State of California

Business, Transportation and Housing Agency

Memorandum

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Division of Structure Design
Design Office 59-232

Attention Mr. Gary Blakesley

Date: January 10, 2001

File: 11-SD-805-KP 45.41
11-0301U1

N805/N5 Truck Connector
Bridge No. 57-1070G

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Revised Foundation Recommendations

Introduction

The proposed N805/N5 Truck Connector (Br. No. 57-1070G) is part of planned Route 5/805 Freeway improvements for the San Diego area. A Request for Final Foundation Recommendations (dated October 22, 1998) for the subject bridge was submitted to the Office of Structure Foundations (OSF) by Mr. Ramin Rashedi. Site specific ARS, liquefaction potential, and methods of liquefaction mitigation were requested in the above memorandum. A list of preliminary column/pile loads and shaft diameters were provided to OSF by Mr. Ramin Rashedi (dated December 18, 1998). As the 5/805 and 5/56 project has progressed, further revisions of the above pile load and shaft diameter list was sent to OSF including Revision 1 (dated February 24, 1999), Revision 2 (dated April 9, 1999), and Revision 3 (dated May 11 and 26, 1999). Final bent pile diameters were confirmed by Mr. Ramin Rashedi (personal communication, September 1999). Abutment pile diameters and axial service loads were provided by Mr. Ramin Rashedi (February 1, 2000) who also requested P-Y curves or COM624 soil profile information at the abutments and final P-Y curves for the bents. Mr. Gary Blakesley provided bottom of footing/pile cutoff elevations for the proposed bridge (Caltrans facsimile copy, dated March 24, 2000). P-Y curves were also requested by Mr. Earl Seaberg (Senior Bridge Engineer, Division of Structure Design) on February 24, 1999. In the same memorandum it was mentioned that in order to mitigate the effects of potential liquefaction, large diameter cast-in-drilled-hole piles (CIDH) would be used for structures at the 5/805 Interchange (Seaberg, February 24, 1999). In preliminary evaluations of the As Built Log of Test Borings (LOTB) from an adjacent structure (Sorrento Valley Blvd Undercrossing, Br. No. 57-0786R/L) performed by the Office of Geotechnical Earthquake Engineering (Perez-Cobo and Abghari, February 10 and April 7, 1999), potentially liquefiable soils are estimated as approximately 7.6 to 9.1 m (25 to 30 ft) thick.

Additional Requests for Revised Foundation Recommendations for Abutments 1 and 9 of the subject bridge was received November 13, 2000, and January 8, 2001, from Mr. Gary Blakesley. Abutment pile diameters were increased from 1.2 m (4 ft) to a revised 1.5 m (5 ft). At Abutment 1, approximate finish grades would remain the same as specified previously, however, the bottom of pile footing elevations would be lowered and pile loads were increased. At Abutment 9, revisions would include both stepping down to the east and lowering all bottom of pile footing elevations, increasing pile load capacities, and revising finish grade (sloping down to the east). This completely revised report is being sent for clarity and convenience regarding foundation recommendations. This revision supersedes the previous Foundation Recommendations (Pratt, May 5, 2000) completed by the OSF.

Subsurface information was obtained by OSF drilling and sampling nine - 94 mm diameter mud rotary borings which also involved extensive coring. Results from the field studies will be shown on the LOTB. In addition to the recent field work, the As Built LOTB for the adjacent Sorrento Valley Blvd. Undercrossing (Br. No. 57-0786R/L) and the Los Penasquitos Creek Bridge

(Br. No. 57-0779), Contract No. 11-045894, dated April 14, 1972, contained additional site and subsurface information and will be included within the new contract plans.

