FOR CONTRACT NO.; 11-2T0824

INFORMATION HANDOUT

WATER SOURCE INFORMATION
(City of Escondido Letter Dated May 25, 2007)

UNITED STATES FISH AND WILDLIFE SERVICE
(Biological Opinion Dated May 8, 2001)
(Biological Opinion Dated January 16, 2003)

MATERIAL INFORMATION

SEISMIC DESIGN RECOMMENDATIONS
Mission Avenue Undercrossing (Widening)
(Bridge No. 57-0813R, Dated February 1, 2007)
Escondido Flood Control Channel
(Bridge No. 0810R/L/K, Dated January 31, 2007
Washington Avenue Overhead (Widening)
(Bridge No. 57-0812R, Dated January 25, 2007)

FOUNDATION REPORT
Valley Parkway Undercrossing (Widen)
(Bridge No. 57-0809R/L, Dated July 9, 2007)
Escondido Flood Control Channel Ramp (Replace)
(Bridge No. 57-0810K, Dated April 25, 2007)
Escondido Flood Control Channel Ramp (Widen)
(Bridge No. 57-0810L/R, Dated May 3, 2007)
Hale Avenue Undercrossing (Widen)
(Bridge No. 57-0811R/L, Dated March 23, 2007)
Washington Avenue Overhead (Widen)
(Bridge No. 57-0812R, Dated March15, 2007)
Mission Avenue Undercrossing (Widen)
(Bridge No. 57-0813 R, Dated February 15, 2007)

REVISED FOUNDATION RECOMMENDATIONS
(Retaining Wall No. 497L, Dated October 29, 2007)

(Retaining Wall No. 498, Dated October 29,2007)
(Retaining Wall No. 506R, Dated October 29,2007)
(Retaining Wall No. 507R1, Dated October 29,2007)
(Retaining Wall No. 509L, Dated October 29,2007)
(Retaining Wall No. 497L, Dated October 29,2007)
(Retaining Wall No. 510R, Dated October 29,2007)

FINAL FOUNDATION DESIGN RECOMMENDATIONS
Hale Avenue Retaining Walls
(Wall No. 57-0507R/L, Dated March 22, 2007)

STRUCTURAL SECTIONS RECOMMENDATIONS-Corrected
(Dated March 3, 2008)

ROUTE: 11-SD-15-R16.9/M24.1

ESCONDIDO
City of Choice

Mary Ann Mann
Utilities Manager
201 North Broadway, Escondido, CA 92025
Phone: 760-839-4528 Fax: 760-839-4597

May 25, 2007

Mr. Dimitar Peev
Department of Transportation
District 11 – MS 333
P.O. BOX 85406
San Diego, CA 92186-5406

Dear Mr. Peev:

Subject: Will Serve Letter – Route 15, 11-SD-15, KP R48.3/R50.7, PM R30.0/R31.5, 11-268-2T0824, Managed Lanes – Unit 2

This letter will confirm water availability for the above-referenced project. The project is located within the City of Escondido (City) water service area. City water facilities are available in the vicinity for this project's use.

At this time, the City is able to provide potable water for this construction project through existing fire hydrants in the area. Exact fire hydrant locations have not been field verified for the issuance of this letter. At a later date, the City will provide a map indicating approximate fire hydrant locations along the project work site.

Construction water connections are to use City fire hydrant meters. All connections are governed by City ordinances and regulations concerning connections, constructions, capacity charges, permit fees and matters pertaining thereto. For more information, please contact the City's Utility Billing Division at (760) 839-4682.

Sincerely,

Nelson Nuezca

Nelson Nuezca
Engineer II

Lori Holt Pfeiler, Mayor Sam Abed, Mayor Pro Tem Ed Gallo Marie Waldron Dick Daniels

United States Department of the Interior
Fish and Wildlife Service
Ecological Services
Carlsbad Fish and Wildlife Office
2730 Loker Avenue West
Carlsbad, California 92008

In Reply Refer to: 1-6-01-F-1328.2

MAY 0 8 2001

Mr. Michael G. Ritchie, California Administrator
Federal Highway Administration
Department of Transportation
980 Ninth Street, Suite 400
Sacramento, CA 95814-2724

Re: Biological Opinion on the Proposed Operational Improvement Projects on Interstate 15 in San Diego County, California;.

Dear Mr. Ritchie:

This document transmits the Fish and Wildlife Service's (Service) biological opinion based on our review of the proposed Operational Improvement Projects on Interstate 15 (I-15) located in San Diego County, California, and its effects on the threatened coastal California gnatcatcher (*Polioptilla californica californica*, gnatcatcher) in accordance with section 7 of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 et seq.). Critical habitat for the gnatcatcher was designated in 2000, however none occurs within the project boundaries and therefore it will not be addressed further. Your January 27, 2001, request for formal consultation was received on January 31, 2001.

This biological opinion is based on information provided in the November 3, 2000, and January 27, Biological Assessment, meetings, and other information available in our files. A complete administrative record of this consultation is on file at the Service's Carlsbad Fish and Wildlife Office.

Focused surveys for the endangered least Bell's vireo(*Vireo bellii pusillus*) and southwestern willow flycatcher (*Empidonax traillii extimus*) detected neither species. A patch of *Plantago erecta*, a larval host plant for the endangered Quino checkerspot butterfly (*Euphydryas editha quino*, Quino), was observed just outside of grading limits on the southbound side of I-15, just south of State Route 56 (SR-56). No Quino were observed during protocol surveys. Suitable habitat for the endangered arroyo toad (*Bufo californicus*) is not currently present within the project area. Designated or proposed critical habitat for the above species does not occur within the project boundaries and therefore none will be adversely modified as a result of the proposed operational improvements project. For these reasons, the proposed operational improvements project is not likely to adversely affect the above species or adversely modify their respective proposed or designated critical habitats. Therefore, these species will not be addressed further in this Biological Opinion except in the Description of the Proposed Action.

CONSULTATION HISTORY

In 1999, several meetings took place between our office and Caltrans to discuss preliminary findings and discuss project review. In 2000, Caltrans and our office discussed project impacts and compensation in meetings and telephone conference calls. We received a draft Biological Assessment dated November 3, 2000, on November 7, 2000, followed by additional information on November 29, 2000. The details of the project were discussed in a January 17, 2001, meeting with Caltrans biologist Bob James at Carlsbad Field Office. On January 18, 2001, Bob James sent additional information regarding the Biological Assessment via e-mail. On January 31, 2001, we received a request for formal consultation. Field notes from Bob James were received on February 15, 2001.

BIOLOGICAL OPINION

DESCRIPTION OF THE PROPOSED ACTION

The Service, Federal Highways, and Caltrans view the proposed action for which this Biological Opinion is written as a part of a larger action along the I-15 Corridor Project. Interstate highway improvement projects are often disjunct individual projects that, when pieced together, constitute a much larger single linear project. Repair and improvement work along an interstate corridor is generally conducted in finite, disjunct construction periods (and areas) to allow for minimum disturbance to the interstate's primary purpose of providing safe transportation. Each distinct project has environmental impacts associated with it that contribute to regional cumulative impacts associated with the improvement and upkeep of the interstate system Impacts associated with the I-15 Corridor Project will be addressed in the Cumulative Impacts section of this Biological Opinion. Due to the duration of the larger I-15 Corridor Project, and the need to stagger the work along the corridor, this Biological Opinion will analyze the impacts within a three year time frame associated with the proposed Operational Improvement Projects for three areas along I-15 between Mercy Road and Carmel Mountain Road. Impacts associated with these three areas will be compensated for by purchase of 86 acres of habitat near Lake Hodges. By rendering a Biological Opinion on the three Operational Improvements addressed in this document, temporal loss of habitat is minimized, and habitat that is near the impact areas can be secured for conservation to compensate for project impacts to coastal California gnatcatcher.

This Description of the Proposed Action will describe the larger I-15 Corridor Project briefly and then describe in detail the three projects within the Component Three Operational Improvements for which this Biological Opinion addresses.

The I-15 Corridor Project is composed of three (3) components:

1. Component One plans for bridge widening and either a Managed Lanes alternative or High Occupancy Vehicle Lanes alternative.

Bridge widening is proposed at Los Peñasquitos Creek, Green Valley Creek, and Lake Hodges. Timber pilings, cofferdams, and piling driving are planned and construction time is anticipated at nine months for Los Peñasquitos, one year for Green Valley and 18 months for Lake Hodges.

The Managed Lanes Alternative of Component One proposes four managed lanes in the freeway median of I-15 throughout the length of the I-15 Corridor. This alternative requires outside widening of the existing freeway lanes on at least one side and sometimes both. Some additional right-of-way will be required for temporary construction and drainage easements and noise barriers. A moveable barrier system is proposed so that the four lanes can be oriented in three different configurations depending upon traffic needs. Three moveable barrier machines would be needed. A maintenance facility is proposed in the I-15 median or adjacent, about 0.5 miles south of State Route-163 junction. Other important project features are bridge removal and reconstruction at Carroll Canyon Road, I-15/SR-56, Carmel Mountain Road, Duenda Road, Highland Valley Road, Via Rancho Parkway, and Del Lago Blvd. Direct access ramps will be constructed into the Managed Lanes at Hillery Avenue, Del Lago Blvd, and Hale Avenue. Up to 21 noise barriers are proposed throughout the I-15 Corridor. The final number will be determined later in the planning process. Potential impacts to CSS and gnatcatchers have been incorporated in the impact analysis (Table 2).

The High Occupancy Vehicle (HOV) Alternative of Component One proposes four lanes located in the freeway median separated from the main lanes by a painted buffer. This alternative also includes the same direct access ramps, bridge modifications, and noise barriers as the Managed Lanes Alternative.

2. Component Two plans for five Bus Rapid Transit Centers.

Transit Centers are proposed with the I-15 Corridor at Mira Mesa (Hillery Drive), Sabre Springs (off Evening Creek Road), Rancho Bernardo (off West Bernardo Road), Del Lago, and Escondido (from Hale Avenue)[1]. Direct Access Ramps are proposed with each of these Transit Centers. To date, the only biological resource identified that occur on these transit center sites are vernal pools present in the vicinity of Miramar College. These pools may support San Diego fairy shrimp (*Branchinecta sandiegonensis*). The Service will be consulted if impacts may affect listed species.

3. Component three plans for Operational Improvements.

Auxiliary/Added Lanes at seven locations (Table 1). Auxiliary lanes extend between entrances/exits, while "added lanes" continue through an interchange.

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[1] The Metropolitan Transit Development Board is the lead agency for all of the proposed transit centers, with the exception of Del Lago.

Table 1. Seven Proposed Auxiliary/Added Lane Projects in I-15 Corridor, Direction, and Kilopost Limits. Projects are listed from south (list top) to north (bottom). The three projects marked in bold type are considered the Proposed Action for this Biological Opinion.

Direction(s)	KP Limits	Street Location Limits
NB, SB	20.7 – 26.1	Miramar Way – Mira Mesa Blvd
NB, SB	28.2 – 29.0	Mercy Rd – Poway Rd
SB	29.6 – 30.9	Rancho Peñasquitos Blvd – State Route 56
NB, SB	30.6 – 32.0	I-15/ State Route 56 Interchange
NB, SB	31.4 – 35.2	Carmel Mt – Camino Del Norte
NB, SB	38.1 – 42.5	Camino Del Norte – Via Rancho Pkwy
NB, SB	46.7 – 49.6	Citracado Pkwy – Valley Pkwy

Interstate 15/ State Route 56 Interchange and Auxiliary Lanes Improvement:

The main features are a loop on-ramp (eastbound SR-56 to northbound [NB] I-15), retaining walls under the Ted Williams Parkway overcrossing, and a soundwall along NB I-15 and connector from southbound [SB] I-15 to westbound SR-56. In the north part, it is proposed to add one auxiliary lane NB from the NB Carmel Mountain Road off-ramp to the end of the existing truck climbing lane. In the SB direction, the proposal is to add one lane from the SB off-ramp to the existing fifth lane located just south of the I-15/SR-56 Interchange. Noise barriers will be included on both the north- and southbound sides adjacent to the highway shoulder north of SR-56/Ted Williams Parkway to Carmel Mountain Road, and adjacent to residences on the southbound side of I-15 near SR-56. Construction is scheduled to begin in December 2001 and take about one year.

Southbound Auxiliary Lane from State Route 56 to Rancho Peñasquitos Blvd:

An auxiliary lane would be extended about 500 meters (1,700 feet) from the southern end of the I-15/SR-56 Project southward to Rancho Peñasquitos Blvd. Construction is scheduled to begin in December 2001, and be completed within the schedule of the above project.

Auxiliary/Added Lanes from Mercy Road to Poway Road:

In the northbound direction, an additional lane would extend from Scripps Poway Parkway to the Rancho Peñasquitos Creek Bridge. The lane would continue from just northbound of the bridge to Ted Williams Parkway. The bridge across Rancho Peñasquitos Creek would be widened. An auxiliary lane would also extend from Rancho Peñasquitos Blvd to Mercy Road in the southbound direction. Noise barriers are proposed on the southbound side adjacent to the highway shoulder between Rancho Penasquitos Creek and just south of Mercy Road, as well as adjacent to residences just south of Rancho Penasquitos Blvd. Construction is proposed to begin in June 2002, with completion by December 2003.

Conservation Measures

The following conservation measures are proposed to minimize impacts to sensitive habitats and species:

4. Environmentally Sensitive Areas (ESA's) and Limited Use Areas (LUA's) will be designated on project plans. ESA designation will best ensure that no impacts occur outside of the project limits. LUA designation (e.g., equipment storage/staging) will also reduce impacts.

5. Erosion control measures will be incorporated, including use of San Diego sagewort at the Los Peñasquitos Creek Bridge. A Caltrans District Biologist will review the Plans, Specifications, and Estimate (PS&E) Package and attend the pre-job meeting with contractors and Caltrans Resident Engineer to explain mitigation requirements. A biologist will provide technical assistance as needed throughout the construction period. San Diego sagewort will be included in the erosion control seed mix, as recommended in the conceptual mitigation plan (Odgen 2000).

6. Impacted San Diego barrel cactus (*Ferocactus viridescens*) will be salvaged and transplanted in appropriate habitat as recommended in the conceptual mitigation plan (Odgen 2000).

7. To compensate for the three operational improvements described in the proposed action, approximately 92.71 acres of habitat was acquired west of I-15 on the north shore of Lake Hodges. The three parcels (Tax Assessor Parcel Nos. 272-111-16, -20, and 272-252-61) were purchased by Caltrans in February 2001, consisting of about 86 acres of coastal sage scrub (CSS) supporting gnatcatchers. There are also 6.2 acres of southern mixed chaparral habitat on the parcels. Cactus wrens (*Campylorhynchus brunneicapillus*), a California State Species of Special Concern, are also present on the parcels. It is likely these lands would be transferred to a land management agency in the future. Parcels Nos. 272-111-16, -20 are within the Multiple Habitat Planning Area of the City's Multiple Species Conservation Program Subarea Plan.

8. The middle parcel Nos. 272-11-20 has four recorded conservation easements totaling 5.46 acres. The Environmental Trust has agreed to manage this area in perpetuity as open space for biological conservation. These easements are clustered at the southern end of the parcel, and are not considered to significantly detract from use of the parcel as mitigation.

The following conservation measures are proposed to minimize incidental take of gnatcatchers:

9. On the three parcels being set aside to compensate for project impacts, there are about 81 acres of CSS, supporting nine pairs and three individual gnatcatchers. To offset adverse impacts to gnatcatchers, a total of 32.2 acres of CSS (a 2:1 ratio) and six gnatcatcher

territories (1:1 ratio) would be encumbered on the parcels for these three proposed projects. The remainder would be available as mitigation for other projects.

10. Vegetation removal, including clearing and grubbing, will be done between August 31 and February 1, outside of the gnatcatcher nesting season.

The proposed I-15 Operational Improvement Projects will have a major impact on a core California gnatcatcher population and a highly important north/south corridor that facilitates gnatcatcher dispersal. The following conservation measures are proposed to minimize the cumulative impacts of Federal Highways and Caltrans actions to gnatcatchers and sensitive biologic communities:

11. To offset cumulative impacts from the proposed I-15 Operational Improvement Projects, and other future Caltrans and Federal Highways projects in San Diego County, to coastal sage scrub vegetation and the California gnatcatcher, Caltrans and/or Federal Highways may establish a mitigation bank with regional conservation value for the California gnatcatcher. The mitigation bank shall be used to offset total project impacts such that an overall net benefit is achieved for conservation of gnatcatchers and coastal sage scrub vegetation.

12. The mitigation bank shall comply with all Federal Highways and State of California Mitigation Banking policy.

13. The mitigation bank or other acquired land must be located in core gnatcatcher habitat (as defined by the Natural Community Conservation Planning Program and Multiple Species Conservation Program (MSCP) or the Multiple Habitat Conservation Program (MHCP)), and shall provide a significant contribution to regional conservation planning efforts.

14. The mitigation bank or other acquired land site shall have high biological value in the San Diego region for target species and habitats. At a minimum, the site(s) should mitigate at no less than in-kind for the amount and quality of coastal sage scrub habitat to support an equal or greater number of gnatcatcher pairs than is proposed to be impacted.

15. The site shall consist of predominantly high or very high value habitat for migratory birds and other rare animal and plants as evaluated by regional conservation planning efforts and may support wetland habitats.

16. The site shall have a high area-perimeter ratio to minimize edge effects, as well as be connected to other preserved lands.

17. Preservation should be consistent with regional conservation plans. The site should be considered by the Service and California Department of Fish and Game to be at risk of future activities that reduce the biological values that contribute to the MSCP or MHCP conservation planning efforts.

18. The mitigation bank lands shall be acquired, preserved, and managed consistent with a Mitigation Bank Implementing Agreement signed by Caltrans, Federal Highways (if applicable), the Service, and the California Department of Fish and Game, prior to initiation of project impacts for the I-15 Managed/HOV Lanes Projects.

19. A land management agency should be prepared to accept the land and have the fiscal and human resources to monitor and manage it properly in perpetuity.

20. The mitigation bank lands should be a good candidate to manage human access and prevent non-native species establishment.

21. There should not be incompatible management constraints (e.g., easements).

STATUS OF THE SPECIES/CRITICAL HABITAT

Listing Status

The Service listed the coastal California gnatcatcher as threatened on March 30, 1993 (*Federal Register* 58: 16742). Critical habitat was finalized October 24, 2000 (*Federal Register* 65: 63681).

Species Description

The coastal California gnatcatcher is a small (length: 11 centimeters; weight: 6 grams), long-tailed member of the old-world warbler and gnatcatcher family *Sylviidae* (American Ornithologists' Union 1998). The bird's plumage is dark blue-gray above and grayish-white below. The tail is mostly black above and below. The male has a distinctive black cap which is absent during the winter. Both sexes have a distinctive white eye-ring.

The coastal California gnatcatcher is one of three subspecies of the California gnatcatcher (*Polioptila californica*) (Atwood 1991). Prior to 1989, the California gnatcatcher was classified as a subspecies of the Black-tailed gnatcatcher (*Polioptila melanura*). Atwood (1980, 1988) concluded that the species was distinct from *P. melanura*, based on differences in ecology and behavior. Recent mitochondrial DNA sequencing confirmed the species-level recognition of the California gnatcatcher (Zink and Blackwell 1998).

Distribution

Gnatcatchers occur on coastal slopes in southern California, ranging from southern Ventura southward through Palos Verdes Peninsula in Los Angeles County through Orange, Riverside, San Bernardino and San Diego Counties into Baja California to El Rosario, Mexico, at about 30 degrees north latitude (Atwood 1991). In 1990, Atwood reported that ninety-nine percent of all gnatcatcher locality records occurred at or below an elevation of 300 meters (m) (984 feet (ft)). Since that time, additional data collected at higher elevation shows that this species may occur as

high as 912 m (3,000 ft) and that more than 99 percent of the known gnatcatcher locations occurred below 770 m (2,500 ft) (U.S. Fish and Wildlife Service 2000).

Habitat Affinities

Gnatcatchers typically occur in or near coastal sage scrub habitat. Coastal sage scrub is patchily distributed throughout the range of the gnatcatcher, and the gnatcatcher is not uniformly distributed within the structurally and floristically variable coastal sage scrub community. Rather, the subspecies tends to occur most frequently within California sagebrush (*Artemisia californica*)-dominated stands on mesas, gently sloping areas, and along the lower slopes of the coast ranges (Atwood 1990). An analysis of the percent gap in shrub canopy supports the hypothesis that gnatcatchers prefer relatively open stands of coastal sage scrub (Weaver 1998). The gnatcatcher occurs in high frequencies and densities in scrub with an open or broken canopy while it is absent from scrub dominated by tall shrubs and occurs in low frequencies and densities in low scrub with a closed canopy (Weaver 1998). Territory size increases as vegetation density decreases and with distance from the coast, probably due to food resource availability.

Gnatcatchers also use chaparral, grassland, and riparian habitats where they occur adjacent to sage scrub (Campbell *et al.* 1998). The use of these habitats appears to be most frequent during late summer, autumn, and winter, with smaller numbers of birds using such areas during the breeding season. These non-sage scrub habitats are used for dispersal, but data on dispersal use are largely anecdotal (Campbell *et al.* 1998). Linkages of habitat along linear features such as highways and power-line corridors may be of significant value in linking populations of the gnatcatcher (Famolaro and Newman 1998). Although existing quantitative data may reveal relatively little about gnatcatcher use of these other habitats, these areas may be critical during certain times of year for dispersal or as foraging areas during drought conditions (Campbell *et al.* 1998). Breeding territories have also been documented in non-sage scrub habitat. Campbell *et al.* (1998) discuss likely scenarios explaining why habitats other than coastal sage scrub are used by gnatcatchers including food source availability, dispersal areas for juveniles, temperature extremes, fire avoidance, and lowered predation rate for fledglings.

Critical Habitat

On October 24, 2000, the Service published the final determination of critical habitat for the gnatcatcher (*Federal Register* 65: 63680), including 207,868 hectares (ha) (513,650 acres (ac)) of Federal, state, local, and private land in Los Angeles, Orange, Riverside, San Bernardino, and San Diego Counties. Primary constituent elements for the gnatcatcher are those habitat components that are essential for the primary biological needs of foraging, nesting, rearing of young, intra-specific communication, roosting, dispersal, genetic exchange, or sheltering (Atwood 1990). Primary constituent elements are provided in (1) undeveloped areas, including agricultural lands, that support or have the potential to support, through natural successional processes, various types of sage scrub, or (2) undeveloped areas that support chaparral, grassland, or riparian habitats where they occur proximal to sage scrub and where they may be utilized for

the biological needs of dispersal and foraging, and (3) undeveloped areas, including agricultural areas, that provide or could provide connectivity or linkage between or within larger core areas, including open space and disturbed areas that may receive only periodic use.

Life History

The California gnatcatcher is primarily insectivorous, nonmigratory, and exhibits strong site tenacity (Atwood 1990). Diet deduced from fecal samples resulted in leaf- and plant-hoppers and spiders predominating the samples. True bugs, wasps, bees, and ants were only minor components of the diet (Burger *et al.* 1999). Gnatcatcher adults selected prey to feed their young that was larger than expected given the distribution of arthropods available in their environment. Both adults and young consumed more sessile than active prey items (Burger *et al.* 1999).

The California gnatcatcher seems to become highly territorial by late February or early March each year, as males become more vocal during this time period (Mock *et al.* 1990). In southwestern San Diego County the mean breeding season territory size ranged from 5 to 11 ha (12 to 27 ac) per pair and non-breeding season territory size ranged from 5 to 17 ha (12 to 42 ac) per pair (Preston *et al.* 1998). During the nonbreeding season, gnatcatchers have been observed to wander in adjacent territories and unoccupied habitat increasing their home range size to approximately 78 percent larger than their breeding territory (Preston *et al.* 1998).

The breeding season of the gnatcatcher extends from mid-February through the end of August, with the peak of nesting activity occurring from mid-March through mid-May. The gnatcatcher's nest is a small, cup-shaped basket usually found 0.3 to 1 m (1 to 3 ft) above the ground in a small shrub or cactus. Clutch sizes range between three and five eggs, with the average being four. Juvenile birds associate with their parents for several weeks (sometimes months) after fledging (Atwood 1990). Nest building begins in mid-March with the earliest recorded egg date of March 20 (Mock *et al.* 1990). Post-breeding dispersal of fledglings occurs between late May and late November. Nest predation is the most common cause of nest failure (Grishaver *et al.* 1998). Gnatcatchers are persistent nest builders and often attempt multiple broods, which is suggestive of a high reproductive potential. This is, however, typically offset by high rates of nest predation and brood parasitism (Atwood 1990). Nest site attendance by male gnatcatchers was determined to be equal to that of females for the first nest attempt and then decline to almost a third of female nest attendance for later nesting attempts (Sockman 1998).

Gnatcatchers typically live for two to three years, although ages of up to five years have been recorded for some banded birds (Dudek and Associates 2000). Observations indicate that gnatcatchers are highly vulnerable to extreme cold, wet weather (Mock *et al.* 1990). Predation occurs in greater proportion in the upper and lower third of the nest shrub. Predation is lower in nests with full clutch sizes (Sockman 1997). Potential nest predators are numerous, and include snakes, raccoons, and corvids (Grishaver *et al.* 1998). The California gnatcatcher also is known to be affected by nest parasitism of the brown-headed cowbird (*Molothrus ater*). Nest parasitism apparently has resulted in earlier nesting dates of the gnatcatcher which may help compensate for the negative effect of parasitism (Patten and Campbell 1998). However, the gains in nest success

from decreased nest parasitism appear to be negated by increased nest abandonment due to predation before cowbirds have migrated into an area (Braden *et al.* 1997).

The natal dispersal, for a nonmigratory bird, such as the gnatcatcher, is an important aspect of the biology of the species (Galvin 1998). The mean dispersal distance of gnatcatchers banded in San Diego County is reported at less than 3 kilometers (km) (1.9 miles (mi)) (Bailey and Mock 1998). Although the mean dispersal distances that have been documented above are relatively low, dispersal of juveniles is difficult to observe and to document without extensive banding studies. Therefore, it is likely that the few current studies underestimate the gnatcatcher's typical dispersal capacity (Bailey and Mock 1998). Juvenile gnatcatchers are apparently able to traverse highly man-modified landscapes for a least short distances (Bailey and Mock 1998). Natural and restored coastal sage scrub habitat along highway corridors is used for foraging and nesting by gnatcatchers and may serve important dispersal functions (Famolaro and Newman 1998). Typically, however, the dispersal of juveniles requires a corridor of native vegetation which provides foraging and cover opportunities to link larger patches of appropriate sage scrub vegetation (Soule 1991). These dispersal corridors may facilitate the exchange of genetic material and provide a path for recolonization of areas from which the species has been extirpated (Soule 1991, Galvin 1998).

Population Trend

The gnatcatcher was considered locally common in the mid-1940's, but by the 1960's this subspecies had declined substantially in the United States owing to widespread destruction of its habitat (Atwood 1990). By 1980, Atwood (1980) estimated that no more than 1,000 to 1,500 pairs remained in the United States. In 1993, at the time the gnatcatcher was listed as threatened, the Service estimated that approximately 2,562 pairs of gnatcatchers occurred in the United States. In 1997, the total number of gnatcatchers in the United States was estimated at 2,899 pairs with two-thirds occurring in San Diego County (U.S. Fish and Wildlife Service 1996), after subtracting out all gnatcatcher pairs authorized for take under Habitat Loss Permits, approved Natural Community Conservation Plans, Habitat Conservation Plans, and section 7 consultations. These population estimates were intended to represent a coarse approximation of the number of gnatcatchers in southern California. Confidence intervals have not been calculated for these estimates and therefore, we can not be sure of their precision.

Threats

The loss, fragmentation, and adverse modification of habitat are the principal reasons for the gnatcatcher's federally threatened status (*Federal Register* 58: 16742). The amount of coastal sage scrub available to gnatcatchers has continued to decrease during the period after the listing of the species. It is estimated that up to 90 percent of coastal sage scrub vegetation has been lost as a result of development and land conversion (Westman 1981a, 1981b, Barbour and Major 1977), and coastal sage scrub is considered to be one of the most depleted habitat-types in the United States (Kirkpatrick and Hutchinson 1977, O'Leary 1990). The fragmentation of habitat may artificially increase populations in adjacent preserved habitat; however, these population

surpluses may be lost in subsequent years due to crowding and lack of resources (Scott 1993). In addition, agricultural use, such as grazing and field crops, urbanization, air pollution, and the introduction of non-native plants have all had an adverse impact on extant sage scrub habitat. A consequence of urbanization that is contributing to the loss, degradation, and fragmentation of coastal sage scrub is an increase in wildfires due to anthropogenic ignitions. High fire frequencies and the lag period associated with recovery of the vegetation may significantly reduce the viability of affected subpopulations (Dudek and Associates 2000). Furthermore, nest-parasitism by the brown-headed cowbird (Unitt 1984) and nest predation threaten the recovery of the gnatcatcher (Atwood 1980, Unitt 1984).

ENVIRONMENTAL BASELINE

Regulations implementing the Act (50 CFR §402.02) define the environmental baseline as the past and present impacts of all Federal, State, or private actions and other human activities in the action area. Also included in the environmental baseline are the anticipated impacts of all proposed Federal projects in the action area that have undergone section 7 consultation, and the impacts of State and private actions which are contemporaneous with the consultation in progress.

I-15 traverses south to north across gnatcatcher habitat in San Diego and Riverside Counties. In the action area, I-15 bisects an important east-west strip of CSS habitat. This linear stretch of CSS is within the Multiple Species Conservation Program's (MSCP) Multiple Habitat Planning Area and represents one of the largest continuous blocks of habitat in the region. This large patch of habitat serves as a major east-west corridor for wildlife, and the area includes core gnatcatcher populations. Lands to both the east and west of the project area have been planned as preserves in both the City and County of San Diego MSCP Subarea Plans. To the west of the proposed project is permanent open space for the 4-S Ranch totaling 1,877 acres, and the Lake Hodges Segment of the City of San Diego Cornerstone Lands. To the east of the proposed project are the Hodges East Segment of the City of San Diego Cornerstone Lands, and unincorporated lands in the Metro/Lakeside/Jamul Segment of the County's Subarea plan.

