Reliz fault crosses the MBL alignment between MP 116+50 and MP 117+00. A recent evaluation of this fault by Rosenberg and Clark (2009) delineates this segment of the fault as the Blanco section. Based on the results of several previous studies, they describe the fault as a high-angle reverse fault with about 300 m (985 feet) of vertical displacement at depth. In the area surrounding the MBL alignment, this fault lacks any geomorphic expression and is mapped as concealed (buried) by young and older dune sand deposits. The Reliz fault is often associated with the Rinconada and King City faults.

Ord Terrace, Seaside and Chupines Faults – The Ord Terrace, Seaside, and Chupines faults cross the alignment just south of MP 122+00, MP 122+90, and MP 123+60, respectively. Bryant (2001, Fault No. 145a), groups these faults into the Chupines fault zone, and describes them as a series of right-lateral strike-slip faults with some reverse movement. Within the fault zone, several structural folds within the older geologic formations underlying the young and older dune sand deposits have been identified. The potential for ground rupture along these faults is not well established.

Monterey Bay-Tularcitos Fault – The Monterey Bay-Tularcitos fault crosses the alignment at about MP 124+70. Bryant (2001, Fault No. 62a) describes this fault as a series of varied fault segments, including the Navy fault, which is the segment that crosses the alignment. The northern segment of this fault extends into Monterey Bay as part of the Monterey Bay fault zone while the southern segments, commonly referred to as the Tularcitos fault, extend about 27 miles to the southeast. According to Bryant (2001, Fault No. 62a), some of the land segments of the Tularcitos fault (south of the

MBL alignment) have displayed evidence of Holocene age displacement; however, recency of activity along the Navy fault segment is not well established.

5.3 HISTORICAL SEISMICITY

The project site and its vicinity are located in an area traditionally characterized by moderate to high seismic activity. A number of moderate to large earthquakes have occurred within 100 kilometers of the MBL alignment during historic time (since 1800). Some of the significant regional earthquake events include: the 1926 (M6.1) Monterey Bay earthquake; the 1926 (M6.1) Monterey Bay earthquake; the 1892 (M5.8) Hollister earthquake; the 1890 (M6.3) Pajaro Gap earthquake; the 1864 (M6.5) S. Santa Cruz earthquake; the 1989 (M6.9) Loma Prieta earthquake; and the 1897 (M6.3) Gilroy earthquake. Other significant regional earthquakes include: the 1882 (M5.8) Hollister Peninsula earthquake; the 1911 (M6.5) Calaveras Fault earthquake; and the 1899 (M5.8) Morgan Hill earthquake.

According to the Working Group for California Earthquake Probabilities (2007), there is a 62 percent chance that an earthquake of magnitude 6.7 or greater will strike the San Francisco Bay Area in the next thirty years. As has been demonstrated recently by the 1989 M6.9 Loma Prieta earthquake, the 1994 M6.7 Northridge earthquake, and the 1995 M6.9 Kobe earthquake, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban areas.

5.4 HYDROGEOLOGY AND GROUNDWATER

According to the California Department of Water Resources (CDWR, 2003), the MBL alignment is located in the Salinas Valley groundwater basin. The alignment crosses over two of the subbasins within Salinas Valley basin. These subbasins are known as the 180/400 Foot Aquifer and Seaside subbasins, whose boundary is located south of MP 120+00. The Salinas Basin aquifer is composed of as much as 2,000 feet of alluvial deposits of Holocene and late Pleistocene age that overlie middle to early Pleistocene deposits at depth (Hanson and others, 2002).

Groundwater levels in the coastal portions of the basin are influenced by daily tidal variations, along with surface input, land use and subsurface lithology. Groundwater level data from previous projects adjacent to the alignment suggest that groundwater is

commonly present at rather shallow depths, within the upper 20 feet of the ground surface. Depending upon local subsurface conditions, it is possible that seasonally perched groundwater could be encountered within surface and near-surface deposits throughout the length of the alignment, particularly in early spring or after periods of prolonged rainfall.

5.5 ALIGNMENT GEOLOGIC CONDITIONS

Geologic formations found along the MBL alignment are of Quaternary age and vary from recent and older sand dune deposits to fine-grained alluvial deposits associated with the Salinas Valley. Man-made fills are also present along the alignment, most notably associated with Monterey Harbor at the south end and those materials placed during construction of the rail line. The geologic conditions along the alignment are presented on Plate 2, which is based on regional mapping by Wagner and others (2002). Following are summaries of the geologic units and their relative distribution along the alignment. The units described are limited to those directly underlying the MBL alignment, and are presented from youngest to oldest. Soil unit descriptions provided in the summaries are based on regional maps prepared by the USDA National Cooperative Soil Survey located at <http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm>.

Artificial Fill (Map Symbol: af): Artificial man-made fills are distributed along the MBL alignment and include those deposits associated with construction of the rail line and other deposits associated with other man-made features. Artificial fills associated with rail line construction include the embankment fills typical of areas where the rail bed is elevated above the surrounding area by a few feet to more significant embankments associated with the Salinas River bridge. Fills associated with man-made features include those along the southern end of the alignment in Monterey Harbor and other fills located northwest of MP 124+00 on the southeast shore of Roberts Lake. Composition of the fill materials vary from silt and clay to sand and gravel and in thickness from a few feet to tens of feet.

Flood Plain Deposits (Map Symbol: Qfl): Holocene age flood plain deposits associated with the Salinas River valley extend between MP 111+00 to about MP 113+70. These deposits consist of fine-grained layers of silt and clay with variable amounts of fine sand. Soil units developed from these deposits include the Diablo Clay, Pacheco Clay,

Metz fine sandy loam and complex, Alviso silty clay loam. These soil units, except for the Metz units, typically contain highly expansive clays and variable amounts of organic material.

Dune Sand (Map Symbol: Qd): Holocene age dune sand deposits form a belt of parabolic dunes directly parallel to the Monterey Bay shoreline and within the study area extend from about MP 114 to MP 125. These deposits directly underlie the alignment between approximately MP 121 to MP 122 and portions of the alignment between MP 118 and 119. The dunes are described by Cooper (1967) as the “Flandrian” dune deposits, which are associated with the general rise in global sea level that began about 18,000 to 19,000 years ago. The dune deposits consist of poorly graded sand, and are prone to wind erosion when not stabilized with vegetation.

Older Dune Sand (Map Symbol: Qod): Late Pleistocene age older dune sand deposits underlie the majority of the MBL alignment and extend from about MP 113+70 to MP 122+50. These deposits lie on the leeward side of the active or younger dune sand deposits previously described. These deposits consist of silty sand and poorly graded sand, which are prone to erosion when not stabilized with vegetation. Soil units associated with the older dune sand include the Oceano loamy sand and the Baywood sand. Both of these units are good for use as roadway fills, but poor for use as sand fill due to excess fines content.

Fan Deposits of Antioch (Map Symbol: Qfa): Middle Pleistocene age fan deposits of Antioch (Dupre and Tinsley, 1980) are located at the northern end of the MBL alignment, and extend from its very northern end to just south of MP 111 at Tembladero Slough. These deposits consist of semi-consolidated layers of well to poorly graded sand and silty sand with variable amounts of fines. The Santa Ynez fine sandy loam is the main soil unit associated with this formation, and consists of lean clay.

Marine Terrace Deposits (Map Symbol: Qmt): Late Pleistocene age marine terrace deposits underlie the southern portions of the MBL alignment from about MP 122+50 to the southern end of the rail line. In this area, Dupre (1990) and Clark and others (1997) divide the terraces into the Oceanview and Lighthouse terraces, which formed between 102,000 and 124,000 years ago. These deposits consist of semi-consolidated layers of sand with variable amounts of fines, and are covered locally by thin deposits of eolian

(wind blown) sand of Holocene age. Soil units associated with the terrace deposits include the Baywood sand and the Tangair fine sand. Locally, the Baywood sand includes duripan deposits in the upper 4 to 5 feet. The deposits are layers of soil, strongly cemented by illuvial silica, and can be difficult to excavate.

6 GEOLOGIC AND SEISMIC HAZARDS

The geologic and seismic conditions along the MBL alignment described previously in this report are based on existing available geologic maps and databases, private consultant reports, and preliminary site reconnaissance. On the basis of these sources, the adverse geologic hazards that could potentially affect the alignment are discussed below.

Hazards associated with geologic conditions and earthquakes include ground rupture along faults, strong ground shaking, liquefaction, tsunami run-up, erosion and sedimentation, landslides, compressible ground, land subsidence, expansive soils, and sea level rise. Other less common geologic hazards and impacts include volcanic eruptions and loss of mineral resources. Geologic and seismic hazards and impacts pose potential constraints to development and often require some level of mitigation in order to reduce risks associated with development to an acceptable level. The following sections describe the more common hazards that are present at the site and provide guidance for reducing their potential impacts. Detailed geologic and geotechnical studies will be required in order to provide specific mitigation measures for future designs.

6.1 FAULTING AND GROUND SURFACE RUPTURE

No known active faults, as defined by the Alquist-Priolo Earthquake Fault Zone Act (Bryant and Hart, 2007), transect the alignment. Several faults that are considered capable of generating earthquakes, but whose ground rupture potential is not well established, cross the MBL alignment (Working Group on California Earthquake Probabilities, 2007). These faults include segments of the Reliz-Rinconada, Ord Terrace, Seaside, Chupines, and Monterey-Tularcitos faults. In general, the surface-rupture potential of these faults is poorly constrained and is considered very low to low. The mapped traces of these faults cross the alignment where the track is at grade. If one or more of these faults were to offset the ground surface, damage would most likely be limited to the track, which could be repaired if surface rupture were to occur. In order to limit the damage to track repair and/or replacement, we recommend that stations and passenger platforms be setback at least 50 feet from the currently mapped fault

locations as shown on Plate 2. Additional studies should be performed if stations or platforms are proposed at the location of the fault crossings.

6.2 EARTHQUAKE GROUND MOTIONS

Data presented by the Working Group on California Earthquake Probabilities (2007) suggests the likelihood of a Magnitude 6.7 or greater seismic event to occur within the greater region adjoining the San Francisco Bay in the next 30 years is approximately 63 percent. Within the region, the Hayward-Rodgers Creek fault has the highest probability at 31%, with the San Andreas fault at 21%. As such, the proposed MBL alignment is expected to experience strong seismic ground shaking resulting from future earthquakes on the San Andreas and other active faults in the region. Time, location, and magnitude of earthquakes are not accurately predictable with existing technology. It is, however, generally agreed that the intensity of ground shaking from future earthquakes will depend on several factors including the distance from the site to the earthquake focus, the magnitude and duration of the earthquake, and the response of the underlying soil and bedrock.

6.3 LIQUEFACTION, LATERAL SPREADING, AND LURCHING

Soil liquefaction is a phenomenon in which saturated, cohesionless or granular soils undergo a substantial loss in strength due to excess build-up of pore water pressure during cyclic loading such as that induced by earthquakes. The primary factors affecting the liquefaction potential of soil include: 1) intensity and duration of seismic shaking, 2) soil type and relative density, 3) overburden pressure, and 4) depth to water. Soils most susceptible to liquefaction are generally clean, loose, fine-grained sands that are saturated and uniformly graded. However, silty and clayey sands have also been known to be susceptible to liquefaction. The occurrence of liquefaction is generally limited to saturated soils located within about 50 feet of the ground surface.

Regional studies conducted by Monterey County (2004) indicate that the liquefaction potential of the soils underlying the alignment varies from low to high (see Plate 4). Areas with moderate to high potentials for liquefaction are located in the northern portions of the alignment from about MP 111 to 114 in the Salinas Valley. Liquefaction occurred in the Salinas Valley during the 1906 San Francisco and 1989 Loma Prieta earthquakes (Dupre, 1990; Dupre and Tinsley, 1980; Youd and Hoose, 1978).

Liquefaction and related lateral spreading occurred along the banks of the Salinas River causing damage to the Salinas River Bridge. Other low lying areas along the MBL alignment, not shown on Plate 4, may also be susceptible to liquefaction and require further evaluation. Additional subsurface investigations and engineering analyses will be necessary in order to quantify the amount of liquefaction-induced settlement during future design-level studies. Potential mitigation measures may include in-place densification of liquefiable layers, removal and recompaction methods, and/or structural mitigations.

Lateral spreading and lurching are potential secondary seismic effects commonly associated with liquefaction, where extensional ground cracking and settlement occur as a response to the lateral migration of liquefied material. These phenomena typically occur adjacent to free faces such as steep slopes and creek channels. As the MBL alignment crosses creek channels and the Salinas River, located in zones characterized as susceptible to liquefaction, the potential for lateral spreading and lurching to occur along or in close proximity to the alignment near channel boundaries is considered to be high.

6.4 TSUNAMI, SEICHE, AND FLOODING

Flooding associated with geologic and seismic hazards are generally considered from two sources that include tsunami and seiche, and seismically induced dam failure. The MBL rail alignment parallels the coast of Monterey Bay and is relatively low lying, making it susceptible to flooding hazards. Tsunami can be generated by offshore earthquakes, submarine landslides, volcanic eruptions, and bolide (meteor) impacts. Recent studies by the California Geological Survey (2009) in conjunction with the University of Southern California Tsunami Research Center and the California Emergency Management Agency indicate that portions of the alignment could be inundated by tsunami run-up. The approximate inundation limits are depicted on Plates 5 and 6. Seiches, which are water waves generated in smaller bodies of water like lakes, could affect the alignment during earthquakes as well. The main locations where a seiche could occur are located between about MP 124 and MP 125 in the area of Roberts Lake and Laguna Grande (see Plate 5). Portions of the alignment that are susceptible to flooding due to dam failure are limited to the northern end of the alignment where it lies within the Salinas Valley. Regional studies conducted by

Monterey County (2004) indicate that the northern portions of the alignment, from about MP 111 to 114, could be affected by flooding due to failure of upstream dams.

It is our understanding the flooding due to seasonal precipitation is being evaluated as part of a hydrology and hydraulics study by other design team members.

6.5 LANDSLIDES AND SLOPE INSTABILITY

The majority of the MBL alignment traverses relatively flat terrain, where landslides and slope instability are not a concern. Landslides and slope instability are a concern along the southern portions of the alignment where slopes composed of dune sand are present adjacent to the track and the abutment slopes for the Salinas River bridge. The dune sand slopes extend from about MP 114 to MP 125, with the highest adjacent slopes from about MP 124 to MP 125. The latter slopes have the potential to cover the track if they were to fail. These slopes are considered moderately stable when covered with vegetation, but are prone to failure when lacking vegetation cover and during moderate to large magnitude earthquakes on nearby active faults or during heavy rains. The stability of these slopes should be evaluated using analytical methods during future geotechnical investigations of the project.