Site Description

During OSF recent foundation investigation, sediments at the site consist of minor preexisting embankment fill slopes from adjacent Rte. 805 in the area of proposed Abutments 1 and 9. Embankment fill material ranges between approximately 0 to 3.7 m (0 to 12 ft) thick in the proposed Abutment 1 area (south) and from 0 to 1.8 m (0 to 6 ft) thick in the proposed Abutment 9 area. Artificial fill, below the embankment material, ranges from approximately 0 to 3.35 m (0 to 11 ft) thick. Underlying native alluvium (Holocene and possible older Quaternary alluvium, undifferentiated) ranges from approximately 17.07 to 21.95 m (56 to 72 ft) thick and thins to the southeast. The undulatory top surface of the underlying Eocene Ardath Shale was encountered from elevations –7.59 m (-24.9 ft) in the proposed Abutment 1 area (south) to –12.68 m (-41.6 ft) at the proposed Abutment 9 area (north).

Approach embankment fill material consists dominantly of stiff to hard, sandy lean clay to clayey sand with minor scattered gravel and cobble-size soft mudstone rock fragments (up to 150 mm diameter). Underlying artificial fill generally consists of loose to medium dense /very soft to very stiff, silty sand and sand with scattered gravel interlayered with sandy silt with minor gravel and rare cobbles [up to 155 mm (6 in) diameter], sandy lean clay, and clayey sand. Native material [mapped as Holocene alluvium and slope wash undifferentiated according to Kennedy (1975) and probably including some older alluvium at depth], can be divided into two units with the upper sediments consisting of dominantly very loose to medium dense/very soft to stiff, silty sand, sand, and sandy silt with intermittent scattered gravel and cobbles [up to 155 mm (6 in) diameter] interbedded with sandy lean clay, clayey sand, minor elastic silt, and rare fat clay. Calcite nodules were observed within these upper alluvial sediments. The underlying native alluvial unit [from 0.91 to 8.53 m (3 to 28 ft) thick] found below elevations ranging from –2.74 to –9.14 m (-9 to –30 ft) consists of generally very dense/hard, gravel/cobble lenses with sand, silty sand, and clayey sand matrix interbedded and overlain by sand with 40% gravel and minor lean clay. Generally the extremely hard, subrounded to subangular gravel/cobbles [up to 215 mm (8.5 in) diameter] composed of metavolcanic, quartzite, and chert rock fragments directly overlie bedrock. Much of the loose and soft native material is considered potentially liquefiable and is being investigated by the Office of Geotechnical Earthquake Engineering for potential mitigation measures or adequacy of proposed mitigation measures. As mentioned earlier, final p-y (lateral resistance) curves are also being developed for use at proposed bridge support locations. The underlying Eocene Ardath Shale generally consists of interbedded very soft to moderately hard, mudstone, claystone, and siltstone. The formation is generally slightly weathered, slightly fractured, often thinly bedded, and contains occasional concentrations of pelecypod debris. The typical Ardath Shale in this area is partially underlain and interfingers with the Eocene Torrey Sandstone at Los Penasquitos Creek. The intertongueing sandstones representative of the Torrey Sandstone are dominantly composed of formational sand (uncemented, soil-like, very dense sand) and minor moderately hard to hard calcite-cemented fine to medium sandstone. The very soft to soft upper formational mudstones of the Ardath Shale [0.3 to 1.5 m (1 to 5 ft) thick], were considered to possess weak rock unconfined compressive strengths of 180 psi. Below this upper zone, generally soft to moderately hard mudstone/claystone/and siltstone (fairly strong rock) show unconfined compressive strengths from at least 300 psi and higher. The two deepest borings for the bridge, Boring 99-3 (near proposed Bent 7) and 99-2 (near proposed Bent 8), extended to 49.68 to 51.76 m (163 to 169.8 ft) below the surface [elevations –37.49 to –40.45 m (-123.0 and –132.7 ft)], respectively. Downhole P-S logging (compression and shear wave) showed that the better quality formational mudstones had shear wave velocities averaging 427 to 457 meters per second (1400 to 1500 fps) which appeared to correlate with unconfined compressive strengths of at least 250 psi and higher. Shear wave velocities ranging from 549 to 792 meters per second (1800 to 2600 fps) in pseudo-rock-like material correlated with unconfined compressive strengths of at least 300 psi and higher in the Ardath Shale below approximate elevations ranging from –24.38 to –26.21 m (-80 to -86 ft). The competent upper mudstone unit of the Ardath Shale is shown to be at least 17.68 to 20.42 m (58 to 67 ft) thick beneath Abutment1 and Bents 2 through 5 and possibly Bent 6. Beneath Bents 7 (Boring 99-3), 8 (Boring 99-2), and Abutment 9, the upper mudstone tongue thins from 11.28 to 7.92 m (37 to 26 ft) thick and interfingers with the Torrey formational sands (uncemented) and cemented sandstones. The LOTBs should be reviewed for more specific details.