The preservation of the lands proposed as mitigation (as described in conservation measure one in the Description of the Proposed Project) contribute to the conservation efforts of the City and County of San Diego to preserve sensitive habitats in the Lake Hodges area. In addition, the purchased lands may contribute to maintaining gnatcatcher demographic and genetic diversity between core populations in San Diego and Riverside Counties along the I-15 cooridor.

Project specific field studies were conducted by Caltrans biologists and Odgen Environmental (and their sub-consultant biologists) from March 1999 to May 2000. Caltrans biologists conducted vegetation mapping from Caltrans Highway Inventory Aerials (1:24,000, black and white, September 1997). Focused gnatcatcher (1999) and Quino surveys (Spring 2000) were done by Robert James (Fish and Wildlife Service Permit No. TE003269-2), in accordance with the latest protocol guidance.

Effects of the action refer to the direct and indirect effects of an action on the species or critical habitat, together with the effects of other activities that are interrelated and interdependent with that action, that will be added to the environmental baseline. Interrelated actions are those that are part of a larger action and depend on the larger action for their justification. Interdependent actions are those that have no independent utility apart from the action under consideration. Indirect effects are those that are caused by the proposed action and are later in time, but are still reasonably certain to occur.

A total of 16.1 acres of coastal sage scrub (CSS) habitat (15 habitat patches) and six territories of coastal California gnatcatchers (*Polioptila californica californica* = gnatcatcher) (two pairs and four individuals) would be affected by construction of the three operational improvements bolded in Table 2. There would be direct loss of habitat likely used by gnatcatchers for breeding, foraging, and sheltering in these areas. The following paragraphs describe the impacts by project feature.

A pair northeast of the I-15/SR-56 Interchange and an individual to the southwest were observed in the direct impact areas for the I-15/SR-56 Interchange and Auxiliary Lane Project (i.e., two territories). A total of 10 CSS habitat patches (consisting of 10.3 acres) would be impacted by construction of the three operational improvements bolded in Table 2. Three of the patches total 8.3 acres (80% of the impact) and are of good habitat quality. The remaining seven areas, totaling 2.0 acres, are within ramp ovals or adjacent to the pavement.

The Auxiliary/Added Lanes from Mercy Road to Poway Road would impact four CSS habitat patches (3.5 acres), supporting three gnatcatcher territories (two on the southbound side, the third on the northbound side). The first patch (0.9acres) is on the northbound side, just north of Poway Road. The habitat quality is fair, with sparse shrub cover, compared to other habitat patches in the area. The remaining areas, all of good quality (fairly dense CSS), are on the southbound side (0.8 acres just north of Los Peñasquitos Creek) and two areas on the slope just south of the Creek (0.8 and 1.0 acres respectively).

The Southbound Auxiliary Lane from State Route 56 to Rancho Peñasquitos Blvd, would impact a single area of CSS habitat (2.2 acres) observed to be used by a pair of gnatcatchers. There would also be direct loss of habitat likely used by gnatcatchers for breeding, foraging, and sheltering in these areas. The shrub cover is dense California sagebrush (*Artemisia californica*) and flat-top buckwheat (*Eriogonum fasciculatum*) and of good quality with few non-native species present.

Planned transportation facilities in the I-15 Corridor with probable Federal involvement that will impact CSS and/or gnatcatchers are listed below in Table 2. Since the biological impacts for all the I-15 Corridor projects will be substantially greater than just the three "operational" projects, the mitigation would need to be correspondingly greater to reflect these associated biological values, and should have increased regional benefit to mitigate for cumulative impacts.

Table 2. I-15 Corridor Project Information and Impacts, by Proposed Project.
Note: The three proposed projects are bolded.

Proposed Projects	CSS (acres)	CSS areas impacted	CAGN territories (pairs, singles)	Planned Construction Dates
[1]ML/HOV Phase 1	12.0	6	2 (2s)	10/2003 - 12/2005
[2]ML/HOV CDN-VR	23.5	18	9 (8p, 1s)	10/2003 - 12/2005
[3]ML/HOV Phase 2	0.2	1	-	2006+
[4]ML/HOV Phase 3	3.4	4	3 (3p)	2006+
[5]ML/HOV BTM Bridge	4.1	1	1 (1s)	10/2003 - 12/2005
ML/HOV Subtotal	43.2	30	15 (11p, 4s)	-
I-15 / SR-56 Interchange	10.3	10	2 (1p, 1s)	8/2002 - 2/2004
Carmel Mt Road	0.8	1	-	12/2001 - 12/2002
SR-56 to Rancho Peñasquitos Blvd	2.2	1	1 (1p)	8/2002 - 2/2004
Poway to Mercy Road	3.5	4	3 (3s)	7/2003 - 12/2004
Miramar Way to Mira Mesa Blvd	5.0	3	-	2/2004- 2/2004
Operational Project Subtotal	21.8	19	6 (2p, 4s)	-
TOTAL	65.0	49	21 (13p, 8s)	

[1]Managed Lanes/High Occupancy Vehicle Lanes from just south of I-15/SR-56 Interchange to Centre City Parkway (central phase)
[2]An operational project between Camino Del Norte to Via Rancho Parkway planned to be built concurrent with ML/HOV Phase 1
[3]From Centre City Parkway to SR-78 (northern phase)
[4]From just south of SR-163 to just south of SR-56 (southern phase)
[5]Barrier Transfer Machine
[6]Coastal California Gnatcatcher

CUMULATIVE EFFECTS

Cumulative effects include the effects of future State, tribal, local or private actions that are reasonably certain to occur in the action area considered in this biological opinion. Future Federal actions that are unrelated to the proposed action are not considered in this section because they require separate consultation pursuant to section 7 of the Act.

The majority of activities anticipated to affect gnatcatchers within the foreseeable future are local urban development projects with no Federal involvement. The conservation of gnatcatchers in the United States is dependent on both the 4 (d) special rule, established at the time of listing, and multi-species Habitat Conservation Plans such as the MSCP, and the Multiple Habitat Conservation Program (MHCP). The 4 (d) rule recognized the State's Natural Community Conservation Planning (NCCP) Program, and several local governments' ongoing multi-species conservation planning efforts that intend to apply Act standards to activities affecting the gnatcatcher. Under the special rule, incidental take of the gnatcatcher by land-use activities

addressed in an approved NCCP Plan would not be considered a violation of section 9 of the Act, provided that the Service determined that the NCCP Plan meets the issuance criteria for an "incidental take" permit, pursuant to section 10(a)(2)(B) of the Act and 50 CFR 17.32(b)(2). Ultimately, conservation of the gnatcatcher is dependent upon implementation of the special rule and HCPs working in concert with conservation on federal lands. Therefore cumulative effects are expected to be minimal with the proper implementation of established conservation strategies.

CONCLUSION

After reviewing the current status of the gnatcatcher, the environmental baseline for the action area, the effects of the proposed operational improvements, and the cumulative effects, it is the Service's biological opinion that the action, as proposed, is not likely to jeopardize the continued existence of the gnatcatcher, and is not likely to destroy or adversely modify designated critical habitat. The Service reached this conclusion for the following reasons:

1) The majority of the CSS habitat impacted is of medium to low quality. Impacted habitats are small oval, or linear, disjunct patches which have high perimeter to area ratios. The majority of impacts will take only portions of gnatcatchers' territories, and this habitat removal will occur outside the breeding season.

2) Impacts to gnatcatchers will be offset by the conservation of occupied habitat inside the Multiple Habitat Planning Area boundary. The CSS which would be preserved is of medium to high quality, occurs in a contiguous habitat patch, connects to other preserved lands, exhibits a low perimeter to area ratio, and will be managed for long-term conservation of the site.

3) To offset cumulative impacts from the proposed I-15 Operational Improvements Projects, Caltrans would establish a mitigation bank or conservation area of regional importance to gnatcatcher populations.

4) Impacts to gnatcathers will be minimized through the implementation of Best Management Practices, such as delineation of Environmentally Sensitive Areas and Limited Use Areas.

INCIDENTAL TAKE STATEMENT

Section 9 of the Act and Federal regulation pursuant to section 4(d) of the Act prohibit the take of endangered and threatened species, respectively, without special exemption. Take is defined as to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, collect, or to attempt to engage in any such conduct. Harm is further defined by the Service to include significant habitat modification or degradation that results in death or injury to listed species by significantly impairing essential behavior patterns, including breeding, feeding, or sheltering. Harass is

defined by the Service as intentional or negligent actions that create the likelihood of injury to listed species to such an extent as to significantly disrupt normal behavior patterns which include, but are not limited to, breeding, feeding, or sheltering. Incidental take is defined as take that is incidental to, and not the purpose of, the carrying out of an otherwise lawful activity. Under the terms of section 7(b)(4) and 7(o)(2), taking that is incidental to and not intended as part of the agency action is not considered to be prohibited taking under the Act provided that such taking is in compliance with the terms and conditions of this Incidental Take Statement.

The measures described below are non-discretionary, and must be undertaken by Federal Highways and Caltrans so that they become binding conditions of any grant or permit issued to the applicant, as appropriate, for the exemption in section 7(o)(2) to apply. Federal Highways and Caltrans have a continuing duty to regulate the activity that is covered by this incidental take statement. If Federal Highways and Caltrans fail to assume and implement the terms and conditions or (2) fail to require the applicant to adhere to the terms and conditions of this incidental take statement through enforceable terms that are added to the permit or grant document, the protective coverage of section 7(o)(2) may lapse. In order to monitor the impact of incidental take, Federal Highways and Caltrans or the applicant must report the progress of the action and its impact on the species to the Service as specified in the incidental take statement. [50 CFR §402.14(i)(3)]

AMOUNT OR EXTENT OF TAKE

Based on Service protocol surveys of the project footprint, the Service anticipates one individual could be harmed as a result of loss of habitat resulting in a significant reduction in the individual's ability to forage. In addition, seven gnatcatchers are expected to be harassed through the loss of small amounts of habitat outside the breeding season which would require the individuals to shift territories.

The Service will not refer the incidental take of any migratory bird or bald eagle for prosecution under the Migratory Bird Treaty Act of 1918, as amended (16 U.S.C. §§ 668-668d), if such take is in compliance with the terms and conditions (including amount and/or number) specified herein.

EFFECT OF THE TAKE

In the accompanying biological opinion, the Service determined that this level of anticipated take is not likely to result in jeopardy to the species or destruction or adverse modification of critical habitat.

REASONABLE AND PRUDENT MEASURES

The Service believes the following reasonable and prudent measure are necessary and appropriate to minimize take of gnatcatchers:

1. The Federal Highways Administration and Caltrans shall adhere to the project description and conservation measures set forth in this Biological Opinion.

2. The Federal Highways Administration and Caltrans shall implement best management practices during construction in order to minimize impacts to gnatcatcher habitat and thereby minimize the amount of take anticipated.

3. The Federal Highways Administration and Caltrans shall use local, endemic plant and seed stock for revegetation efforts resulting from the proposed project. Revegetation efforts shall be intended to recreate the impacted native community. This shall apply to all areas which are temporarily cleared, graded or otherwise adversely impacted by the proposed project. Areas between main lanes of the freeway and ramps, and inside ramp ovals, may be seeded or replanted with non-invasive ornamental plants.

4. To offset impacts from the proposed I-15 Operational Improvement Projects, and other future Caltrans and Federal Highway projects in San Diego County, to coastal sage scrub vegetation and the California gnatcatcher, Caltrans and/or FHWA shall establish a mitigation bank with regional conservation value for the California gnatcatcher.

TERMS AND CONDITIONS

In order to be exempt from the prohibitions of section 9 of the Act, Federal Highways and Caltrans must comply with the following terms and conditions, which implement the reasonable and prudent measures described above and outline required reporting/monitoring requirements. These terms and conditions are non-discretionary.

The Federal Highways and Caltrans shall implement reasonable and prudent measure one through the following terms and conditions:

1.1 The Federal Highways Administration and Caltrans shall implement conservation measures set forth in the project description portion of this Biological Opinion.

The Federal Highways and Caltrans shall implement reasonable and prudent measure two through the following terms and conditions:

2.1 The project work area, consistent with that described in the project description, will be delineated with plastic orange fencing in the field before construction, and a map depicting the project work area will be provided to the construction crews. Construction crews will be educated as to the importance of containing impacts within the project

delineation boundary. Environmentally Sensitive Areas, and Limited Use Areas shall be clearly marked by fencing on site and clearly shown and labeled on the map depicting the project work area. Before construction begins, the Service shall be provided with a copy of the project work area map and education materials presented to the construction crews.

2.2 The Service shall be allowed to access and inspect the project site for compliance with the proposed project description and with the terms and conditions of this biological opinion. Any habitat destroyed that is not in the identified project footprint should be disclosed immediately to the Service for possible reinitiation of consultation. Compensation for such habitat loss will be requested at a minimum ratio of 5:1.

2.3 Before construction begins, a storm water prevention plan (SWPP) shall be developed and implemented to minimize erosion. The Service shall be notified when this plan is developed, and shall be provided with a copy upon request.

The Federal Highways and Caltrans shall implement reasonable and prudent measure three through the following terms and conditions:

3.1 All cut and fill slopes, or other areas which have been cleared of vegetation, shall be seeded or planted with appropriate native, local stock from the immediate area. No exotic, non-native plants shall be used in any portion of this project. The seed list, or planting pallet, as well as revegetaion and monitoring plans, shall be submitted to the Service for approval prior to any revegetation efforts. Areas between main lanes of the freeway and ramps, and inside ramp ovals, may be seeded or replanted with non-invasive ornamental plants.

The Federal Highways and Caltrans shall implement reasonable and prudent measure four through the following terms and conditions:

4.1 Federal Highways and Caltrans shall work in collaboration with the Service and California Fish and Game to identify possible mitigation bank locations. Within three months of this opinion, these parties shall meet to discuss possible mitigation bank site locations.

4.2 Within six months of this opinion, a mitigation site, or sites, shall be identified and agreed upon by Federal Highways, Caltrans, the Service and California Fish and Game. A description of the property, and its conformance with regional conservation planning efforts (such as the MSCP, MHCP), shall be provided to the Service and California Fish and Game. This deadline may be extended with written consent of the Service.

4.3 Within nine months of this opinion, a land management agency shall be identified to accept the land and have the fiscal and human resources to monitor and manage it properly in perpetuity. This land management agency shall be subject to the approval of

the Service and California Fish and Game. This deadline may be extended with written consent of the Service.

4.4 Within one year of this opinion, a draft Mitigation Bank Implementing Agreement shall be in review by Caltrans, Federal Highways (if applicable), the Service, and the California Department of Fish and Game. This deadline may be extended with written consent of the Service.

4.5 Prior to initiation of project impacts for the I-15 Managed/HOV Lanes Project, the final Mitigation Bank Implementing Agreement shall be signed by Caltrans, Federal Highways (if applicable), the Service, and the California Department of Fish and Game.

The Service's Carlsbad Office is to be notified within three working days should any endangered or threatened species be found dead or injured during this project. Notification must include the date, time, and location of the carcass, and any other pertinent information. Dead animals may be marked in an appropriate manner, photographed, and left on-site. Injured animals should be transported to a qualified veterinarian. Should any treated animals survive, the Service should be contacted regarding the final disposition of the animals. The Service contact person is Bill Ostheimer and he may be contacted at the letterhead address or at (760) 431-9440.

The Service believes that no more than one gnatcatcher will be harmed and seven harassed as a result of the proposed action. The reasonable and prudent measures, with their implementing terms and conditions, are designed to minimize the impact of incidental take that might otherwise result from the proposed action. If, during the course of the action, this level of incidental take is exceeded, such incidental take represents new information requiring reinitiation of consultation and review of the reasonable and prudent measures provided. The Federal agency must immediately provide an explanation of the causes of the taking and review with the Service the need for possible modification of the reasonable and prudent measures.

CONSERVATION RECOMMENDATIONS

Section 7(a)(1) of the Act directs Federal agencies to utilize their authorities to further the purposes of the Act by carrying out conservation programs for the benefit of endangered and threatened species. Conservation recommendations are discretionary agency activities to minimize or avoid adverse effects of a proposed action on listed species or critical habitat, to help implement recovery plans or to develop information.

- In an effort to assist with regional conservation, Federal Highways and Caltrans should develop guidelines for planting road cut and fill areas with local endemic species. The Service invites Federal Highways and Caltrans to informal discussions of potential cooperative efforts to ensure the continuation of coastal sage scrub and locally endemic plants along highway corridors.

In order for the Service to be kept informed of actions minimizing or avoiding adverse effects or benefitting listed species or their habitats, the Service requests notification of the implementation of any conservation recommendations.

REINITIATION NOTICE

This concludes formal consultation on the I-15 Operational Improvements at and near the I-15/SR 56 Interchange as outlined in the January 27, 2001, initiation package. As provided in 50 CFR §402.16, reinitiation of formal consultation is required where discretionary Federal agency involvement or control over the action has been retained (or is authorized by law) and if (1) the amount or extent of incidental take is exceeded; (2) new information reveals effects of the agency action that may affect listed species or critical habitat in a manner or to an extent not considered in this opinion; (3) the agency action is subsequently modified in a manner that causes an effect to the listed species or critical habitat not considered in this opinion; or (4) a new species is listed or critical habitat designated that may be affected by the action. In instances where the amount or extent of incidental take is exceeded, any operations causing such take must cease pending reinitiation. If you have any questions or concerns about this biological opinion, please contact Bill Ostheimer of my staff at (760) 431-9440.

Sincerely,

Nancy Gilbert

Nancy Gilbert
Assistant Field Supervisor

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United States Department of the Interior
Fish and Wildlife Service
Ecological Services
Carlsbad Fish and Wildlife Office
6010 Hidden Valley Road
Carlsbad, California 92009

In Reply Refer To:
FWS-SDG-1328.5

JAN 16 2003

Mr. Gary N. Hamby
Division Administrator
U.S. Department of Transportation
Federal Highway Administration
980 Ninth Street, Suite 400
Sacramento, California 95814-2724

Re: Reinitiation of the Biological Opinion for the Proposed Operational Improvements on Interstate 15, San Diego County, California (EA #: 064800)

Dear Mr. Hamby:

This letter is in response to the Federal Highway Administrations (FHWA) request for reinitiation of consultation under section 7 of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C 1531 *et seq.*) on the biological opinion (opinion) for the Interstate 15 (I-15) Operational Improvements Project issued by the U.S. Fish and Wildlife Service (Service) on May 8, 2001. Your letter requesting reinitiation of formal consultation dated December 19, 2002, was received in our office on December 20, 2002. Reinitiation for formal consultation was requested because of an increase in the proposed amount of incidental take of the coastal California gnatcatcher (*Polioptila californica californica*; gnatcatcher) in association with the proposed operational improvements along Interstate 15 (I-15). Besides requesting reinitiation of formal consultation, your letter, dated December 19, 2002, requested our concurrence that the proposed project is not likely to adversely affect the southwestern willow flycatcher (*Empidonax traillii extimus*; flycatcher) or least Bell's vireo (*Vireo bellii pusillus*; vireo). Focused surveys conducted in 1999 and 2001 for the vireo and flycatcher detected seven pairs of vireo and two individual flycatchers ranging in distances of 350 to 1400 feet southeast and northeast of the Lake Hodges Bridge within the action area. However, the conservation measures listed as part of the modified project description will ensure that implementing the proposed Project will result in no adverse effect to the flycatcher and vireo. Therefore, these two species will not be addressed further in this biological opinion, except in the Description of the Proposed Action.

Section 7(a) of the Act requires Federal agencies, including the Service, to ensure that actions they fund, or carry out are not likely to jeopardize the continued existence of a threatened or endangered species, or result in the destruction or adverse modification of critical habitat to the extent that the action appreciably diminishes the value of the critical habitat for the survival and recovery of the species.

Reinitiation of formal consultation is required if: (1) the amount or extent of taking specified in the incidental take statement is exceeded; (2) new information reveals effects of the action that may affect listed species or critical habitat in a manner or to an extent not previously considered; (3) the identified action is subsequently modified in a manner that causes an effect to listed species or critical habitat that was not considered in the biological opinion; or (4) a new species is listed or critical habitat designated that may be affected by the identified action. This document transmits the Service's reinitiated biological opinion based upon our review of the increased take of the gnatcatcher, in accordance with section 7 of the Act.

This reinitiated biological opinion is based on information from your letter requesting reinitiation, dated December 19, 2002; the I-15 Managed Lanes Project Draft Initial Study/Environmental Assessment and Proposed Mitigated Negative Declaration (MND), dated October 2002; and electronic mail messages from Caltrans staff containing information regarding effects of the proposed action.

CONSULTATION HISTORY

Initially, Caltrans and FHWA submitted a request to consult on only a portion of the I-15 operational improvements. However, they were in the planning stages for larger scale improvements to I-15. We requested that they consult on the entire planning scope to ensure that cumulative effects from all improvements to I-15 were addressed. Although the Bus Rapid Transit Centers Project (Transit Centers) and Managed Lanes Project were still in the early planning stages, they were included in the original biological opinion.

The original biological opinion discussed the proposed I-15 operational improvements in three (3) components. The first component provided two alternatives, Managed Lanes and High Occupancy Vehicle Lanes. The MND presents the Managed Lanes as the preferred alternative and is the proposed project discussed in this reinitiated biological opinion. The second component presents two plans for five Transit Centers. The Transit Centers are in the planning stages, and it has not yet been determined whether the implementation of this component will affect federally listed species. The third component presents three plans for operational improvements including: auxiliary/added lane projects from Mercy Road to Poway Road; the I-15/State Route (SR) 56 Interchange; and from Camino Del Norte to Via Rancho Parkway. These three operational improvements are the only projects covered under the incidental take statement in the original opinion.

On May 8, 2001, the Service issued the original opinion (Service Log No. 1-6-01-F-1328.2) which considered impacts of the proposed operational improvements along I-15 on the federally threatened gnatcatcher and its associated habitat.

On November 5, 2002, the Service met with Caltrans staff to discuss the original opinion and the fact that it only provided incidental take for eight gnatcatchers, occupying six territories and 16.1 of the 65 acres of coastal sage scrub to be impacted by the project. Reinitiation was discussed.

On December 2, 2002, the Service submitted a comment letter to Caltrans on the MND for the Project. Included within our comment letter was a recommendation to reinitiate formal section 7 consultation to address impacts to additional gnatcatchers that were not addressed in the original opinion.

On December 11, 2002, the Service met with Caltrans staff to discuss the Service's comments on the MND, FHWA's draft reinitiation letter, and the timeline for reinitiating formal section 7 consultation under the Act. At the meeting, the Service agreed to expedite the biological opinion and provide FHWA with the reinitiated opinion on or before January 15, 2003.

On December 20, 2002, we received an official request for reinitiation of formal section 7 consultation dated December 19, 2002. Your letter, dated December 19, 2002, also requested both concurrence that Terms and Conditions 4.1 and 4.2 of the original biological opinion (opinion) have been completed by Caltrans, and an extension of time to complete Terms and Conditions 4.3, 4.4, and 4.5 of the opinion.

BIOLOGICAL OPINION

DESCRIPTION OF THE PROPOSED ACTION

This reinitiated opinion addresses the Managed Lanes alternative of the first component. Components two (2) and three (3) remain unchanged from the original biological opinion issued on May 8, 2001, and are hereby incorporated by reference. Changes to the project description are reflected by strike-overs, and added language is marked by underlining.

1. Component one plans for bridge widening or replacement and ~~either~~ a Managed Lanes Project ~~alternative or High Occupancy Vehicle Lanes alternative~~.

Bridge widening is proposed at Los Peñasquitos Creek~~,~~ and Green Valley Creek~~, and Lake Hodges~~. Bridge replacement is proposed at Lake Hodges. Timber pilings, cofferdams, and piling driving are planned and construction time is anticipated at nine months for Los Peñasquitos, one year for Green Valley, and 18 months for Lake Hodges.

The Managed Lanes Project is anticipated to be constructed in three phases and is proposed to begin in early 2004. All three phases of construction should be completed by 2010. Phase One extends just south of the I-15/SR56 Interchange, north to Centre City Parkway. Included in Phase One is an operational improvement project between Camino Del Norte, north to Via Rancho Parkway. Phase Two extends from Centre City Parkway north to SR78 in northern San Diego County. Phase Three extends from just south of SR163 to just south of I-15/SR56.

The Managed Lanes Project ~~Alternative~~ of Component One proposes four managed lanes in the freeway median of I-15 ~~throughout the length of the I-15 Corridor~~ from 1.5 miles south of SR 163 in the City of San Diego to 0.3 mile north of SR78 in the City of Escondido for a total project length of 21.1 miles. This alternative will primarily be constructed within the State right-of-way. However, the project also requires outside widening of the existing freeway lanes on at least one side, and sometimes both. Some additional new right-of-way will be required for temporary construction, grading and drainage easements, retaining wall footing, and soil-nail and tieback (wall support) easements ~~and noise barriers~~. Widening will require major cut and fill slopes along the corridor. The project also proposes auxiliary and/or added lanes at various areas within the corridor. Auxiliary lanes are lanes that extend from one intersection to the next, while added lanes extend through intersections.

The cross-section for the managed lanes will include standard portland cement concrete (PCC) lanes and PCC shoulders for all widening and new construction. Standards for new construction on freeways is 12 feet for lanes and 10 feet for shoulders. There will be some exceptions to using PCC lanes and shoulders in order to match existing lanes. The managed lanes will be separated from the general use freeway lanes by a type 60 series median concrete barrier. A standard lane width of 14 feet will be used to allow for half of the 2 foot width of the moveable concrete barrier. This allows for a 13 foot lane width adjacent to the moveable barrier.

A moveable barrier system is proposed so that the four lanes can be oriented in three different configurations depending upon traffic needs, with access openings occurring at two to three mile intervals. The three configurations are: (1) one lane northbound and three lanes southbound; (2) two lanes northbound and two lanes southbound; and (3) three lanes northbound and one lane southbound. ~~Three~~ Two moveable barrier machines will be needed to rearrange the barriers and a third machine would be available when a machine is under maintenance. A new overcrossing structure will be required south of "H" Avenue for the machines so they can move from the median to the west side of I-15.

A maintenance facility is proposed in the I-15 median or adjacent, about 0.5 mile south of SR163 junction. Other important project features are bridge removal and reconstruction at Carroll Canyon Road, I-15/SR56, Carmel Mountain Road, Duenda Road, Highland Valley Road, Via Rancho Parkway, and Del Lago Blvd. Two types of access into and out of the managed lanes will be incorporated. The first type would be called intermediate access points (IAP). The IAP are access points that are at grade and adjacent to the freeway main lanes. To exit the managed facility using the IAP, traffic will enter a dedicated weaving lane of 305 to 610 meters (100 to 2000 feet) in length, depending on traffic volumes. Traffic will then weave from this lane into the fast lane on the freeway main lanes. The other type of access is a Direct Access Ramp (DAR) into the managed lanes from a grade separated interchange. The proposed DARs have been located to accommodate the Transit System proposed by Metropolitan Transit Development ~~Board. The DAR will also encourage carpooling by offering easy access into the managed lanes~~ as an incentive for carpooling. The proposed DARs ~~Direct access ramps~~ will be constructed into the managed lanes at Hillery Avenue, Sabre Springs, Del Lago Blvd., Rancho Bernardo, and Hale Avenue.

Up to 21 noise barriers are proposed throughout the I-15 Corridor. The final number will be determined later in the planning process. Ramp realignments will be required at several locations to accommodate additional widening on these ramps and to accommodate widening of the main lanes. Road widening will require utilities to be relocated in several locations and will extend outside of the State right-of-way. Existing drainage culverts will need to be extended, replaced or lined depending on their condition. Potential impacts to CSS and gnatcatchers have been incorporated in the impact analysis (Table 2 of the original biological opinion).

~~The High Occupancy Vehicle (HOV) Alternative of Component One proposes four lanes located in the freeway median separated from the main lanes by a painted buffer. This alternative also includes the same direct access ramps, bridge modifications, and noise barriers as the Managed Lanes Alternative.~~

Conservation Measures

The original biological opinion contains detailed conservation measures four (4) through twenty-one (21). Conservation measures four (4) through eight (8) and ten (10) remain unchanged and are hereby incorporated by reference. Conservation measure nine (9) is changed below. In addition, conservation measures eleven (11) through twenty-one (21) discuss the need for FHWA and California Department of Transportation (Caltrans) to acquire a regionally significant conservation bank that meets a list of requirements. Caltrans has purchased the 231.43 acre Bonita Meadows Open Space Preserve (Bonita Meadows) to satisfy conservation measures eleven (11) through twenty-one (21). Conservation measures eleven (11) through twenty-one (21) are no longer valid and are replaced below:

9. ~~On the three parcels being set aside to compensate for project impacts, there are about 81 acres of CSS, supporting nine pairs and three individual gnatcatchers. To offset adverse impacts to gnatcatchers, a total of 32.2 acres of CSS (a 2:1 ratio) and six gnatcatcher territories (1:1 ratio) would be encumbered on the parcels for these three proposed projects. The remainder would be available as mitigation for other projects.~~

   To offset impacts to 65 acres of coastal sage scrub (CSS) and twenty-one (21) gnatcatcher territories (13 pairs and 8 individuals) from all Operational Improvement Projects including the Managed Lanes Project, a total of 130 acres (a 2:1 ratio) of CSS and fourteen gnatcatcher territories will be encumbered on both the Walsh properties at Lake Hodges and at Bonita Meadows. To satisfy this requirement, 78.3 acres of CSS at the Walsh properties and 51.7 acres of CSS at Bonita Meadows will be debited. In addition a total of 20 territories (16 pairs and 4 individuals) will be debited from the two properties. A 1:1 ratio for gnatcatcher territories would require an additional territory. However, the territories of 16 pairs and 4 individuals includes an actual increase in the number of birds since three pairs will be available to offset four of the individual gnatcatcher territories affected.