Also of concern are the abutment slopes for the Salinas River Bridge. Portions of these slopes are mantled with large boulders (rip-rap). The stability of the abutment slopes should be evaluated using analytical methods during future geotechnical investigations of the project.

6.6 VOLCANIC ERUPTIONS

The MBL alignment is not located in close proximity to an active eruptive center, and therefore the potential for damage or detrimental impact due to volcanic eruption is considered to be low to non-existent.

6.7 EROSION, SEDIMENTATION, AND SCOUR

Erosion of soil deposits along the MBL alignment along with redeposition of the sediments, or sedimentation, has the potential to affect the long-term performance of the rail line. These processes are most likely to occur in areas where the alignment

transects the younger and older dune sand deposits. Regional studies conducted by Monterey County (2004) indicate that the majority of the alignment, from about MP 114 to its southern terminus in Monterey Harbor is located in areas where moderate to high rates of soil erosion are likely to occur (see Plate 7). The highest rates of erosion are likely to occur in the younger dune sand deposits, especially where they are not covered by vegetation. Deposition of sand along the MBL alignment was noted during our reconnaissance mapping and by Shannon & Wilson (2001) at approximately MP 122. Additional field mapping should be performed to refine the limits of sedimentation along the track, specifically from about MP 121 to the southern terminus of the alignment.

Scour is a fluvial process through which an established channel increases its depth of incision through removal and transport of sediment bed load material during periods of moderate to high flow. Unanticipated or excessive scour at a structure location can undermine the structure foundation elements, resulting in distress and possible collapse. Scour is a concern at the Salinas River Bridge area per the report by Shannon & Wilson (2001) and should be assessed during future evaluations of the bridge.

6.8 COMPRESSIBLE SOILS

Compressible soils are typically saturated, fine-grained soils that possess low density and are incapable of supporting significant vertical loads without excessive settlement. Compressible soils tend to coincide with younger, Holocene age deposits that have not had sufficient time to densify (poorly consolidated or unconsolidated soil deposits). Compressible soils underlying the rail line can cause soft track conditions, especially where poor surface drainage conditions exist. Compressible soils are most likely to occur along the northern portion of the MBL alignment, where the track crosses the Salinas River Valley, from about MP 111 to MP 114.

Areas with suspected compressible soils should be investigated by performing site-specific subsurface investigation, laboratory analysis and engineering evaluations.

6.9 EXPANSIVE SOILS

Expansive soils have the capacity to undergo large volume changes with changes in moisture content and typically are associated with moderate to high plasticity clays.

Expansive soil forces can cause damage to rigid structures, such as shallow foundations, slab on grades, and sidewalks. Expansive soils are known to be present in the northern portion of the alignment within the Salinas River Valley, from about MP 110 to MP 114, but could also be present in other reaches of the MBL rail corridor.

Areas with suspected expansive soils should be investigated by performing site-specific subsurface investigation, laboratory analysis and engineering evaluations.

6.10 SEA LEVEL RISE AND COASTAL RECESSION

Regional studies on sea level rise and coastal erosion along the Monterey Bay coastline, in conjunction with global concerns regarding greenhouse gas emissions suggest that the sea level will rise along the coast in the next century. Local studies of coastal regression by Rogers E. Johnson (2004) indicate that the Marina portion of the Monterey Bay coast has been receding at a rate of about 5 feet per year. During heavy storm years, such as the 1982/1983 winter storm season, the amount of coastal bluff recession can be on the order of tens of feet.

Based on these factors, it is possible that the coastline could recede on the order of 250 to 300 feet in the next 50 years. In addition, the anticipated rise in sea level will also increase the liquefaction susceptibility of sediments bordering the coastline, and the susceptibility of low lying coastal areas to flooding. Considering these preliminary estimates, there is a low to moderate potential for portions of the alignment within 300 to 500 feet of the current coastline to be affected by regional rise in sea level and related coastal recession. The segment of the alignment from about MP 125 to its western terminus (within the City of Monterey) appears to be the portion most susceptible to future sea level rise. We recommend that potential mitigation measures for this portion of the alignment be evaluated during future design phases.

7 DESIGN CONSIDERATIONS FOR THE CONCEPTUAL ENGINEERING PHASE

7.1 IMPACT OF IDENTIFIED GEOLOGIC AND SEISMIC HAZARDS

7.1.1 Faulting and Ground Surface Rupture

Five faults cross the central and southern portions of the MBL alignment, however, none of these faults are currently considered active and their potential for surface-rupture is considered very low to low. If surface rupture were to occur during a seismic event it could cause track and ballast section settlement, displacement, and/or separation, resulting in disruption or termination of active service.

7.1.2 Earthquake Ground Motions

Structures that are not engineered or constructed to account for seismic ground shaking are likely to suffer significant distress and/or collapse. As the MBL rail alignment is underlain by potentially liquefiable alluvial deposits in some segments, the impact is compounded, in that 1) shaking is typically amplified in areas of relatively thick unconsolidated sediments, and 2) consolidation and settlement of alluvium can occur at an irregular and accelerated rate, particularly when under vertical load. The impact is not limited to structures, as the track section is equally as susceptible to the differential settlements, potentially causing track irregularity, displacement and/or separation, resulting in disruption or termination of active service.

The following paragraphs provide preliminary recommendations regarding seismic ground motion hazard based on AREMA guidelines (used for railroad trackway structures) and also the California Building Code - CBC (for buildings and similar features) for the project. These preliminary recommendations are only considered suitable for use during the current conceptual engineering design phase, and will need to be reevaluated during the next design phase using site specific data obtained from the upcoming geotechnical investigation program.

7.1.2.1. AREMA Seismic Parameters for Conceptual Design

We evaluated preliminary seismic design parameters for the rail alignment based on the requirements of American Railway Engineering and Maintenance-of-Way Association

design manual (AREMA, 2007). We evaluated the available subsurface soils information using limited data from existing geotechnical reports in the project vicinity and available geologic maps.

According to AREMA (2007), seismic design parameters in terms of base acceleration coefficient (A_R) are estimated for three different ground motion levels. According to Table 9-1-3 of AREMA (2007), three different ground motion levels are associated with three different performance criteria. Ground motion levels 1 (occasional), 2 (rare), and 3 (very rare) correspond to serviceability, ultimate, and survivability performance criteria, respectively. Ranges of average earthquake return periods have been assigned to these ground motion levels. According to Table 9-1-4 of AREMA (2007), return periods between 50 and 100 years, between 200 and 500 years, and between 1,000 and 2,400 years are assigned to ground motion levels 1, 2, and 3, respectively. Appropriate return period for each level depends on and should be calculated using many risk factors provided in the manual. In absence of actual return periods for each performance level, seismic design parameters are estimated for return periods of 100, 475, and 2,400 years. We understand that actual return periods corresponding to each performance level have not been established by the design team at this time. Therefore, seismic design parameters for assumed return periods of 100, 475, and 2,400 years are provided at this time for purposes of conceptual design.

Preliminary seismic ground motion design parameters in terms of base accelerations and site coefficients have been developed as per AREMA (2007), Part 1, Sections 1.3.2.2.4, 1.3.2.3, and 1.4.4.1 at four selected locations along the alignment. The approximate coordinates for the four locations, named as “North End”, “Salinas River”, “Center”, and “South End” of the MBL alignment are presented in Table 2 as follows:

Table 2
Location Coordinates for AREMA Seismic Parameters

Location	Latitude	Longitude
North End	36.7605N	121.7447W
Salinas River	36.7294N	121.7760W
Center	36.6762N	121.8095W
South End	36.6025N	121.8930W

As described above, for purposes of our preliminary analysis return periods were assumed/selected as 100, 475 and 2,400 years. Based on these assumed return periods, the base accelerations coefficients were evaluated using 2008 USGS data and using USGS java calculator (available at the website <http://earthquake.usgs.gov/research/hazmaps/design/>) instead of Figures 9-1-1 through 9-1-3 of the AREMA (2007) manual. Using the java calculator is considered to be more appropriate as it provides better estimate of base accelerations for a specific site compared to the maps provided in AREMA (2007). Using the existing exploration data, geologic maps, and experience and engineering judgment we assigned the following preliminary Soil Types and Site Coefficients per AREMA (2007), Table 9-1-6 for each of the four selected locations. Tables 3.A through 3.D summarize preliminary return periods, base accelerations, soil type and site coefficients for the four selected locations along the MBL alignment.

Table 3.A – Seismic Design Parameters – North End

Ground Motion Level	Frequency	Average Return Period (yr.)	Base Acceleration Coefficient, A_R (g)	Soil Type	Site Coefficient
1	Occasional	100	0.19	3	1.5
2	Rare	475	0.37		
3	Very rare	2,400	0.59		

Table 3.B – Seismic Design Parameters – Salinas River

Ground Motion Level	Frequency	Average Return Period (yr.)	Base Acceleration Coefficient, A_R (g)	Soil Type	Site Coefficient
1	Occasional	100	0.17	3	1.5
2	Rare	475	0.33		
3	Very rare	2,400	0.55		

Table 3.C – Seismic Design Parameters – Center

Ground Motion Level	Frequency	Average Return Period (yr.)	Base Acceleration Coefficient, A_R (g)	Soil Type	Site Coefficient
1	Occasional	100	0.16	2	1.2
2	Rare	475	0.31		
3	Very rare	2,400	0.54		

Table 3.D – Seismic Design Parameters – South End

Ground Motion Level	Frequency	Average Return Period (yr.)	Base Acceleration Coefficient, A_R (g)	Soil Type	Site Coefficient
1	Occasional	100	0.14	3	1.5
2	Rare	475	0.32		
3	Very rare	2,400	0.60		

References

AREMA (2007), American Railway Engineering and Maintenance-of-Way Association, Part 1, Seismic Design for Railway Structures, 2007

7.1.2.2. 2007 CBC Seismic Parameters for Conceptual Design

For ‘building’ structures that will be designed according to the CBC, we estimated the 2007 CBC seismic parameters for three selected locations along the MBL alignment, with assigned locations named as follows – “North”, “Center”, and “South”. The approximate coordinates for the three locations along the MBL alignment are presented in Table 4 as follows:

Table 4
Location Coordinates for CBC Seismic Parameters

Location	Latitude	Longitude
North End	36.760462	-121.744736
Center	36.676228	-121.809484
South End	36.602495	-121.892992

The Maximum Considered Earthquake (MCE) mapped spectral accelerations for the 0.2 second and 1.0 second periods (S_S and S_1) were estimated using Section 1613.5 of 2007 CBC. The mapped acceleration values and associated soil amplification factors (F_a and F_v) based on 2007 CBC are presented in Table 5 below. Corresponding design spectral accelerations (SD_S and SD_1) are also presented in Table 5¹.

¹ Based on the mapped acceleration values using the USGS’s Java Ground Motion Parameter Calculator available at <http://earthquake.usgs.gov/research/hazmaps/design/>

Table 5
Ground Motion Parameters Based On 2007 CBC

CBC Seismic Parameters for Engineering Design	North End	Center	South End	2007 CBC Reference
S_s	1.416	1.287	1.444	Section 1613.5.1
S_1	0.6	0.564	0.602	Section 1613.5.1
Site Class	D	D	D	Table 1613.5.2
F_a	1.0	1.0	1.0	Table 1613.5.3(1)
F_v	1.5	1.5	1.5	Table 1613.5.3(2)
SM_s	1.416	1.287	1.444	Section 1613.5.3
SM_1	0.9	0.846	0.903	Section 1613.5.3
SD_s	0.944	0.858	0.962	Section 1613.5.4
SD_1	0.6	0.564	0.602	Section 1613.5.4
PGA	0.38	0.34	0.39	Section 1802.7

According to Section 1802.7 of 2007 CBC, Peak Ground Acceleration (PGA) can be taken as $SD_s/2.5$, where SD_s is determined using Section 1613.5.4.

These preliminary seismic ground motion recommendations are only suitable for use during the current conceptual engineering phase, and will require reanalysis during the next design phase using site specific data obtained from the upcoming geotechnical investigation program.

7.1.3 Liquefaction, Lateral Spreading, and Lurching

Liquefiable eolian deposits and alluvium will be subject to differential settlement due to the previously discussed ground effects during a seismic event, particularly under the load of the track bed and structures. A liquefaction susceptibility map is presented on Plate no. 4. The effects of the seismically induced differential settlement will be track settlement, structure distress and/or collapse. The effects are likely to be immediate, potentially resulting in disruption or termination of active service and could also pose a threat to rail traffic safety.

The potential impacts due to lateral spreading and lurching include structure distress and collapse as well as track disruption due to lateral movement, but will be limited to areas proximal to ponds, lakes, channels, and free faces.

7.1.4 Tsunami, Seiche, and Flooding

Recent studies by the California Geological Survey (2009) in conjunction with the University of Southern California Tsunami Research Center and the California Emergency Management Agency indicate that portions of the rail alignment could be affected by a tsunami. The approximate limits of the areas of potential impact are depicted on Plates 5 and 6. The potential impacts due to tsunami and/or seiche inundation include flooding and displacement of track, resulting in temporary disruption or termination of active service.

7.1.5 Landslides and Slope Instability

Shallow failure of drainage channel banks, ballast sections, cut slopes and natural slopes in close proximity to the track bed could result in debris inundation of the rail line, or loss of rail line confinement and support. Both impact scenarios would result in temporary disruption or termination of active service and could also pose a threat to rail and adjacent traffic safety.

The potential for landslides and slope instability to affect the alignment are limited to the area between MP 124 and MP 125, and at the abutment slopes for the Salinas River Bridge. There is a low to moderate potential that the dune sand slopes between about MP 124 and MP 125 could fail under static or seismic (pseudo-static) conditions and cover the rail alignment. Such an occurrence would bury the track and could result in disruption or termination of active service. In addition, there is a moderate to high potential that the abutment slopes for both approaches of the Salinas River Bridge could become unstable during an earthquake. Failure of the abutments could cause damage to the bridge.

7.1.6 Volcanic Eruptions

No impact on the proposed project due to volcanic eruptions is anticipated.

7.1.7 Erosion, Sedimentation, and Scour

Impacts due to scour and/or erosion include undermining of structures, track bed and ballast sections, resulting in structure distress and collapse, as well as track settlement and disruption. Any of these factors could cause temporary termination of service at a minimum.

Regional studies conducted by Monterey County (2004) indicate that the majority of the alignment, from about MP 114 to its southern terminus in Monterey Harbor is located in areas where moderate to high rates of soil erosion are likely to occur (see Plate 7). Erosion is most likely to occur where younger and older dune sand deposits are exposed and not covered by vegetation. Erosion of these deposits adjacent to the track can create unstable embankment conditions. Redeposition of the sand deposits, such as that occurring along the track at about MP 122, could result in temporary disruption or termination of active service.