Surface Water and Scour

Surface water was often stagnant within Los Penasquitos Creek with only minor flow observed during the field investigation. Following a wet period on March 2, 2000, the author observed more substantial surface flow, however, flows were not enough to cause apparent substantial scour.

The proposed bridge will span over Los Penasquitos Creek with Bents 7 and 8 located outside of the stream channel and protected by embankment levees with rock slope protection. Only minor erosion or scour was found at the existing pier 2 for the nearby Los Penasquitos Creek Bridge, Br. No. 57-0779, during OSF field investigation and brief survey of the stream bottom in the area. The As Built Boring B-1 (Br. No. 57-0779) shows up to 6.1 m (20 ft) of very loose to loose, silty sand with little clay binder below the surface on the east side of the bridge at existing Pier 2. OSF survey of the bottom of the channel matched the elevation of the original Boring B-1 at approximately the same location. This may indicate that scour is not substantial at the site.

Scour will not effect the foundations for the N805/N5 Truck Connector as axial support is gained within unscourable rock material and levees will protect the supports from erosion and loss of lateral support within soils.

For further information, refer to Preliminary Investigations and Hydraulics reports in this area.

Ground Water

Static ground water was last measured on January 11, 2000, within Boring 99-6 (near proposed Bent 2) at elevation +8.93 m (+29.3 ft). Also, nearby Boring 99-5 (Bent 2 – Right side) for the Rte. 5/805 Separation (Br. No. 57-0512) revealed static ground water at elevation +8.84 m (+29.0 ft) measured March 2, 2000 (shortly after rains). In the area of Boring 99-3 (proposed Bent 7), the water table may be estimated as high as +7.2 m (+23.6 ft) elevation based on P-wave velocity (measured June 30, 1999, during the dry season). The bottom of Los Penasquitos Creek in the area ranges from approximately +7.73 to +8.24 m (+25.4 to +27.0 ft) elevations and can be flooded. The ground water level fluctuated approximately 0.3 m (1 ft) during OSF's recent investigation.

The As Built LOTB for the Sorrento Valley Blvd. Undercrossing (Br. No. 57-0786R/L) shows ground water was encountered from approximate elevations +8.29 to +7.74 m (+27.2 to +25.4 ft) based on NGVD29 elevations, which requires a +0.588 m (+1.93 ft) add (Schuh, Caltrans Memorandum, March 7, 2000) to adjust to the current metric elevations (NAVD 88) upon which the recent plans and boring program are based. The adjusted to metric As Built elevations would then show ground water was encountered at elevations +8.88 to +8.33 m (+29.1 to +27.3 ft) for the earlier foundation investigation, with measurements taken during May 1969. The As Built LOTB for the Los Penasquitos Creek Bridge (Br. No. 57-0779) reveals ground water was encountered from elevations ranging from +6.58 to +6.40 m (21.6 to 21.0 ft). Again correcting English to metric

(NAVD88) elevations would show ground water encountered from elevations +7.17 to +6.99 m (+23.5 to +22.9 ft).

Seismicity

See the memorandum (dated February 10, 1999) concerning Preliminary Seismic Design Recommendations sent to Mr. Earl Seaberg from Mr. Angel Perez-Cobo and Mr. Abbas Abghari. Final Seismic Design Recommendations and Lateral Resistance, p-y Curves will be submitted by the OGEE.