11. The 130 acres of CSS and the twenty (20) territories utilized to offset impacts to gnatcatchers are part of larger parcels that will be preserved in perpetuity and managed for the long term preservation of core gnatcatcher populations.

The following conservation measures are proposed to avoid incidental take of the southwestern willow flycatcher and the least Bell's vireo:

12. All pile driving occurring during construction will be conducted outside the breeding season for the flycatcher and vireo. The breeding season for the flycatcher and vireo begins March 15 and continues through September 30.

13. Protocol level surveys for the flycatcher and vireo will be conducted in spring of 2003, and the spring of each following year that Lake Hodges Bridge construction continues. Protocol level surveys will be conducted in all areas surrounding the Lake Hodges Bridge that are currently supporting habitat and were not included in the original surveys because the lake-bed was inundated.

14. If flycatchers and/or vireos are detected in the habitat adjacent to the proposed project at Lake Hodges, noise attenuation structures will be constructed, if necessary, to reduce noise levels to 60 d(B)A Leq or existing ambient noise levels if ambient noise levels exceed 60 d(B)A Leq. Those construction activities that are creating the excess noise (> 60 d(B)A Leq or ambient noise levels) will cease operation until effective noise attenuation structures are in place.

15. Any night lighting for the construction of the Lake Hodges Bridge will be selectively placed, shielded, and directed away from all suitable habitat.

STATUS OF THE SPECIES/CRITICAL HABITAT

The original biological opinion contains a detailed status of the species/critical habitat for the gnatcatcher, and is hereby incorporated by reference.

ENVIRONMENTAL BASELINE

Changes in the environmental baseline from the original biological opinion are reflected by strike-overs, and added language is marked by underlining.

Regulations implementing the Act (50 CFR §402.02) define the environmental baseline as the past and present impacts of all Federal, State, or private actions and other human activities in the action area. Also included in the environmental baseline are the anticipated impacts of all proposed Federal projects in the action area that have undergone section 7 consultation, and the impacts of State and private actions which are contemporaneous with the consultation in progress.

I-15 traverses south to north across gnatcatcher habitat in San Diego and Riverside Counties. In the action area, I-15 bisects an important east-west strip of CSS habitat. This linear stretch of CSS is within the Multiple Species Conservation Program's (MSCP) Multiple Habitat Planning Area and represents one of the largest continuous blocks of habitat in the region. This large patch of habitat serves as a major east-west corridor for wildlife, and the area includes core gnatcatcher populations. Lands to both the east and west of the project area have been planned as preserves in both the City and County of San Diego MSCP Subarea Plans. To the west of the proposed project is permanent open space for the 4-S Ranch totaling 1,877 acres, and the Lake Hodges Segment of the City of San Diego Cornerstone Lands. To the east of the proposed project are the Hodges East Segment of the City of San Diego Cornerstone Lands, and unincorporated lands in the Metro/Lakeside/Jamul Segment of the County's Subarea plan.

The preservation of the lands proposed as mitigation (as described in conservation measures seven (7) through nine (9) ~~one~~ in the Description of the Proposed Project in the original and this reinitiated biological opinion) contribute to the conservation efforts of the City and County of San Diego to preserve sensitive habitats in the Lake Hodges area. In addition, the purchased lands may contribute to maintaining gnatcatcher demographic and genetic diversity between core populations in San Diego and Riverside Counties along the I-15 corridor.

Gnatcatcher surveys conducted between 1999 and 2000, found a number of patches of occupied CSS occur along the proposed I-15 Managed Lanes corridor. Many of these patches are

contiguous and/or adjacent to larger CSS communities. The Managed Lanes Project will directly affect fifteen (15) gnatcatcher territories. Many of these territories are adjacent to and/or part of larger core populations of gnatcatchers. Since gnatcatcher surveys for the Managed Lanes Project focused on the area of direct impact, the actual population sizes of gnatcatchers along the corridor is unknown.

Project specific field studies were conducted by Caltrans biologists and Odgen Environmental (and their sub-consultant biologists) from March 1999 to May 2000. Caltrans biologists conducted vegetation mapping from Caltrans Highway Inventory Aerials (1:24,000, black and white, September 1997). Focused gnatcatcher (1999) and Quino surveys (Spring 2000) were done by Robert James (Fish and Wildlife Service Permit No. TE003269-2), in accordance with the latest protocol guidance.

EFFECTS OF THE ACTION

Except as discussed below, the effects of the action remain the same as discussed in the original biological opinion, which is hereby incorporated by reference.

The Operational Improvements Projects (Managed Lanes Project and the operational improvement projects) will impact 65 acres of CSS containing 21 gnatcatcher territories (13 pairs and 8 individuals). The original biological opinion discussed the effects to 6 territories (2 pairs and individuals) occupying 16 acres of CSS. The direct effects to 49 acres of CSS covering 15 territories (11 pairs and 4 individuals) from the Managed Lanes Project and two operational improvement projects not discussed in the effects section of the original biological opinion are discussed below.

A total of 43.2 acres of CSS and fifteen (15) gnatcatcher territories (seven pairs and eight individuals) would be affected by the construction of the Managed Lanes Project. There would be a direct loss of habitat likely used by gnatcatchers for breeding, foraging, and sheltering in these areas. Large CSS areas subject to permanent impacts are north of both sides of Mira Mesa Boulevard to Mercy Road/Scripps Poway Parkway ("Mercy Hill"), near the I-15/SR-56 Interchange, south of Carmel Mountain Road (northbound side), and near Lake Hodges and the Green Valley Creek Bridge. The habitat quality is variable from densely vegetated, high-quality gnatcatcher habitat to more sparse CSS without gnatcatchers.

Phase One will impact 12 acres of CSS within six CSS patches that support two gnatcatcher territories comprised of two single individuals.

- The first habitat patch is located on the southbound side of I-15, just north of Mira Mesa, and consists of 1.5 acres of CSS. No gnatcatchers were observed in this patch during biological surveys conducted for this project.

- The second habitat patch is located to the north of the I-15/SR 56 Interchange, on the northbound side of I-15 and consists of 4.5 acres of CSS. One pair of gnatcatchers was observed adjacent to this location, but will not be directly impacted by the Managed Lanes Project.

- The third and fourth CSS habitat patches are located just north of Carmel Mountain Road, on the southbound side of I-15, and consist of 2.7 and 1.1 acres. No gnatcatchers were observed in these two patches during biological surveys conducted for this project.

- One single gnatcatcher was observed on the southbound side of I-15, just south of West Bernardo Road. Although no CSS will be permanently disturbed, this gnatcatcher will be affected by construction of Phase One of the Managed Lanes Project.

- The remaining habitat patches that will be impacted consist of 1.7 and 0.5 acres of CSS and are located just north of Del Lago Drive on both the north and southbound sides of I-15. One male gnatcatcher was observed in the CSS patch located on the northbound side of I-15 adjacent to Kit Carson Park.

The operational improvements between Camino Del Norte and Via Rancho Parkway (CDN-VR) that are included as part of Phase One will impact 23.5 acres of CSS in 18 patches that support nine gnatcatcher territories (eight pairs and one individual).

- The first four CSS patches (1.8, 1.0, 2.6, and 0.8 acres) are found on both the north and southbound sides of I-15, just north of Camino del Norte. No gnatcatchers were observed within these patches during biological surveys conducted for this project.

- Three CSS habitat patches are found adjacent to the Bernardo Center Drive Overcross. One area consisting of 0.5 acre is located on the southbound side of I-15, to the south of Bernardo Center Drive. The remaining two are small, linear patches (1.0 and 0.5 acre) found within the median of I-15 from Bernardo Center Drive extending to the north. No gnatcatchers were observed within these patches during biological surveys conducted for this project.

- ~~Two small CSS habitat patches consisting of 0.1 and 0.3 acre are located on both the~~ south and northbound side of I-15 adjacent to the Duenda Road Overcross. No gnatcatchers were observed within these patches during biological surveys conducted for this project.

- Three of the largest CSS habitat patches (0.5, 3.2, and 1.6 acres) impacted by the Managed Lanes project are located adjacent to Green Valley Creek Bridge on both the north and southbound sides of I-15. All three of these habitat patches consist of good habitat quality CSS. Approximately three pairs and one single gnatcatcher were observed within these three CSS habitat patches.

- Four CSS habitat patches are found adjacent to the Highland Valley Road/West Bernardo Road Interchange. Three of these, consisting of 1.1, 6.5, and 0.1 acres, are located to the south of the interchange. A total of two pairs and one family of gnatcatchers were observed within these three habitat patches. The remaining CSS habitat patch (0.7 acre) is located on the northbound side of I-15, north of the interchange. One single gnatcatcher was observed at the far eastern portion of this patch, but outside the limits of impacts.

- The remaining two CSS habitat patches impacted are located to the north of Lake Hodges and consist of 0.6 and 0.6 acres, respectively. Two pairs of gnatcatchers were observed within each of these patches.

Phase Two will impact 0.2 acre of CSS in one patch on the northbound side of I-15 north of Centre City Parkway. No gnatcatchers were observed within this habitat patch during surveys.

Phase Three will impact 3.4 acres of CSS in four patches and three gnatcatcher territories consisting of three pairs.

- Two of the CSS patches (0.5 and 0.4 acre) are located on the southbound side of I-15, south of Miramar Way. One pair of gnatcatchers was observed within the 0.5-acre patch.

- The third habitat patch (1.7 acres) is located on the southbound side of I-15 to the north of Miramar Way and is comprised of roughly equal amounts of drought-deciduous sage scrub species and woody chaparral species. No gnatcatchers were observed within this habitat patch during biological surveys conducted for this project.

- The last CSS habitat patch (0.8 acre) is located on the southbound side of I-15, north of Mira Mesa Boulevard, to the southwest of Mercy Hill. One pair of gnatcatchers was observed within this 0.8-acre patch. An additional pair of gnatcatchers included in the impacts for Phase Three of the Managed Lanes Project is located on the northbound side of I-15, east of Mercy Hill.

The proposed Managed Lanes Barrier Transfer Machine Bridge will be located just south of "H" Street, adjacent to the I-15/SR163 Interchange. One single gnatcatcher was observed within a 4.1 acre patch of CSS within the direct impact area..

~~Additional effects to gnatcatchers not discussed in the original biological opinion include two~~ operational improvement projects at Carmel Mt. Road and between Miramar Way and Mira Mesa Blvd. Approximately 0.8 acre of CSS on the northbound side of I-15, north of Carmel Mt. Road will be directly impacted by the proposed Auxiliary Lane Project. No gnatcatchers were observed within this habitat patch during biological surveys conducted for this project. The Auxiliary/ Added Lanes from Miramar Way to Mira Mesa Boulevard would impact three CSS habitat patches (5 acres). The first patch (0.7 acre) is on the southbound side of I-15, west of the southbound on-ramp to I-15 from Miramar Road. The remaining two patches (3 and 1.3 acres) are on the northbound and southbound sides of I-15, just north of the Carroll Canyon road. The habitat quality of these two patches is fair, with sparse shrub cover, compared to other habitat patches in the area. No gnatcatchers were observed within any of the three habitat patches during biological surveys conducted for his project.

CUMULATIVE EFFECTS

The cumulative effects remain unchanged from the original biological opinion, and are hereby incorporated by reference.

CONCLUSION

After reviewing the current status of the gnatcatcher, the environmental baseline for the action area, the effects of the proposed operational improvements, and the cumulative effects, it is the Service's biological opinion that the action, as proposed, is not likely to jeopardize the continued existence of the gnatcatcher, and is not likely to destroy or adversely modify designated critical habitat. The Service reached this conclusion for the following reasons:

1) The majority of the CSS habitat impacted is of medium to low quality. Impacted habitats are small oval, or linear, disjunct patches which have high perimeter to area ratios. The majority of impacts will take only portions of gnatcatchers' territories, and this habitat removal will occur outside the breeding season.

2) Impacts to gnatcatchers will be offset by the conservation of occupied habitat inside the Multiple Habitat Planning Area boundary. The CSS which would be preserved is of medium to high quality, occurs in a contiguous habitat patch, connects to other preserved lands, exhibits a low perimeter to area ratio, and will be managed for long-term conservation of the site.

3) To offset cumulative impacts from the proposed I-15 Operational Improvements Projects, Caltrans has purchased the 231.43 acre Bonita Meadows property and established Bonita Meadows as a conservation area of regional importance to gnatcatcher populations.

4) Impacts to gnatcatchers will be minimized through the implementation of Best Management Practices, such as delineation of Environmentally Sensitive Areas and Limited Use Areas.

INCIDENTAL TAKE STATEMENT

The incidental take statement remains the same as specified in the original biological opinion, and is hereby incorporated by reference, except as amended below.

AMOUNT OR EXTENT OF TAKE

~~Based on Service protocol surveys of the project footprint, the Service anticipates one individual could be harmed as a result of loss of habitat resulting in a significant reduction in the individual's ability to forage. In addition, seven gnatcatchers are expected to be harassed through the loss of small amounts of habitat outside the breeding season which would require the individuals to shift territories.~~

The Service anticipates the following levels of take could occur as a result of the proposed Operational Improvements including the Managed Lanes Project:

1. Thirteen (13) pairs and eight (8) individual coastal California gnatcatchers may be harmed by the removal of 65 acres of gnatcatcher habitat.

EFFECT OF THE TAKE

The effect of the take remains unchanged from the original biological opinion, and is hereby incorporated by reference.

REASONABLE AND PRUDENT MEASURES

The reasonable and prudent measures remain unchanged from the original biological opinion, and are hereby incorporated by reference:

TERMS AND CONDITIONS

The terms and conditions remain the same as specified in the original biological opinion, and are hereby incorporated by reference, except as amended below.

2.4 All lighting for nighttime construction activities will be selectively placed, shielded, and directed away from all adjacent habitat.

4.1 ~~Federal Highways and Caltrans shall work in collaboration with the Service and California Fish and Game to identify possible mitigation bank locations. Within three months of this opinion, these parties shall meet to discuss possible mitigation bank site locations.~~ FHWA and Caltrans have identified and purchased the 231.43 acre Bonita Meadows property as a regionally significant conservation area to offset impacts to gnatcatchers.

4.2 ~~Within six months of this opinion, a mitigation site, or sites, shall be identified and agreed upon by Federal Highways, Caltrans, the Service and California Fish and Game. A description of the property, and its conformance with regional conservation planning efforts (such as the MSCP, MHCP), shall be provided to the Service and California Fish and Game. This deadline may be extended with written consent of the Service.~~ Bonita Meadows, a regionally significant conservation area, has been purchased to protect a core population of gnatcatchers.

4.3 ~~Within nine months of this opinion, a land management agency shall be identified to accept the land and have the fiscal and human resources to monitor and manage it properly in perpetuity. This land management agency shall be subject to the approval of the Service and California Fish and Game. This deadline may be extended with written consent of the Service.~~ Caltrans has identified the California Department of Fish and Game (CDFG) and San Diego County (County) as potential managers of Bonita Meadows. Caltrans prefers to transfer the site to CDFG with the County being the second option. Caltrans will complete two seasons of surveys for natural resources before developing and forwarding Preliminary Terms of Agreement to CDFG or the County for review.

4.4 ~~Within one year of this opinion~~By September 30, 2003, a draft Mitigation Bank Implementing Agreement shall be in review by Caltrans, Federal Highways (if applicable), the Service, and the California Department of Fish and Game. ~~This deadline may be extended with written consent of the Service.~~

4.5 ~~Prior to initiation of project impacts for the I-15 Managed/HOV Lanes Project, t~~The final Mitigation Bank Implementing Agreement shall be signed by Caltrans, Federal Highways (if applicable), the Service, and the California Department of Fish and Game no later than the end of March 2004.

4.6 Caltrans and FHWA shall provide annual reports regarding the status of the Walsh Property and Bonita Meadows Conservation Area. These reports shall include information regarding mitigation debits and remaining credits, species status, restoration updates and overall site conditions.

Reporting Requirements

In order to demonstrate compliance with the foregoing Terms and Conditions, FHWA, or its designated contact, shall submit an annual report to the Service that describes and summarizes the implementation of the proposed project and its associated conservation measures.

Disposition of Sick, Injured, or Dead Specimens

The Service's Division of Law Enforcement, San Diego, California (619) 557-5063 is to be notified within three working days should any vireo(s), flycatcher(s), or gnatcatcher(s) be found sick, injured, or dead in the project area. The Service's Carlsbad Fish and Wildlife Office should be notified concurrently at (760) 431-9440. Written notification to both offices must be made within five calendar days and include the collection date and time, location of the bird(s), and any other pertinent information. Care must be taken in handling sick or injured bird(s) to ensure effective treatment and care. And in handling dead specimens to preserve biological material in the best possible state. The remains of intact vireo(s), willow flycatcher(s), or gnatcatcher(s) shall be placed with educational or research institutions holding appropriate State and Federal permits.

CONSERVATION RECOMMENDATIONS

The conservation recommendations remain unchanged from the original biological opinion, and are hereby incorporated by reference.

REINITIATION NOTICE

The reinitiation notice remains unchanged from the original biological opinion, and is hereby incorporated by reference.

Should you have any questions regarding our comments, please call John DiGregoria of my staff at (760) 431-9440.

Sincerely,

For Peter Sorensen
Acting Assistant Field Supervisor

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 497L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Revised Foundation Recommendations

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number and to change the name from Retaining Wall #496L to Retaining Wall # 497L. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a foundation investigation and recommendations for the proposed Retaining Wall #496L, located on southbound Route 15, north of 9th Avenue Undercrossing. This report is based on Layouts provided by the district dated 10/06/06. The foundation study consisted of: a field investigation including 1 rotary mud boring, 4 CPT borings, and a review of "As-built" plans for nearby bridge 9th Avenue Undercrossing. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on State Route 15 north of 9th Avenue Undercrossing on the right shoulder of southbound Route 15. The proposed retaining wall is Type 1, 243m long, with a maximum height of 5.7m, and an average height of approximately 3.5m.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic

rocks of the Southern California batholith. The second unit is composed of mostly medium to coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

Current borings and as-built data from nearby structure, 9th Avenue Undercrossing (Br #57-0808) indicate these sites are underlain by dense to very dense alluvium overlaying native granitic rock in various states of decomposition to the depth of the borings. The alluvium consists of dense, sandy silt and hard sandy clay.

Groundwater was recorded in two borings on the as-built LOTBs. Groundwater was encountered at approximate elevation of 213m and 208.7m in February 1970. It is not anticipated that groundwater will be encountered during construction of the retaining wall, however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #496L, as shown on the retaining wall Layout plans dated 10/06/06. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified maximum wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #496L Spread Footing Data

Approximate Support Location of Retaining Wall #496L on the "SD-15L" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}^*)
21 m LT Sta 495+57 to 21 m LT Sta 498+00	Type 1	6100 mm	3350 mm	230 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date: 10/29/07

GINA W. PURSELL, R.C.E. # 54512
Associate Materials & Research Engineer
Office of Geotechnical Design – South 2, Branch C

REGISTERED PROFESSIONAL ENGINEER
GINA W. PURSELL
No. 54512
Exp. 12-31-07
CIVIL
STATE OF CALIFORNIA

c: R.E. Pending File
OGDS2 - SWei
Geology – North
Geology - South

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 498L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Revised Foundation Recommendations

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a foundation investigation and recommendations for the proposed Retaining Wall #498L, located on Route 15, between 9th Avenue and Valley Parkway. This report is based on Layouts provided by the district dated 10/06/06. The foundation study consisted of: a field investigation including 2 rotary mud borings and 7 CPT borings, and a review of "As-built" plans for nearby bridges- 9th Avenue Undercrossing and Valley Parkway Undercrossing. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on State Route 15 between 9th Avenue Undercrossing and Valley Parkway Undercrossing on the right shoulder of southbound Route 15. The proposed retaining wall is Type 1, 546 m long, with a maximum height of 6.7m, and an average height of approximately 5.5m.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic rocks of the Southern California batholith. The second unit is composed of mostly medium to

coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

As-built data from two nearby structures, 9th Avenue Undercrossing (Br #57-0808R/L) and Valley Parkway Undercrossing (Br #57-0809R/L) indicate these sites are underlain by dense alluvium overlaying native granitic rock in various states of decomposition to the depth of the borings. The alluvium consists of dense sandy silt and coarse sand with occasional gravel lenses. The field data gathered February and March 2007 from the 2 rotary borings and 7 CPT borings indicate a layer of fill varying from 15 to 25 ft thick overlaying the alluvium. This fill is described as; well graded fine to coarse sand with clay (SW), dense, brown to dark brown, dry, micaceous.

Groundwater was not reported on any of the as-built LOTB nor was groundwater reported in the current field investigations. It is not anticipated that groundwater will be encountered during construction however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #498L, as shown on the retaining wall Layout plans dated 10/06/06. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified maximum wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #510 Spread Footing Data

Approximate Support Location of Retaining Wall #498L on the "R-5NA" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}^*)
6 m LT Sta 497+51.117 to 6 m LT Sta 502+96.641	Type 1	6700 mm	3700 mm	260 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date:

10/29/07

GINA W. PURSELL, R.C.E. # 54512
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Office of Geotechnical Design – South 2, Branch C

REGISTERED PROFESSIONAL ENGINEER
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c: R.E. Pending File
OGDS2 - SWei
Geology – North
Geology - South

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 506R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Revised Geotechnical Design Recommendations

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a geotechnical design investigation and recommendations for the proposed Retaining Wall #506R, located on Route 15, between Escondido Flood Control Channel and Valley Parkway. This report is based on Layouts provided by the district dated 05/27/07. The Geotechnical design study consisted of as-built plans for nearby bridge Escondido Flood Control Channel. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on the right shoulder of the northbound onramp of Valley Parkway and Route 15, between Escondido Flood Control Channel and Valley Parkway. The proposed retaining wall is Type 1, approximately 14.6m long, with a maximum height of 6100mm, and a minimum height of approximately 1200mm.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic rocks of the Southern California batholith. The second unit is composed of mostly medium to coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

As-built data from nearby structure, Escondido Flood Control Channel (Br #57-0810R/L) indicate this site is underlain by medium dense alluvium overlaying native granitic rock in various states of decomposition to the depth of the borings. The alluvium consists of medium dense sandy silt, silty sand, with occasional coarse sand lenses.
Groundwater was encountered in the 1970 and the 2006 subsurface exploration for Escondido Flood Control Channel. The groundwater elevation was reported varying from 187.4m to 187.6m in the 1970 borings and 188.3m to 188.5m in the 2006 borings. It is not anticipated that groundwater will be encountered during construction of the retaining wall however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #506R, as shown on the Retaining Wall Plans dated 05/27/07. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified maximum wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #506R Spread Footing Data

Approximate Support Location of Retaining Wall on the "R-1VA" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult})
7.2m RT Sta 505+13.8 to 6.9m LT Sta 506+28.4	Type 1	6100 mm	3350 mm	230 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date:

G. W. Pursell 10-29-07

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Office of Geotechnical Design – South 2, Branch C

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Geology – North
Geology - South

State of California Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 507R1

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Foundation Recommendations Revision

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a foundation investigation and recommendations for the proposed Retaining Wall #507R1, located on Route 15, between Valley Parkway and Hale Avenue. This report is based on Layouts provided by the district dated 10/06/06. The foundation study consisted of a review of "As-built" plans for nearby bridges; Valley Parkway Undercrossing and Hale Avenue Undercrossing. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on State Route 15 between Valley Parkway and Hale Avenue, on the right shoulder of northbound Route 15. The proposed retaining wall is Type 1, 224m long, with a maximum height of 5.5m, and an average height of approximately 4.9m.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic rocks of the Southern California batholith. The second unit is composed of mostly medium to coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

As-built data from two nearby structures, Valley Parkway Undercrossing (Br #57-0809R/L) and Hale Avenue Undercrossing (Br #57-0811) indicate these sites are underlain by dense alluvium overlaying native granitic rock in various states of decomposition to the depth of the borings. The alluvium consists of dense clayey sandy silt, clayey sand and silty sand with occasional gravel lenses/pockets.

Groundwater was reported in the 1972 Hale Avenue Undercrossing subsurface exploration. The reported groundwater elevation ranged from 188.4m to 187.5m. Groundwater is not anticipated to be encountered during construction of the retaining wall however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #507R1, as shown on the retaining wall Layout plans dated 10/06/06. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified maximum wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #507R1 Spread Footing Data

Approximate Support Location of Retaining Wall #507R1 on the "SD-15R" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}*)
28.34 m RT Sta 506+96 to 24.599 m RT Sta 509+20.096	Type 1	5500 mm	3050 mm	205 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^*).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date:

G. W. Pursell 10/29/07

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State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 509L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Foundation Recommendations Revision

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a foundation investigation and recommendations for the proposed Retaining Wall #509L, located on Route 15, between Hale Avenue and Santa Fe Avenue. This report is based on Layouts provided by the district dated 10/06/06. The foundation study consisted of as-built plans for nearby bridges Hale Avenue and Santa Fe Avenue. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on State Route 15 between Hale Avenue and Santa Fe Avenue on the right shoulder of southbound Route 15. The proposed retaining wall is Type 1, 208m long, with a maximum height of 6.1m, and an average height of approximately 4.9m.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic rocks of the Southern California batholith. The second unit is composed of mostly medium to coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

As-built data from two nearby structures, Hale Avenue Undercrossing (Br #57-0811) and Santa Fe Overhead (Br #57-0812) indicate these sites are underlain by dense alluvium overlaying native granitic rock in various states of decomposition to the depth of the borings. The alluvium consists of dense clayey sandy silt, clayey sand, sandy silt with coarse sand with occasional gravel lenses. Groundwater was encountered in the 1972 subsurface exploration for Hale Avenue Undercrossing. The groundwater elevation was reported varying from 188.4m to 187.5m. It is not anticipated that groundwater will be encountered during construction of the retaining wall however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #509L, as shown on the retaining wall Layout plans dated 10/06/06. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified maximum wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #509L Spread Footing Data

Approximate Support Location of Retaining Wall #509L on the "SD-15L" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}^{*})
24.6 m LT Sta 509+33.0 to 24.6 m LT Sta 511+41.344	Type 1	6100 mm	3350 mm	230 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^{*}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^{*}).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date:

10/29/07

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Geology - South

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. DAVID STEBBINS – D11
Department of Transportation
Office of Design

Date: October 29, 2007

File: 11-SD-15-KP 48.3/50.1
11-2T0821

Attention: Mr. Dimitar Peev

Retaining Wall # 510R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2, Branch C

Subject: Revised Foundation Recommendations

This Revision is written in response to a request dated October 26, 2007 from District 11 design, to change the expenditure authorization number. This report shall supersede any previous recommendations for this structure.

As requested, our Office has completed a foundation investigation and recommendations for the proposed Retaining Wall #510R, located on Route 15, northbound, between Hale Avenue and Washington Avenue. This report is based on Layouts dated 10/06/06, and partial Retaining Wall Plans dated 3/27/06. The foundation study consisted of a field investigation and a review of "As-built" plans for nearby bridges Hale Ave. and Santa Fe Avenue Overhead. All elevations referenced in this memo and shown on the Log of Test Boring (LOTB) sheets are in meters, and are referenced to the NAVD88 datum. All the "As-built" borings are referenced to the NGVD29 datum.

Project / Site Description

The current project site is located in Escondido, on State Route 15 between Hale Avenue and Santa Fe Overhead (W. Washington Avenue) on the right shoulder of northbound Route 15. The proposed retaining wall is a Type 1, 109m long, with a maximum height of 4.8m, and has an average height of approximately 3.6m.

Geology

The City of Escondido lies on the boundary of the Peninsular Range and the Coastal Plain Region. The terrain is characterized by rocky, moderately hilled topography, and is further surrounded by mountains that rise several hundred meters above the valley floor. The local area consists mainly of two units of geology. The first is the Santiago Peak Volcanics. These volcanics consist of a structurally complex, somewhat metamorphosed unit composed of volcanic and volcanoclastic rocks of the Southern California batholith. The second unit is composed of mostly medium to

coarse grained leucocratic granodiorite and tonalite of the Cretaceous era. Depth to bedrock is considered to be shallow.

As-built data from two nearby structures was used for this report, Hale Avenue Undercrossing (Bridge #57-0811) and Santa Fe Avenue OH (Bridge #57-0812R). These sites are underlain by dense alluvium overlaying native granitic rock in various states of decomposition. The alluvium consists of dense clayey sandy silt, clayey sand and silty sand with occasional gravel lenses.

The as-built log of test borings of Hale Avenue Undercrossing indicate ground water was encountered in the exploratory borings at an elevation of 187.5m to 188.4m in the 1970 and 1972 borings. The as-built log of test borings of Santa Fe Avenue OH indicate ground water was encountered in the exploratory borings at an elevation of 189.6m to 190.0m in the 1970 and 1972 borings. It is not anticipated that groundwater will be encountered during construction however, surface flows may occur due to seasonal rain run off.

Corrosion

The Materials Engineering and Testing Services, Testing and Technology Branch, has performed corrosion tests on soil samples from several nearby structures field investigations of 2006. Laboratory test results from boring samples indicate that soils at the site are considered non-corrosive. Normal design and construction techniques may be used.

Note: Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 ppm of chlorides, more than 2000 ppm of sulfates, has a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Foundation Recommendations

The following foundation recommendations are for the proposed Retaining Wall #510, as shown on the retaining wall Layout plans dated 10/06/06. The proposed structure may be supported on spread footings at all support locations. Refer to the recommendations provided in the Table below for the minimum footing width for the specified wall height and the recommended Gross Allowable Soil bearing pressure to be used in design. The Recommended Soil Bearing Pressures shown in the Table below are based on Case III loading. The soil supporting the proposed wall will be new fill material. If normal construction techniques and inspections for this fill section are observed, the soil supporting the proposed wall foundations will be dense and capable of supporting any standard type 1 retaining wall for any given wall height shown in the Standard Plans, sheet B3-1. The gross allowable soil bearing pressure will exceed the maximum footing pressure listed for any loading case in the Standard Plans (July 2004).