7.1.8 Compressible Soils

When loaded by fill placement and/or structural loads, compressible soil undergoes settlement, due to consolidation of the soil, and may potentially experience both vertical and lateral displacement due to plastic deformation. Settlement can cause cracking in walls and floor slabs of structures as well as misalignment of the rail and roadbed. Rarely is this settlement uniform, due to the variability in thickness and composition of the deposit.

As such, improvements constructed on un-remediated compressible soil can be subject to differential settlement which can approach several inches or more, occurring over 100 years, depending upon the thickness of the compressible soil deposit and the vertical load placed upon it. In the case of the rail alignment, this would result in differential settlement of the track bed and structures, over time.

7.1.9 Expansive Soils

If not properly mitigated, the cyclic volume changes capable of expansive soils (i.e. shrink-swell) can cause distress and failure of structures, platforms, asphaltic and concrete pavements, slabs-on-grade, and other surfaces. In areas where the track bed

is supported by a minimum of 2-3 feet of non-expansive ballast, the effects of expansive soils are expected to be marginal. Areas where the track bed is underlain by a marginal ballast section, however should anticipate localized movement (and thus track disruption) if expansive soils are present. Lightly loaded structures on founded on shallow spread footing foundations may be subject to comparable effects.

7.1.10 Sea Level Rise and Coastal Recession

Future rise in sea level and recession of the coastline could affect the low lying portions of the alignment located within 300 to 500 feet of the existing coastline over the next 50 years. Rise in sea level could ultimately inundate the alignment requiring raising or relocation of the rail bed.

7.2 POTENTIAL MITIGATION ALTERNATIVES FOR GEOLOGIC AND SEISMIC HAZARDS

The identified geologic and seismic hazards can be evaluated through site-specific investigation, laboratory testing and engineering analysis. Given the limited right of way available to the project, avoidance of some or all of the hazards identified would generally prove logistically prohibitive. As such, mitigation of the hazards can generally be achieved through risk assessment, site grading and structure foundation design. The following section discusses possible mitigation alternatives for the geologic and seismic hazards identified.

7.2.1 Faulting and Ground Surface Rupture

As the potential displacement associated with active fault creep and surface rupture can not be avoided, the likely mitigation alternative becomes facilitation of track maintenance. This could be achieved through widening of the track railbed and ballast section in the vicinity of the fault traces, to provide sufficient width for track repair with minimal regrading, should rupture occur.

7.2.2 Earthquake Ground Motions

Seismic design criteria will be implemented for engineering design of light rail infrastructure facilities as required by codes and standards described above. Further

information and preliminary ground motion criteria for use in seismic design for the conceptual engineering phase is provided in Section 7.1.2 of this report.

7.2.3 Liquefaction, Lateral Spreading and Lurching

Standard mitigative solutions for the detrimental effects of liquefaction, cyclic softening, lateral spreading and lurching on surface improvements and structures include, but are not limited to the following:

- Ground improvement, including techniques such as pressure grouting, jet grouting, etc.
- Deep soil mixing of liquefiable soils with cement or lime for their full depth.
- Construction of proposed structures on mat type foundations, or deep foundation elements (such as driven piling).

7.2.4 Tsunami, Seiche, and Flooding

For geologic or seismic-related hazards from these extreme but rare events - tsunami, seiche or flooding from upstream dam failures - advance mitigative solutions are not considered to be practical for the potentially affected light rail infrastructure facilities. Since the potential impacts include temporary flooding/inundation of light rail track, stations etc. and possible displacement and damage of infrastructure features resulting in temporary disruption or termination of active service, then mitigation is expected to consist of repair, reconstruction or replacement after the event occurs.

7.2.5 Landslides and Slope Instability

The stability of the dune sand slopes adjacent to portions of the rail alignment should be verified by slope stability analysis for both static and seismic conditions as part of a future design-level geotechnical investigation. The stability of cut and fill slopes (existing and proposed) and potential mitigation measures include establishment and maintenance of vegetation, regrading of the slopes utilizing industry accepted grading techniques, and construction of retaining structures. Mitigation of the Salinas River Bridge abutment slopes, if deemed unstable, should be based on a design-level geotechnical investigation including slope stability analysis. The impacts of liquefaction

and lateral spreading should be evaluated as part of the analyses for embankment slopes.

7.2.6 Erosion, Sedimentation and Scour

Potential mitigation measures for addressing erosion and sedimentation include establishment of vegetation in areas where dune sands lack vegetation, installation of silt fences, construction of graded berms, installation of drainage facilities, and regular track maintenance.

The impacts of scour at the Salinas River bridge can be mitigated through applying anticipated scour depths and flow patterns to engineering analyses and design of the structures involved. The design team should consider 100-year scour criteria at bridge piers and scour protection should be provided at bridge abutments. Hydraulic studies to determine required bridge openings should be performed during evaluation of the existing structure.

7.2.7 Compressible Soils

The impact of compressible soil deposits on structures or the track section can be mitigated using alternatives comparable to those described for liquefiable soils. Alternatively, advance mitigation of post-construction settlement could be performed by either surcharging programs (with waiting periods that may last several weeks to months) or surcharging in combination with pre-fabricated vertical (wick) drains prior to construction of the fills and rail track roadbeds, or by using lightweight fill material such as EPS Geofoam, lightweight aggregates (volcanic pumice or tuff), or light weight Tire-Derived Aggregate (TDA) to raise site grades. Some areas affected by compressible soils could only become apparent after service of the rail line is reestablished and can be identified by the work crews doing maintenance-of-way activities.

7.2.8 Expansive Soil

Standard mitigative solutions for the detrimental effects of expansive soils acting on surface improvements and structures include, but are not limited to the following:

- chemical (lime) treatment of the expansive soil within the zone (depth) of influence to reduce its plasticity and expansion potential to an acceptable level.
- removal of the expansive soil within the zone of influence, and replacement with non-expansive import soil.
- construction of subsurface moisture barriers along the perimeter of improvements, which extend to depths below the zone of influence.
- construction of foundation elements which inherently resist the differential uplift pressures of expansive soils, such as post-tensioned slabs, cast in place pier foundations, and driven pile foundations.

7.2.9 Sea Level Rise and Coastal Recession

Potential measures to mitigate the impact of future rise in sea level and recession of the coastline include elevating the railbed in the areas most likely to be affected, or relocation of the alignment to higher ground. The necessity of the former option should be based on a quantitative assessment of future sea level rise and coastal recession in the areas along the alignment most likely to be affected. The latter option is most likely not feasible due to the limited nature of project boundaries.

8 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS FOR CONCEPTUAL DESIGN PHASE

Preliminary evaluation of the geotechnical aspects of the project corridor conditions, and geotechnical recommendations for conceptual design are presented below.

8.1 EMBANKMENTS AND TRACK SECTIONS

Low and eroded embankments; shallow, unmaintained, and overgrown ditches; poor surface drainage; and isolated areas where water ponds adjacent to the rail embankment were observed along the MBL alignment. An exception to this is the approximate 2-mile long reach of track (restored/resurfaced in the 1960s) through the former Fort Ord military reservation which is generally in good condition. Plugged or damaged culverts were also observed during our reconnaissance visit. These conditions may cause infiltration of water into the rail embankment, which could wet and weaken subgrade soils and foul ballast, and contribute to poor embankment and track performance, and "soft track." Historic soft track locations could not be identified during our site reconnaissance because the track is out of service and there are no records for the historic maintenance-of-way activities or trackway repairs.

Embankments with relatively steep side slopes, poorly constructed embankments (e.g., poor compaction), and embankments constructed with or over fine-grained and poorly-drained soils are the most likely locations to experience instability. All of these conditions were observed or are suspected to occur along the alignment; however, evidence of past instability at these locations was not noted. Steep slopes that do occur may result from overly steep initial construction, widening of embankment shoulders, undercutting of the bottom of embankment slopes, and other causes.

In general, embankment side slopes should be 2 horizontal to 1 vertical (2H:1V) or flatter. Steeper slopes may be acceptable for embankments consisting of and underlain by well-drained granular soils. Flatter slopes may be required for embankments constructed of or founded on other soils. In general, embankment slopes should be uniform from the shoulder to the toe of the slope and constructed at an inclination appropriate for the embankment and foundation soils. Where the shoulder is of insufficient width to retain ballast, as occurs at some locations along the track, we

recommend that the existing embankment should be widened to provide adequate shoulders to retain ballast.

We recommend that embankments that are undercut be repaired and the toe areas protected from erosion. Overbuilt embankment shoulders should be graded and trimmed until they are approximately 12 feet from track centerline. Excavated material should not be deposited high on embankment slopes, it should instead be placed near the toe area of slopes such that the slope is flattened, or disposed of off site. Where this is not possible or practical, site-specific geotechnical recommendations should be developed. Stability of embankment locations that have experienced past instability should be improved by flattening or buttressing slopes, improving surface and subsurface drainage (as appropriate), and draining water from ballast pockets that may be exposed during reconstruction.

Additional site settlement estimates should be evaluated on the proposed embankment and rail track roadbed sections when the project design is more complete, and updated grading plans have been developed. Maintenance and releveling of the rail track roadbeds should be anticipated if these settlements are beyond the tolerance of the proposed rail system. As discussed above, mitigation alternatives for post-construction settlement could include surcharging or surcharging in combination with pre-fabricated vertical (wick) drains prior to construction of the fills and rail track roadbeds, or by using lightweight fill material such as EPS Geofoam, lightweight aggregates (volcanic pumice or tuff), or light weight Tire-Derived Aggregate (TDA) to raise site grades in settlement prone areas.

Much of the track ballast that we observed appeared fouled, of poor quality and low durability, consists of sub-rounded particles, and is likely of inadequate thickness. This ballast condition is unacceptable for the required safe, low maintenance needs of a passenger rail transit operation. Ballast breakdown, pushing of ballast into underlying soils, pumping of soils upward into the ballast, windblown soil, transport of soil in runoff water that flows into the track section, and other factors likely contributed to ballast fouling. Some of the ballast also appeared to be of poor quality rock that, in our opinion, would likely not meet American Railway Engineers and Maintenance-of-Way Association (AREMA) standards.

Historically, the previous corridor owners used sub-standard ballast rock that resulted in providing lower level of lateral resistance for the track structure to stay in alignment. As this is the case on most reaches of the MBL line, most of the existing ballast rock section is somewhat “cemented” with the mixture of fines, small aggregate, and the ballast rock. The existing ballast section does not perform its intended function for resistance to track movement, drainage of the track structure, or holding of alignment and cross level; in addition, the ballast is now fouled with organic material from vegetation.

The presence of subballast material was not confirmed, and, based on our experience, it is probable that no subballast is present beneath the ballast for all or the majority of the rail line for the MBL segment.

8.1.1 Roadbed Subgrade

A stable roadbed is critical to provide the foundation upon which ballast, rails, and ties of the railroad are laid and for support of the track structure with limited deflections. The design and construction of new roadbed, and reconstruction or maintenance of existing roadbed shall conform to guidelines given in AREMA guidelines, Chapter 1-Part 1, Roadbed. The minimum data needed for to evaluate the subgrade soils should be classification and strength as specified in AREMA Chapter 1, Section 2.11.2.1, Subgrade Soils. The current ASTM test designations given in AREMA Section 2.11.2.1 shall be used. We recommend that subgrade soils should be evaluated to a depth of at least 5 feet below the existing ballast sections. The level of stress in the subgrade should not exceed an allowable bearing pressure that includes a safety factor. A minimum safety factor of at least 2 and as much as 5 or more should be provided to prevent bearing capacity failure or undue creep under the loaded area. When subgrade support is marginal and/or where the liquid limit of the subgrade soil exceeds a value of 30 or the plasticity index exceeds 12, special attention should be given to that soil.

We recommend that the embankment subgrade conditions be evaluated during the upcoming preliminary engineering design phase through completion of geotechnical test pits along the project alignment. We recommend that an allowance for a minimum of one test pit per one-quarter mile of track be included in the design phase budget. The

actual number of test pits required may be greater or less and will depend on subsurface conditions and observations made during future site visits by design team members.

Replacement of subgrade soil or stabilization of the subgrade material may be considered to obtain a more reliable support for the ballast/sub-ballast section. Placing geotextile fabric, geogrid reinforcement, or lime treatment of soils, cement slurries injected at relatively shallow depths and close spacing, or increasing the thickness of the ballast/sub-ballast section are some of the methods to improve roadbed stability. This may be done in new construction or in combination with undercutting, sledding, or other track raise techniques that avoids the total removal or shifting of the track. Geotextiles and geogrids used in this manner must possess the strength and other material properties necessary to act as reinforcements capable of bridging over the unstable area or soft spot. The geotextile and/or geogrid should be placed at least 8 inches and preferably 12 inches beneath the bottom of the tie, at least deep enough to avoid damage by track surfacing equipment. Geotextile and/or geogrid should extend the entire interface zone between the ballast and subballast. Geotextile and/or geogrid should be placed under all special trackwork (on the mainline and in yard areas) and tracks with potential subgrade stability issues. The application and physical requirements for geosynthetics shall meet the requirements given in AREMA Chapter 1, Part 10, Geosynthetics. Maintenance of existing roadbed shall conform to the guidelines given in AREMA, Chapter 1, Section 1.4, Maintenance.

The subgrade to 12 feet on both sides of track centerline should be compacted to a minimum density of 95 percent of the maximum density determined in accordance with ASTM D1557. The subgrade shall be in a moist condition, at (or up to 2 percent above) the optimum moisture content as determined by ASTM D1557. If laboratory results indicate that existing subgrade material is unsuitable, then the material must be treated or removed and replaced with clean, sound and properly compacted engineered fill per our recommendations.

The compacted subgrade shall be sloped at 40:1 downward and away from the center point located midway between two tracks in double track reaches. In single track areas, the compacted subgrade shall slope toward the underdrain (if provided) at 40:1. The conceptual plans for track restoration prepared by Parsons (Figures 8-8, 8-9 and 8-10) show illustrations of typical subgrade configurations. Configurations other than these

may be adopted if drainage requirements or specific locations dictate a special constraint or treatment.

8.1.2 Sub-Ballast

The presence of subballast material was not confirmed during our site reconnaissance, and based on past experience with similar projects it is probable that no subballast is present beneath the ballast for all or the majority of the rail line for the MBL segment.