As mentioned above (Perez-Cobo and Abghari, February 10, 1999) the proposed "structures are located approximately 5 km from the Newport-Inglewood-Rose Canyon fault which has a maximum credible earthquake moment magnitude of M=7.0 and based on the Caltrans California Seismic Hazard Map (Mualchin, 1995), these structures are within the peak horizontal bedrock acceleration zone of 0.5 g."

As mentioned above pseudo-rock-like material [Vs ranging from 549 to 792 meters per second (1800 to 2600 fps) occurs below approximate elevations ranging from –24.38 to –26.21 m (-80 to -86 ft).

Liquefaction

Liquefaction potential is considered moderate to high. Holocene and older Quaternary alluvium (undifferentiated) at the site is dominantly composed of very loose to medium dense/very soft to stiff, silty sand, sand, and sandy silt with intermittent scattered gravel and cobbles [up to 155 mm (6 in) diameter] interbedded with sandy lean clay, clayey sand, minor elastic silt, and rare fat clay. Ground water is also rather shallow [measured within the recent investigation at 2.04 m (6.7 ft) below the surface at Bent 2 and estimated as shallow as 5.0 m (16.4 ft) below the surface at Bent 8 based on P-S logging]. Preliminary analysis (Perez-Cobo and Abghari, February 10, 1999) estimates that the top 7.62 to 9.14 m (25 to 30 ft) of soils are considered potentially liquefiable. As mentioned above, final liquefaction potential is being determined by the Office of Geotechnical Earthquake Engineering.

Foundation Recommendations

The following recommendations are based on the N805/N5 Truck Connector, Br. No. 57-1070G, General Plan (revised March 10, 1999), Foundation Plans (3 sheets, revised October 3, 2000), the Abutment 9 Layout (revised October 13, 2000), the above mentioned memorandums and personal communications from Messrs Ramin Rashedi (Caltrans facsimile copy dated May 26, 1999, personal communications regarding pile loads and pile diameter, September 1999, and a memorandum supplying abutment pile diameters and service loads, February 1, 2000), and Gary Blakesley (Caltrans facsimile copy with bottom of footing elevations, dated March 24, 2000). Requested revisions (Blakesley, November 13, 2000, and January 8, 2001) at both Abutments 1 and 9 include increasing pile diameters and lowering the bottom of pile footing (pile cutoff) elevations. At Abutment 1, approximate finish grades would remain the same as specified previously, however, pile loads would increase. At Abutment 9, the bottom of pile footing elevations would be stepped down to the east, pile loads would be increased, and finish grade would slope down to the east.

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. At the Abutment 1 area, additional fill is estimated at 6.71 m (22 ft) maximum height. Existing embankment for the Sorrento Valley Blvd. Undercrossing (Br. No. 57-0786R/L), adjacent to the Abutment 1 area, has been in place since 1972 so settlement will be reduced somewhat from the calculated maximum settlement of 380 mm (15 in). The settlement period is estimated at approximately 180 days, however the actual settlement period will be determined by the project engineer on the basis of settlement data in the field.

At the Abutment 9 area, additional fill is estimated at 14.33 m (47 ft) maximum height. Existing fill for the adjacent Abutment 5 (5/805 Separation, Br. No. 57-0512) has been in place since 1964 so settlement should be reduced from the original settlement of 0.95 ft measured at settlement platform #7 where similar fill height was added. Some additional fill has already been in place for months due to parallel ongoing improvements by San Diego County. Estimated settlement for the original conditions (encountered during OSF's recent field investigation) was calculated at approximately 183 mm (7.2 in). Again, all fills can be placed in accordance with the Section 19-6 of the Standard Specifications. OSF recommends a fill settlement of up to 90 days in this area; however, the actual settlement period will be determined by the project engineer on the basis of settlement data in the field. According to Mr. Arturo Jacobo (District 11 Project Engineer, April 11, 2000) and as shown on the Abutment 9 Layout Plans (revised October 13, 2000) a retaining wall (Retaining Wall No. 470) is planned for the east side of the bridge along the proposed San Diego County frontage road extending from Abutment 9 to the north to prevent encroachment of embankment upon the road.