Retaining Wall #510 Spread Footing Data

Approximate Support Location of Retaining Wall #510 on the "SD-15R" Line	Wall Type	Maximum Wall Height	Minimum Footing Width for Max Wall Height	Recommended Soil Bearing Pressures	
				ASD [1]	LFD [2]
				Gross Allowable Soil Bearing Pressure (q_{all})	Ultimate Soil Bearing Pressure (q_{ult}^{*})
24.6 m RT Sta.510+00.00 to 24.6 m RT Sta 511+08.6	Type 1	4800 mm	2750 mm	180 kPa	N/A

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).
2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^{*}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Ultimate Soil Bearing Pressure, (q_{ult}^{*}).

General Notes

1. Should there be any reduction in the minimum spread footing width, or an increase in the wall heights or maximum allowable slope, our Office shall be notified to reevaluate the recommended gross allowable bearing capacity.

2. All support locations are to be plotted on the Log of Test Borings in plan view. The plotting of support locations should be made prior to requesting a foundation review.

Construction Considerations

1. The footing concrete shall be placed neat against the recompacted soil at the bottom of the footing excavation. If the soils at the bottom of the excavation are disturbed or loosened, they shall be re-compacted to 95% relative density prior to placing any concrete or steel.

The recommendations contained in this report are based on specific project information regarding retaining wall heights, and wall locations that have been provided to Office of Geotechnical Design – South 2, Branch C. If any conceptual changes are made during final project design, our Office should review those changes to determine if the foundation recommendations contained in this report are still applicable.

Any questions regarding the above recommendations should be directed to Gina Pursell, (916) 227-4539, or Shawn Wei, (916) 227-5252, of Office of Geotechnical Design – South 2, Branch C.

Report by: Date:

Gina W Pursell 10/29/07

GINA W. PURSELL, R.C.E. # 54512
Associate Materials & Research Engineer
Office of Geotechnical Design – South 2, Branch C

REGISTERED PROFESSIONAL ENGINEER
GINA W. PURSELL
No. 54512
Exp. 12-31-07
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STATE OF CALIFORNIA

c: R.E. Pending File
OGDS2 - SWei
Geology – North
Geology - South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Office of Bridge Design South
DIVISION OF ENGINEERING SERVICES

Date: January 25, 2007

File: 11-SD-15-R31.17
11-2T0821

Attention:

Washington Avenue OH
(Widening)
Bridge No. 57-0812R

From: OFFICE OF GEOTECHNICAL DESIGN SOUTH 2
GEOTECHNICAL SERVICES - MS 5
DIVISION OF ENGINEERING SERVICES

Subject: Seismic Design Recommendations

This is a follow up to our memorandum of June 8, 2001 and supersedes the preliminary seismic design recommendations for the above bridge.

Seismicity

Based on the California Seismic Hazard Map1996, the site is located between the Newport-Inglewood-Rose Canyon/E Fault (NIE, strike-slip) of maximum credible moment magnitude $M_w = 7.0$ and Whittier-Elsinore Fault (WEE, strike slip) of $M_w = 7.5$. The site distances to the above faults are about 27 and 23 km, respectively. The site is located within the peak horizontal bedrock acceleration zone PBA = 0.3g. The value of PBA was verified using Sadigh et al. (1997) attenuation relation. A copy of the local seismic map is attached.

Soil Profile Type

From the As-built log of test borings (1970) and the field notes from recent subsurface investigations (B1-06 and B1-07), the site is classified as soil profile type C as defined in Appendix B of Seismic Design Criteria (SDC).

ARS Curve

SDC standard acceleration response spectrum, Figure B-5, having PBA = 0.3g, $M_w = 7.5$, and soil profile type C is recommended for design. A copy of the design response spectrum is attached for your reference.

Liquefaction

Layers of fill material consisting of medium dense to very dense silty and clayey sand and gravel underlie the abutments. Depths from ground surface to bedrock at Abutments 1 and 5 are about 45 ft and 40 ft, respectively. Ground water was encountered during subsurface investigations at elevations 622 to 626 ft. The potential for soil liquefaction under strong ground shaking is considered low.

Surface Fault Rupture

The site is not located inside Earthquake Fault Zones as delineated by the California Geological Survey Maps developed under the Alquist-Priolo Earthquake Fault Zoning Act. Hence the potential for surface fault rupture hazard is considered low.

If you have any questions, please call Mahmoud Khojasteh at (916) 227-7211.

M. Khojasteh
MAHMOUD KHOJASTEH
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Attachments:

c: E. Neupert – GDS2
M. DeSalvatore – GDS2
D. Stebbins – D11
File

DEPARTMENT OF TRANSPORTATION

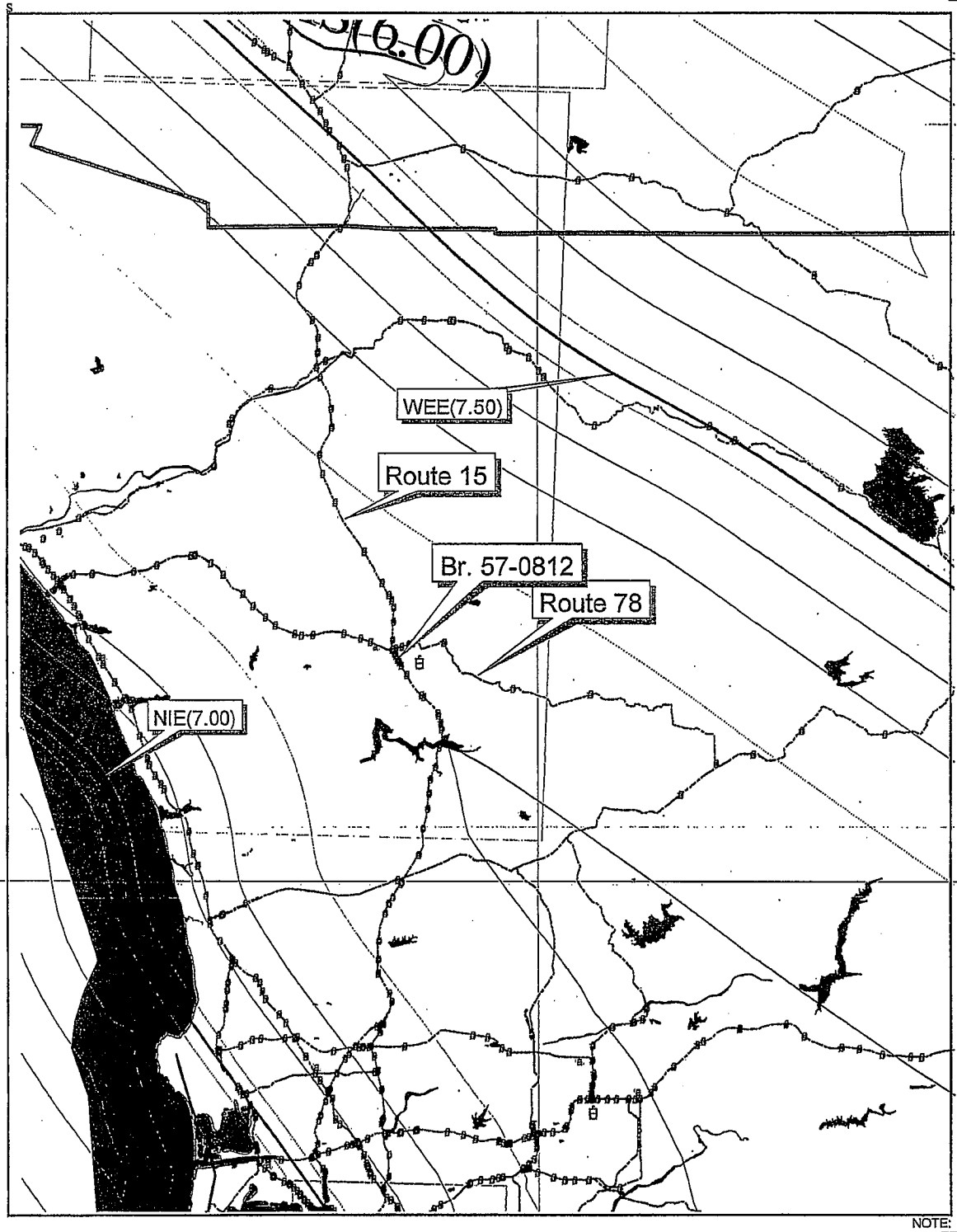
STATE OF CALIFORNIA
BUSINESS, TRANSPORTATION AND HOUSING AGENCY

ENGINEERING SERVICE CENTER

CALIFORNIA SEISMIC HAZARD MAP 1996 - DETAIL AREA

BASED ON MAXIMUM CREDIBLE EARTHQUAKES (MCE)

Caltrans

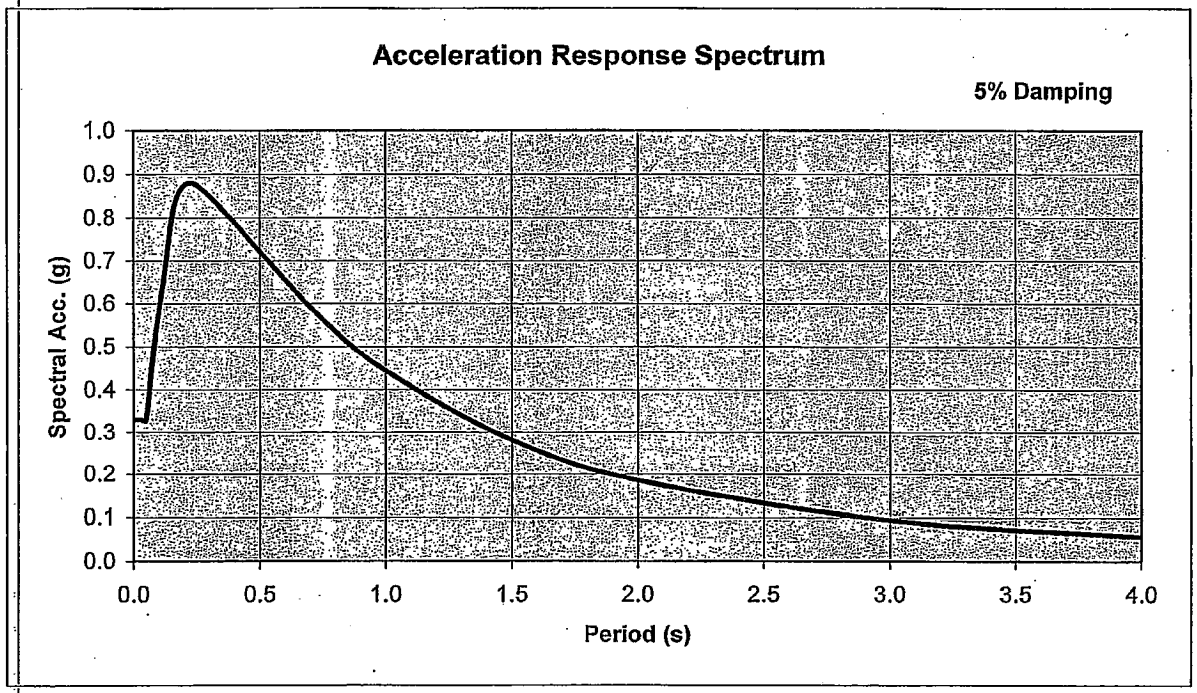


NOTE: SEE ACCOMPANYING TECHNICAL REPORT FOR NAMES OF FAULTS CORRESPONDING TO FAULT CODES

Wahington Ave OH

Bridge No. 57- 0812R

Period (s)	ARS (g)
0.010	0.330
0.020	0.330
0.030	0.330
0.050	0.330
0.075	0.467
0.100	0.574
0.120	0.656
0.150	0.789
0.170	0.842
0.200	0.874
0.240	0.880
0.300	0.851
0.400	0.791
0.500	0.723
0.750	0.564
1.000	0.445
1.500	0.282
2.000	0.186
3.000	0.094
4.000	0.056



Fault	Newport-Inglewood-Rose Canyon/E
Style	Strike-Slip
Magnitude	7
R (km)	23
Soil Type	C
Note:	

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Office of Bridge Design South
DIVISION OF ENGINEERING SERVICES

Date: February 1, 2007

File: 11-SD-15-R31.32
11-2T0821

Mission Avenue UC
(Widening)
Bridge No. 57-0813R

From: OFFICE OF GEOTECHNICAL DESIGN SOUTH 2
GEOTECHNICAL SERVICES - MS 5
DIVISION OF ENGINEERING SERVICES

Subject: Seismic Design Recommendations

This is a follow up to our memorandum of June 8, 2001 and supersedes the preliminary seismic design recommendations for the above bridge.

Seismicity

Based on the California Seismic Hazard Map1996, the site is located between the Newport-Inglewood-Rose Canyon/E Fault (NIE, strike-slip) of maximum credible moment magnitude $M_w = 7.0$ and Whittier-Elsinore Fault (WEE, strike slip) of $M_w = 7.5$. The site distances to the above faults are about 23 and 27 km, respectively. The site is located within the peak horizontal bedrock acceleration zone PBA = 0.3g. The value of PBA was verified using Sadigh et al. (1997) attenuation relation. A copy of the local seismic map is attached.

Soil Profile Type

From the As-built log of test borings (1970, 1972) and considering that the abutments are on compacted fill, the site is classified as soil profile type C as defined in Appendix B of Seismic Design Criteria (SDC).

ARS Curve

SDC standard acceleration response spectrum, Figure B-5, having PBA = 0.3g, $M_w = 7.5$, and soil profile type C is recommended for design. A copy of the design response spectrum is attached for your reference.

Liquefaction

Based on the As-Built log of test borings, alluvial soil consisting of medium dense to very dense silty and clayey sand over decomposed granite underlies the bents. Approximately about 40 ft of fill material was placed at the abutments as part of freeway construction. The composition of the fill material is unknown. The elevation of the top of bedrock varies from 625 to 630 ft. Ground water was encountered during subsurface investigations at elevations 621 to 624 ft. The potential for soil liquefaction under strong ground shaking is considered low.

Surface Fault Rupture

The site is not located inside Earthquake Fault Zones as delineated by the California Geological Survey Maps developed under the Alquist-Priolo Earthquake Fault Zoning Act. Hence the potential for surface fault rupture hazard is considered low.

If you have any questions, please call Mahmoud Khojasteh at (916) 227-7211.

M. Khojasteh

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Attachments:

c: E. Neupert – GDS2
M. DeSalvatore – GDS2
D. Stebbins – D11
File

Memorandum

Flex your power!
Be energy efficient!

To: MS. CHUNG-YUAN WEN
Office of Bridge Design South 2
Diamond Bar, CA

Date: January 31, 2007

File: 11-SD-15-R30.81
11-081001

Attention: Mr. Charles Chen

Escondido Flood Control Channel
Bridge No. 57-0810R/L/K

From: OFFICE OF GEOTECHNICAL DESIGN SOUTH 2
GEOTECHNICAL SERVICES - MS 5
DIVISION OF ENGINEERING SERVICES

Subject: Seismic Design Recommendations

This is a follow up to our memorandum of June 8, 2001 and supersedes the preliminary seismic design recommendations for the above bridges.

Seismicity

Based on the California Seismic Hazard Map1996, the site is located between the Newport-Inglewood-Rose Canyon/E Fault (NIE, strike-slip) of maximum credible moment magnitude $M_w = 7.0$ and Whittier-Elsinore Fault (WEE, strike slip) of $M_w = 7.5$. The site distances to the above faults are about 23 and 27 km, respectively. The site is located within the peak horizontal bedrock acceleration zone PBA = 0.3g. The value of PBA was verified using Sadigh et al. (1997) attenuation relation. A copy of the local seismic map is attached.

Soil Profile Type

From the As-built log of test borings (1970) and the field notes from recent subsurface investigations (B1-06, B1-07), the site is classified as soil profile type D as defined in Appendix B of Seismic Design Criteria (SDC).

ARS Curve

SDC standard acceleration response spectrum, Figure B-8, with PBA = 0.3g, $M_w = 7$, and soil profile type D is recommended for design. A copy of the design response spectrum is attached for your reference.

Liquefaction

Soil consisting of layers of loose to medium dense silty sand and poorly graded sand underlies the abutments. The depths to bedrock at Abutment1 and 2 are about 21 ft and 23 ft, respectively. Ground water has been reported at depths varying from 10 to 15 ft. Due to the existence of layers of loose sand there is a potential for soil liquefaction and lateral spreading under strong ground shaking.

Surface Fault Rupture

The site is not located within Earthquake Fault Zones as delineated by the California Geological Survey Maps developed under the Alquist-Priolo Earthquake Fault Zoning Act. Hence the potential for surface fault rupture hazard is considered low.

If you have any questions, please call Mahmoud Khojasteh at (916) 227-7211.

M. Khojasteh

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c: E. Neupert – GDS2
M. DeSalvatore – GDS2
D. Stebbins – D11
File

DEPARTMENT OF TRANSPORTATION

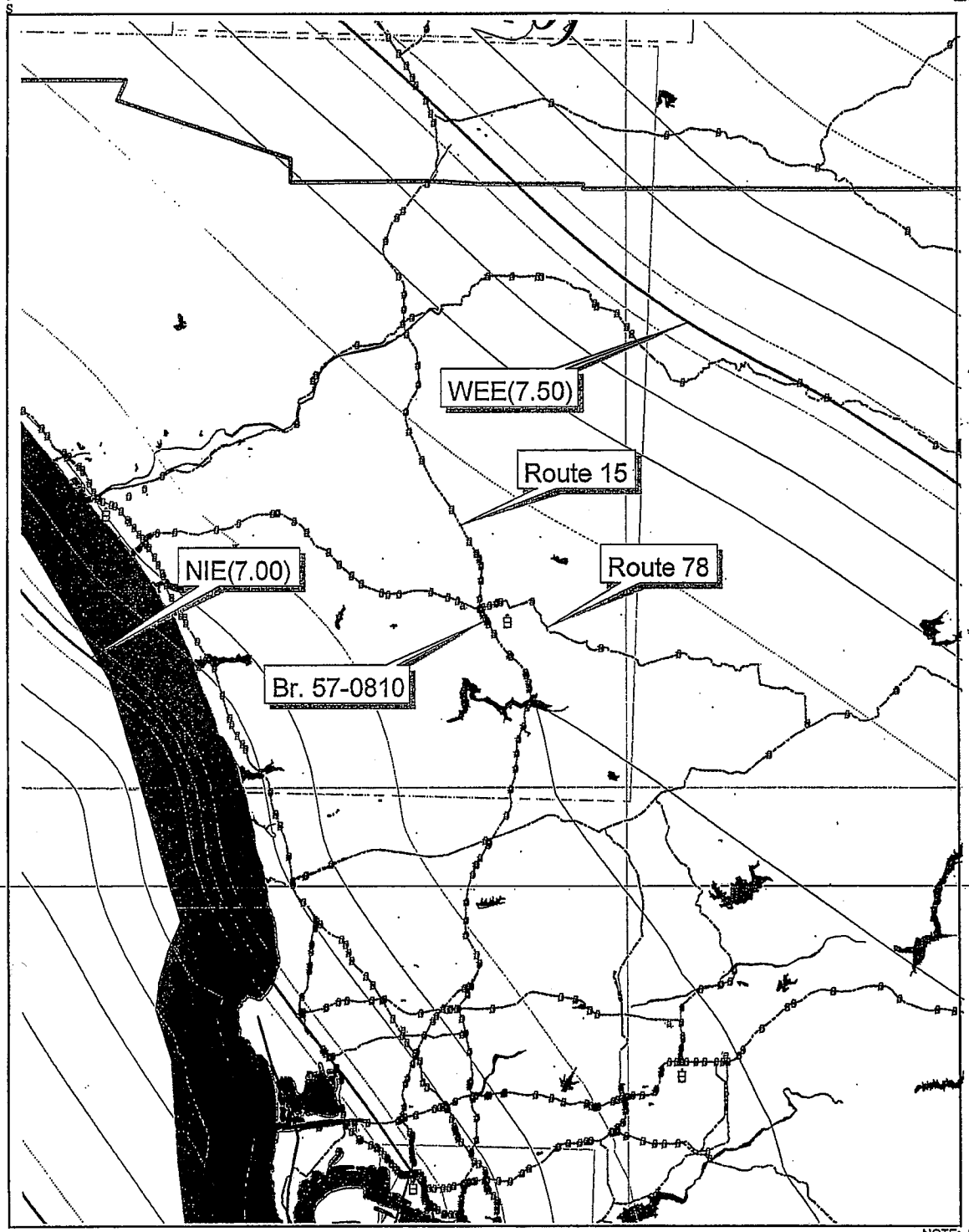
STATE OF CALIFORNIA
BUSINESS, TRANSPORTATION AND HOUSING AGENCY

ENGINEERING SERVICE CENTER

CALIFORNIA SEISMIC HAZARD MAP 1996 - DETAIL AREA

BASED ON MAXIMUM CREDIBLE EARTHQUAKES (MCE)

Caltrans

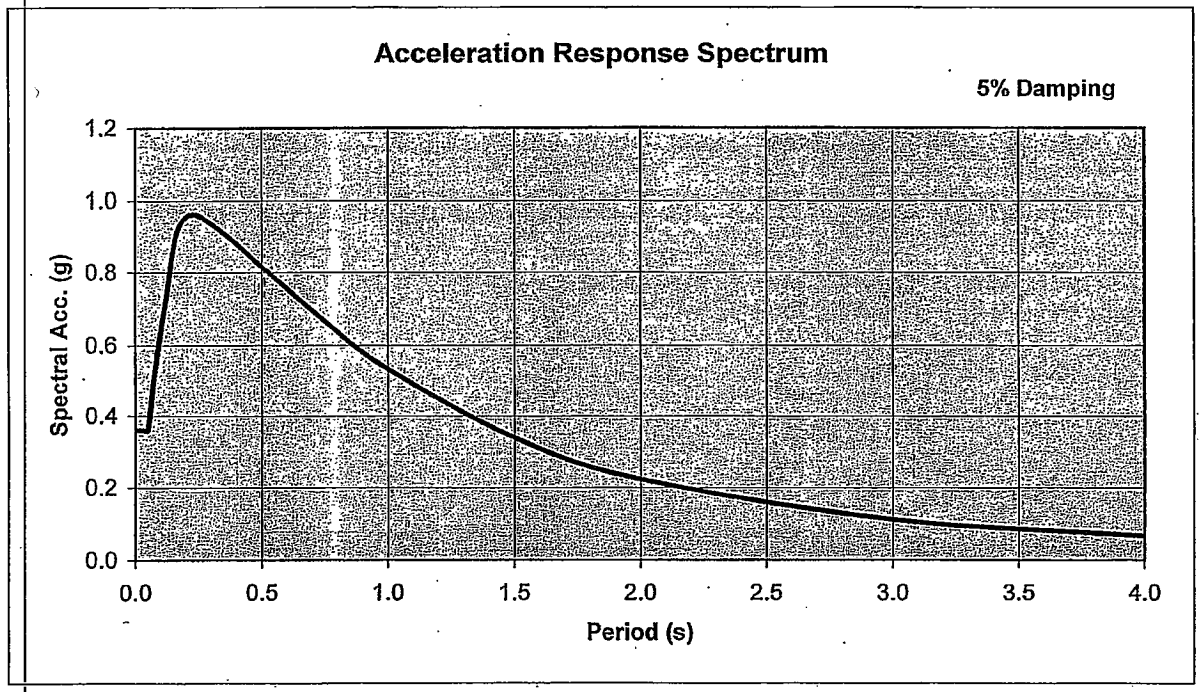


NOTE: SEE ACCOMPANYING TECHNICAL REPORT FOR NAMES OF FAULTS CORRESPONDING TO FAULT CODES

Escondido Flood Control Channel

Bridge No. 57- 0810R/L/K

Period (s)	ARS (g)
0.010	0.360
0.020	0.360
0.030	0.360
0.050	0.360
0.075	0.509
0.100	0.626
0.120	0.716
0.150	0.861
0.170	0.918
0.200	0.954
0.240	0.960
0.300	0.937
0.400	0.881
0.500	0.817
0.750	0.666
1.000	0.534
1.500	0.339
2.000	0.223
3.000	0.113
4.000	0.067



Fault	Newport-Inglewood-Rose Canyon/E
Style	Strike-Slip
Magnitude	7
R (km)	23
Soil Type	D
Note:	

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. CHUNG-YUAN WEN
Branch Chief
Design Branch 21

Date: July 9, 2007

File: 11-Sd-15-PM 27.7
11-2T0821
Valley Parkway UC
(Widen)
Br. No. 57-0809R/L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 2

Subject: Foundation Report

The Division of Bridge Design, Bridge Design Branch 21, has requested a final foundation report for widening of the Valley Parkway Undercrossing on Interstate Route 15 located in Escondido in San Diego County.

The recommendations provided in this report are based on the Request for Final Foundation Report dated December 26, 2006, the Bridge Type Selection Report dated September 8, 2006, the Preliminary Foundation Report prepared by Earth Mechanics, Inc. dated June 19, 2006, the Foundation Report dated April 23, 1970, the LOTBs dated March 9, 1970, the four penetration borings advanced on February 6 and 7, 2007, and the Preliminary Foundation Report dated June 14, 2001.

Project Description/History

The bridge was originally constructed in 1977 and consists of two-span cast-in-place reinforced concrete box girder bridges. The abutments are open-end diaphragm type and the two-column bents are two-way flared octagonal columns integral with bent caps. The existing structures are supported on 400 mm Cast-In-Drilled-Hole (CIDH) piles at the abutments and spread footings at the bents.

All elevations referenced in this report are based on the NAVD88 vertical datum. As-Built elevations have been converted from the NGVD29 datum.

Geology

Escondido lies in a valley within the foothills of the Peninsular Ranges. The terrain is characterized by moderately hilly and rocky topography. The area of the project site consists primarily of igneous and metamorphic basement rocks of the Santiago Peak Volcanics.

Two rotary borings and two penetration borings were advanced in February of 1970. One rotary boring extended approximately 7.6 meters (25 feet) below grade, and the other rotary boring extended approximately 4.6 meters (15 feet) below grade with the deepest elevation being 189.9 meters (623 feet). The borings show about one to seven feet of sandy silt underlain by decomposed granite grading to weathered granite for the full depth of investigation. Four penetration borings were advanced in February of 2007. Fill in the area is between approximately 20 feet and 30 feet, but exact placement of fill is not available. Variable resistance demonstrated in the penetrations borings advanced in February of 2007 indicated the likelihood of encountering boulders within the fill. For site-specific soil and rock information, see the LOTBs.

Ground Water

Ground water was not encountered during the investigations. This may vary seasonally and with recent rainfall totals.

Scour Potential

There is no scour potential at this site as the structures do not span a watercourse.

Fault and Seismic Data

See Seismic Design Recommendations prepared by Mr. Asef Wardack dated July 9, 2007, and the Soil Springs For Spread Footing prepared by Mr. Asef Wardack dated July 9, 2007.

Corrosion

Due to the predominance of coarse-grained soils, corrosion is not expected at this site. The District 11 Materials Report indicates the soil samples are non–corrosive (Preliminary Foundation Report dated June 11, 2001). Further testing was performed in February, 2007 and verified that the site is non-corrosive.

Liquefaction

Groundwater was not encountered during the investigations. Thus, liquefaction is not anticipated.

Settlement

Due to the granular nature of the soils at the site, the settlement induced by the fill will occur in a relatively short time period. There is no need for an embankment surcharge.

As-Built Foundations

As-Built Pile data and spread footing data are shown in Tables 1 and 2.

Table 1
As-Built Shaft Data for Bridge 57-0809R/L

Location	CIDH Diameter	Working Stress Design	As-Built Tip Elevations			
			Design Tip Elevation	Average Tip Elevation	Maximum Elevation	Minimum Elevation
Abut 1 Right east	16 inch 406.4 mm	70 Ton 623 kN	635.1 ft 193.6 m	639.3 ft 194.8 m	641 ft 195.4 m	638.4 ft 194.5 m
Abut 1 Right west	16 inch 406.4 mm	70 Ton 623 kN	635.1 ft 193.6 m	634.5 ft 193.4 m	634.9 ft 193.5 m	634.3 ft 193.3 m
Abut 3 Right east	16 inch 406.4 mm	70 Ton 623 kN	646.1 ft 196.9 m	647.5 ft 197.3 m	653 ft 199 m	644.3 ft 196.4 m
Abut 3 Right west	16 inch 406.4 mm	70 Ton 623 kN	646.1 ft 196.9 m	646.4 ft 197 m	647.3 ft 197.3 m	645.3 ft 196.7 m
Abut 1 Left west	16 inch 406.4 mm	70 Ton 623 kN	632.1 ft 192.7 m	631.7 ft 192.5 m	635.8 ft 193.8 m	626 ft 190.8 m
Abut 3 Left west	16 inch 406.4 mm	70 Ton 623 kN	632.1-641.1 ft 192.7-195.4 m	632.3 ft 192.7 m	633.7 ft 193.1 m	631.3 ft 192.4 m

Note: As-Built elevations have been converted from NGVD29 to NAVD88.

Table 2
As-Built Spread Footing Data for Bridge 57-0809R/L

Location	Gross Allowable Bearing pressure	Left Column Bottom of Footing Elevation	Right Column Bottom of Footing Elevation
Bent 2 Right	5 tsf 478 kPa	641.1 feet 195.4 m	642.1 feet 195.7 m
Bent 2 Left	5 tsf 478 kPa	637.1 feet 194.2 m	637.1 feet 194.2 m

Note: As-Built elevations have been converted from NGVD29 to NAVD88

Foundation Recommendations

At abutments 1 and 3 support locations 750 mm (29.5 inch) diameter CIDH piles may be used. The specified pile tip elevations are shown in Table 3.