The purpose of sub-ballast is to form a transition zone between the ballast and subgrade to avoid migration of subgrade soil into the ballast, and to reduce the stress applied to the subgrade. This material plays an integral role in the overall track section, and the quality of the subballast has a direct relationship to the overall performance of the track structure. This layer also acts as a drainage median for the track bed. Based on the range of subgrade soil types anticipated, it is recommended that a sub-ballast section be used and consist of at least 8 inches of compacted well-graded crushed rock on top of the subgrade, which is consistent with design recommendations presented in AREMA Chapter 1, Section 2.11, Sub-Ballast Specifications. Subballast should extend the full width of the subgrade, and extend laterally a minimum 24 inches past the toe of the ballast.

8.1.3 Ballast

Ballast is the selected crushed and graded aggregate material which is placed upon the railroad roadbed. The principal purpose of the ballast section is to anchor the track and provide resistance against lateral, longitudinal and vertical movement of ties and rail, i.e., stability. Additionally, the ballast section bears and distributes the applied load with diminished unit pressure to the subgrade beneath, gives immediate drainage to the track, facilitates maintenance, and provides a necessary degree of elasticity and resilience.

Ideal qualities in ballast materials are hardness and toughness, durability or resistance to abrasion and weathering, freedom from deleterious particles (dirt), workability, compactability, cleanability, and availability. Shape of the ballast particle, degree of sharpness, angularity, and surface texture or roughness are important ballast properties. These factors have been shown to have a significant effect upon the

stability and compactability of aggregates in general. Ballast material properties and placement shall conform to requirements in AREMA Chapter 1- Part 2, Ballast or in AREMA Section 10.3, Ballast. Crushed slag ballast should not be permitted.

The gradation of a ballast material is a prime consideration for the in track performance of ballast materials. The gradation must provide the means to develop the compaction or density requirements for the ballast section and provide necessary void space to allow proper run off of ground water. The ballast materials used in track construction shall meet the requirements specified in the AREMA Chapter 1, Section 2.4.4, Gradations, and Section 2.10.4, Ballast Gradations.

The ballast sizes recommended in the AREMA Manual for Railway Engineering are time-proven and acceptable. However, a number of AASHTO and ASTM gradations are similar to AREMA's and may be acceptable for use in some situations. This may be more cost-effective in locales where AREMA gradations are not readily available but highway rock gradations are available. Various acceptable gradations listed in the Gradation Chart in AREMA Practical Guide to Railway Engineering, Chapter 3 could be used.

The depth of ballast section required is a function of the supporting capacity of the subgrade. It should be sufficient to distribute the pressures to within the bearing capacity of the subgrade. Uniform distribution of pressures is another factor that varies with depth. Usually, a minimum depth of 18 inches is necessary to achieve uniform distribution. This depth may be distributed between ballast and sub-ballast. The greater the height of ballast around the tie, the greater is the resistance to vertical displacement. The same holds true for shoulder and lateral displacement.

According to Parsons conceptual plans for track restoration, ballast type No. 4 (1-1/2 inches to 3/4 inches) and/or No. 3 (2 inches to 1 inch) ballast conforming to AREMA specifications shall be used on all mainline tracks except for those in streets and yards, where No. 5 (1 inch to 3/8 inch) ballast shall be used. A minimum depth of 8 to 9 inches of ballast shall be used between the bottom of ties and top of the subballast. The ballast shoulder shall extend 18 inches beyond the ends of the ties parallel to the plane formed by the top of the rails. Ballast shoulder shall then slope downward to the subballast at a 2:1 slope. The final top of ballast elevation shall be one inch below the

top of tie, when compacted. (Refer to Figures 8-8, 8-9 and 8-10 of the conceptual plans for track restoration by Parsons.) Ballast shall be placed in-between track, around platforms and other areas where the tracks are splayed out. All ballast is to be thoroughly washed and or re-screened (0.5 percent maximum passing #200 sieve) as necessary to remove fine particles prior to placement.

8.1.4 General Engineered Fill

Structural fill may be partially composed of on-site excavated materials that meet the requirements in this section and as approved by the Geotechnical Engineer. If imported fill is required, on-site excavated materials should be used at the lowest lifts of the backfill, and imported fill should be used near the finished subgrade. Imported fill should meet the following requirements for Structural fill:

- the material should be a soil or soil-rock mixture free of organic matter or other deleterious substances,
- it should not contain rocks or lumps over 6 inches in greatest dimension and not more than 15 percent by weight larger than 2-½ inches,
- it should not contain more than 40 percent by weight passing the No. 200 sieve,
- it should have a maximum plasticity index of 15,
- any materials used to backfill behind retaining walls should be granular, free-draining sand or combinations of sand and gravel.

Fill should be spread in lifts not to exceed a maximum uncompacted thickness of 8 inches, moisture conditioned, and compacted using appropriate compaction equipment. Fill should be compacted to a minimum of 95 percent relative compaction in all areas, except within five feet behind retaining walls where a minimum of 90 percent relative compaction is recommended. Compaction acceptance shall be based on test method ASTM D1557.

8.1.5 Material Borrow Sources

Potential material borrow sources for select engineered fill, sand, aggregate base, ballast, sub-ballast will be evaluated during subsequent design phases. Local quarry and borrow sources could include the following:

- Granite Rock Company – A. R. Wilson Quarry – P.O. Box 50001, Watsonville, CA (has provided railroad ballast since the late 1800s)
- Syar Industries, Inc. – Stonewall Canyon Quarry – P.O. Box 867 Stonewall Canyon Quarry, Soledad, CA
- Granite Construction Company – Handley Ranch Quarry- 25485 Iverson Road, Gonzales, CA, and several other locations in site vicinity, Gilroy, Pebble Beach, Salinas, Chualar, CA
- Assured Aggregates - 520-A Crazy Horse Canyon Rd, Salinas, CA – Sand and Gravel
- RMC Cemex - Lapis Industrial Sand – HWY 1, 2 miles north of Marina, Ca - Sand

8.2 TRACKWAY BRIDGES AND STRUCTURES

All structures that are subject to railroad loading will need to be evaluated per the requirements of the American Railway Engineering and Maintenance of Way Association (AREMA), Manual of Railway Engineering (MRE) as well as any project specific requirements. Foundation design, materials and construction shall conform to AREMA Guidelines, Chapter 8, Concrete Structures and Foundations.

There are 6 existing bridges/trestles within the MBL rail segment corridor including the ‘major’ bridge over the Salinas River (with spans of over 140 feet in length) and some minor culvert structures. Most of the bridges are either timber open deck trestles or timber ballast deck trestles constructed around 100 years ago. Recommendations by Parsons for repairs or replacements of these structures were based on a visual inspection performed several years ago, which revealed that the bridges were generally in poor to fair condition. The bridge inspection reports and recommendations for

rehabilitation or replacement are presented in the Bridge Strategy Report prepared by Parsons, dated July 2005 and updated in May 2010. Original construction of most of the bridges appears to be based on standard drawings common to the railroads, with timber piles supporting the abutments and pier bents.

It was recommended in the strategy report that bridge/trestle structures be replaced with ballast-deck prestressed, prefabricated concrete trestle spans supported on driven piles. This pile type is generally cost-effective, practical to install, appropriate for the range of anticipated site conditions, and has been used successfully on similar structures to achieve relatively high axial compression capacities. Where feasible, it was recommended that smaller bridge structures be replaced with box culverts and embankment fills, which will also reduce maintenance.

Tembladero Slough Bridge (MP 111.05)

This bridge is a 150-foot, 10-span timber trestle and is the northernmost structure along the MBL alignment. Based on our site reconnaissance, major cracks and deformations in the bridge abutments were observed. The bridge is located in a medium liquefaction potential area (Plate 5) which can experience further ground deformation and movement in the event of a severe earthquake. We understand that the existing structure will be replaced with a new bridge as funding permits.

Bridges at MP 112.54, MP 112.80, and MP 113.04

These timber trestle bridges range from 90 to 225 feet in length, and appear to be supported on timber piles and bents. Foundations for all the bridges seem to have experienced various levels of ground displacement occurrences including liquefaction, lateral spreading, and consolidation settlement. These bridges are located in a medium liquefaction potential area (Plate 5) which can experience further ground deformation and movement (liquefaction and lateral spreading) in the event of a severe earthquake. We understand that the existing structures will be replaced with new box or round culverts.

Salinas River Bridge (MP 113.46)

The through-truss, steel bridge crossing the Salinas River is a 715-foot long structure which has experienced damage from liquefaction and lateral spreading during the 1906 San Francisco earthquake and 1989 Loma Prieta earthquake. The 5-span bridge is located in an area of high liquefaction potential, and pier numbers 1 and 2 (numbered from north) experienced relative displacement during the 1989 Loma Prieta earthquake which created a permanent offset between the two piers. Based on Parsons Bridge Strategy Report, if the TAMC agency agrees to replace or rehabilitate the bridge then foundations for the bridge would either be replaced or seismically retrofitted (similar to the nearby Highway 1 bridges that have been seismically upgraded in recent years).

The nearby bridges immediately to the west of the railroad bridge over the river were upgraded by driving 4- to 6-foot diameter steel pipe piles (filled with concrete) to depths of more than 100 feet. The most recent exploratory soil borings performed by Shannon & Wilson at the north end of the rail bridge conformed well to the Caltrans borings at the nearby bridges. Additional soil investigations will be necessary at the south end of the rail bridge, and may also be necessary in the mid-river area, to characterize the soil conditions further. Shannon & Wilson borings in 2001 encountered possible gravel, cobbles, and large rocks at depths between 15 to 43 feet with poor sample recovery. These particles could cause obstructions/obstacles to pile driving using similar large diameter steel pipes as were used for the nearby bridges. Further subsurface investigations with exploratory borings would provide a better understanding of the extent of these zones.

8.2.1 Recommended Foundations for Bridge Structures

Based on the anticipated range of bridge load demands and variety of subsurface conditions (including liquefaction and lateral spreading potential) a deep foundation system is recommended for the rail bridges for conceptual design purposes.

Driven steel pipe piles can be considered to support the proposed structures, where foundations may be exposed to tidal and brackish water (with moderate to high corrosion potential). For preliminary design considerations, we anticipate that pipe piles should have minimum wall thickness of 0.5 inches, and will be driven closed-ended (with closure plates welded to the pile tips). Pile lengths of approximately 100 to 130

feet for Salinas River Bridge, and 70 to 100 feet (below the existing ground surface, or mud line in the creek and channel crossings) for other bridges are estimated to achieve the range of geotechnical/soil capacities that will probably be required. Piles will achieve their capacity in a combination of both in skin friction and end bearing.

Driven steel H-piles can be considered to support bridge structures where foundations will not be exposed to tidal and brackish water, but may be exposed to mildly corrosive soil conditions. Steel H-pile sections such as HP14x89 and HP14x117 can be considered, where the larger H-pile section may be required to provide sacrificial steel thickness in mildly corrosive soil conditions. H-pile lengths of approximately 70 to 110 feet (below the existing ground surface, or mud line in the creek and channel crossings) for bridges are estimated to achieve the range of geotechnical/soil capacities that will probably be required. Piles will achieve their capacity in a combination of both in skin friction and end bearing in dense sand and/or stiff clay alluvial soils.

The lateral resistance of a pile is a function of the surrounding soil strength and stiffness, size and stiffness of the pile, pile top connection, and induced moments and forces at the top of the pile. Resistance to lateral loads on piles will be provided by passive soil pressure against the pile and by the bending strength of the pile itself. We anticipate that there will be a wide variety of pile lateral loading conditions along the rail alignment, due to the varying configurations of the proposed structures and the geometries of the creeks and crossings. Site specific pile lateral load analyses should be performed when the structural design details are more clearly defined.

In addition to the piles, lateral loads will also be resisted by passive earth pressures that develop (mobilize) against the pile cap sidewalls under lateral translation. This assumes the pile cap excavation sidewalls remain stable during construction, and the concrete is placed neat with the sides of the excavation. For passive pressure design, an ultimate equivalent fluid pressure of 600 pounds per cubic foot (pcf) is recommended against the forward facing sidewall of the pile cap. Mobilization of this ultimate passive pressure will require a lateral displacement of approximately 0.02 of the pile cap height for sandy fill soils that will be placed and compacted around the pile cap. For fixed head pile connections, the displacement of the piles will be equal to the lateral pile cap translation. A linear reduction in the passive equivalent fluid pressure should be assumed for pile head displacements less than those needed to develop the ultimate

passive capacity against the pile cap sidewall. The appropriate factor of safety will depend on the design condition. We recommend using a minimum factor of safety of at least 1.5 for extreme wind load design and 1.1 for the design-level seismic event.

Consideration should be given to performing indicator pile programs at the bridge spans prior to production pile driving in order to help in evaluating driving resistances and developing capacities across the site and obtaining data for the selection of production pile lengths. The indicator pile programs could include dynamic pile testing and monitoring with a pile driving analyzer (PDA) during initial pile installation and restriking to evaluate setup (increase in capacity with time, and static load testing. Recommendations and specifications for the indicator pile programs should be developed during final design of the foundation systems for the proposed structures.

Predrilling may be required to penetrate rock fragments and/or obstructions in any existing site fills. The drill auger and predrill hole should not be greater than the diameter or width of the piles. Jetting is not recommended due to the presence of sandy materials and shallow groundwater conditions. Relief drilling through the open-ended pipe piles to facilitate pile installation may be considered with consultation from Kleinfelder.

Ground vibrations will occur as a result of pile driving. Prior to the start of pile driving, we recommend that pre-condition surveys be conducted at nearby existing facilities and structures in the vicinity, and that procedures be established to monitor ground vibrations and the response of nearby structures and facilities.

For culvert structures that are to carry rail track loads, design shall conform to the guidelines provided in AREMA Chapter 1, Part 4, Culverts, and AREMA Chapter 8, Section 10.4, Installation. All concrete culverts shall have the structural backfill thoroughly compacted to 90% minimum relative density value.

8.2.2 Retaining Walls

The materials, design, and construction of retaining walls and abutments should conform to AREMA Chapter 8, Part 5, Retaining Walls, Abutments and Piers. Bridge abutments, retaining walls and below-grade structures such as culverts will experience

lateral pressures due to the retained earth on the exterior of the walls. These walls or structures must be designed to resist static and seismic earth pressures due to the adjacent soil, and any surcharge effects caused by loads adjacent to the walls. Analysis of backfill pressures behind the wall and active, passive, and at-rest earth pressures should conform to AREMA Manual Chapter 8, Part 5, Section C-5.3.2, Computation of Backfill Pressure. Passive earth pressures in native soils or select fill and base friction mobilized at the base of retaining walls can be used to resist lateral loads. Earth pressures and related parameters for final design of retaining structures will be assessed based on site specific data during a future phase of geotechnical investigation. For conceptual design purposes, the following paragraphs present our preliminary recommendations for retaining walls.