Table 3
CIDH DATA for Bridge 57-0809R/L

Support Location	CIDH Diameter	Maximum Factored Loading for Limit States in Compression			Controlling Nominal Resistance[4]	Design Tip Elevation[5]
		Service Loading [1]	Strength Loading[2]	Extreme Loading[3]		
Abut 1 Right east	29.5 inch 750 mm	145 kips 650 kN	230 kips 1030 kN	-	330 kips 1500 kN	638.4 ft 194.6 m
Abut 1 Right west	29.5 inch 750 mm	143 kips 640 kN	278 kips 1240 kN	-	400 kips 1800 kN	634.3 ft 193.3 m
Abut 3 Right east	29.5 inch 750 mm	137 kips 610 kN	229 kips 1020 kN	-	330 kips 1500 kN	644.3 ft 196.4 m
Abut 3 Right west	29.5 inch 750 mm	93 kips 420 kN	176 kips 790 kN	-	260 kips 1200 kN	645.3 ft 196.7 m
Abut 1 Left west	29.5 inch 750 mm	179 kips 800 kN	244 kips 1090 kN	-	350 kips 1600 kN	626 ft 190.8 m
Abut 3 Left west	29.5 inch 750 mm	177 kips 800 kN	238 kips 1060 kN	-	340 kips 1550 kN	631.3 ft 192.4 m

Notes: (1) Service Loading Case III controlled, (2) Strength Loading Case II controlled, (3) Load not provided because earthquake case will not control a diaphragm abutment on piles with pin connectors, 4) The Controlling Nominal Resistance listed are for the controlling loading case. To determine the Controlling Nominal Resistance a Strength Reduction Factor (ϕ) of 0.7 was used with Strength Loading, (5) Design tip elevations are based on the controlling Nominal Resistance in Compression.

At bent 2 spread footings may be used. Bearing pressures and estimated settlements are shown in tables 5 and 7. Spread footing loading are shown in Tables 4 and 6.

Table 4
Spread Footing Loading (Br. #57-0809 L)

Location	Limit State[1]	Vertical Loading[2]	Moment[3]		Shear[4]	
			M_T	M_L	S_T	S_L
Bent 2 Left Bridge (West)	Service	1341 kips 5970 kN	282 kip-ft 382 kN-m	2097 kip-ft 2843 kN-m	0	0
	Strength	2085 kips 9300kN	319 kip-ft 432 kN-m	2932 kip-ft 3975 kN-m	489 kips 2225 kN	373 kips 1697 kN
	Extreme (T)	1357 kips 6050 kN	5861 kip-ft 7946 kN-m	0	767 kips 3489.85 kN	0
	Extreme (L)	868 kips 3900 kN	0	5374 kip-ft 7286 kN-m	0	614 kips 2794 kN

Notes:
1) (T) Transverse direction. (L) Longitudinal direction.
2) (M_T) Moment in transverse direction at top of footing. (M_L) Moment in longitudinal direction at top of footing.
3) (S_T) Shear in transverse direction at top of footing. (S_L) Shear in longitudinal direction at top of footing.

Table 5
Spread Footing Soil Resistance (Br. #57-0809/L West)

Location	Limit State	Equivalent Uniform Contract Pressure (q_{equ})	Required Nominal [2] Bearing Pressure (q_n)	Ultimate Soil Bearing[3] Pressure (q_{alt})	Estimated Total Settlement
Bent 2 Left Bridge (West)	Service	10.09 k/ft^2 483.11 kPa	Not applicable	Not applicable	<13 mm
	Strength	14.00 k/ft^2 670.32 kPa	31.11 k/ft^2 1489.55 kPa	263.48 k/ft^2 12615.42 kPa	N/A
	Extreme (T)	21.03 k/ft^2 1006.92 kPa	46.73 k/ft^2 2237.43 kPa	184.47 k/ft^2 8279.01 kPa	<13 mm
	Extreme (L)	21.42 k/ft^2 1025.59 kPa	47.60 k/ft^2 2279.01 kPa	160.70 k/ft^2 7694.32 kPa	<13 mm

Notes:
1) (T) Transverse direction. (L) Longitudinal direction.
2) Required Nominal Bearing Pressure, (q_n) was determined by dividing the Equivalent Uniform Contact Pressure, (q_{equ}), by the Strength Reduction Factor, (ϕ). A Strength Reduction factor (ϕ) of 0.45 was used with Strength Limit and a Strength Reduction Factor (ϕ) of 1 was used for Extreme Limits.
3) Load and Resistance Factor Design, (LRFD). The Required Nominal Bearing Pressure, (q_n), is not to exceed the Ultimate Soil Bearing Pressure, (q_{ult})

Table 6
Spread Footing Loading (Br. #57-0809/R)

Location	Limit State[1]	Vertical Loading[2]	Moment[3]		Shear[4]	
			M_T	M_L	S_T	S_L
Bent 2 Right Bridge West	Service	1122 kips 5000 kN	118 kip-ft 160 kN-m	893 kip-ft 1211 kN-m	0	0
	Strength	1836 kips 8200 kN	84 kip-ft 114 kN-m	1439 kip-ft 1951 kN-m	0	379 kips 1700 kN
	Extreme (T)	944 kips 4200 kN	5259 kip-ft 7130 kN-m	0	696 kips 3100 kN	0
	Extreme (L)	944 kips 4200 kN	0	5259 kip-ft	0	680 kips 3050 kN
Bent 2 Right Bridge East	Service	1078 kips 4800 kN	155 kip-ft 210 kN-m	413 kip-ft 560 kN-m	0	0
	Strength	1767 kips 7900 kN	374 kip-ft 507 kN-m	521 kip-ft 706 kN-m	296 kips 1350 kN	277 kips 1250 kN
	Extreme (T)	1357 kips 6040 kN	5861 kip-ft 7946 kN-m	0	767 kips 3450 kN	0
	Extreme (L)	868 kips 3900 kN	0	5374 kip-ft 7286 kN-m	0	614 kips 2750 kN

Notes:
1) (T) Transverse direction. (L) Longitudinal direction.
2) (M_T) Moment in transverse direction at top of footing. (M_L) Moment in longitudinal direction at top of footing.
3) (S_T) Shear in transverse direction at top of footing. (S_L) Shear in longitudinal direction at top of footing.

Table 7
Spread Footing Soil Resistance (Br. #57-0809/ R)

Location	Limit State	Equivalent Uniform Contract Pressure (q_{equ})	Required Nominal[2] Bearing Pressure (q_n)	Ultimate Soil Bearing[3] Pressure (q_{ult})	Estimated Total Settlement
Bent 2 Right Bridge West	Service	9.21 k/ft^2 440.97 kPa	Not applicable	Not applicable	<13 mm
	Strength	12.71 k/ft^2 608.55 kPa	28.25 k/ft^2 1343.29 kPa	358.69 k/ft^2 17174.08 kPa	N/A
	Extreme (T)	18.80 k/ft^2 900.14 kPa	41.78 k/ft^2 2000.43 kPa	246.69 k/ft^2 11811.52 kPa	<13 mm
	Extreme (L)	18.61 k/ft^2 891.05 kPa	41.35 k/ft^2 1979.84 kPa	247.80 k/ft^2 11864.66 kPa	<13 mm
Bent 2 Right Bridge East	Service	9.83 k/ft^2 470.66 kPa	Not applicable	Not applicable	<13 mm
	Strength	13.09 k/ft^2 626.75 kPa	29.10 k/ft^2 1393.31 kPa	429.31 k/ft^2 20555.36 kPa	N/A
	Extreme (T)	20.32 k/ft^2 972.92 kPa	45.15 k/ft^2 2161.78 kPa	323.02 k/ft^2 15466.19 kPa	<13 mm
	Extreme (L)	17.87 k/ft^2 855.62 kPa	39.70 k/ft^2 1900.84 kPa	309.32 k/ft^2 14810.24 kPa	<13 mm

Notes: 1) (T) Transverse direction. (L) Longitudinal direction.
2) Required Nominal Bearing Pressure, (q_n) was determined by dividing the Equivalent Uniform Contact Pressure, (q_{equ}), by the Strength Reduction Factor, (ϕ). A Strength Reduction factor (ϕ) of 0.45 was used with Strength Limit and a Strength Reduction Factor (ϕ) of 1 was used for Extreme Limits.
3) Load and Resistance Factor Design, (LRFD). The Required Nominal Bearing Pressure, (q_n), is not to exceed the Ultimate Soil Bearing Pressure, (q_{ult})

Table 8
Spread Footing Data (Br. #57-0809R/L)

Location	Bottom of Footing Elevation	Footing Dimensions			Depth of Footing[3] (D_f)
		Length (L_L)[1]	Length (L_T)[2]	Thickness (t)	
Bent 2 Left Bridge (West)	636 ft 193.8 m	15 ft 4.6 m	15 ft 4.6 m	3.5 ft 1.1 m	5.5 ft 1.7 m
Bent 2 Right Bridge (West)	640 ft 195.1 m	15 ft 4.6 m	15 ft 4.6 m	3.5 ft 1.1 m	5.7 ft 1.7 m
Bent 2 Right Bridge (East)	642 ft 195.7 m	15 ft 4.6 m	15 ft 4.6 m	3.5 ft 1.1 m	7.05 ft 2.1 m

Notes: 1) (L_L) Longitudinal direction
2) (L_T) Transverse direction.
3) (D_f) Footing embedment depth taken from ground surface to bottom of footing.

General Notes:

All elevations are based on NAVD88.

All support locations are to be plotted on plan view on the LOTB (MTD 4-2)

Recommendations are based on the foundation geometry and load data provided by Structure Design in the Foundation Design Data Sheet. The uniform contact pressure on the effective footing area is assumed to account for the effects of eccentric loading.

CIDH Notes

The tip elevations will lie at least 10 feet within the decomposed/weathered bedrock with the tips embedded at least two feet within the weathered bedrock.

The axial capacity of the drilled shafts is developed from skin friction and end bearing.

Calculations for the design tip elevation for CIDH shafts are based on the methods and equations presented in Federal Highway Administration publication No. FHWA HI 88-042.

All shafts are designed as single shafts.

The bottom of the new CIDH footing elevations match the slope of the bottom of the existing footing elevations.

No tension capacity is being requested.

Spread Footing Notes

The bottoms of new spread footings are not to be above 195. 7 meters (642.1) feet in elevation. The spread footings shall be placed neat against the undisturbed bedrock material.

Calculations for the spread footing elevations are based on the methods and equations presented in Federal Highway Administration publication No. FHWA HI 88-009.

Construction Considerations

At all support locations where CIDH piles are to be constructed, due to the weak and intensely fractured nature of the upper zones of the underlying granitic material, caving conditions should be anticipated during CIDH pile construction. The amount of caving the contractor will experience will be dependent upon the method of excavation the contractor chooses, and the speed which the contractor chooses to advance the drilled hole. Temporary casing may be necessary to control caving during construction. Use of temporary casings shall conform to Standard Specifications 49-4.03. All temporary casing is to be removed during concrete placement.

Variations in the weathering, fracturing, and hardness of the bedrock occurred both laterally and vertically across the site, as shown on the LOTB sheets. The contractor should anticipate both soft and hard rock drilling conditions across the bridge site. The amount of difficulty the contractor will experience will be dependent upon the methods and means the contractor chooses to construct the CIDH piles.

Most of the calculated geotechnical capacity of the CIDH piles is based upon skin friction and relies upon a concrete bond to the rock walls of the drilled shaft. It is imperative that the borehole walls are not contaminated with drill cuttings or loose materials. The use of rotator or oscillator methods for drilling of CIDH piles will allow for alluvium or rock cuttings to be trapped between the borehole walls and

Rotator or oscillator drilling methods are not to be allowed to drill the shafts for the CIDH piles.

Concrete for the support footings shall be placed neat against the undisturbed bedrock material along the trimmed walls and bottom of footing excavations. Should the bottom of footing excavations be disturbed, then the bottom of the footing excavations shall be extended down at 0.15-meter (0.5 ft) intervals until undisturbed bedrock material is observed and approved by the Engineer. The sub-excavated material is to be replaced with lean concrete. The disturbed native material is not to be recompacted.

Obstructions, consisting partially of hard rock, are likely to be encountered while drilling through the existing fill. Fill is likely to consist of soils and rock.

If you have any questions or need additional information, please call Bill Levine at 916-227-0505 or Angel Perez-Cobo at 916-227-7167.

Bill Levine
Engineering Geologist
Geotechnical Design-South 2
Design Branch A

Angel Perez-Cobo
Senior Engineer
Geotechnical Design-South 2
Design Branch A

cc: APerez-Cobo
R.E. Pending File
Specs & Estimates
Proj Mgmt
BLevine
File

REGISTERED PROFESSIONAL ENGINEER
ANGEL PEREZ-COBO
No. C 053543
Exp. 6/30/09
CIVIL
STATE OF CALIFORNIA

REGISTERED GEOLOGIST
W.J. LEVINE
NO. 2108
Exp. 7/31/07
CERTIFIED ENGINEERING GEOLOGIST

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MS. CHUNG-YUAN WEN
Division of Engineering Services
Office of Bridge Design-South 2
Diamond Bar, CA

Attention: Mr. Charles Chen

Date: April 25, 2007

File: 11-SD-15-KP R30.81
11-2T0821
Escondido Flood Control
Channel Ramp (Replace)
Br. #57-0810K

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: Foundation Report

This report presents the foundation recommendations for the proposed replacement of the Escondido Flood Control Channel Ramp (Br. #57-0810 K). The Office of Geotechnical Design-South 2, Design Branch B completed a foundation investigation pursuant to a request by the Office of Bridge Design-South 2 for a foundation investigation and recommendations for the proposed replacement structure.

The following foundation recommendations are based on subsurface information gathered during the recent foundation investigation (December 2006/January 2007) performed by Caltrans, as well as "As-built" information from a 1970 subsurface investigation. With regards to the current foundation recommendations, all elevations referenced within this report and shown on the recent Log of Test Boring (LOTB) sheets are based on the NAVD 1988 vertical datum (as-built elevations have been converted from NGVD 1929 datum to NAVD 1988 datum using a conversion factor of +0.63 m (+2.07 ft.)).

Project Description/History

The project site is located on Interstate 15 in the city of Escondido, San Diego County. The bridge site is in an area of moderately low to steep rolling hills. The existing Escondido Flood Control Channel Ramp bridge is a single span structure built in 1977, which is supported on 625 kN (70 ton) driven HP 250x85 (HP 10x57) steel "H" piles at Abutments 1 and 2 locations. The current project proposes to replace the existing Escondido Flood Control Channel Ramp bridge with a new structure to be located just west of the existing structure. This is due to the proposed construction of managed lanes down the center of the existing Interstate 15, which will require widening or replacement of existing bridges.

The proposed replacement Escondido Flood Control Channel Ramp bridge is proposed to be a single-span, cast-in-place, prestressed, concrete box-girder structure with seat abutments.

Geology

The foundation investigation performed in February 1970 consisted of four mud-rotary borings and one dynamic cone penetration sounding. The foundation investigation performed in December 2006/January 2007 consisted of two mud-rotary borings, one drilled with a Mobile B-47 drill rig, and one drilled with a CME 750 drill rig.

Both the 1970 and the 2006/2007 foundation investigations, at the bridge site, revealed alluvial material overlying granitic bedrock. During the December 2006/January 2007 foundation investigation, in boring B-1-06, drilled near the proposed Abutment 1 location of the replacement bridge, it was revealed the alluvial material extends down approximately 6.4 m (21 ft.) where intensely weathered to decomposed granitic bedrock was encountered (approx. elev. 187.1 m (614.0 ft)). The alluvial material is generally a loose to medium dense, silty sand to poorly graded sand. Below the alluvial material, the granitic bedrock is intensely weathered to decomposed to approximately 10.6 m (34.8 ft) (approx. elev. 182.6 m (599.0 ft)) where it becomes moderately to slightly weathered, moderately hard to hard.

In boring B-1-07, drilled near the proposed Abutment 2 location of the replacement bridge, it was revealed the alluvial material extends down approximately 7.0 m (23.0 ft) where decomposed granitic bedrock was encountered (approx. elev. 185.0 m (607.0 ft)). The alluvial material is generally a loose to medium dense, silty sand and well-graded sand with scattered gravels. Below the alluvial material, the granitic bedrock is decomposed to the bottom of boring B-1-07 (elev. 175.4 m (575.3 ft.)). Refer to the Log of Test Borings for site specific soil and rock data.

Ground Water

Ground water was encountered during both the 1970 and 2006/2007 subsurface investigations. The measured static groundwater levels are summarized below in Table 1. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions at time of construction.

Table 1: Groundwater Measurements

Boring No.	Date Measured	Groundwater Elevation
B-3-70	02/22/70	188.1 m (617.1 ft)
B-4-70	02/22/70	188.2 m (617.5 ft)
B-5-70	02/20/70	187.8 m (616.0 ft)
B-1-06	01/08/07	188.3 m (617.8 ft)
B-1-07	01/08/07	188.5 m (618.4 ft)

Scour Potential

The Escondido Flood Control Channel is concrete lined, therefore scour is not considered to be an issue at this location.

Corrosion

Corrosion test results for soil samples collected from borings B-1-06 and B-1-07, are shown below in Table 2. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2: Corrosion Test Summary

Boring	SIC Number	Sample Depth	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1-06	C654037	0-6.46 m (0-21.2 ft)	7.73	4600	N/A	N/A
B-1-07	C654038	0-7.71 m (0-25.3 ft)	7.74	2700	N/A	N/A

Note: Caltrans currently defines a corrosive environment as an area where the soil has a minimum resistivity of less than 1000 ohm-cm, and either contains greater than or equal to 500 ppm of chlorides, greater than or equal to 2000 ppm of sulfates, or has a pH of 5.5 or less

Fault and Seismic Data

The structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Office of Geotechnical Design – South 2, has provided Seismic Design Recommendations for the site in a memorandum dated January 31, 2007. The controlling faults for the site are the Newport-Inglewood-Rose Canyon Fault/E (NIE, strike-slip) located approximately 23 km (14.3 mi.) southwest of the site, which is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.0, and the Whittier-Elsinore Fault (WEE, strike-slip), located approximately 27 km (16.8 mi.) northeast of the site, which is capable of generating a Maximum Credible Earthquake moment magnitude (Mw) of 7.5. The site is located within the peak horizontal bedrock acceleration zone of 0.3g.

Liquefaction Potential

Seismic Design Recommendations, dated January 31, 2007, states that due to the existence of layers of loose sand, beneath the site, there is the potential for soil liquefaction and lateral spreading under strong ground shaking. Based on "As-built" LOTB's and information from borings drilled during the 2006/2007 subsurface investigation, there is approximately 3.7 m (12 ft) of potentially liquefiable material beneath the Abutment 1 location, extending from approximate elevation 190.8 m (626.0 ft) to elevation 187.1 m (614.0 ft). Beneath the Abutment 2 location, there is approximately 2.7 m (9.0 ft) of potentially liquefiable material, extending from approximate elevation 187.8 m (616.0 ft) to elevation 185.0 m (607.0 ft). The seismic specialist from the Office of Geotechnical Design South-2 has provided lateral load due to lateral spreading information to the district designer via an email, dated March 1, 2007. The

email states that estimates for lateral load on 625 kN (70 ton) design load 250x85 (10x57) "H" piles due to lateral spreading of the abutment soils are 33.4 kN (7.5 kips) at Abutment 1, with a point of application 2.74 m (9.0 ft) from the bottom of the footing, and 66.8 kN (15 kips) at Abutment 2, with a point of application 4.0 m (13.0 ft) from the bottom of the footing.

Foundation Recommendations

The following recommendations are for the proposed replacement of the Escondido Flood Control Channel Ramp (Br. #57-0810 K) as shown on the General Plan dated December 27, 2006. Abutments 1 and 2 as well as the associated wingwall piles may be supported on 625 kN (70 ton) design load 250x85 (10x57) "H" piles. The specified pile tip elevations (SPTE) for the "H" piles are shown below in Table 3. The ultimate geotechnical capacity for the piles will meet or exceed the required nominal resistance in compression shown below in Table 3.

Table 3: Abutments 1 and 2, and Wingwalls Driven Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
			Compression	Tension		
Abutment 1	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	185.6 m (1) (609.0 ft)	185.6 m (609.0 ft)
Abutment 2	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	183.5 m (1) (602.0 ft)	183.5 m (602.0 ft)
Abutment 1 Right Side Wingwall Pile	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	185.6 m (1) (609.0 ft)	185.6 m (609.0 ft)
Abutment 2 Left Side Wingwall Pile	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	183.5 m (1) (602.0 ft)	183.5 m (602.0 ft)
Abutment 2 Right Side Wingwall Pile	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	183.5 m (1) (602.0 ft)	183.5 m (602.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

General Notes:

1. All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

Construction Considerations:

1. The calculated geotechnical capacity of the "H" piles is based predominantly on end bearing.

2. Due to variations of the top of bedrock elevation as well as the hardness and degree of weathering of the bedrock material beneath the proposed Abutments 1 and 2 support loctions, hard and erratic driving of the "H" piles should be anticipated, and the contractor should anticipate that some piles will require splicing or cutting-off in the field.

3. Pile acceptance criteria for the "H" piles will be in accordance with Standard Specifications Section 49-1.08 "Bearing Value and Penetration". Any pile that achieves two times the required nominal resistance in compression, within 1.5 m (5 ft) of the SPTE, may be considered good and cut off with the Engineer's written approval. Two times the required nominal resistance in compression is 2500 kN (280.0 tons).

4. At the contractor's option, drilling to assist driving may be utilized prior to driving each "H" pile. All drilling to assist driving shall be done in accordance with Standard Specification Section 49-1.05 "Driving Equipment". Drilling to assist driving shall not extend beyond the elevations shown in Table 4, below.

Table 4: Abutments 1 and 2 and Wingwall Piles
Drilling To Assist Driving Elevations

Location	Drilling To Assist Driving Elevation
Abutment 1	187.1 m (614.0 ft)
Abutment 2	184.7 m (606.0 ft)
Abutment 1 Right Side Wingwall Pile	187.1 m (614.0 ft)
Abutment 2 Left Side Wingwall Pile	184.7 m (606.0 ft)
Abutment 2 Right Side Wingwall Pile	184.7 m (606.0 ft)

5. After driving the first few "H" piles, utilizing drill-to-assist driving methods, if it is found that two (2) times the nominal resistance in compression is achieved above the SPTE, then the drill-to-assist elevation may be modified, with the written approval of the engineer, to aid in reaching the SPTE shown in Table 3, above.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by the Office of Bridge Design-South 1. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the

above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 4/25/07

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Supervised by: ~~Date:~~ 4/25/07

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REGISTERED PROFESSIONAL ENGINEER
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cc: R.E. Pending File
John Stayton - Specs & Estimates (4)
David Stebbins – District 11 (Project Manager)
Abbas Abghari – OGDS-II
Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE
Division of Engineering Services
Office of Bridge Design-South 1
Bridge Design Branch 18

Attention: Mr. Prem Rimal

Date: May 3, 2007

File: 11-SD-15-KP R30.81
11-2T0821
Escondido Flood Control
Channel Br. (Widen)
Br. #57-0810 L/R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: Foundation Report

This report presents the foundation recommendations for the proposed widening of the left and right Escondido Flood Control Channel Bridges (Br. #57-0810 L/R). This is pursuant to a request by the Office of Bridge Design-South 1 for foundation recommendations for the proposed widening of the existing structures.

The following foundation recommendations are based on subsurface information gathered during the recent foundation investigation (December 2006/January 2007) performed by Caltrans, as well as "As-built" information from a 1970 subsurface investigation. With regards to the current foundation recommendations, all elevations referenced within this report and shown on the recent Log of Test Boring (LOTB) sheets are based on the NAVD 1988 vertical datum (as-built elevations have been converted from NGVD 1929 datum to NAVD 1988 datum using a conversion factor of +0.63 m (+2.07 ft.)).

Project Description/History

The project site is located on Interstate 15 in the city of Escondido, San Diego County. The bridge site is located in a small valley surrounded by moderately low to steep rolling hills. The existing left and right Escondido Flood Control Channel Bridges are single span, cast-in-place, prestressed, concrete box-girder structures with end-diaphragm abutments, which were built in 1977. Abutments 1 and 2 are supported on 625 kN (70 ton) driven HP 250x85 (HP 10x57) steel "H" piles. The current project proposes to widen the left side of the existing left bridge by approximately 7.62 m (25 ft.), the right side of the existing left bridge by approximately 6.1 m (20 ft.), and the right side of the existing right bridge by approximately 17.4 m (57 ft.). The proposed widenings are due to the proposed construction of managed lanes down the center of existing Interstate 15.

Geology

Both the 1970 and the 2006/2007 foundation investigations, at the bridge site, revealed alluvial material overlying granitic bedrock. The foundation investigation performed in February 1970 consisted of four mud-rotary borings and one dynamic cone penetration sounding. Boring B-1, drilled near the right bridge Abutment 1 location, revealed the alluvial material extends down approximately 6.1 m (20.0 ft.) where decomposed granitic bedrock was encountered (approx. elev. 187.1 m (614.0 ft)). The alluvial material is described as a compact to slightly compact, sandy silt and silty fine to coarse sand. Below the alluvial material, the granitic bedrock is decomposed to approximate elevation 186.8 m (613.0 ft), where it is becomes slightly weathered to the bottom of boring B-1 (elev. 183.8 m (603.0 ft)).

Boring B-2, drilled near the Escondido Ramp Abutment 1 location, revealed the alluvial material extends down approximately 5.6 m (19.0 ft.) where "weathered" granitic bedrock was encountered (approx. elev. 186.8 m (613.0 ft)). The alluvial material is described as a compact to slightly compact, sandy silt with slight clay binder, and silty fine to coarse sand, and medium to coarse sand. Below the alluvial material, the granitic bedrock is described as "weathered" to elevation 185.6 m (609.0 ft), below which it becomes slightly weathered to the bottom of boring B-2 (elev. 183.5 m (602.0 ft)).

Boring B-3, drilled near the Escondido Ramp Abutment 2 location, revealed the alluvial material extends down approximately 7.6 m (25.0 ft.) where decomposed granitic bedrock was encountered (approx. elev. 184.1 m (604.0 ft)). The alluvial material is described as a very loose, fine to coarse, silty sand to elevation 190.0 m (623.0 ft), where it is underlain by a very loose, sandy silt with clay binder to elevation 187.8 m (616.0 ft). The sandy silt is underlain by a slightly compact to compact, coarse to very coarse sand, to elevation 185.0 m (607.0 ft), where it is underlain by a layer of cobbles to elevation 184.1 m (604.0 ft). At elevation 184.1 m (604.0 ft) the decomposed granitic bedrock is encountered which extends to the bottom of boring B-3 (elev. 182.6 m (599.0 ft)).

Boring B-4, drilled near the right bridge Abutment 2 location, revealed the alluvial material extends down approximately 7.9 m (26.0 ft.) where granitic bedrock was encountered (approx. elev. 183.8 m (603.0 ft)). The alluvial material is described as a sandy silt to elevation 190.5 m (625.0 ft), where it is underlain by a slightly compact, medium to coarse sand, to elevation 185.3 m (608.0 ft). The sand is underlain by very dense granitic gravels to elevation 183.8 m (603.0 ft), where granitic bedrock is encountered, which extends to the bottom of boring B-4 (elev. 183.2 m (601.0 ft)).

The cone penetration sounding, B-5, located near the left bridge, Abutment 1 location, penetrated approximately 6.4 m (21.0 ft) where it hit refusal at approximate elevation 186.2 m (611.0 ft).

The foundation investigation performed in December 2006/January 2007 consisted of two mud-rotary borings, one drilled with a Mobile B-47 drill rig, and one drilled with a CME 750 drill rig.

During the December 2006/January 2007 foundation investigation, boring B-1-06, drilled near the proposed replacement Escondido Ramp Abutment 1 location, revealed the alluvial material extends down approximately 6.4 m (21 ft.) where intensely weathered to decomposed granitic bedrock was encountered (approx. elev. 187.1 m (614.0 ft)). The alluvial material is generally a loose to medium dense, silty sand to poorly graded sand. Below the alluvial material, the granitic bedrock is intensely weathered to decomposed to approximately 10.6 m (34.8 ft) (approx. elev. 182.6 m (599.0 ft)) where it becomes moderately to slightly weathered, moderately hard to hard to the bottom of boring B-1-06 (elev. 176.3 m (578.6 ft)).

Boring B-1-07, drilled near the proposed replacement Escondido Ramp Abutment 2 location, revealed the alluvial material extends down approximately 7.0 m (23.0 ft) where decomposed granitic bedrock was encountered (approx. elev. 185.0 m (607.0 ft)). The alluvial material is generally a loose to medium dense, silty sand and well-graded sand with scattered gravels. Below the alluvial material, the granitic bedrock is decomposed to the bottom of boring B-1-07 (elev. 175.4 m (575.3 ft.)). Refer to the Log of Test Borings (LOTB's) for site specific soil and rock data.

Ground Water

Ground water was encountered during both the 1970 and 2006/2007 subsurface investigations. The measured static groundwater levels are summarized below in Table 1. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions at time of construction.

Table 1: Groundwater Measurements

Boring No.	Date Measured	Groundwater Elevation
B-3-70	02/22/70	188.1 m (617.1 ft)
B-4-70	02/22/70	188.2 m (617.5 ft)
B-5-70	02/20/70	187.8 m (616.0 ft)
B-1-06	01/08/07	188.3 m (617.8 ft)
B-1-07	01/08/07	188.5 m (618.4 ft)

Scour Potential

The Escondido Flood Control Channel is concrete lined, therefore the scour is not considered to be an issue at this site.

Corrosion

Corrosion test results for soil samples collected from borings B-1-06 and B-1-07, are shown below in Table 2. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2: Corrosion Test Summary

Boring	SIC Number	Sample Depth	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1-06	C654037	0-6.46 m (0-21.2 ft)	7.73	4600	N/A	N/A
B-1-07	C654038	0-7.71 m (0-25.3 ft)	7.74	2700	N/A	N/A

Note: Caltrans currently defines a corrosive environment as an area where the soil has a minimum resistivity of less than 1000 ohm-cm, and either contains greater than or equal to 500 ppm of chlorides, greater than or equal to 2000 ppm of sulfates, or has a pH of 5.5 or less

Fault and Seismic Data

The structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Office of Geotechnical Design – South 2, has provided Seismic Design Recommendations for the site in a memorandum dated January 31, 2007. The controlling faults for the site are the Newport-Inglewood-Rose Canyon Fault/E (NIE, strike-slip) located approximately 23 km (14.3 mi.) southwest of the site, which is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.0, and the Whittier-Elsinore Fault (WEE, strike-slip), located approximately 27 km (16.8 mi.) northeast of the site, which is capable of generating a Maximum Credible Earthquake moment magnitude (Mw) of 7.5. The site is located within the peak horizontal bedrock acceleration zone of 0.3g.

Liquefaction Potential

Seismic Design Recommendations, dated January 31, 2007, states that due to the existence of layers of loose sand, beneath the site, there is the potential for soil liquefaction and lateral spreading under strong ground shaking. Based on "As-built" LOTB's and information from borings drilled during the 2006/2007 subsurface investigation, there is approximately 3.7 m (12.0 ft) of potentially liquefiable material beneath the Abutment 1 location, of the left and right bridges, extending from approximate elevation 190.8 m (626.0 ft) to elevation 187.1 m (614.0 ft). This zone would also have the potential for lateral spreading under seismic conditions.

"As-built" plans show that prior to construction of the existing left and right bridges, it was proposed to remove unsuitable material from just north of the Escondido Flood Control Channel, near the existing Abutment 2 locations of the left and right bridges, north to Hale Avenue. "As-built" plans show that just north of the Abutment 2 locations unsuitable material was removed to an approximate depth of 7.0 m (23.0 ft), and replaced with fill material. Further

north, toward Hale Avenue, unsuitable material was removed to depths ranging from 3.0 m (10.0 ft) to 2.1 m (7.0 ft), and replaced with fill material.

Based on "As-built" plans, north of the Escondido flood control channel, unsuitable material was subexcavated down to elevation 186.2 m (611.0 ft), and replaced with fill material. The actual limits of the removal of unsuitable material did not extend as far south to be directly under the Abutment 2 locations of the left and right bridges, nor as far east as the proposed right side widening. Therefore, some of the original liquefiable alluvial material may still be present beneath those locations. Due to removable of the unsuitable material and replacement with fill material, just north of the Abutment 2 locations, liquefaction and lateral spreading at the Abutment 2 locations, of the left and right bridges, is considered less likely than at the Abutment 1 locations. The seismic specialist for the Office of Geotechnical Design South-2, is in the process of providing recommendations related to liquefaction and lateral spreading issues, for the site, which will be forwarded to your office when completed.

Foundation Recommendations

The following recommendations are for the proposed left and right side widening for the left bridge, and the proposed right side widening of the right bridge, of the Escondido Flood Control Channel Bridges (Br. #57-0810 L/R), as shown on the undated General Plan. At Abutments 1 and 2 locations of the widenings, 625 kN (70-ton) design load HP 250x85 (HP 10x57) driven steel "H" piles may be used for support. The recommended specified pile tip elevations are shown below in Tables 3 and 4. The ultimate geotechnical capacity for the piles will meet or exceed the required nominal resistance in compression listed below in Table 3 and 4.

Table 3: Left Bridge, Left and Right Side Widening, Abutments 1 and 2 Pile Data Table

Location	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
			Compression	Tension		
Abutment 1 Left Bridge Left Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	185.9 m (1) 610.0 ft	185.9 m 610.0 ft
Abutment 1 Left Bridge Right Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	185.6 m (1) 609.0 ft	185.6 m 609.0 ft
Abutment 2 Left Bridge Left Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	182.9 m (1) (600.0 ft)	182.9 m (600.0 ft)
Abutment 2 Left Bridge Right Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	183.2 m (1) (601.0 ft)	183.2 m (601.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 4: Right Bridge, Right Side Widening, Abutments 1 and 2 Pile Data Table

Location	Pile Type	Design Loading	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
			Compression	Tension		
Abutment 1 Right Bridge Right Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	186.2 m (1) (611.0 ft)	186.2 m (611.0 ft)
Abutment 2 Right Bridge Right Side Widening	HP 250x85 "H" Piles	625 kN (70.0 tons)	1250 kN (140.0 tons)	-0-	182.9 m (1) (600.0 ft)	182.9 m (600.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

General Notes:

1. All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

Construction Considerations:

1. The calculated geotechnical capacity of the "H" piles is based predominantly on end bearing.

2. Due to variations of the top of bedrock elevation as well as the hardness and degree of weathering of the bedrock material beneath the Abutments 1 and 2 widenings, hard and erratic driving of the "H" piles should be anticipated, and the contractor should anticipate that some piles will require splicing or cutting-off in the field.

3. Pile acceptance criteria for the "H" piles will be in accordance with Standard Specifications Section 49-1.08 "Bearing Value and Penetration". Any pile that achieves two times the required nominal resistance in compression, within 5 ft (1.5 m) of the SPTE, may be considered good and cut off with the Engineer's written approval. Two times the required nominal resistance in compression is 2500 kN (280.0 tons).

4. At the contractor's option, drilling to assist driving may be utilized prior to driving each "H" pile. All drilling to assist driving shall be done in accordance with Standard Specification Section 49-1.05 "Driving Equipment". Drilling to assist driving shall not extend beyond the elevations shown in Tables 5 and 6, below.

Table 5: Left Bridge, Left and Right Side Widening, Abutments 1 and 2 Drilling To Assist Driving Elevations

Location	Bottom of Predrilled Hole Elevation
Abutment 1 Left Bridge Left Side Widening	187.5 m (615.0 ft)
Abutment 1 Left Bridge Right Side Widening	187.1 m (614.0 ft)
Abutment 2 Left Bridge Left Side Widening	184.4 m (605.0 ft)
Abutment 2 Left Bridge Right Side Widening	184.7 m (606.0 ft)

Table 6: Right Bridge, Right Side Widening, Abutments 1 and 2 Drilling To Assist Driving Elevations

Location	Bottom of Predrilled Hole Elevation
Abutment 1 Right Bridge Right Side Widening	187.8 m (616.0 ft)
Abutment 2 Right Bridge Right Side Widening	184.4 m (605.0 ft)

5. After driving the first few "H" piles, utilizing drill-to-assist driving methods, if it is found that two (2) times the nominal resistance in compression is achieved above the SPTE, with the written approval of the engineer, the drill-to-assist driving elevation may be modified to aid in reaching the SPTE shown in Tables 3 and 4, above.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by the Office of Bridge Design-South 1. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565, or Mark DeSalvatore, (916) 227-5391, at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 5/3/07

Erich Neupert

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REGISTERED PROFESSIONAL ENGINEER
MARK DeSALVATORE
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CIVIL
STATE OF CALIFORNIA

cc: R.E. Pending File
John Stayton - Specs & Estimates (4)
David Stebbins – District 11 (Project Manager)
Abbas Abghari – OGDS-II
Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN SERVICES
Office of Special Funded Projects – MS 9-2/7G

Attention: Norbert Gee

Date: March 23, 2007

File: 11-SD-15-KP 49.9
11-2T0821
Hale Avenue UC (Widen)
Bridge No. 57-0811R/L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS 5
OFFICE OF GEOTECHNICAL DESIGN – *SOUTH 2*

Subject: Final Foundation Design Recommendations

This memorandum presents the final foundation design recommendations for the above referenced project on Route 15 in San Diego County. The proposed project includes widening the existing bridge (Bridge No. 57-0811R/L) that is a single-span 55.3m long and 2x28.2m wide concrete box girder superstructure supported on 16-inch diameter CIDH piles at the abutments, as shown on the General Plan and the Foundation Plan for the project in Appendix A. The structure after widening will be 2x36.2m wide minimum. The widened portion of the structure is to be supported on CIDH piles with a diameter of 30 inches.

A subsurface investigation for the proposed Hale Avenue UC widening project was conducted in January 2007. The field investigation was performed in conjunction with that for the Hale Avenue DAR (Direct Access Ramp) retaining wall structures. The investigation consisted of a review of existing site information that includes the as-built Log of Test Borings (LOTB) drilled in the early 1970's, and drilling/logging of five mud-rotary soil borings including Standard Penetration Test (SPT) performed in 2007. Boring locations at the site are shown on a sheet in Appendix B. Soil information from the recent investigation is included on the Plan of Log of Test Borings. The locations and elevations shown on the LOTB for the recently completed borings and referenced in this document are based on the NAD 83 and NAD88 survey benchmarks. Elevations for the as-built LOTBs that are based on NGVD 1929 datum need add 0.63m (2.07ft) to be in consistency with NAD 88 at this site.

Subsurface Conditions

The Log of Test Borings completed in 1970, 1972, and 2007 for this bridge are used for the final foundation design recommendations. The as-built and new plans of Log of Test Borings will be included in the structure construction plans. Subsurface materials below the original ground (Elevation +192.6m or +632 ft) at the site mainly consist of top layers of medium dense to dense silty sand or sandy silt with clay overlying highly decomposed granitic rock at depths from 1.2 to

6m (5 to 15 ft). At Abutments 1R and 2L, a layer (about 1 m or 3 ft thick) of stiff silty clay was encountered within Borings B-2-07 and B-3-07, as well as B-1-70, within a depth of approximately 1.5 meters (5 ft) below the ground surface. The embankment materials (up to 8 meters high) are import soil of mainly fine to medium sand with streaks of clay and decomposed granitic rock.

Groundwater

Groundwater was encountered at a depth of 1.8 to 3.4m (6 to 11ft) below the original ground (Elevation +192.6m or +632 ft) during the drilling in February 1970 and October 1972. Recent groundwater observation was conducted on January 17, 2007 at this site. At the end of drilling, slotted PVC tubes were placed in two bored holes (B-1-07 and B-5-07). After placement of PVC tubes, these holes were flushed with water to ensure less fine soil component/debris was left in the holes and groundwater at different depths freely permeates/flows into the PVC tubes. During the groundwater observation in January 2007, groundwater was encountered at a depth of 10.8 meters (35.3 ft) within B-1-07 and a depth of 3.8 meters (12.5 ft) within B-5-07, which corresponds elevation +190.7 meters (+625.8 ft) within B-1-07 at Abutment 1L and +189.3 meters (+621.2 ft) within B-5-07 at Abutment 2R. Since the two observation holes are located near Abutments 1L and 2R, 70m (230 ft) apart diagonally crossing the site, the two recently measured water tables may be used for the left and right bridges, separately. Thus, a ground water table at elevation +189.3 meters (+621.2 ft) is assumed for soil liquefaction analysis and foundation design for the left bridge and elevation +190.7 meters (+625.8 ft) for the right bridge.

Scour Potential

There is no scour concern since the bridge site is not located in a creek/river channel or in any flood/drainage watercourse.

Corrosion

In general, a minimum resistivity value of less than 1000 ohm-cm for soil and/or water indicates presence of high quantities of soluble salts and a higher propensity for corrosion. Soil and/or water with a resistivity of greater than 1000 ohm-cm may be considered to be non-corrosive. Chloride and sulfate tests are not required if the minimum resistivity is greater than 1,000 ohm-cm. Soil and water that have a minimum resistivity less than 1,000 ohm-cm need to be tested for chlorides and sulfates. As per Caltrans Corrosion Guidelines (September 2003), a bridge site is considered to be corrosive to R/C structure foundations, if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

(a) Chloride concentration is 500 part per million (ppm) or greater,
(b) Sulfate concentration is 2000 ppm or greater,
(c) The pH index is 5.5 or less.

To determine whether the bridge site is corrosive to reinforced concrete structure foundations embedded into ground, eight soil samples were collected at various depths during the foundation investigation and tested at the Caltrans Laboratory in accordance with the applicable California Test Methods. Corrosion test results are summarized below in Table 1. A summary sheet of corrosion test results for the eight soil samples is attached in Appendix C.

The results for one of the eight soil samples collected at this site indicate that the bridge site is corrosive to R/C structure foundations. We recommend that standard design procedures for structure foundations in corrosive areas be used.

Table 1 - Summary Results of Corrosion Test
Hale Avenue UC (Widening) Bridge No. 57-0811R/L

Sample Type	Sample Depth (Boring) meter	pH	Minimum Resistively ohm-cm	Soluble Salts (ppm)		Corrosive
				Sulfates	Chlorides	
Soil	15' (B-1)	7.72	2700	N/A	N/A	No
Soil	48' (B-1)	8.58	4700	N/A	N/A	No
Soil	20' (B-2)	7.52	7100	N/A	N/A	No
Soil	40' (B-2)	8.30	1600	N/A	N/A	No
Soil	30' (B-3)	9.14	650	49	15	No
Soil	15' (B-4)	7.84	10000	N/A	N/A	No
Soil	10' (B-5)	8.02	500	508	522	Yes
Soil	20' (B-5)	8.00	5700	N/A	N/A	No

Controlling Active Fault and Surface Rupture

As shown on the attached California Seismic Hazard Map 1996 by the California Department of Transportation in Appendix D, within 27 kilometers of the bridge site, there are two active faults that both stretch from northwest to southeast: the Newport-Inglewood-Rose Canyon/E (NIE, Mw = 7, style Strike-slip) fault on the southwest side and the Whittier-Elsinore (WEE, Mw = 7.5, style Strike-slip) fault on the northeast side. The distances between the bridge site and the two faults (NIE & WEE) are respectively 22.9 and 26.7 kilometers. The controlling fault for the bridge structure design is the WEE fault which is capable of generating a maximum credible earthquake of moment magnitude Mw = 7.5. The horizontal Peak Bedrock Acceleration (PBA) at this site is estimated to be 0.2g (gravity acceleration) based on the 1996 Geomatrix attenuation relationship and to be 0.3g according to the Caltrans Hazard Map. For structural design of the bridge, a PBA = 0.3g is conservatively recommended as was in our previous preliminary seismic report. However, based on the estimated PBA = 0.2g with the attenuation, a Peak Ground Acceleration of 0.3g may be used for liquefaction evaluation and retaining wall design.

The surface rupture due to fault movement is not a concern since there is no known fault crossing through the bridge site.

Potential for Liquefaction and Lateral Spreading

The Log of Test Borings (LOTB) completed in 1970, 1972, and 2007 for this bridge are used for the seismic design recommendations. Subsurface materials at the site consist of top layers of medium dense to dense silty sand or sandy silt with clay overlying highly decomposed granitic rock. Different groundwater tables are assumed for the left and right bridges as discussed above. A ground water table at elevation +189.3 meters (+621.2 ft) is assumed for the left bridge and elevation +190.7 meters (+625.8 ft) for the right bridge. Based on the logs, the site is not susceptible to soil liquefaction during earthquake shaking (PGA = 0.3g) and the potential for lateral spreading at the bridge site is considered to be very low.

Horizontal ARS Curve

For this seismic design recommendation, the soil profile at this site may be classified as a Type D as defined in the Caltrans Seismic Design Criteria (SDC). For structural design of the bridge, a SDC Acceleration Response Spectrum (ARS) corresponding to Type D soil profile, $M_w = 7.5$, and PBA of 0.3g is recommended as provided in Appendix D.

Slope Stability of Embankment at Abutments

Slope instability of approach fills under seismic shaking may displace abutments and cause significant differential settlement and structural damage. Approach settlement slabs providing structural support between approach fills and abutments should be provided for this bridge since the site is one with $M_w = 7.5$, and PGA = 0.3g. Slabs should be adequately linked to abutments using flexible ties. Caltrans Standard Structure Approach Type EQ may well serve this purpose.

The global slope stability of approach embankment under seismic shaking is evaluated by performing pseudo-static slope stability analysis using the program XSTABL. It is assumed in the pseudo-static analyses that a seismic acceleration coefficient, $k_h = 0.1$, equal to one third of the peak ground acceleration coefficient is used while the effect of vertical acceleration is ignored. It is found from the XSTABL results as presented in Appendix E for the approach slope at Abutment 1 (decomposed bedrock at Abutment 1 is deeper than that at Abutment 2) that the factor of safety FS = 1.55 for static case and FS = 1.29 for seismic case: $k_h = 0.1$. As the factor of safety for seismic loading is greater than 1.1, the approach slope at sides of the abutment is considered to be stable during the designated earthquake shaking.

Settlement of Embankment at Abutments

Before constructing the new widened structure, approximately 10 to 20 meters wide and up to 8 meters high new approach fills are to be placed on each side of the existing embankment at

Abutments 1 and 2. Placement of new fills should conform to the provisions in Section 16, "Clearing and Grubbing" and Section 19, "Earthwork" of Caltrans Standard Specifications. The new "patched" approach fills, as surcharge on the foundation soils on both sides of the embankment will cause differential settlement relative to the existing embankment at the center. The maximum differential settlement between the toe of the new slope and the center of the embankment is estimated to range from 60 to 90 mm. Since subsurface materials at the site mainly are medium dense to dense silty sand or sandy silt, this settlement is primarily "elastic" and is expected to finish within 30 days after placement of the approach fill.

Soil Passive Pressure on Abutment Walls

According to Caltrans SDC (2001), Section 7.8, an ultimate/maximum soil passive pressure of 5 kip/ft^2 (239 kPa) for a wall height h = 5.5 ft (1.7 m) may be utilized for resisting the movement at the abutment in seismic analysis. A height proportionality factor of h/5.5 ft (h/1.7 m) should be applied to reduce the maximum soil pressure if the effective wall retaining height is different from 5.5 ft (1.7 m). It is advised to apply a limit of soil passive pressure, p_{ult} = 5 kip/ft^2 (239 kPa) since the height proportionality factor of h/5.5 ft (h/1.7 m) results in a linear increase in soil pressure and a higher value than 5 kip/ft^2 when the effective wall height is greater than 5.5 ft (1.7 m). It is typically estimated that soil passive pressure behind an abutment wall is mobilized as the wall moves laterally two percent of the wall height toward the backfill.

Static Earth Pressures on Abutment Walls

Static earth pressures on abutment walls should be considered as per Caltrans Bridge Design Specifications (BDS, 2004), Section 3.20. A minimum equivalent fluid pressure of 5.65 kPa per meter of wall height (36 lbs/ft^2 per linear foot of height) should be applied for design of the wall. A static earth pressure in terms of an equivalent fluid pressure of 9.42 kPa per meter of wall height (60 lbs/ft^2 per linear foot of height) may be applied if the wall movement is restrained by piles or the superstructure. To account for the effect of highway traffic/vehicle loads on the abutment wall, an additional uniform lateral pressure of 3.5 kPa (72 lbs/ft^2) should be applied, which is equivalent to a live load surcharge of 0.6m (2 ft) of earth. This live load surcharge may cause an additional uniform lateral pressure of up to 5.75 kPa (120 lbs/ft^2) on a wall if its lateral movement is restrained by piles or the superstructure. However, this live load surcharge need not be considered when an adequately designed reinforced concrete approach slab supported at one end by the bridge is provided. The values of earth pressure do not account for any hydrostatic pressure from standing water in the backfill materials behind the wall since weep holes and a wall drain with filter or drainage materials are to be provided as per Caltrans Standard Plans or Specifications.

Seismic Earth Pressures on Abutment Walls

Seismic earth pressure on abutment walls may be estimated using the Mononobe-Okabe method as recommended by AASHTO (1996) – Section 6.4.3. Seismic earth pressure/thrust on a wall is a

function of the lateral displacement or stiffness of the wall among other factors. Two extreme displacement cases are usually considered to estimate seismic earth pressure on a wall: (a) large displacement or low stiffness case in which an abutment or retaining wall may move laterally (say, a displacement of up to 250A (mm), in which A is the peak ground acceleration coefficient) without much restrain from superstructure or supporting foundations, and (b) small displacement or high stiffness case in which a wall is restrained from any significant lateral movement by superstructure or/and anchors or/and batter piles. A reduced seismic acceleration coefficient equal to one half the peak ground acceleration coefficient, $k_h = 0.5A$ (k_h is the seismic acceleration coefficient) for case (a) and an enlarged coefficient up to one and half the peak ground acceleration coefficient, $k_h = 1.5A$ for case (b) are assumed for use as an input in the Mononobe-Okabe analysis. Since A = 0.3g at the bridge site, $k_h = 0.15$ for case (a) and $k_h = 0.45$ for case (b) may be used for the pseudo-static analyses. For case (a) in which the wall may experience a displacement of up to 75 mm (3 inches) under seismic loading, seismic active earth pressure (not including static earth pressure) on the wall is estimated as $0.94H^2B$ kN, in which H is the wall height in meters and B (in meters) the effective retaining width of the wall. A seismic active earth pressure of $4.21H^2B$ kN, which is more than twice as much as the pressure for case (a), may be applied to case (b) where high stiffness is expected from the superstructure restraints or pile support. The total seismic earth thrust on the wall may be calculated as the seismic earth pressure ($p_{eq} = 0.94H$ or $4.21H$) times the retaining wall area (HB). The point of application for the seismic lateral soil thrust may be assumed at 0.5H – mid-height of the wall.

Pile Tip Elevations

750mm (30-inch) diameter CIDH piles are proposed for support at Abutments 1 and 2. The design load per pile ranges from 925 kN to 1160 kN as shown in the following table.

Table 2 – 750mm CIDH Pile Data Table for Abutments

Abutment Location	Pile Type	Design Load (kN)	Nominal Resistance (kN)		Cut-Off Elevation (m)	Design Tip Elevation (m)	Specified Tip Elevation (m)
			Compression	Tension			
1L	750^{mm} CIDH	1070	2140	0	+195.165	+183.460	+183.460
2LL	750^{mm} CIDH	1160	2320	0	+191.345	+178.460	+178.460
2LC	750^{mm} CIDH	1160	2320	0	+191.845	+178.460	+178.460
2LR	750^{mm} CIDH	1160	2320	0	+192.445	+178.460	+178.460
1R	750^{mm} CIDH	925	1850	0	+195.425	+185.080	+185.080
2R	750^{mm} CIDH	950	1900	0	+195.575	+185.530	+185.530

Note: Design tip elevation based on: (1) compression

The specified pile tip elevations (SPTE) are determined so that the ultimate geotechnical capacities of the piles will not be less than the required nominal resistance in compression as shown above in Table 2, as well as in Appendix F. The compressive nominal resistance for piles at Abutments 1 & 2 is obtained by multiplying the working stress design load by a safety factor of 2. The cut-off elevation is the bottom of footing elevation plus 125mm (5-inch) embedment into the pile cap for the left bridge and 75mm for the right bridge. Skin resistance of the pre-drilled portion is ignored for piles penetrating through the embankment fill.

To determine the pile lateral capacity under service and seismic loading conditions, the lateral load – deflection curves for 750mm (30-inch) diameter CIDH piles can be provided on request.

Soil Parameters for Lateral Pile Analysis using LPILE

Soil parameters were provided in Appendix G for lateral pile analysis using the LPILE program. The soil parameters were estimated from correlations with Standard Penetration Testing (SPT) blow counts and soil models as presented in the Technical Manual for LPILE 5 (October, 2004). The calculations were conducted based on the Log of Test Borings completed in 1970 and 2007 for this bridge site. The tabulated soil data and layer elevations are given in the tables in Appendix G for each of the support locations. Specifically, the soil effective unit weight, undrained shear strength, angle of internal friction, initial modulus of subgrade reaction, and soil type of p-y model are given in Column 8, 12, 13, 15, and 16 respectively. The ground elevation at the abutments is assumed at the pile cut-off elevation to not consider the overburden effect from the approach fills at the abutments. LPILE seems to overestimate the lateral pile stiffness for a pile with an embedded pile head below the ground. Thus it is advised that a reduced embedment depth below the ground surface to the embedded pile head be used to discount the overburden effect in an LPILE analysis.

Construction Considerations

1. Placement of concrete for structure footings should conform to the provisions in Section 19-3, "Structure Excavation and Backfill" and Section 51, "Concrete Structures" of Caltrans Standard Specifications. The concrete for pile caps shall be placed neat against suitable foundation material as determined by the Engineer at the bottom of footing elevation. If over-excavation is required to remove unsuitable materials at the bottom of footing, Section 19-3.06 of Caltrans Standard Specifications should be applied.

2. The 750mm (30") diameter CIDH piles are to be constructed in subsurface materials ranging from medium dense to dense silty sand or sandy silt with clay, to highly or moderately decomposed granitic rock. The contractor should anticipate difficult drilling conditions that exceed those normally encountered in alluvial soil and should be advised to use suitable drilling or coring equipment for the construction of the CIDH piles. Augering or drilling through sandy soil or decomposed rock may cause caving condition in hole being drilled. The contractor's attention should be drawn to Section 49-4.01 and 49-4.03 of Caltrans Standard Specifications. A

drilled hole should be filled with concrete on the same day it is drilled, or the hole should be protected from caving.

3. The SPTE of the CIDH piles at the abutments is below the observed groundwater that is approximately 1.8 – 3.4 meters (6 – 11 ft) below the current local street surface. The contractor should anticipate that groundwater may be higher during pile construction if it is in wet or rainy season. Any temporary casing used to construct the CIDH piles shall be removed during concrete placement.

4. The calculated geotechnical capacity of the CIDH piles is based on skin friction only. All drilled holes are to be cleaned out and inspected prior to placement of reinforcing cage or placing the concrete. Any wet-constructed CIDH piles should be inspected by Gamma-Gamma logging through pre-installed PVC tubes. Memos to Designers 3-1, Attachment B should be referenced for layout details of inspection PVC tubes.

5. The Engineer shall specify in the special provisions the requirements of Tunnel Safety Orders for the CIDH pile work that meets the definition of a tunnel or shaft, as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Discussion

The foundation design recommendations contained in this report are based on specific project information regarding the proposed structure, loading conditions and foundation dimensions provided by the Structure Designer. If any conceptual changes are made during final project design, this Office should be notified of those changes and review the changes to determine whether the foundation recommendations are still applicable.

If there are any questions or comments, please contact Jin-xing Zha at (916) 227-4519 or Calnet 8-498-4519 or Angel Perez-Cobo at (916) 227-7167 or Calnet 8-498-7167.

JIN-XING ZHA
Transportation Engineer – Civil
Office of Geotechnical Design – *South 2*

ANGEL PEREZ-COBO
Senior Transportation Engineer
Office of Geotechnical Design – *South 2*

Attachments

c: File Room in Trabslab

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Structures Design
Office of Bridge Design-South 1
MS #9 - 5/6G

Date: March 15, 2007

File: 11-SD-15-KP R50.2
11-2T0821
Washington Ave. OH
(Widen)
Br. #57-0812R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South II MS #5
Design Branch B

Subject: Foundation Report

This report presents the foundation recommendations for the proposed right side widening to the Washington Ave. OH right bridge (Br. #57-0812R). The Office of Geotechnical Design-South 2, Design Branch B completed a foundation investigation pursuant to a request by District 11 for a foundation investigation and recommendations for the proposed structure.

The following foundation recommendations are based on "as-built" information from 1970 and 1972 subsurface investigations, foundation recommendations from 1970, 1971, and 1973, as well as information gathered during the recent foundation investigation (December 2006/January 2007) performed by Caltrans. With regards to the current foundation recommendations, all elevations referenced within this report and shown on the recent Log of Test Boring sheets are based on the NAVD 1988 vertical datum (as-built elevations have been converted from NGVD 1929 datum to NAVD 1988 datum using a conversion factor of +0.63 m (+2.07 ft.)).

Project Description/History

The project site is located on Interstate 15 in the city of Escondido, San Diego County. The bridge site is located in a small valley surrounded by moderately low to steep rolling hills. The existing Washington Ave. OH bridges are four span, cast-in-place, prestressed, concrete box-girder structures with end-diaphragm abutments, which were built in 1977. For the left and right bridges, Abutments 1 and 5 are supported on 625 kN (70 ton) design load 400 mm (16 in.) Cast-In-Drilled-Hole (CIDH) piles, and Bents 2, 3, and 4 are supported on 5800 kN (650 ton) design load 2100 mm (84 in.) CIDH piles. The current project proposes to widen the right side of the right bridge by approximately 4.6 m (15 ft). The proposed widening is due to the proposed construction of managed lanes down the center of existing Interstate 15.

Geology

The foundation investigation performed in 1970 consisted of three mud-rotary borings and two dynamic cone penetration soundings. The 1972 investigation consisted of two bucket auger borings, and the 2006/2007 subsurface investigation consisted of two mud-rotary borings, one drilled with a B-47 drill rig and one drilled with a CME 750 drill rig. The 1970, 1972, and 2006/2007 subsurface investigations revealed that the bridge site is underlain by alluvial material overlying granitic bedrock. The alluvial material varies in thickness across the bridge site, ranging from approximately 1.5 m (4.8 ft) thick in boring B-4 (1970) to approximately 4.1 m (13.6 ft) thick in boring B-2 (1970). Generally, the alluvial material consists of medium dense to very dense sandy silt, silty sand, clayey sand, with some gravel. For specific details regarding alluvium thickness and descriptions, refer to the LOTB sheets.

Underlying the alluvium is granitic bedrock, which is generally decomposed to intensely weathered, near the surface, and then becomes harder with depth. The structure of the bedrock is chaotic, and results in discontinuous, non-fractured, moderately hard to very hard cobble to boulder sized blocks of more competent rock within an intensely fractured, very intensely weathered to decomposed, moderately soft to very soft, rock material. During the 2006/2007 field investigation, rock samples were collected from borings B-1-06 and B-1-07 at five-foot (1.52 m) intervals, when possible, and were submitted to the laboratory for strength testing. Some of the samples were so weak that they were unable to be tested. The ranges of unconfined compressive strength, for the rock samples that were tested, are shown below in Table 1. Refer to the LOTB's for site-specific soils and rock data.

Table 1: Rock Unconfined Compressive Strengths (q_u)

Boring	Ranges of Sample Depth	Ranges of Laboratory Unconfined Compressive Strength Values
B-1-06	10.5-20.6 m (34.5 – 67.5 ft)	57,456 –79,000 kPa (600 to 825 tsf)
B-1-07	12.0-17.2 m (39.5 – 56.5 ft)	31,697 – 115,679 kPa (331 to 1208 tsf)

Note: Unconfined compressive strength values shown are the ranges for only the samples that were able to be tested by the laboratory. Some samples submitted to the laboratory were too soft to be tested, or disintegrated upon preparation for testing.