Active soil pressure may be used for design of retaining walls if the wall is allowed to move, the backfill is level and fully drained and backfill settlement is not a concern. The static active lateral soil pressure will be a triangular pressure distribution calculated using a preliminary equivalent fluid weight of 40 pcf for conceptual design. In addition, one-third of any live loads behind the walls should be applied as additional design surcharge.

For walls that will retain soil as a "restrained" wall system, the walls should be designed using at-rest soil pressure. To calculate the static lateral at-rest soil pressures on a fully-drained wall, a triangular pressure distribution based on a preliminary equivalent fluid weight of 60 pcf may be assumed for conceptual design. In addition, one-half of any live loads behind the restrained walls should be applied as additional design surcharge. These preliminary values need to be verified/modified based on the site specific results from a future geotechnical investigation program.

The 'dynamic' increment of earth pressure acting on retaining structures (applies to unrestrained and restrained walls) due to seismic load case will be evaluated during a future phase of investigation and geotechnical analysis using site-specific data.

Backfill behind the retaining walls should consist of granular, imported soil or approved on-site soils of a low expansion potential. Clays with moderate to high expansion potential should not be used as backfill behind retaining walls. Over-compaction of wall

backfill should be avoided because increased compaction effort can result in lateral pressures significantly greater than those recommended in this report.

The new retaining walls may be designed without hydrostatic pressures if they are fully drained. Backdrainage should consist of either a prefabricated drainage material or a layer of drain rock. Prefabricated drainage material (such as Miradrain® 2000 or an approved alternative) may be used behind retaining walls. Prefabricated drainage material should be installed in accordance with the manufacturer's recommendations. As an alternative to prefabricated drainage material, a drain rock layer may be used. The drain rock layer should be at least 12 inches thick and extend to within 2 feet of the ground surface. A four-inch diameter, perforated, schedule 40 PVC (or equivalent) pipe should be installed (with perforations facing down) along the base of the wall. Drain pipes should rest on a 2-inch thick bed of drain rock. Drain pipes should be sloped to drain by gravity to a sump or other drainage facility. Alternately, weep holes at least 3 inches in diameter and spaced no farther than 8 feet apart may be used where drainage from the holes does not create a hazard.

Drain rock should conform to Caltrans specifications for Class 2 Permeable material. Alternatively, clean, 1/2 to 3/4-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. A 12 to 18-inch thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration.

Retaining walls (for low abutments and wing walls) should be founded in properly-prepared subgrade soils at a minimum embedment depth of 24 inches below nearest adjacent finished grade. Excavations for retaining wall foundations should expose undisturbed and competent stiff clayey natural or compacted fill soils that are free of significant organic materials or other deleterious materials. Retaining wall footings may be designed for a net allowable bearing pressure of 2,000 psf due to dead plus live loads, with a one-third increase for all loads including seismic. Kleinfelder should observe the exposed subgrade for retaining wall foundations to compare the exposed soils with the design assumptions described above. If exposed subgrade soils are found to differ from those assumed, we may need to review the net allowable bearing pressure for applicability or recommend remedial earthwork.

8.3 PASSENGER STATIONS, AND MAINTENANCE FACILITY BUILDING

8.3.1 Foundations

The project will include construction of at-grade passenger Station features (including platforms and small/lightly loaded structures) and also the Maintenance Facility. Shallow spread footing foundations could be considered for these structures with lighter loading as compared to the bridges. Station and maintenance facility buildings are anticipated to be one-story buildings. Building foundation loading information was not available when this report was prepared for the conceptual design phase. Therefore, based on the proposed construction and our experience with similar buildings, we anticipate the buildings will have maximum column loading (dead plus live loads) of about 50 kips.

Based on the review of the existing subsurface data in the vicinity of the proposed structures and preliminary evaluation, we conclude that the loads for the proposed buildings can be supported by continuous and isolated shallow spread footings. The recommended allowable soil bearing pressures, depth of embedment, and width of footings are presented below for conceptual design purposes. These values need to be verified/modified based on the site specific results of a future geotechnical investigation program.

Table 6
Shallow Foundations

FOUNDATION BEARING CAPACITY RECOMMENDATIONS		
Footing Type	Allowable Bearing Pressure (ksf)*	Minimum Embedment (in)**
Continuous Footing	2 to 2.5	12 to 18
Isolated Footing	2.5 to 3.5	18 to 24

* Pounds per square foot, dead plus live load. Includes a factor of safety (FS) of 3.

** Below lowest adjacent grade defined as bottom of capillary break on the interior and finish grade at the exterior.

Allowable soil bearing pressures may be increased by one-third for transient loads such as wind and seismic loads. Total settlement from building loads is anticipated to be less

than 1 inch with 1/2 inch of differential settlement over 50 feet or between columns. These estimates do not include the potential settlement due to liquefaction.

Additional subsurface soil investigation is needed to better characterize the underlying soils. Additional investigation will also be required to estimate the potential for liquefaction at the building and platform locations. If liquefaction potential exists and the associated settlement is in excess of what the structure can tolerate, alternative foundation systems (such as mat foundation) or ground improvement (using stone columns or other methods) can be used to mitigate the effects. Potential liquefaction settlement will be evaluated during the next phase of geotechnical investigation using site specific data on subsurface conditions.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundations, including grade beams. An allowable friction coefficient of about 0.30 to 0.35 between the foundation and supporting subgrade may be used. For passive resistance, an allowable equivalent fluid pressure of 300 pounds per cubic foot may be used. Passive pressure should be neglected in the upper one foot unless the adjacent surface is confined by paving or flatwork. The friction coefficient and passive resistance may be used concurrently, and the passive resistance can be increased by one-third for wind and/or seismic loading. These preliminary recommendations are for conceptual design purposes, and will need to be reassessed during the next phase of geotechnical investigation using site specific data on subsurface conditions.

8.3.2 Slabs-on-Grade – General

Concrete slabs for the project are expected to include interior slabs-on-grade (such as at the Maintenance Building), Station platforms, ramps and shelters, curb and gutter, walkways and other exterior flatwork. New concrete slabs-on-grade should be supported on crushed rock, angular gravel, or aggregate base material over properly prepared subgrade soils. The upper portion of the soil subgrade is expected to consist of poor (clayey) soil in the area from approximately Castroville to Station 150+00 and should consist of properly compacted “non-expansive” fill material to protect and reduce potential damage to the slabs as a result of seasonal shrink/swell of the expansive

subgrade soils. Based on existing data we estimate that this layer of “non-expansive” engineered fill material will need to be at least 18 inches in total thickness.

8.3.3 Interior Concrete Slabs

Concrete floors should be supported on at least 6 inches of angular gravel or crushed rock to improve subgrade support for the slab. The capillary break material should be 3/4-inch maximum size with no more than 10 percent by weight passing the #4 sieve. The section thickness of angular gravel or crushed rock may be counted towards the recommended “non-expansive” fill material thickness described in the previous section.

For conceptual design purposes, slabs-on-grade planned for the rail segment located between Castroville and Station 150+00 (or other areas where fine-grained subgrade is encountered), supported as described above may be designed using a modulus of subgrade reaction (KV1) value of 140 pounds per cubic inch. For the segment between approximately Station 150+00 and the south end of the project in Monterey, a preliminary modulus of subgrade reaction value (KV1) of 200 pounds per cubic inch may be designed used for slabs-on-grade. These values need to be refined during future soil investigations.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) the current industry standard is to place a vapor retarder on the compacted crushed rock layer. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least 20 mil in thickness. Lapped joints and perforations in the vapor barrier should be kept to a minimum, and should be sealed. To provide protection for the membrane, 2 inches of slightly moistened clean fine sand should be placed on top of the membrane prior to placement of concrete. Where crushed rock is used as the capillary break material, seating of the rock with a vibratory plate compactor may aid in reducing the potential for damage to the vapor barrier as the reinforcing steel and the concrete are placed.

It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture

problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

8.3.4 Exterior Concrete Slabs

Exterior concrete slabs-on-grade for the project may include new slabs, or repairs to existing flatwork. New exterior concrete slabs-on-grade should be supported on crushed rock, angular gravel, or aggregate base material at least 4 inches thick. The

rock layer should be supported on properly prepared and compacted subgrade soils. In the MBL segment located between Castroville and approximately Station 150+00 (or other areas where clayey subgrade is encountered), the upper portion of subgrade should consist of properly compacted “non-expansive” fill material. For conceptual design purposes, preliminary modulus of subgrade reaction values (by geographic reach) are provided in the previous section of this report. The preliminary values need to be refined during future soil investigations. Exterior concrete slabs-on-grade exposed to more than occasional light vehicle traffic should be designed as rigid pavements.

8.4 DRAINAGE AND GROUNDWATER

8.4.1 Surface Drainage

Water is the principal influence on soil stability in roadbed, subgrade and slopes, therefore, control of surface and subsurface water is the most important factor in rail roadbed design and maintenance. Individual ballast particles must provide a free-draining and clean section for proper drainage of surface water to parallel side ditches or runoff areas. Excessive moisture in subgrades and ballast sections are a primary source of track roadway problems. Side ditches should be free draining and prevent standing water which could saturate the roadway subgrade. A wet ballast section reduces the shear strength of the assembly of ballast particles and dirty, moist ballast sections will support the growth of vegetation which reduces the drainage capability of the ballast material.

Surface drainage for the track section shall conform to the requirements provided in AREMA Chapter 1, Section 1.2.4 and 1.4.5 for the construction and maintenance of roadbed, respectively. Lateral and longitudinal subdrains consisting of perforated pipes, geotextiles, and free draining backfill materials may be used in combination to improve the roadbed drainage.

Due to the lack of maintenance, some of the culverts that were observed appeared to be plugged and not draining. Sediment has washed over the tracks (or accumulated due to wind-blown action) in various locations. Commercial and residential construction along the southern reaches of the right-of-way has taken place over the last few years. Due to the lack of funding and thus property management and oversight of the right-of-way in past years, drainage has been accelerated onto the railroad right-of-way by

these commercial and residential developments. The growth of commercial and residential buildings, driveways, and parking lots has increased impervious soil areas with alterations to the overall hydrology of the region. Due to the corridor's location, the additional runoff must be channelized along or through the right-of-way. No accommodation for this additional runoff has been provided.

Adequate drainage structures and drainage maintenance are critical to track performance and longevity. Surface drainage should be improved by raising the embankment, sloping embankment shoulders to drain, and grading surrounding land so that water flows away from the track embankment. Ditches and culverts should be cleaned and upgraded where necessary to accommodate runoff. Adequate drainage ditches must be provided as part of the design for track improvements.

Particular attention should be directed toward proper drainage of trackwork that crosses local street. The adjacent surface pavement should be designed so surface water will drain away from the track. Track drains should be used to prevent water from standing. In areas of special trackwork, particular attention will be needed to provide drainage for any special trackwork units and switch-throwing mechanisms. When possible, track drains should be located in tangent track.

Future evaluations by the civil designers for preliminary engineering should include a detailed inspection of all the culverts to determine which, if any, require replacement or rehabilitation. The inlets of all culverts should be cleared of all debris and brush during construction of other track improvements. In addition, track ditches should be restored during construction and cleared of brush periodically.

An economical means of improving embankment stability and decreasing soft track related maintenance is to improve surface drainage so that runoff water is directed away from the embankment. Water should not be permitted to pond on shoulders or adjacent to embankments.

- We recommend that the embankment be raised as necessary so that it is well above the surrounding ground. We recommend that the top of subballast (approximately 18 to 24 inches below the bottom of ties) should be a minimum of 6 inches above adjacent ground.

- We recommend that surface drainage be improved along the railroad so that water flows away from the embankment and does not pond next to it. The top of the embankment should be graded so that water will flow away from the track and so that it does not sit or pond on the shoulders.
- We recommend that ditches be cleaned, deepened, widened, and regraded, as appropriate, to convey water away from the track. Hydrologic evaluations may be appropriate in some locations to evaluate the adequacy of ditches. Ditch inverts should be a minimum of 6 inches below the bottom of subballast and a minimum of 3 feet below top of tie.
- We recommend that existing culverts be cleaned, repaired, and replaced as necessary so that water will flow downgrade and away from the embankment. Hydrologic evaluation should be performed to check existing culvert sizes and to size new and replacement culverts. We recommend that the embankments near culvert headwalls and ditches upstream and downstream of culverts be evaluated for past occurrences of erosion and potential to erode. Erosion protection should be provided where the potential for erosion exists.

8.4.2 Sub-Surface Groundwater Considerations

Because much of the track has experienced about 100 years of train traffic and was likely originally poorly constructed (e.g., inadequate compaction, poor quality soils), we anticipate that ballast pockets are present in the embankments along much of the line. Ballast pockets are depressions that form in railroad embankments as a result of train loading, deformation of the embankment or foundation soils, and subsequent addition of ballast to raise or restore the track. Ballast pockets in the embankments may be only a few inches deep to many feet deep. Water trapped in ballast pockets is a major contributor to track settlement and failures, ballast fouling, and other "soft track" conditions.

Subsurface drainage may be appropriate in some locations, e.g., trench drains or pipe subdrains parallel to or away from the track. We recommend that site-specific evaluations and recommendations be developed when candidate locations are identified. We recommend that further assessment of the potential need for these alternative subsurface drainage systems be performed during the subsequent design

phases of this project after existing topography and approximate proposed track embankment configurations are developed.

For slope areas, under certain conditions seepage gradients at slopes can cause soils to become completely "liquid" or "quick." Under this condition, high seepage gradients reduce the soil effective stress to zero and the soil has no shear strength. The phenomena of piping and "quick" behavior are of concern where silt, silty sand and fine sand are present in areas of seepage discharge. Clays, particularly those of medium to high plasticity, are not usually susceptible to piping.

The most effective method of reducing the detrimental effects of water on stability of slopes is to remove any water hazard that may exist upslope in the form of ponds, blocked ditches, beaver dams or similar sources of recharge. If further improvement is required, it can be achieved through the installation of subsurface drainage. Where seepage outcrops on a lower slope or can be economically reached with a backhoe, trench drains can provide a considerable improvement to the stability. If drains can be extended to a sufficient depth to excavate and replace a portion of the shear zone, additional positive benefit will be achieved. Alternatively, where seepage outcrops on a slope and the soil is competent to allow flow without damage, buttress may be effective.

8.5 PAVEMENT RECOMMENDATIONS FOR CONCEPTUAL DESIGN

All road and street design should be in accordance with the current specifications and design guidelines of the involved local jurisdictions. For those cases where the local jurisdictions have no design guidelines, Caltrans Design Standards should be used.

New asphalt concrete pavements are planned for the project. Subgrade support characteristics for asphalt concrete pavements vary along the alignment. Near-surface soils located in Castroville from approximately the realignment of the Monterey Branch Line Track in Castroville to about Station 5+00 consist of fine sandy loam. Based on our previous work in the Castroville area we estimate that the subgrade R-value will range from about 5 to 40. Areas along the alignment on the north side of the Salinas River, from approximately Station 5+00 to Station 150+00, consist of clay and clay loam with some areas of fine sandy loam. In these areas we estimate that the subgrade R-value will range from about 5 to 10. Areas along the alignment starting on the south

side of the Salinas River at approximately Station 153+00 and extending to the area of Lake El Estero at approximately Station 784+50 in Monterey consist of sands and sandy loams. In these areas we estimate that the subgrade R-value will range from about 55 to 80. From the area of Lake El Estero at approximately Station 784+50 to the end of the project at the east side of the Custom House in Monterey, subgrade soils are anticipated to consist of loamy fine sands. Previous work in the site vicinity on similar soils suggests that R-values will be about 20 on this material. These R-value estimates are provided for preliminary evaluation only.

Preliminary estimates for asphalt concrete pavement sections are presented below. These estimates are based on a conservative application of the above R-value estimates herein. Where new asphalt concrete pavements are anticipated, we recommend performing R-value testing on bulk samples collected the existing subgrade soils in the areas of the new pavements. These samples may be collected during the proposed subsurface exploration. It is expected that this will allow revision/reduction of some of the pavement sections and a corresponding savings in materials.

Pavements for this project are expected to consist primarily of reconstructed grade crossings, vehicle access lanes, parking areas and access pathways. For preliminary pavement section design estimates we used R-values of 5, 20 and 55 to develop the preliminary pavement sections presented in the table below. Our preliminary pavement sections are based on the Caltrans design method, which includes a gravel equivalent safety factor of 0.2 feet applied to the asphalt thickness. Based on our experience with similar projects, we suggest using a Traffic Index (TI) of at least 4.5 for automobile parking areas, a TI of at least 5.5 for automobile and light truck traffic lanes, and a TI of at least 6.5 for garbage truck areas. The design traffic index should be determined by the civil engineer. For heavy vehicle areas, a minimum asphalt concrete section of 3 inches is recommended.

Table 7
AC Pavement Sections, Assuming R-Value = 5

TAMC PRELIMINARY PAVEMENT SECTIONS CASTROVILLE STATION TO STATION 150+00 R-VALUE = 5					
T.I.	Alternative 1		Alternative 2		
	AC	AB	AC	AB	ASB
4.5	2.5	9.0	2.5	4.5	5.0
5.0	2.5	11.0	2.5	5.0	6.5
5.5	3.0	12.0	3.0	5.5	7.0
6.0	3.0	13.5	3.0	6.5	8.0
6.5	3.5	14.5	3.5	6.5	9.0
7.0	4.0	15.5	4.0	6.5	10.0
7.5	4.5	16.5	4.5	7.0	10.5
Note: Thicknesses shown are in inches. AC = Type A or B Asphalt Concrete AB = Aggregate Base (Minimum R-Value = 78) ASB = Aggregate Subbase (Minimum R-Value = 50)					

Table 8
AC Pavement Sections, Assuming R-Value = 20

TAMC PRELIMINARY PAVEMENT SECTIONS STATION 784+50 TO CUSTOM HOUSE R-VALUE = 20					
T.I.	Alternative 1		Alternative 2		
	AC	AB	AC	AB	ASB
4.5	2.5	7.0	2.5	4.5	2.5
5.0	2.5	8.5	2.5	5.0	3.5
5.5	3.0	9.0	3.0	5.5	4.0
6.0	3.0	10.5	3.0	6.5	4.5
6.5	3.5	11.5	3.5	6.5	5.5
7.0	4.0	12.0	4.0	6.5	6.0
7.5	4.5	12.5	4.5	7.0	6.5
Note: Thicknesses shown are in inches. AC = Type A or B Asphalt Concrete AB = Aggregate Base (Minimum R-Value = 78) ASB = Aggregate Subbase (Minimum R-Value = 50)					

Table 9
AC Pavement Sections, Assuming R-Value = 55

TAMC PRELIMINARY PAVEMENT SECTIONS STATION 153+00 TO STATION 784+50 R-VALUE = 55		
T.I.	AC	AB
4.5	2.5	4.0
5.0	2.5	4.0
5.5	3.0	4.0
6.0	3.0	4.0
6.5	3.5	4.0
7.0	4.0	4.0
7.5	4.5	4.0
Note: Thicknesses shown are in inches. AC = Type A or B Asphalt Concrete AB = Aggregate Base (Minimum R-Value = 78) Minimum 4 inches AB thickness used		

The anticipated traffic and the alternate pavement sections presented above should be reviewed by the project civil engineer in consultation with the owner during the development of the final grading and paving plans.

Subgrade preparation may require some remedial excavations in areas where loose/weak near-surface soils are anticipated, based on the results of our previous subsurface explorations. We anticipate that loose/ weak near-surface soils will be encountered in the sandy soils in undeveloped areas between approximately Stations 153+00 to about 280+00 and between 370+00 to about 608+00. In these areas we anticipate that excavations of approximately 1 foot below the bottom of the proposed aggregate base section will be needed to provide a firm surface for construction of backfill and the pavement section. After stripping and after excavations as needed, the exposed subgrade should be scarified in-place to a depth of 12 inches, moisture conditioned to near optimum moisture and compacted to at least 95 percent relative compaction based on ASTM D 1557, except in the clayey area between about Stations 5+00 to 150+00, where the soil should be compacted to 92 percent relative compaction at 2 to 5 percent over the optimum moisture content. Backfill and other fills in the pavement areas should likewise be moisture conditioned and compacted to at least 95 percent relative compaction near optimum moisture, except clay backfill or fills which

should be compacted to 92 percent relative compaction at 2 to 5 percent over the optimum moisture content. Compacted pavement subgrade should be non-yielding. Removal and subsequent replacement of some material (i.e., areas of excessively wet materials, unstable subgrade, or pumping soils) may be required to obtain the required compaction.

Asphalt concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type A or Type B asphalt concrete in vehicle areas. Asphalt concrete should comply with the specifications presented in Section 39 of the Caltrans Standard Specifications, latest edition. Class 2 aggregate base materials should conform with Section 26 of the Caltrans Standard Specifications, latest edition. Class 2 aggregate base should be compacted to at least 95 percent relative compaction at near the optimum moisture content. Asphalt concrete should be compacted to a minimum of 96 percent of the maximum laboratory compacted (Hveem) unit weight. ASTM test procedures should be used to assess the percent relative compaction of the pavement subgrade soils, aggregate base and asphalt concrete.

Typically the pavement surface should be sloped at a minimum of 2 percent and drainage gradients maintained to carry all surface water off the site due to the slightly porous or permeable nature of asphalt concrete. Surface water ponding should not be allowed anywhere on the site during or after construction.

Rigid concrete pavements are anticipated along portions of the proposed rail project. Our review of the published soil surveys and R-value tests in the area between Castroville Station to Station 150+00 suggest that the near-surface soil beneath the this area will primarily consist of clays with a low R-Value. As such, flexible pavements are preferred to rigid concrete pavements. The Caltrans Highway Design Manual recommends that where expansive basement soil (with a Plasticity Index > 12) and/or a low Resistance Value (R-Value < 10) occur, an asphalt concrete or flexible pavement should be specified, as opposed to a Portland cement concrete pavement (PCC pavement). An exception to this would be if "the R-Value of the basement soil can be raised to 10 or above by treatment, to a minimum depth of 0.65 feet, with an approved stabilizing agent such as lime, cement, asphalt, or fly ash, PCC pavement can be specified." We recommend that Atterberg Limits (Plasticity Index) tests and R-Value tests be performed on the proposed basement soils to determine if these PCC

pavement restrictions will apply to the site. If these restrictions exist and it is desired to further investigate the use of PCC pavement, a treated R-Value test should be performed to evaluate the applicability of using a treated basement soil. Alternatively, for short sections of PCC pavement, subgrade soils may be replaced with soils with a Plasticity Index ≤ 12 and an R-value ≥ 10 .

For purposes of this feasibility study, if site conditions or treatment/improvement allow the use of PCC pavements, the following PCC pavement sections are anticipated for the project:

Table 10
PCC Rigid Pavement Sections, Assuming TI <9.0

TAMC PRELIMINARY PAVEMENT SECTIONS RIGID PAVEMENTS ^{(1), (2),(3)} FOR TRAFFIC INDICES <9								
Location	Rigid Pavement Structural Depth							
	With Lateral Support (ft)				Without Lateral Support (ft)			
Castroville Station to Station 150+00 Soil Type III*	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.75 JPCP	0.75 JPCP
	0.35 LCB	0.35 HMA-A	1.00 AB	0.35 ATPB	0.35 LCB	0.35 HMA-A	1.00 AB	0.35 ATPB
	0.50 AS	0.50 AS		0.80 AB	0.50 AS	0.50 AS		0.80 AB
Station 153+00 to Station 784+50 Soil Type I	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.75 JPCP	0.75 JPCP
	0.35 LCB	0.35 HMA-A	0.50 AB	0.35 ATPB	0.35 LCB	0.35 HMA-A	0.50 AB	0.35 ATPB
				0.35 AB				0.35 AB
Station 784+50 to Custom House Soil Type II	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.70 JPCP	0.75 JPCP	0.75 JPCP
	0.35 LCB	0.35 HMA-A	1.00 AB	0.35 ATPB	0.35 LCB	0.35 HMA-A	1.00 AB	0.35 ATPB
	0.50 AS	0.50 AS		0.80 AB	0.50 AS	0.50 AS		0.80 AB

* Requires subgrade to be treatable (or replaceable) to improve to soil Type II, otherwise alternate pavement type.

Notes:

(1) Thicknesses shown for JPCP are for doweled pavement only. The thickness shown in these tables are not valid for non-doweled JPCP

(2) Includes 0.03 ft sacrificial wearing course for future grinding of JPCP

(3) Portland cement concrete may be substituted for LCB when justified for constructability or traffic handling. If Portland cement concrete is used in lieu of LCB, it must be placed in a separate lift than JPCP and must not be bonded to the JPCP

Legend:

JPCP = Jointed Plain Concrete Pavement

LCB = Lean Concrete Base

HMA-A = Hot Mix Asphalt (Type A)

ATPB = Asphalt Treated Permeable Base

AB = Class 2 Aggregate Base

AS = Class 2 Aggregate Subbase

TI = Traffic Index

From Caltrans Highway Design Manual Table 623.1A, 623.1D, 623.1E, July 1, 2008

See Caltrans Highway Design Manual Chapters 600 and 620 for additional information.

If site soil conditions allow the use of untreated basement soils, the upper 12 inches of subgrade soil in Portland cement concrete pavement areas should be moisture conditioned and compacted to at least 95 percent relative compaction at a soil moisture content within 2 percent of the optimum moisture content. The compacted subgrade should be non-yielding.

If treatment is required, we estimate that subgrade treatment would include moisture conditioning and blending in approximately 4 percent Dolomitic quick lime (DQM) by weight into the upper 0.65 feet of the exposed subgrade soils and compacting to 95 percent relative compaction. This treatment would be considered subgrade preparation and no additional subgrade preparation would be required provided the finished treated basement soil is firm and unyielding.

We assume that the PCC pavements will have doweled joints and a modulus of rupture for the concrete of at least 625 pounds per square inch. It should be noted that the modulus of rupture for concrete is based on flexural strength, not compressive strength, and should be specified accordingly. Concrete with a compressive strength of 3,000 psi is not expected to provide the desired flexural strength. Our experience is that the compressive strength would need to be in the range of 4,500 to 5,500 psi to achieve the required flexural strength. Laboratory testing to evaluate the design strength is recommended. Alternatives to this conceptual design may be considered, based on the site grading plans and more accurate traffic data.

8.6 EXPANSIVE SOILS – RECOMMENDATIONS FOR CONCEPTUAL DESIGN

Expansive soils have the capacity to undergo large volume changes with changes in moisture content and typically are associated with moderate to high plasticity clays. Our review indicates expansive soil should be anticipated from approximately Station 5+00 to Station 124+50. Some areas of expansive soil may also occur at approximately the MBL track in Castroville to Station 5+00, and from Station 124+50 to approximately Station 150+00. We recommend that Atterberg limits tests be performed on samples of soils retrieved from these areas, as well as the sites for all buildings/platforms, paved areas, passenger Stations, and the Maintenance Facility to evaluate the expansion potentials of these soils. Additionally, Atterberg limits (plasticity) tests of soils from

approximately Station 784+50 to the east side of the Custom House in Monterey typically exhibit high plastic indices and liquid limits, especially where consisting of native or fill materials derived from Monterey formation parent material. However, these soils usually also exhibit a low expansion index and may not be expansive. In this area we would recommend that an expansion index be performed on the proposed subgrade materials in addition to the Atterberg limits tests to avoid potential over-design of the foundations, slabs-on-grade and other improvements.

If not properly mitigated, the cyclic volume changes capable of expansive soils (i.e. shrink-swell) can cause distress and failure of structures, platforms, asphaltic and concrete pavements, slabs-on-grade, and other surfaces. In areas where the track bed is supported by a minimum of 2 to 3 feet of non-expansive ballast, the effects of expansive soils are expected to be marginal. Areas where the track bed is underlain by a marginal (thinner) ballast section, however should anticipate localized movement and thus track movement/disruption if expansive soils are present. Lightly loaded structures founded on shallow spread footing foundations may be subject to comparable effects.

Standard mitigation solutions for the detrimental effects of expansive soils acting on surface improvements and structures include, but are not limited to the following:

- chemical (lime) treatment of the expansive soil within the zone (depth) of influence to reduce its plasticity and expansion potential to an acceptable level.
- removal of the expansive soil within the zone of influence, and replacement with non-expansive import soil.
- construction of subsurface moisture barriers along the perimeter of improvements, which extend to depths below the zone of influence.
- construction of foundation elements which inherently resist the differential uplift pressures of expansive soils, such as post-tensioned slabs, cast-in-place drilled pier foundations, and driven pile foundations.

8.7 CORROSIVE SOILS – RECOMMENDATIONS FOR CONCEPTUAL DESIGN

Based on the results of previous corrosivity tests performed in the vicinity of the alignment, near surface soils are anticipated to be potentially corrosive to ferrous metals

(i.e. iron, steel, cast iron, ductile iron, and galvanized steel and dielectric coated steel or iron). This conclusion is predominantly based on the results of the reviewed saturated resistivity tests. Some limited areas also indicated corrosivity based on the soil pH and redox. The limited number of tests did not indicate sufficient chloride ion or sulfate ion concentrations to damage reinforced concrete structures and/or cement mortar coated/embedded steel. It must be noted that these conclusions are based on a limited number of corrosivity tests taken in the vicinity of the alignment and may not represent conditions throughout the project area. Laboratory analyses should be performed on potentially corrosive soils in general accordance with ASTM Test Methods for pH, resistivity, and for soluble sulfates and chlorides.