Ground Water

Ground water was encountered during all subsurface investigations at the bridge site. In February 1970, ground water was measured at elevations 191.0 m (626.7 ft) and 190.3 m (624.3 ft) in borings B-1 and B-2, respectively. In October 1972, ground water was measured at elevations 188.5 m (618.5 ft) and 188.7 m (619.0 ft) in borings B-1a and B-2a, respectively. In December 2006 ground water was measured in boring B-1-06 at elevation 190.9 m (626.3 ft), and in January 2007 ground water was measured in boring B-1-07 at elevation 190.8 m (626.0 ft). Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions at time of construction.

Scour Potential

There is no scour potential at the site, since the structure does not span any watercourse.

Corrosion

Corrosion test results for soil samples collected from borings B-1-06 and B-1-07 are shown below in Table 2. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 2: Corrosion Test Summary

Boring	SIC Number	Sample Depth	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1-06	C654039	0-3.6 m (0-11.8 ft)	8.47	1200	N/A	N/A
B-1-07	C654040	0-2.9 m (0-9.6 ft)	8.07	2400	N/A	N/A

Note: Caltrans currently defines a corrosive environment as an area where the soil has a minimum resistivity of less than 1000 ohm-cm, and either contains greater than or equal to 500 ppm of chlorides, greater than or equal to 2000 ppm of sulfates, or has a pH of 5.5 or less.

Fault and Seismic Data

The structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Office of Geotechnical Design-South 2 has provided final seismic design recommendations in a memorandum dated January 25, 2007. The controlling faults for the site are the Newport-Inglewood-Rose Canyon Fault/E (NIE, strike-slip), located approximately 23 km (14 miles) southwest of the site, and the Whittier-Elsinore Fault (WEE, strike-slip) located approximately 27 km (17 miles) northeast of the site. The Newport-Inglewood-Rose Canyon Fault/E fault is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.0 and a corresponding Peak Bedrock Acceleration is estimated to be 0.3g. The Whittier-Elsinore Fault is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.5 and a corresponding Peak Bedrock Acceleration is estimated to be 0.3g.

Liquefaction Potential

Based on the Final Seismic Recommendations, dated January 25, 2007, the potential for soil liquefaction under strong ground shaking is considered to be low.

"As-built" Foundations

The existing Washington Ave. UC left and right bridges are four span structures completed in 1977. "As-built" pile tip elevations for the CIDH piles are shown below in Table 3.

Table 3: "As-built" CIDH Pile Data

Location	Pile Type	Design Loading	Design Tip Elevation	Average "As-built" Pile Tip Elevation	Maximum "As-built" Pile Tip Elevation	Minimum "As-built" Pile Tip Elevation
Abutment 1 Left Bridge	400 mm (16 inch) CIDH	625 kN (70 tons)	189.00 m (620.0 ft)	188.90 m (619.7 ft)	189.00 m (620.1 ft)	188.6 m (618.8 ft)
Abutment 1 Right Bridge	400 mm (16 inch) CIDH	625 kN (70 tons)	189.00 m (620.0 ft)	188.10 m (617.2 ft)	188.9 m (619.7 ft)	187.9 m (616.6 ft)
Bents 2, 3, 4 Left & Right Bridges	2100 mm (84 inch) CIDH	5800 kN (650 tons)	187.50 m (615.0 ft)	187.50 m (615.0 ft)	187.5 m (615.0 ft)	187.5 m (615.0 ft)
Abutment 5 Left Bridge	400 mm (16 inch) CIDH	625 kN (70 tons)	189.00 m (620.0 ft)	188.90 m (619.9 ft)	189.3 m (621.0 ft)	188.5 m (618.6 ft)
Abutment 5 Right Bridge	400 mm (16 inch) CIDH	625 kN (70 tons)	189.00 m (620.0 ft)	188.90 m (619.7 ft)	189.4 m (621.5 ft)	188.5 m (618.6 ft)

Note: Elevations is Table 3 are based on NGVD 1929 vertical datum.

Foundation Recommendations

The following recommendations are for the proposed right-side widening of the Washington Ave. OH right bridge (Br. #57-0812 R), as shown on the undated General Plan received on March 14, 2007 from TY Lin, Int'l. At Abutments 1 and 5 support locations either 600 mm (24 in.) or 750 mm (30 in.) diameter Cast-In-Drilled-Hole (CIDH) piles may be used for support. At Bents 2, 3, and 4 locations, 2400 mm (96 in.) diameter CIDH piles may be used for support. At the Bent 2 location, due to the proximity to a railroad track, a permanent steel casing, installed through the alluvial material, will be used to facilitate construction of the CIDH shaft. At Bents 3 and 4 locations, spread footings may also be used for support as an alternative to large diameter CIDH piles. The specified pile tip elevations for 600 mm and 750 mm CIDH piles, with design loads of 625 kN (70 tons) and 900 kN (100 tons) are shown below in Tables 4, 5, 6, and 7. The specified pile tip elevations for 2400 mm (96 in.) CIDH piles at Bents 2, 3, and 4, are shown below in Table 8. The ultimate geotechnical pile capacity for the CIDH piles will meet or exceed the required nominal resistance in compression. Refer to Tables 4, 5, 6, 7 and 8, below, for the required nominal resistances and specified pile tip elevations.

Table 4: Abutments 1 and 5, 600 mm (24 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance (CIDH)		Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	N/A	185.0 m(1) (607.0 ft)	185.0 m (607.0 ft)
Abutment 5	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	N/A	186.8 m(1) (613.0 ft)	186.8 m (613.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 5: Abutments 1 and 5, 600 mm (24 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance (CIDH)		Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	N/A	183.8 m(1) (603.0 ft)	183.8 m (603.0 ft)
Abutment 5	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	N/A	185.6 m(1) (609.0 ft)	185.6 m (609.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 6: Abutments 1 and 5, 750 mm (30 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance (CIDH)		Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	N/A	185.5 m(1) (608.5 ft)	185.5 m (608.5 ft)
Abutment 5	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 7: Abutments 1 and 5, 750 mm (30 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance (CIDH)		Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension			
Abutment 1	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	N/A	184.6 m(1) (605.5 ft)	184.6 m (605.5 ft)
Abutment 5	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	N/A	186.4 m(1) (611.5 ft)	186.4 m (611.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 8: Bents 2, 3, and 4 CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance (CIDH)		Nominal Resistance (Steel Casing)		Pile Cut-off Elevation	Specified Tip Elevation (Casing)*	Design Tip Elevation (CIDH)	Specified Tip Elevation (CIDH)
			Compression	Tension	Compression	Tension				
Bent 2	2400 mm (96 inch) CIDH w/ Steel Casing	N/A	5700 kN (640 tons)	-0-	-0-	-0-	193.4 m (634.5 ft)	189.0 m (620.0 ft.)	181.4 m(1) (595.0 ft)	181.4 m (595.0 ft)
Bent 3	2400 mm (96 inch) CIDH	N/A	7100 kN (793 tons)	-0-	N/A	N/A	193.8 m (635.8 ft)	N/A	182.9 m(1) (600.0 ft)	182.9 m (600.0 ft)
Bent 4	2400 mm (96 inch) CIDH	N/A	6600 kN (740 tons)	-0-	N/A	N/A	193.0 m (633.2 ft)	N/A	182.9 m(1) (600.0 ft)	182.9 m (600.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression
*Permanent Steel Casing specified tip elevation is based on the lowest estimated top-of-bedrock elevation from boring nearest the support location.

Bents 3 and 4 Spread Footing Option

At Bents 3 and 4, spread footings may also be used for support. The recommended Nominal Bearing Resistances to be used for design, bottom of footing elevations and minimum footing width dimensions are summarized below in Table 9.

Table 9: Bents 3 and 4 - Spread Footing Data

Support Location	Minimum Footing Width	Bottom of Footing Elevation	Recommended Bearing Limits	
			WSD [1]	LFD [2]
			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Bent 3	4.27 m (14.0 ft)	190.8 m (626.0 ft)	N/A	766.1 kPa (16.0 ksf)
Bent 4	4.27 m (14.0 ft)	190.2 m (624.0 ft)	N/A	766.1 kPa (16.0 ksf)

Notes: 1) Working Stress Design (WSD): the Maximum Contact Pressure (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity (q_{all}). 2) Load Factor Design, (LFD): The Maximum Contact Pressure (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance (q_n).

The recommended Nominal Bearing Resistances to be used for design, provided in Table 9, above, are based on the following design criteria:

1) Bents 3 and 4 footings have a minimum width as shown in Table 9.

2) Bents 3 and 4 footings are to be constructed at or below the recommended elevation as shown in Table 9.

If the above minimum footing widths are reduced or bottom of footing elevation are raised, the Office of Geotechnical Design-South 2, Branch B, is to be contacted for reevaluation.

General Notes:

1. All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

2. When applicable, the structure engineer shall show on the plans, in the pile data table, the design pile tip elevation required to meet the lateral load demands. If the design pile tip elevation required to meet lateral load demands exceeds the specified pile tip elevations given within this report, the Office of Geotechnical Design-South 2, Branch B shall be contacted for further recommendations.

3. The District engineer shall specify in the special provisions the requirements of Tunnel Safety Orders for the CIDH pile work that meets the definition of a tunnel or shaft, as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Construction Considerations:

Rock Cores:

- Rock core samples from the 2006/2007 Caltrans foundation investigation are available for viewing by bidders at the California Department of Transportation, Transportation Laboratory, 5900 Folsom Blvd., Sacramento, CA. The bidders requesting to view rock cores are to allow the State five (5) working days to prepare and display the cores.

- Specific rock strength test data is available upon request.

CIDH Piles:

- Groundwater was encountered during the 1970, 1972, and 2006/2007 field investigation and it is anticipated that the contractor will encounter groundwater during CIDH pile construction. Groundwater levels indicated on the LOTB reflect the measured groundwater levels at the time of the Caltrans investigation. At the time of construction, the groundwater surface elevations may be significantly higher or lower than those shown on the LOTB due to seasonal fluctuations.

- Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity of all CIDH piles is based on skin friction and no end-bearing was considered. The skin friction zones used to calculate geotechnical capacity of the CIDH piles are summarized below in Table 10.

Table 10: CIDH Pile Skin Friction Zone Elevations

Location	Pile Diameter	Design Loading	Nominal Resistance		Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
			Compression	Tension		
Abutment 1	600 mm (24 in)	625 kN (70 tons)	1250 kN (140 tons)	-0-	189.0 m (620.0 ft)	185.6 m (609.0 ft)
Abutment 1	600 mm (24 in)	900 kN (100 tons)	1800 kN (200 tons)	-0-	189.0 m (620.0 ft)	184.4 m (605.0 ft)
Abutment 1	750 mm (30 in)	625 kN (70 tons)	1250 kN (140 tons)	-0-	189.0 m (620.0 ft)	186.2 m (611.0 ft)
Abutment 1	750 mm (30 in)	900 kN (100 tons)	1800 kN (200 tons)	-0-	189.0 m (620.0 ft)	185.3 m (608.0 ft)
Bent 2	2400 mm (96 inch)	N/A	5700 kN (640 tons)	-0-	189.0 m (620.0 ft)	183.8 m (603.0 ft)
Bent 3	2400 mm (96 inch)	N/A	7100 kN (793 tons)	-0-	191.4 m (627.8 ft)	185.3 m (608.0 ft)
Bent 4	2400 mm (96 inch)	N/A	6600 kN (740 tons)	-0-	190.6 m (625.2 ft)	185.3 m (608.0 ft)
Abutment 5	600 mm (24 in)	625 kN (70 tons)	1250 kN (140 tons)	-0-	190.8 m (626.0 ft)	187.5 m (615.0 ft)
Abutment 5	600 mm (24 in)	900 kN (100 tons)	1800 kN (200 tons)	-0-	190.8 m (626.0 ft)	186.2 m (611.0 ft)
Abutment 5	750 mm (30 in)	625 kN (70 tons)	1250 kN (140 tons)	-0-	190.8 m (626.0 ft)	188.1 m (617.0 ft)
Abutment 5	750 mm (30 in)	900 kN (100 tons)	1800 kN (200 tons)	-0-	190.8 m (626.0 ft)	187.1 m (614.0 ft)

- The contractor should anticipate having to use slurry displacement methods to construct the CIDH piles. Allowable slurries shall consist of mineral or synthetic slurry only. Use of water as slurry shall not be allowed.

- At Bent 2 support location, to prevent caving conditions which may adversely affect the nearby railroad track, permanent steel casing will be installed through the alluvial material to the specified casing tip elevation stated in Table 8, above.

- Due to there being no axial geotechnical capacity demand on the permanent steel casing, at Bent 2, the method to install the permanent steel casing down to the specified casing tip elevation is left to the contractor. Due to lateral demands on the CIDH piles, the method chosen by the contractor to place the permanent steel casing shall result in the permanent steel casing being installed tight within the undisturbed native material. No voids surrounding the casing shall be allowed.

- At the Washington Ave OH location, the granitic bedrock, beneath the bridge site, is extremely variable in hardness, ranging from very soft to very hard. The structure of the bedrock is chaotic, and results in discontinuous, non-fractured, moderately hard to very hard cobble to boulder sized blocks of more competent rock within a very intensely weathered to decomposed, intensely fractured, moderately soft to very soft, rock material. Therefore, at all support locations where CIDH piles are to be constructed, the contractor should anticipate variable drilling conditions in the bedrock similar to the drilling conditions of very dense soils with very to extremely hard cobbles and boulders. The contractor should also anticipate the need for soft and hard rock drilling techniques to extend the drilled holes for the CIDH piles to the SPTE.

- If the contractor decides to utilize a simple auger method to drill the CIDH piles at Abutments 1 and 5, and Bents 2 through 4 locations, the contractor should be aware that the rotating action of the auger within the fractured rock matrix will produce material being removed from the excavation that will appear to be cobbles and boulders.

- At all support locations where CIDH piles are to be constructed, due to the weak and intensely fractured nature of the upper zones of the underlying granitic material, caving conditions should be anticipated during CIDH pile construction. The amount of caving the contractor will experience will be dependent upon the method of excavation the contractor chooses, and the speed which the contractor chooses to advance the drilled hole. Temporary casing may be necessary to control caving during construction. Use of temporary casings shall conform to Standard Specifications 49-4.03. All temporary casing is to be removed during concrete placement.

- The contractor will need to use care while drilling the shafts for the CIDH piles using slurry. Due to the decomposed nature of the upper zones of the granitic bedrock, rapid insertion and removal of the drilling tools during the drilling process can cause excessive scouring and caving of the walls of the drilled shaft.

- Variations in the weathering, fracturing, and hardness of the bedrock occurred both laterally and vertically across the site, as shown on the LOTB sheets. The contractor should anticipate both soft and hard rock drilling conditions across the bridge site. The amount of difficulty the contractor will experience will be dependent upon the methods and means the contractor chooses to construct the CIDH piles.

- The calculated geotechnical capacity of the CIDH piles is based upon skin friction and relies upon a concrete bond to the rock walls of the drilled shaft. It is imperative that the borehole walls are not contaminated with drill cuttings or loose materials. The use of rotator or oscillator methods for drilling of CIDH piles will allow for alluvium or rock cuttings to be trapped between the borehole walls and rotator or oscillator drill rod resulting in a capacity reduction of the CIDH piles. Rotator or oscillator drilling methods are not to be allowed to drill the shafts for the CIDH piles.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by TY Lin, Int'l. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 3/15/07

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John Stayton - Specs & Estimates (4)
Jeremy LaHaye-TY Lin
David Stebbins – District 11 (Project Manager)
Abbas Abghari – OGDS-2
Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Structures Design
Office of Bridge Design-South 1
MS #9 - 5/6G

Date: March 28, 2007

File: 11-SD-15-KP R50.2
11-2T0821
Washington Ave. OH
(Widen)
Br. #57-0812R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: Supplemental Foundation Report

This memorandum contains supplemental recommendations to the "original" Foundation Report, dated March 15, 2007, for the proposed right side widening to the Washington Ave. OH right bridge (Br. #57-0812R). This Supplemental Foundation Report is to provide additional construction considerations addressing the spread-footing option at Bents 3 and 4 locations.

Construction Considerations:

At the proposed right bridge, right-side widening Bents 3 and 4 footing locations, where the bottom of footings will be located on native bedrock material:

- Concrete for the support footings shall be placed neat against the undisturbed bedrock material along the trimmed walls and bottom of footing excavations. Should the bottom of footing excavations be disturbed, then the bottom of the footing excavations shall be extended down at 0.15-meter (0.5 ft) intervals until undisturbed bedrock material is observed and approved by the Engineer. The subexcavated material is to be replaced with lean concrete. The disturbed native material is not to be recompacted.

- The footing excavations are to be inspected and approved by a representative of the Office of Geotechnical Design-South 2, Branch B, prior to placing any steel or concrete for the support footings in the excavations. The required inspection is to verify that the concrete for the support footings are being placed neat on top of the native bedrock material. Once the excavations have been completed to the specified elevations, the contractor is to allow the Office of Geotechnical Design–South 2, Branch B, five (5) working days to perform the inspection. The structures representative is to provide the Office of Geotechnical Design–South 2, Branch B, a one-week notification prior to beginning the five-day contractor waiting period.

All other recommendations, general notes, and construction considerations in the "original" Foundation Report, dated March 15, 2007, are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 3/28/07

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cc: R.E. Pending File
John Stayton - Specs & Estimates (4)
Jeremy LaHaye-TY Lin
David Stebbins – District 11 (Project Manager)
Abbas Abghari – OGDS-2
Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Structures Design
Office of Bridge Design-South 1
MS #9 - 5/6G

Date: February 15, 2007

File: 11-SD-15-KP R50.4
11-2T0821
Mission Ave. UC (Widen)
Br. #57-0813 R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South II MS #5
Design Branch B

Subject: Foundation Report

This report presents the foundation recommendations for the proposed tapered widening to the right side of the Mission Ave. UC right bridge (Br. #57-0813 R) pursuant to a request by District 11 for foundation recommendations for the proposed structure.

The following foundation recommendations are based on a review of "as-built" plans, the foundation investigation and foundation report from 1970, the field investigation of 1972, and corrosion samples collected in January 2007. With regards to the current foundation recommendations, all elevations referenced within this report are based on the NAVD 1988 vertical datum (as-built elevations have been converted from NGVD 1929 datum to NAVD 1988 datum using a conversion factor of +0.63 m (+2.07 ft)).

Project Description/History

The project site is located on Interstate 15 in the city of Escondido, San Diego County. The bridge site is located in a small valley surrounded by moderately low to steep rolling hills. The existing Mission Ave. UC bridges are three span, cast-in-place, prestressed, concrete box-girder structures with end-diaphragm abutments, which were built in 1977. For the left and right bridges, Abutments 1 and 4 are supported on 625 kN (70 ton) design load 400 mm (16-inch) Cast-In-Drilled-Hole (CIDH) piles, and Bents 2 and 3 are supported on spread footings founded on the underlying bedrock, and designed for an allowable bearing capacity of 383 kPa (4.0 tsf).

The current project proposes to construct a tapered widening to the right side of the right bridge ranging from approximately 1.52 m – 3.35 m (5 ft to 11 ft). The proposed tapered widening is due to the proposed construction of managed lanes down the center of existing Interstate 15.

Geology

The foundation investigation performed in 1970 consisted of two mud-rotary borings and three dynamic cone penetration soundings. The 1972 investigation consisted of five 600 mm (24 in.) bucket holes. In January 2007 soil samples were collected for corrosion testing from the fill material at Abutment 1 and the alluvial material underlying the bridge site near Bent 2. The subsurface investigations revealed that the bridge site is underlain by alluvial material overlying granitic bedrock. The alluvial material varies in thickness across the bridge site, ranging from approximately 1.2 m (4.0 ft.) thick in boring B-4 (1970) to approximately 3.4 m (11.0 ft.) thick in boring B-1a (1972). Generally, the alluvial material consists of loose to dense sandy silt, silty sand, sandy clay, with some gravels. Underlying the alluvium is granitic bedrock, which is generally decomposed to intensely weathered near the surface, then becomes harder with depth. For specific details regarding alluvium thickness and descriptions, refer to the LOTB sheets.

Ground Water

Ground water was not encountered during the 1970 subsurface investigation, but was encountered during the 1972 investigation at elevations ranging from 190.8 m (626.1 ft.) to 189.9 m (623.1 ft.) in borings B-1a and B-2a, respectively. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions at time of construction.

Scour Potential

There is no scour potential at the site, since the structure does not span any watercourse.

Corrosion

Corrosion test results for soil samples collected from the fill material at Abutment 1 and the alluvium beneath the bridge site, near Bent 2, are shown below in Table 1. All of the soil samples tested are considered non-corrosive by current Caltrans standards.

Table 1: Corrosion Test Summary

Location	SIC Number	Sample Depth	pH	Minimum Resistivity (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
Abutment 1 Fill	C654034	(0-.91 m) 0-3.0 ft	7.56	1200	-	-
Alluvium Near Bent 2	C654033	(0-.91 m) 0-3.0 ft	7.70	1200	-	-

Note: Caltrans currently defines a corrosive environment as an area where the soil has a minimum resistivity of less than 1000 ohm-cm, and either contains greater than or equal to 500 ppm of chlorides, greater than or equal to 2000 ppm of sulfates, or has a pH of 5.5 or less.

Fault and Seismic Data

The structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The Office of Geotechnical Design-South 2 has provided Final Seismic Design Recommendations in a memorandum dated February 1, 2007. The controlling faults for the site are the Newport-Inglewood-Rose Canyon Fault/E (NIE, strike-slip), located approximately 23 km (14 miles) southwest of the site, and the Whittier-Elsinore Fault (WEE, strike-slip) located approximately 27 km (17 miles) northeast of the site. The Newport-Inglewood-Rose Canyon Fault/E fault is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.0 and a corresponding Peak Bedrock Acceleration is estimated to be 0.3g. The Whittier-Elsinore Fault is capable of generating a Maximum Credible Earthquake moment magnitude (M_w) of 7.5 and a corresponding Peak Bedrock Acceleration is estimated to be 0.3g.

Liquefaction Potential

Final Seismic Design Recommendations, dated February 1, 2007, state that due to the site being underlain by alluvial soil consisting of medium dense to very dense silty and clayey sand overlying decomposed granite, the potential for soil liquefaction under strong ground shaking is considered to be low.

"As-built" Foundations

The existing Mission Ave. UC left and right bridges are three span structures completed in 1977. "As-built" pile tip elevations for the CIDH piles at Abutments 1 and 4 locations, and spread footing data for Bents 2 and 3 locations, are shown below in Tables 2 and 3, respectively.

Table 2: "As-built" CIDH Pile Data

Location	Pile Type	Design Tip Elevation	Average "As-built" Pile Tip Elevation	Maximum "As-built" Pile Tip Elevation	Minimum "As-built" Pile Tip Elevation
Abutment 1 Left Bridge	400 mm (16 inch) CIDH	189.00 m (620.0 ft)	188.70 m (619.2 ft)	189.00 m (620.1 ft)	188.5 m (618.3 ft)
Abutment 1 Right Bridge	400 mm (16 inch) CIDH	189.00 m (620.0 ft)	189.20 m (620.7 ft)	191.2 m (627.4 ft)	188.6 m (618.8 ft)
Abutment 4 Left Bridge	400 mm (16 inch) CIDH	189.00 m (620.0 ft)	188.90 m (619.9 ft)	189.6 m (622.1 ft)	188.1 m (617.0 ft)
Abutment 4 Right Bridge	400 mm (16 inch) CIDH	189.00 m (620.0 ft)	188.90 m (619.7 ft)	189.1 m (620.3 ft)	188.6 m (618.9 ft)

Table 3: "As-built" Spread Footing Data

Location	Allowable Bearing Capacity	Bottom of Footing Elevation
Bent 2 Left & Right Bridges	383 kPa (4 tsf)	192.0 m (630.0 ft)
Bent 3 Left & Right Bridges	383 kPa (4 tsf)	192.0 m (630.0 ft)

Foundation Recommendations

The following recommendations are for the proposed tapered widening to the right side of the Mission Ave. UC right bridge (Br. #57-0813 R), as shown on the General Plan dated January 29, 2007. At Abutments 1 and 4 support locations either 600 mm (24 in.) or 760 mm (30 in.) diameter Cast-In-Drilled-Hole (CIDH) piles may be used for support. At Bents 2 and 3 locations, spread footings are recommended for support. The specified pile tip elevations for 600 mm and 760 mm CIDH piles, with design loads of 625 kN (70 tons) and 900 kN (100 tons) are shown below in Tables 4, 5, 6, and 7. The ultimate geotechnical pile capacity for the CIDH piles will meet or exceed the required nominal resistance in compression. Refer to Tables 4, 5, 6, and 7 below, for the required nominal resistances.

Table 4: 600 mm (24 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	187.8 m(1) (616.0 ft)	187.8 m (616.0 ft)
Abutment 4	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	188.1 m(1) (617.0 ft)	188.1 m (617.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 5: 600 mm (24 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	186.5 m(1) (612.0 ft)	186.5 m (612.0 ft)
Abutment 4	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	186.8 m(1) (613.0 ft)	186.8 m (613.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 6: 760 mm (30 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	760 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	188.2 m(1) (617.5 ft)	188.2 m (617.5 ft)
Abutment 4	760 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	188.5 m(1) (618.5 ft)	188.5 m (618.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 7: 760 mm (30 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	760 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)
Abutment 4	760 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

The recommendations for bottom of footing elevations and Nominal Bearing Resistances to be used for design, at Bents 2 and 3 support locations, are listed below in Table 8.

Table 8 - Spread Footing Data (Br. #57-0813 R)

Support Location	Minimum Footing Width	Bottom of Footing Elevation	Recommended Bearing Limits	
			WSD [1]	LFD [2]
			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Bent 2	4.3 m (14.0 ft)	192.0 m (630.0 ft)	N/A	1149 kPa (24 ksf)
Bent 3	4.3 m (14.0 ft)	192.0 m (630.0 ft)	N/A	1149 kPa (24 ksf)

Notes: 1) Allowable Bearing Capacity (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Bearing Capacity, (q_{all}). 2) Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Bearing Resistance, (q_n).

The recommended Nominal Bearing Resistances provided in Table 8, above, are based on the following design criteria:

1) The bent footings have a minimum width as shown in Table 8.

2) All bent spread footings are to be constructed at or below the recommended elevations as shown in Table 8.

If any of the above minimum footing widths are reduced, or bottom of footing elevations raised, the Office of Geotechnical Design-South II, Branch B, is to be contacted for reevaluation.

General Notes:

- All support locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

Construction Considerations:

CIDH Piles

- Ground water was not encountered during the 1970 subsurface investigation, but was encountered during the 1972 investigation. It is anticipated that the contractor will encounter groundwater during CIDH pile construction. Groundwater levels indicated on the LOTB reflect the measured groundwater levels at the time of the subsurface investigation. At the time of construction, the groundwater surface elevations may be significantly higher or lower than those shown on the LOTB, due to seasonal fluctuations.

- The contractor should anticipate having to use slurry displacement methods to construct the CIDH piles. Allowable slurries shall consist of mineral or synthetic slurry only. Use of water shall not be allowed.

Spread Footings

- At the proposed right bridge, right-side widening Bents 2 and 3 footing locations, where the bottom of footings will be located on native bedrock material, the concrete for the support footings shall be placed neat against the undisturbed bedrock material along the trimmed walls and bottom of footing excavations. Should the bottom of footing excavations be disturbed, then the bottom of the footing excavations shall be extended down at 0.15-meter (0.5 ft) intervals until undisturbed bedrock material is observed and approved by the Engineer. The subexcavated material is to be replaced with lean concrete. The disturbed native material is not to be recompacted.

- At the proposed right bridge, right-side widening Bents 2 and 3 footing locations, where footing excavations extend into native bedrock material, the footing excavations are to be inspected and approved by a representative of the Office of Geotechnical Design-South II, Branch B, prior to placing any steel or concrete for the support footings in the excavations. The required inspection is to verify that the concrete for the support footings are being placed neat on top of the native bedrock material. Once the excavations have been completed to the specified elevations, the contractor is to allow the Office of Geotechnical Design–South II, Branch B, five (5) working days to perform the inspection. The structures

representative is to provide the Office of Geotechnical Design–South II, Branch B, a one-week notification prior to beginning the five-day contractor waiting period.

The recommendations contained in this report are based on specific project information regarding structure type, location, and design loads that have been provided by TY Lin International. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South II, Design Branch B should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South II, Branch B.

Prepared by: Date: 2/15/07

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Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. TEJPAL BHATIA
Structures Design
Office of Bridge Design-South 1
MS #9 - 5/6G

Date: February 27, 2007

File: 11-SD-15-KP R50.4
11-2T0821
Mission Ave. UC (Widen)
Br. #57-0813 R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South II MS #5
Design Branch B

Subject: Revised Foundation Report

This Revised Foundation Report supercedes and supplements the original Foundation Report, dated February 15, 2007, for the proposed widening to the Mission Ave. UC right bridge (Br. #57-0813R). This Revised Foundation Report addresses a typographical error in Tables 6 and 7 of the original Foundation Report, from a CIDH pile diameter of 760 mm, which should be 750 mm. As well, this report includes the addition of a construction consideration addressing the need for the special provisions to specify the requirements of Tunnel Safety Orders when the use of CIDH piles is recommended.

The revised CIDH Pile Data tables are presented below in Tables 1 and 2, which replace Tables 6 and 7 in the original Foundation Report, dated February 15, 2007.

Table 1: 750 mm (30 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	188.2 m(1) (617.5 ft)	188.2 m (617.5 ft)
Abutment 4	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	203.11 m (666.4 ft)	N/A	188.5 m(1) (618.5 ft)	188.5 m (618.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Table 2: 750 mm (30 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)
Abutment 4	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	203.11 m (666.4 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Construction Considerations:

- The District engineer shall specify in the special provisions the requirements of Tunnel Safety Orders for the CIDH pile work that meets the definition of a tunnel or shaft, as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

All other recommendations, general notes, and construction considerations in the original Foundation Report, dated February 15, 2007, which have not been superceded by this report, are still applicable.

Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South II, Branch B.

Prepared by: Date: 2/27/07

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Project File-South
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State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. TEJPAL BHATIA
Structures Design
Office of Bridge Design-South 1
MS #9 - 5/6G

Date: March 13, 2007

File: 11-SD-15-KP R50.4
11-2T0821
Mission Ave. UC (Widen)
Br. #57-0813 R

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: 2nd Revised Foundation Report

This 2nd Revised Foundation Report supercedes and supplements the original Foundation Report, dated February 15, 2007, and the Revised Foundation Report, dated February 27, 2007 for the proposed widening to the Mission Ave. UC right bridge (Br. #57-0813R). This 2nd Revised Foundation Report addresses a change in the bottom of pile cap elevations at Abutments 1 and 4 locations. The change in bottom of pile cap elevations, at Abutments 1 and 4 locations, has a negligible effect on the pile capacity, therefore the specified pile tip elevations remain unchanged.

The revised CIDH Pile Data Tables 4, 5, 6, and 7 are presented below, which replace Tables 4, 5, 6 and 7 in the original Foundation Report, dated February 15, 2007. Tables 6 and 7, below, also replace Tables 1 and 2 of the Revised Foundation Report, dated February 27, 2007.

Revised Table 4: 600 mm (24 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	201.90 m (662.4 ft)	N/A	187.8 m(1) (616.0 ft)	187.8 m (616.0 ft)
Abutment 4	600 mm (24 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	202.60 m (664.7 ft)	N/A	188.1 m(1) (617.0 ft)	188.1 m (617.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Revised Table 5: 600 mm (24 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	201.90 m (662.4 ft)	N/A	186.5 m(1) (612.0 ft)	186.5 m (612.0 ft)
Abutment 4	600 mm (24 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	202.60 m (664.7 ft)	N/A	186.8 m(1) (613.0 ft)	186.8 m (613.0 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Revised Table 6: 750 mm (30 in.), 625 kN (70 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	201.90 m (662.4 ft)	N/A	188.2 m(1) (617.5 ft)	188.2 m (617.5 ft)
Abutment 4	750 mm (30 inch) CIDH	625 kN (70 tons)	1250 kN (140 tons)	-0-	202.60 m (664.7 ft)	N/A	188.5 m(1) (618.5 ft)	188.5 m (618.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

Revised Table 7: 750 mm (30 in.), 900 kN (100 ton) CIDH Pile Data

Location	Pile Type	Design Loading	Nominal Resistance		Bottom of Pile Cap Elevation	Pile Cut-off Elevation	Design Tip Elevation	Specified Tip Elevation
			Compression	Tension				
Abutment 1	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	201.90 m (662.4 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)
Abutment 4	750 mm (30 inch) CIDH	900 kN (100 tons)	1800 kN (200 tons)	-0-	202.60 m (664.7 ft)	N/A	187.3 m(1) (614.5 ft)	187.3 m (614.5 ft)

Note: Design tip elevation is controlled by the following demands: (1) Compression

All other recommendations, general notes, and construction considerations in the original Foundation Report, dated February 15, 2007, as well as the Revised Foundation Report, dated February 27, 2007, which have not been superceded by this report, are still applicable.

Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565 (CALNET 498-4565), or Mark DeSalvatore, (916) 227-5391 (CALNET 498-5391), at the Office of Geotechnical Design-South 2, Branch B.

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Abbas Abghari – OGDS-2
Mark DeSalvatore-OGDS-2
Project File-North
Project File-South

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. TEJPAL BHATIA
DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN SERVICES
Office of Special Funded Projects – MS 9-2/7G

Attention: Norbert Gee

Date: March 22, 2007

File: 11-SD-15-KP R50.2
11-2T0821
Hale Avenue RWalls
Wall No. 57-0507R/L

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS 5
OFFICE OF GEOTECHNICAL DESIGN – *SOUTH 2*

Subject: Final Foundation Design Recommendations

This memorandum presents the final foundation design recommendations for the above referenced project on Route 15 in San Diego County. The proposed project includes constructing two more than 124 meters long retaining walls (Wall No. 57-00507R/L) to create direct access ramp (DAR) lanes on/off I-15 in the median area south of the Hale Avenue Undercrossing. The types of walls include: tieback anchored walls, cantilevered and anchored soldier pile walls at different areas based on the retaining height and the tieback interference, as shown on the General Plan for the project in Appendix A.

A subsurface investigation for the proposed Hale Avenue DAR (Direct Access Ramp) retaining wall structures was conducted in January 2007. The field investigation was performed in conjunction with that for Hale Avenue UC widening. The investigation consisted of a review of existing site information that includes the as-built Log of Test Borings (LOTB) drilled in the early 1970's for the Have Avenue UC, and drilling/logging of five mud-rotary soil borings including Standard Penetration Test (SPT) performed in 2007. Boring locations at the site are shown on a sheet in Appendix B. Soil information from the recent investigation is included on the Plan of Log of Test Borings. The locations and elevations shown on the LOTB for the recently completed borings and referenced in this document are based on the NAD 83 and NAD88 survey benchmarks. Elevations for the as-built LOTBs that are based on NGVD 1929 datum need add 0.63m (2.07ft) to be in consistency with NAD 88 at this site.

Subsurface Conditions

The Log of Test Borings completed in 1970, 1972, and 2007 for this site are used for the final foundation design recommendations. The new plans of Log of Test Borings will be included in the structure construction plans. Subsurface materials below the original ground (Elevation +192.6m or +632 ft) at the site mainly consist of top layers of medium dense to dense silty sand or sandy silt with clay overlying highly decomposed granitic rock at depths from 1.2 to 6m (5 to

15 ft). At Abutments 1R and 2L, a layer (about 1 m or 3 ft thick) of stiff silty clay was encountered within Borings B-2-07 and B-3-07, as well as B-1-70, within a depth of approximately 1.5 meters (5 ft) below the ground surface. The embankment materials (up to 8 meters high) are import soil of mainly fine to medium sand with streaks of clay and decomposed granitic rock.

Groundwater

Groundwater was encountered at a depth of 1.8 to 3.4m (6 to 11ft) below the original ground (Elevation +192.6m or +632 ft) during the drilling in February 1970 and October 1972. During the groundwater observation in January 2007, groundwater was encountered at a depth of 10.8 meters (35.3 ft) within B-1-07 and a depth of 3.8 meters (12.5 ft) within B-5-07, which corresponds elevation +190.7 meters (+625.8 ft) within B-1-07 at Abutment 1L and +189.3 meters (+621.2 ft) within B-5-07 at Abutment 2R. Since the two observation holes are located near Abutments 1L and 2R, 70m (230 ft) apart diagonally crossing the site, the two recently measured water tables may be used for the left and right bridges, separately. Thus, a ground water table at elevation +189.3 meters (+621.2 ft) is assumed for soil liquefaction analysis and foundation design for the left bridge and elevation +190.7 meters (+625.8 ft) for the right bridge. An average water table of the two, elevation +190.0 meters (+623.5 ft) may be used for design of the DAR wall structures in the median area.

Scour Potential

There is no scour concern since the bridge site is not located in a creek/river channel or in any flood/drainage watercourse.

Corrosion

In general, a minimum resistivity value of less than 1000 ohm-cm for soil and/or water indicates presence of high quantities of soluble salts and a higher propensity for corrosion. Soil and/or water with a resistivity of greater than 1000 ohm-cm may be considered to be non-corrosive. Chloride and sulfate tests are not required if the minimum resistivity is greater than 1,000 ohm-cm. Soil and water that have a minimum resistivity less than 1,000 ohm-cm need to be tested for chlorides and sulfates. As per Caltrans Corrosion Guidelines (September 2003), a bridge site is considered to be corrosive to R/C structure foundations, if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

(a) Chloride concentration is 500 part per million (ppm) or greater,
(b) Sulfate concentration is 2000 ppm or greater,
(c) The pH index is 5.5 or less.

To determine whether the bridge site is corrosive to reinforced concrete structure foundations embedded into ground, eight soil samples were collected at various depths during the foundation investigation and tested at the Caltrans Laboratory in accordance with the applicable California

Test Methods. Corrosion test results are summarized below in Table 1. A summary sheet of corrosion test results for the eight soil samples is attached in Appendix C.

The results for one of the eight soil samples collected at this site indicate that the bridge site is corrosive to R/C structure foundations. We recommend that standard design procedures for structure foundations in corrosive areas be used.

Table 1 - Summary Results of Corrosion Test
Hale Avenue UC (Widening) Bridge No. 57-0811R/L

Sample Type	Sample Depth (Boring) meter	pH	Minimum Resistively ohm-cm	Soluble Salts (ppm)		Corrosive
				Sulfates	Chlorides	
Soil	15' (B-1)	7.72	2700	N/A	N/A	No
Soil	48' (B-1)	8.58	4700	N/A	N/A	No
Soil	20' (B-2)	7.52	7100	N/A	N/A	No
Soil	40' (B-2)	8.30	1600	N/A	N/A	No
Soil	30' (B-3)	9.14	650	49	15	No
Soil	15' (B-4)	7.84	10000	N/A	N/A	No
Soil	10' (B-5)	8.02	500	508	522	Yes
Soil	20' (B-5)	8.00	5700	N/A	N/A	No

Controlling Active Fault and Surface Rupture

As shown on the attached California Seismic Hazard Map 1996 by the California Department of Transportation in Appendix D, within 27 kilometers of the bridge site, there are two active faults that both stretch from northwest to southeast: the Newport-Inglewood-Rose Canyon/E (NIE, Mw = 7, style Strike-slip) fault on the southwest side and the Whittier-Elsinore (WEE, Mw = 7.5, style Strike-slip) fault on the northeast side. The distances between the bridge site and the two faults (NIE & WEE) are respectively 22.9 and 26.7 kilometers. The controlling fault for the bridge structure design is the WEE fault which is capable of generating a maximum credible earthquake of moment magnitude Mw = 7.5. The horizontal Peak Bedrock Acceleration (PBA) at this site is estimated to be 0.2g (gravity acceleration) based on the 1996 Geomatrix attenuation relationship and to be 0.3g according to the Caltrans Hazard Map. For structural design of the bridge, a PBA = 0.3g is conservatively recommended as was in our previous preliminary seismic report. However, based on the estimated PBA = 0.2g with the attenuation, a Peak Ground Acceleration of PGA = 0.3g may be used for liquefaction evaluation and retaining wall design.

The surface rupture due to fault movement is not a concern since there is no known fault crossing through the bridge site.

Potential for Liquefaction and Lateral Spreading

The Log of Test Borings (LOTB) completed in 1970, 1972, and 2007 for this bridge are used for the seismic design recommendations. Subsurface materials at the site consist of top layers of medium dense to dense silty sand or sandy silt with clay overlying highly decomposed granitic rock. Different groundwater tables are assumed for the left and right bridges and the DAR walls in the median area as discussed above. A ground water table at elevation +189.3 meters (+621.2 ft) is assumed for the left bridge and elevation +190.7 meters (+625.8 ft) for the right bridge. An average water table of the two, elevation +190.0 meters (+623.5 ft) may be used for design of the DAR wall structures in the median area. Based on the logs, the site is not susceptible to soil liquefaction during earthquake shaking (PGA = 0.3g) and the potential for lateral spreading at the bridge site is considered to be very low.

Horizontal ARS Curve

For this seismic design recommendation, the soil profile at this site may be classified as a Type D as defined in the Caltrans Seismic Design Criteria (SDC). For structural design of the bridge, a SDC Acceleration Response Spectrum (ARS) corresponding to Type D soil profile, $M_w = 7.5$, and PBA of 0.3g is recommended as provided in Appendix D.

Static Earth Pressures on Walls

Static earth pressures on walls should be considered as per Caltrans Bridge Design Specifications (BDS, 2004), Section 3.20. A minimum equivalent fluid pressure of 5.65 kPa per meter of wall height (36 lbs/ft^2 per linear foot of height) should be applied for design of the wall. A static earth pressure in terms of an equivalent fluid pressure of 9.42 kPa per meter of wall height (60 lbs/ft^2 per linear foot of height) may be applied if the wall movement is restrained by piles or the superstructure. To account for the effect of highway traffic/vehicle loads on the abutment wall, an additional uniform lateral pressure of 3.5 kPa (72 lbs/ft^2) should be applied, which is equivalent to a live load surcharge of 0.6m (2 ft) of earth. This live load surcharge may cause an additional uniform lateral pressure of up to 5.75 kPa (120 lbs/ft^2) on a wall if its lateral movement is restrained by piles or the superstructure. The values of earth pressure do not account for any hydrostatic pressure from standing water in the embankment materials behind the wall since weep holes and a wall drain with filter or drainage materials are to be provided as per Caltrans Standard Plans or Specifications.

The DAR retaining walls at the Hale Avenue UC are to be constructed in three types of walls: tieback anchored walls, cantilevered and anchored soldier pile walls at different areas based on the retaining height and the tieback interference. The anchored retaining walls are assumed to be placed from the top down and restrained by ground anchors (tieback anchors). The lateral earth pressures on the walls should be determined in accordance with Caltrans Bridge Design Specifications (BDS) (2003) Section 5 – Retaining Walls. Three diagrams of lateral earth pressure distributions for the three types of walls are presented in Figures 1 to 3 on Appendix E. A soil unit weight of $\gamma = 120$ lbs/ft^3, an active pressure coefficient of $K_a = 0.3$, and a passive

coefficient of $K_P = 3$ are recommended in calculating the active and passive soil pressures on the walls and the soldier piles. The effective widths of soldier piles installed in cohesionless soils at this site may be increased by an adjustment factor (passive arching capability) of 0.08 times the internal friction angle of the soil, but not to exceed a value of 3.00 (i.e. $\beta = 0.08\phi \leq 3.0$, $\phi = 30$). When a program using p-y soil spring models such as LPILE is to be used for lateral pile analysis, the effective pile width should be the diameter of the pile. Examples of soldier pile analysis and design can be found in Caltrans Trenching And Shoring Manual Revision 12 (January, 2000) Section 10 – Soldier Pile Systems.

A static earth pressure diagram with a maximum ordinate of p_a as shown in Figures 1 & 2 may be applied to the anchor wall. The maximum ordinate, p_a, may be calculated by Equations (5.5.5.7.1-1 & -2) by substituting P_{Total} with the at-rest static earth pressure, since the wall movement is restrained by anchors. The following table lists the maximum ordinates for different number of tieback anchors even spaced vertically per effective retaining width of the wall.

Table 1. Maximum Ordinate versus Number of Anchors

n	1	2	3	4	5
p_a/H (kN/m^3)	7.07	6.06	5.66	5.44	5.30

n – number of tieback anchors
p_a – maximum ordinate of earth pressure, in kN/m^2
H – anchor wall height from the top of wall to the design grade (not including any embedment into ground), in meters

The design reaction force, R, at the design grade is resisted by the passive earth pressure on the embedded portion of the wall as shown in Figure 1. A static passive earth pressure in terms of an equivalent fluid pressure of 56.57 kPa per meter of wall height (360 lbs/ft^2 per linear foot of height) may be applied to determine the design reaction force. It should be noted that the static passive earth pressure is the ultimate resistance for the design reaction force and it may not be fully mobilized due to limited lateral movement.

To account for the effect of highway traffic/vehicle loads on the anchor wall, an additional uniform lateral pressure of 5.75 kPa (120 lbs/ft^2) should be applied, which is equivalent to a live load surcharge of 0.6m (2 ft) of earth. The values of earth pressure do not account for any hydrostatic pressure from standing water in the backfill materials behind the wall since weep holes and a wall drain with filter or drainage materials are to be provided as per Caltrans Standard Plans or Specifications.

Seismic Earth Pressures on Retaining Walls

Seismic earth pressure on the walls may be estimated using the Mononobe-Okabe method as recommended by AASHTO (1996) – Section 6.4.3. Seismic earth pressure/thrust on a wall is a function of the lateral displacement or stiffness of the wall among other factors. Two extreme

displacement cases are usually considered to estimate seismic earth pressure on a wall: (a) large displacement or low stiffness case in which an abutment or retaining wall may move laterally (say, a displacement of up to 250A (mm), in which A is the peak ground acceleration coefficient) without much restrain from superstructure or supporting foundations, and (b) small displacement or high stiffness case in which a wall is restrained from any significant lateral movement by superstructure or/and anchors or/and batter piles. A reduced seismic acceleration coefficient equal to one half the peak ground acceleration coefficient, $k_h = 0.5A$ (k_h is the seismic acceleration coefficient) for case (a) and an enlarged coefficient up to one and half the peak ground acceleration coefficient, $k_h = 1.5A$ for case (b) are assumed for use as an input in the Mononobe-Okabe analysis. Since $A = 0.3g$ at the bridge site, $k_h = 0.15$ for case (a) and $k_h = 0.45$ for case (b) may be used for the pseudo-static analyses. For case (a) in which the wall may experience a displacement of up to 75 mm (3 inches) under seismic loading, seismic active earth pressure (not including static pressure) on the wall is estimated as $0.94H^2B$ kN, in which H is the wall height in meters and B (in meters) the effective retaining width of the wall (between soldier piles, if any). A seismic active earth pressure of $4.21H^2B$ kN, which is more than twice as much as the pressure for case (a), may be applied to case (b) where high stiffness is expected from the tieback anchors. The total seismic earth thrust on the wall may be calculated as the seismic earth pressure ($p_{eq} = 0.94H$ or $4.21H$) times the retaining wall area (HB). The point of application for the seismic lateral soil thrust may be assumed at 0.5H – mid-height of the wall. If the anchor wall is to be designed with little or no lateral movement under seismic thrust, a uniform seismic earth pressure of $p_{eq} = 4.21H$ corresponding to case (b) may be applied for design.

Other Loads on Walls

Load from vehicle collision on barrier should be determined in accordance with Caltrans BDS by the structure designer and directly applied to the top of the wall for design of the anchor wall.

Settlement of the anchor wall with inclined tieback anchors should be limited to a minimum, since any such settlement will cause distressing in pre-stressed tendons. The inclination angle of the tieback anchors should be designed at approximately 10° to minimize the vertical component of the pre-stressing force in tendon while facilitating anchor construction. The vertical load on the wall should be transferred into competent soil through soldier piles. For check of pile compressive capacity, a plot of pile vertical ultimate resistance versus tip elevation is provided in Appendix I for 36-inch diameter CIDH soldier piles near Abutment 1R/L. A factor of safety of 3.0 should be used for determining the allowable pile resistance for service design loads.

Seismic load due to transverse movement of the abutment may be considered as concentrated load/impact on the wall.

Bond Resistance of Tieback Anchors

The DAR retaining wall at the abutment is to be tieback-anchored within a depth of 2 to 4 meters (7 to 13 ft) below the top of wall by one anchor per pile. The required anchored resistance is developed through soil bond resistance on the bonded length beyond the Rankine wedge as

shown in Figure 1 of Appendix E. The ultimate soil bond resistance may be assumed to be 120 kPa (2.5 ksf) and a minimum factor of safety of 3.0 should be used for determining the allowable bond resistance. It is current Caltrans' practice that the construction contactor is to design and proof-test the tieback anchors for the required anchored resistance. The above recommended ultimate soil bond resistance may only be used for the purpose of cost estimate.

Alternative Analysis Method for Soldier Pile Walls

For seismic design of soldier pile retaining walls, the Mononobe-Okabe method may be used to determine the soil pressures including dynamic inertia effect by applying assumed pseudo-static horizontal and/or vertical accelerations. The minimum tieback force and soldier pile embedment depth can be found based on a semi-empirical method in which pile deflection shape, and full mobilization of the ultimate soil passive resistance and active pressure are assumed. This approach may be appropriate for short rigid piles, but cannot predict the lateral response of long flexible piles. In addition, one major shortcoming with the use of the Mononobe-Okabe method is the uncertainty in determining the seismic coefficient according to the restraints that prevent the wall from freely moving. Seismic earth pressures on walls are given above for two extreme cases of wall movement. The pressures with four times difference are hard to be used for design of the wall. To eliminate the uncertainty, a simple two-curve method (a Newmark-type method as referenced in Caltrans BDS – Section 5.2.2.3) may be alternatively used for determining the force demand on the soldier pile and the minimum tieback force. It has shown from our previous projects that the simple numerical analysis method utilizing p-y soil spring model can more accurately predict the response of a pile-supported tieback retaining wall than the conventional empirical or semi-empirical methods.

The simplified procedure that can be utilized for computing lateral earth forces on piles/walls due to cut slope instability during earthquake shaking is to develop two curves of load versus displacement behavior to consider the soil-pile-wall interaction in a "decoupling" way. A curve of lateral earth force versus displacement is first established by slope stability analysis for the soil mass behind the wall based on the Newmark chart that relates the standardized maximum displacement of the soil mass to the limiting acceleration coefficient. A second curve of lateral load versus displacement for the soldier pile wall is then obtained by LPILE analysis under the lateral earth force. The intersection between the two load-displacement curves is considered as the lateral earth force on the wall in the sense that equal loads are applied to the sliding soil mass and the pile/wall retaining system and the displacement compatibility is maintained between the soil and the pile/wall.

Slope Stability of DAR Walls at Abutments

The wall height at the abutment is approximately 7.3 meters (=200.6-193.3m) (24 ft). Slope instability of backfills behind DAR walls under seismic shaking may displace the walls and cause significant differential settlement and cracks/damage in highway pavement.

The global slope stability of embankment materials behind DAR walls under seismic shaking is evaluated by performing pseudo-static slope stability analysis using the program XSTABL. It is assumed in the pseudo-static analyses that a seismic acceleration coefficient, $k_h = 0.06$ to 0.3, up to the peak ground acceleration coefficient, is used while the effect of vertical acceleration is ignored. It is found from the XSTABL results as presented in Appendix F for the embankment cut slope that the retaining wall should be designed adequately to provide minimum lateral resistance to maintain the cut slope stability.

The required minimum lateral resistance is calculated as the force demand on the soldier pile when a factor of safety FS = 1.3 for static case and FS = 1.0 for seismic cases: $k_h = 0.06$ to 0.3 is applied. The lateral static and seismic forces as shown in the table of Appendix G are for a soldier pile with an effective retaining width of 3 meters (10 ft). For other retaining width, the minimum force demand on a pile can be calculated based on the data of the resistance of the soil wedge and the driving force as presented in the table. The first curve of lateral earth force versus displacement for the specific retaining width of 3 meters (10 ft) is generated by slope stability analysis for the soil mass behind the wall based on the Newmark chart, as plotted in the chart of Appendix G.

Soil Parameters for Lateral Pile Analysis using LPILE

Soil parameters are provided in Appendix H for lateral pile analysis using the LPILE program. The soil parameters were estimated from correlations with Standard Penetration Testing (SPT) blow counts and soil models as presented in the Technical Manual for LPILE 5 (October, 2004). The calculations were conducted based on the Log of Test Borings completed in 1970 and 2007 for this bridge site. The tabulated soil data and layer elevations are given in the table in Appendix H for soldier piles near Abutment 1R/L. Specifically, the soil effective unit weight, undrained shear strength, angle of internal friction, initial modulus of subgrade reaction, and soil type of p-y model are given in Column 8, 12, 13, 15, and 16 respectively. The ground elevation for piles near the abutments is assumed at the design grade of the wall to not consider the overburden effect from the approach fills at the abutments. LPILE seems to overestimate the lateral pile stiffness for a pile with an embedded pile head below the ground. Thus it is advised that a reduced embedment depth below the ground surface to the embedded pile head be used to discount the overburden effect in an LPILE analysis.

Construction Considerations

1. Placement of concrete for wall structure should conform to the provisions in Section 19-3, "Structure Excavation and Backfill" and Section 51, "Concrete Structures" of Caltrans Standard Specifications.

2. The 915mm (36") diameter CIDH soldier piles are to be constructed in subsurface materials ranging from medium dense to dense silty sand or sandy silt with clay, to highly or moderately decomposed granitic rock. The contractor should anticipate difficult drilling conditions that exceed those normally encountered in alluvial soil and should be advised to use suitable drilling

or coring equipment for the construction of the CIDH piles. Augering or drilling through sandy soil or decomposed rock may cause caving condition in hole being drilled. The contractor's attention should be drawn to Section 49-4.01 and 49-4.03 of Caltrans Standard Specifications. A drilled hole should be filled with concrete on the same day it is drilled, or the hole should be protected from caving.

3. The tip elevation of the CIDH piles at the abutments may be below the observed groundwater that is approximately 1.8 – 3.4 meters (6 – 11 ft) below the current local street surface. The contractor should anticipate that groundwater may be higher during pile construction if it is in wet or rainy season. Any temporary casing used to construct the CIDH piles shall be removed during concrete placement.

4. All drilled holes for CIDH piles are to be cleaned out and inspected prior to placement of reinforcing cage or placing the concrete. Any wet-constructed CIDH piles should be inspected by Gamma-Gamma logging through pre-installed PVC tubes. Memos to Designers 3-1, Attachment B should be referenced for layout details of inspection PVC tubes.

5. The Engineer shall specify in the special provisions the requirements of Tunnel Safety Orders for the CIDH pile work that meets the definition of a tunnel or shaft, as described in the Highway Design Manual, Section 110.12 "Tunnel Safety Orders."

Discussion

The foundation design recommendations contained in this report are based on specific project information regarding the proposed structure, loading conditions and foundation dimensions provided by the Structure Designer. If any conceptual changes are made during final project design, this Office should be notified of those changes and review the changes to determine whether the foundation recommendations are still applicable.

If there are any questions or comments, please contact Jin-xing Zha at (916) 227-4519 or Calnet 8-498-4519 or Angel Perez-Cobo at (916) 227-7167 or Calnet 8-498-7167.

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Attachments

c: File Room in Trabslab

State of California

Business, Transportation and Housing Agency

Memorandum

To : DIMITAR PEEV (MS 333)
Project Engineer
Design

Date: March 3, 2008

File: 11-SD-15
KP R48.3/R50.7
(PM R30.0/R31.5))
EA 11-2T0821

From : **DEPARTMENT OF TRANSPORTATION - DISTRICT 11**
PAVEMENT ENGINEERING SECTION

Subject: **STRUCTURAL SECTIONS RECOMMENDATIONS - Corrected**

The previous structural section recommendation furnished in the October 30, 2007 memo has been corrected for the HOV Managed Lane portion of this project.

The Highway Design Manual, revised June 26, 2007 is the guideline for pavement design.

The Traffic Forecasting Branch has furnished Traffic Indices (TIs), used in the design.

The design criterion for the soil is based on an R (Resistance)- value of 15 for the existing subgrade soils. This value is based on previous projects in the area and is representative of the soil strength encountered in the project vicinity.

Outside Main Lane (TI = 14.5, R-value = 15)

Preferred PCC Option	AC Option
300 mm PCC	60 mm AC (Type A)
150 mm ACB	165 mm ACB
195 mm AB – Class 2	750 mm AB – Class 2

Inside Main Lane (TI = 12.5, R-value = 15)

270 mm PCC
150 mm ACB
195 mm AB – Class 2

Main lane PCC shoulder option:

The Highway Design Manual, Section 603.4, recommends that the shoulders be constructed of the same material as the main lane pavement in order to facilitate construction, improve pavement performance, and reduce maintenance costs.

Main lane AC shoulder option:

For the outside shoulder, AC may be used when the adjacent outside PCC lane is 4.27 m wide.

For the inside shoulder, AC may be used with the standard 3.2 m wide adjacent PCC lane.

Main Lane Outside Shoulder (TI = 9.0, R-value = 15)

PCC Alternate	AC Alternate
300 mm PCC	135 mm AC (Type A)
150 mm ACB	450 mm AB – Class 2
195 mm AB – Class 2	

Main Lane Inside Shoulder (TI = 9.0, R-value = 15)

PCC Alternate	AC Alternate
270 mm PCC	135 mm AC (Type A)
150 mm ACB	450 mm AB – Class 2
195 mm AB – Class 2	

HOV Lane (TI = 11.0, R-value = 15)

No Lateral Support

240 mm PCC
120 mm ACB
165 mm AB – Class 2

HOV Shoulder (TI = 8.0, R-value = 15)

PCC Alternate	AC Alternate
240 mm PCC	120 mm AC (Type A)
120 mm ACB	390 mm AB – Class 2
165 mm AB – Class 2	

Ramp Traveled Way (TI = 10.0, R-value = 15)

Alternate 1	Alternate 2
60 mm AC (Type A)	150 mm AC (Type A)
300 mm AC Base	510 mm AB – Class 2

Ramp Shoulder (TI = 6.5, R-value = 15)

Alternate 1	Alternate 2
60 mm AC (Type A)	90 mm AC (Type A)
180 mm AC Base	315 mm AB – Class 2

Local Streets Traveled Way (TI = 11.5, R-value = 20)
(Clarence Lane, Felicita Road & Ninth Avenue)

Alternate 1	Alternate 2
60 mm AC (Type A)	180 mm AC (Type A)
360 mm AC Base	540 mm AB – Class 2

Local Streets Traveled Way (TI = 11.5, R-value = 5)
(Valley Parkway & Hale Avenue)

Alternate 1	Alternate 2
60 mm AC (Type A)	180 mm AC (Type A)
405 mm AC Base	690 mm AB – Class 2

CHP Enforcement Pads/Maintenance Vehicle Pullouts

90 mm AC (Type A)
300 mm AB – Class 2

Design Notes:

1) If the shoulder width is less than 1.2 m wide, then the traveled way structural section shall be used in lieu of the shoulder structural section.
2) Local Streets are designed, per regional standards, with a uniform structural section from flowline of gutter to flowline of gutter.
3) The aggregate gradation for the AC (Type A) is 19.0 maximum, coarse.
4) The recommended PG grade of asphalt for the AC is PG 64-10.
5) To achieve the 40 year long life pavement design, PCC pavement will have dowel bars at the transverse joints and tie bars in the longitudinal joints as shown in the Standard Plans.

If you have questions with regards to this memorandum, please contact me at 858-467-4056 or FAX at 858-467-4063.

David Evans
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