Kleinfelder is not a corrosion expert. We recommend that long-term corrosion control design recommendations be provided from a qualified corrosion engineer to: 1) evaluate the corrosion potential of the site soils to proposed improvements; 2) recommend further testing as required; and 3) provide specific methods for corrosion mitigation that are appropriate for the project. The corrosion potential for any imported fill and backfill should also be checked. Corrosion preventive measures must be utilized on all embedded track components.

In our experience with deep foundation projects in northern California along the bay margins, we have seen steel pile and H-pile designs ranging from (1) a complete discounting of the pipe pile shell in the long-term (full corrosion), to (2) an increase in the steel pipe pile wall thickness or H-pile section to account for a predicted loss of steel due to corrosion over time.

8.8 RECOMMENDATIONS FOR FUTURE GEOLOGIC AND SEISMIC HAZARD EVALUATION INVESTIGATIONS

The following future geologic and seismic hazard investigations are recommended, in addition to site specific subsurface investigation of structures and improvements:

- Detailed geologic mapping of the dune sand slopes, and zones of sedimentation over the trackway and proposed Station areas along the southern portions of the alignment.

- Slope stability analysis of the dune sand slopes under static and pseudo-static conditions.
- Analyses for liquefaction and lateral spreading.
- Refined analyses of seismic ground motions (both AREMA and CBC based methods) to develop final seismic design criteria.
- Surface-rupture evaluations if Stations, platforms or other buildings for the project are proposed across the mapped traces of the faults described in this report.

8.9 PRELIMINARY RECOMMENDATIONS FOR FUTURE SUBSURFACE EXPLORATION PROGRAM

A program for subsurface exploration and testing will be required for characterization of subsurface conditions to support the next phase of “preliminary engineering” design work for the various rail facilities and trackwork. A detailed Work Plan, including a thorough review of available geotechnical information, objectives of the investigation, proposed test boring and cone penetration test (CPT) exploration points, frequencies and spacing, depths, prioritized locations, descriptions of drilling and sampling methods, laboratory testing program, permitting, and a Health and Safety Plan will need to be prepared during a future task for subsequent design phases of the project.

The proposed geotechnical subsurface exploration program should generally follow the AREMA guidelines, Chapter 8 – Concrete Structures and Foundation, Part 22 – Geotechnical Subsurface Investigation.

As discussed above, for rehabilitation of track roadbed sections, we recommend that the embankment subgrade conditions also be evaluated during a future engineering design phase through completion of a number of test pits along the rail alignment. We recommend that an allowance for a minimum of one test pit per one-quarter mile of track be included in the upcoming design phase budget. The actual number of test pits required may be greater or less, and will depend on subsurface conditions and observations made during future site investigations by design team members.

9 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, expressed or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was bas on project information provided by the Client. If Client does not retain Kleinfelder to review any plans and specification, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do no affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

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Source: Pacific Aerial Surveys
Black & White

Date	Flight No.	Line No.	Scale
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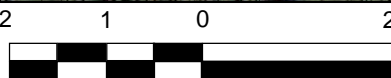
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PLATES



Approximate Scale (miles)

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Original in Color

EXPLANATION

— Proposed Alignment

◆ Milepost



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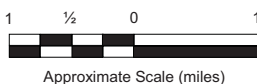
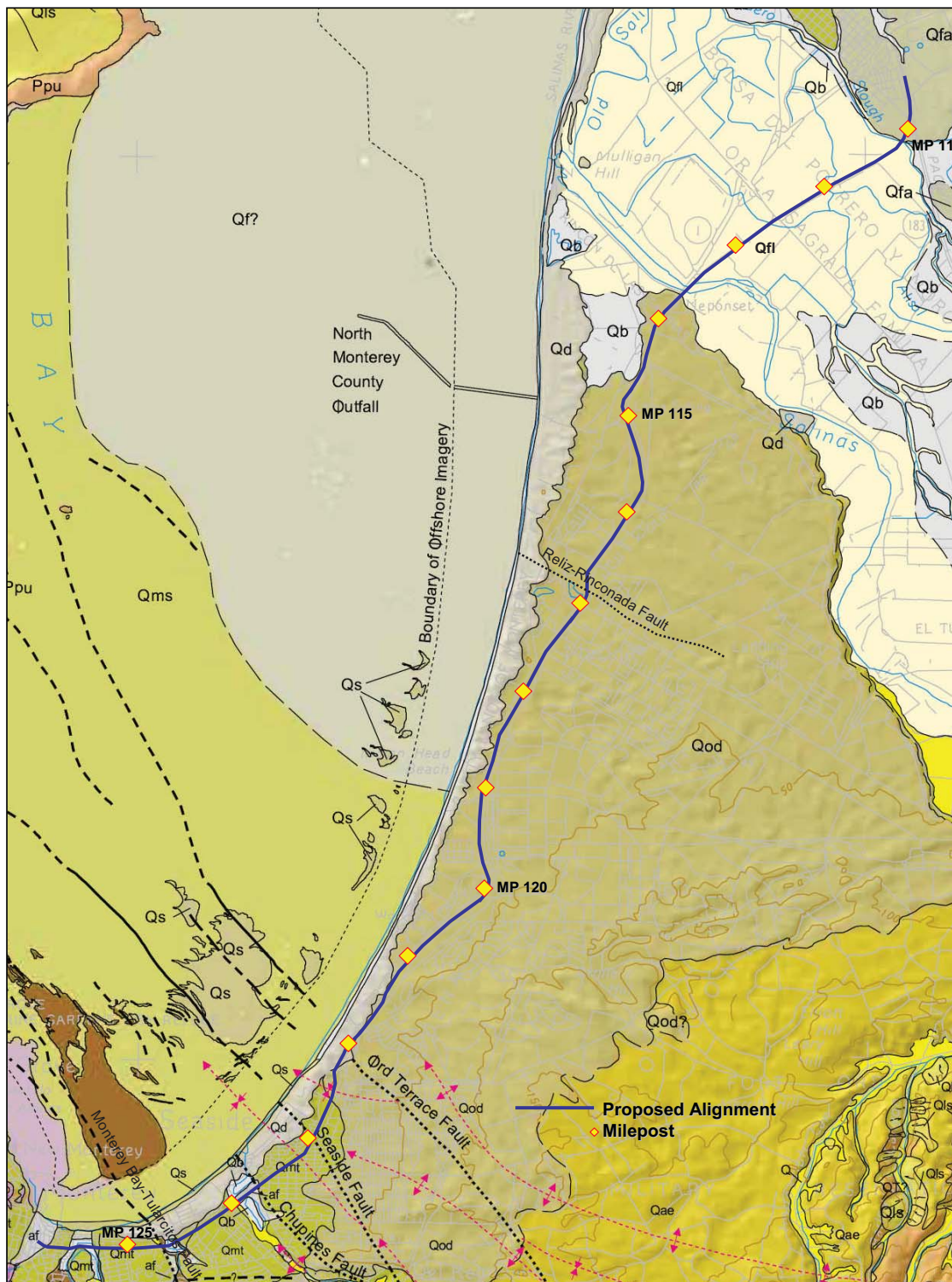
ALIGNMENT LOCATION

Monterey Peninsula
 Light Rail Project
 Monterey County, California

PLATE

1

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Map Source: Wagner, D.L., Greene, H.G., Saucedo, G.J., and Pridmore, C.L., 2002, Geologic Map of the Monterey 30'x60' Quadrangle and Adjacent Areas, California: A Digital Database, California Geologic Survey, map scale 1:100000.

Original in Color

EXPLANATION	
(Selected Units – Not All Units Listed)	
af	Artificial Fill (Recent)
Qfi	Floodplain Deposits (Holocene)
Qb	Basin Deposits (Holocene)
Qd	Dune Sand (Holocene)
Qod	Older Dune Sand (Pleistocene)
Qae	Aromas Sand (Pleistocene)
Qfa	Fan Deposits of Antioch (Pleistocene)
Qmt	Marine Terrace Deposits (Pleistocene)
---	Fault (dashed where inferred, dotted where concealed, queried where uncertain)
- - -	Geologic Contact (dashed where inferred, queried where uncertain)

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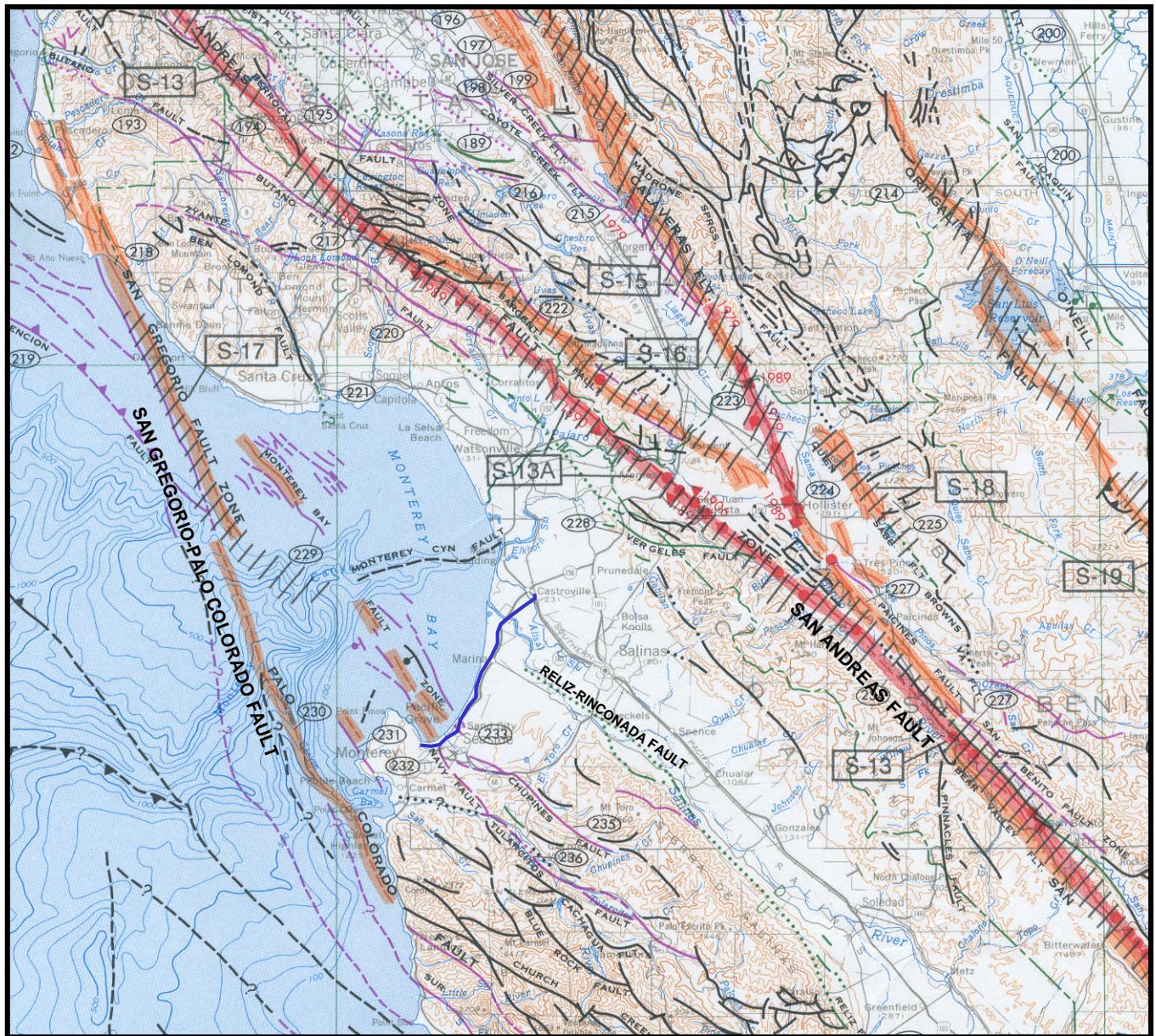
GEOLOGIC MAP

Monterey Peninsula
Light Rail Project
Monterey County, California

PLATE

2

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10 5 0 10



Approximate Scale (miles)

Reference

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EXPLANATION

- Fault with historic movement
- Fault within last 10,000 years
- Faults with movement older than 10,000 years or undifferentiated
- Proposed Alignment

Original in Color



PROJECT NO: 109205
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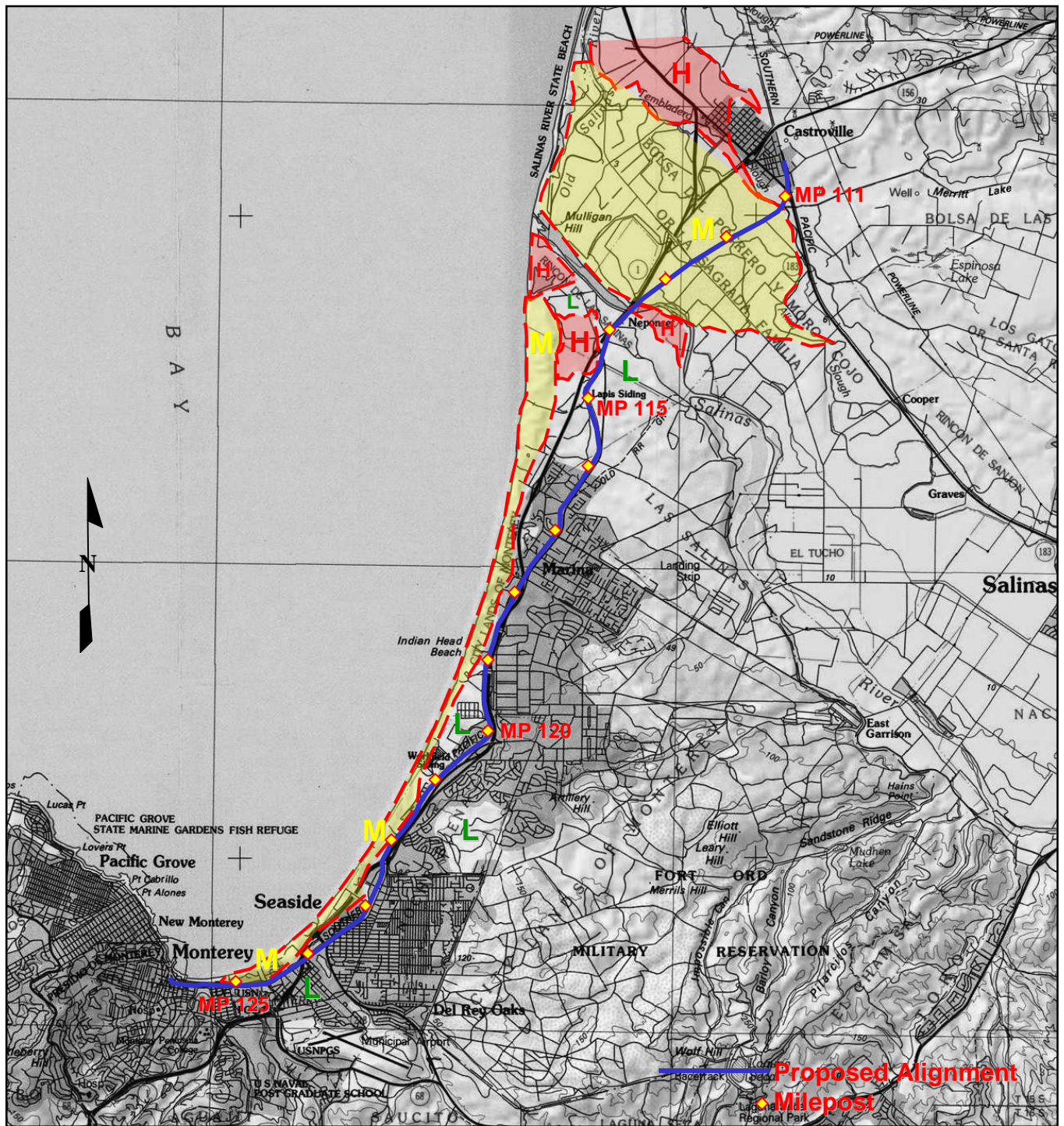
REGIONAL FAULT MAP

Monterey Peninsula
 Light Rail Project
 Monterey County, California

PLATE

3

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Original in Color

EXPLANATION

- H High
- M Moderate
- L Low

Reference
County of Monterey, 2004,
General Plan Update, Relative
Earthquake Induced
Liquefaction Susceptibility



PROJECT NO: 109205
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FILE NAME:

LIQUEFACTION SUSCEPTIBILITY MAP

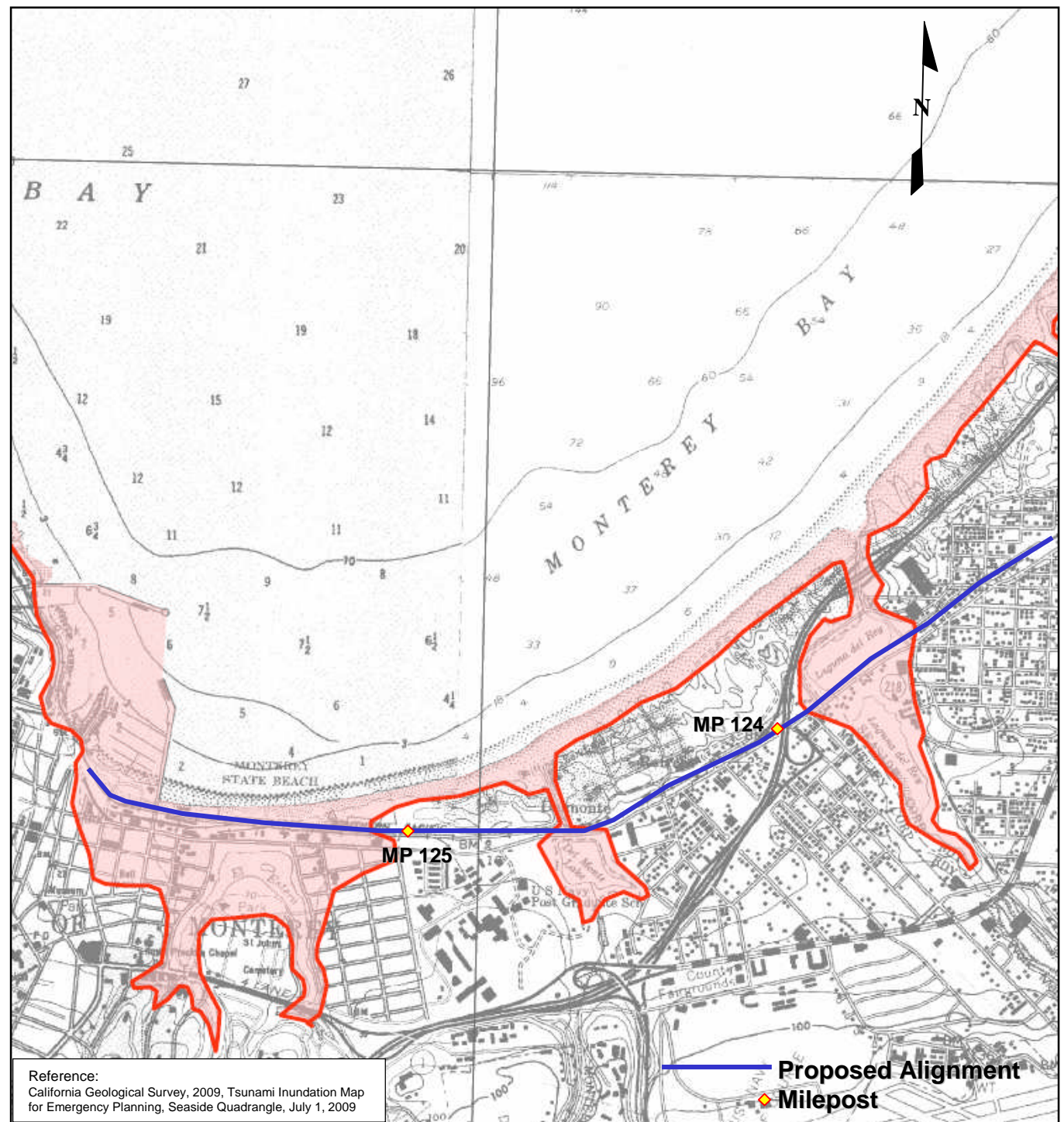
Monterey Peninsula
Light Rail Project
Monterey County, California

PLATE

4

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2000 1000 0 2000



Approximate Scale (feet)

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EXPLANATION

- Tsunami Inundation Line
- Tsunami Inundation Area



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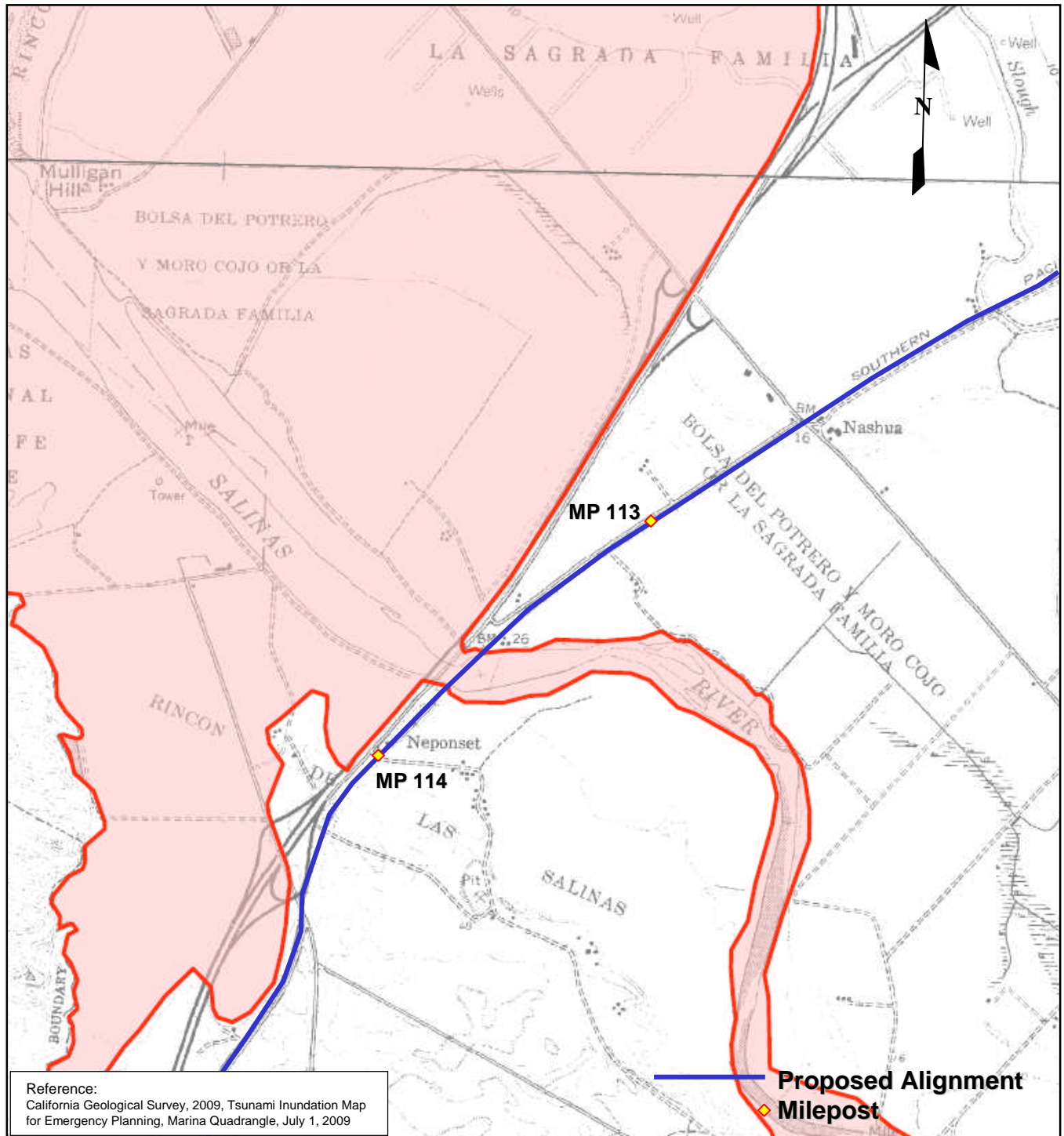
Tsunami Inundation Map (Seaside Quadrangle)

Monterey Peninsula
Light Rail Project
Monterey County, California

PLATE

5

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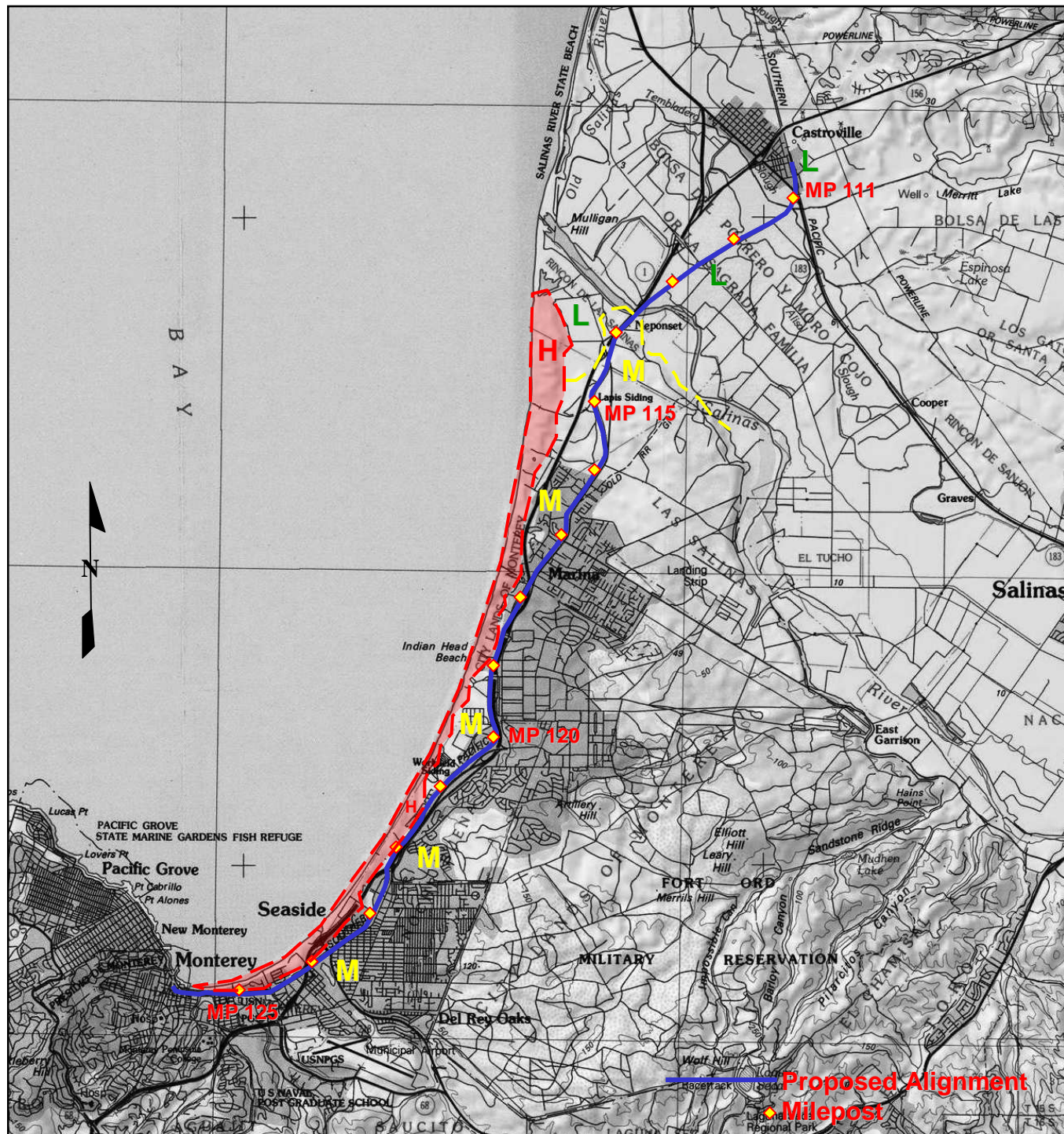
Tsunami Inundation Map (Marina Quadrangle)

Monterey Peninsula
Light Rail Project
Monterey County, California

PLATE

6

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Original in Color

EXPLANATION

- H High
- M Moderate
- L Low

Reference
County of Monterey, 2004,
General Plan Update, Relative
Soil Erosion Hazards



PROJECT NO: 109205
DRAWN: April 2010
DRAWN BY: DS
CHECKED BY:
FILE NAME:

SOIL EROSION MAP

Monterey Peninsula
Light Rail Project
Monterey County, California

PLATE

7

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