Due to high fills OSF assumes that structure approach slabs will be incorporated within the new bridge, but approach slabs were not denoted on available plans.

Plumb, 1.5m (5 ft) diameter, Cast-in-Drilled-Hole (CIDH) Piles can be used to support the bridge abutments. Plumb, 3.0 m (10 ft) diameter drilled shafts will be used at the bent supports as shown below. Cast-in-Drilled-Hole Pile capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-HI-88-042) published July 1988. Permanent casing is recommended to be placed into bedrock to facilitate construction of the drilled shafts, prevent caving of loose soils and gravel/cobble lenses into the pile borings, and seal off ground water from entering the pile borings. OSF feels that permanent steel casing can be emplaced at least near specified tip elevation using a vibratory hammer. In discussions between Mr. Ron Jones (Geotechnical Earthquake Engineering) and the author (for nearby Sorrento Viaduct, Br. No. 57-0513R/L, March and April, 2000) the practice of drilling ahead of the casing before dropping the casing into place is considered undesireable as caving of loose soils and gravel/cobble lenses would create voids between the casing and surrounding soil, thus compromising the lateral capacity of the pile. However, drilling slightly ahead of casing in the basal gravel/cobble lenses and within bedrock will probably be necessary. OSF assumes no additional axial geotechnical capacity for permanent steel casing that will be installed to aid in construction of CIDH piles shown below.

N805/N5 Truck Connector, Br. No. 57-1070G:

	Design Loading			Nominal Resistance						
Support Location/ Type & Diameter	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)	Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/ Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
Abut 1/ CIDH 1.5 m (5 ft)	**1800 (202.3)**			3600 (404.6)	0	3.97 (13.0)	+14.0 (+45.9)	-8.53 (-28.0)	-12.50(1) (-41.0)(1)	-12.50 (-41.0)
Bent 2/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.28 (37.0)	+4.68 (+15.4)	-7.92 (-26.0)	-19.20(1) (-63.0)(1)	-19.20 (-63.0)
Bent 3/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.28 (37.0)	+6.12 (+20.1)	-9.75 (-32.0)	-21.03(1) (-69.0)(1)	-21.03 (-69.0)
Bent 4/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.28 (37.0)	+8.39 (+27.5)	-10.06 (-33.0)	-21.34(1) (-70.0)(1)	-21.34 (-70.0)
Bent 5/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.28 (37.0)	+8.89 (+29.2)	-9.14 (-30.0)	-20.42(1) (-67.0)(1)	-20.42 (-67.0)
Bent 6/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.58 (38.0)	+10.89 (+35.7)	-11.28 (-37.0)	-22.86(1) (-75.0)(1)	-22.86 (-75.0)
Bent 7/ CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	11.28 (37.0)	+9.17 (+30.1)	-10.97 (-36.0)	-22.25(1) (-73.0)(1)	-22.25 (-73.0)
Bent 8/* CIDH 3.0 m (10 ft)				**26,250 (2950)**	**0**	13.87 (45.5)	+9.17 (+30.1)	-12.34 (-40.5)	-26.21(1) (-86.0)(1)	-26.21 (-86.0)
Abut 9 Lt/ CIDH 1.5 m (5 ft)	**1700 (191)**			3400 (382)	0	3.81 (12.5)	+19.1 (+62.7)	-13.26 (-43.5)	-17.07(1) (-56.0)(1)	-17.07 (-56.0)
Abut 9 Center/ CIDH 1.5 m (5 ft)	**2275 (256)**			4550 (512)	0	4.72 (15.5)	+16.3 (+53.5)	-13.26 (-43.5)	-17.98(1) (-59.0)(1)	-17.98 (-59.0)
Abut 9 Rt/ CIDH 1.5 m (5 ft)	**2850 (320)**			5700 (640)	0	5.33 (17.5)	+13.4 (+44.0)	-13.26 (-43.5)	-18.59(1) (-61.0)(1)	-18.59 (-61.0)

Notes: Design tip elevation is controlled by the following demands:(1) Compression;(2) Tension;(3) Lateral Loads

*Note - Special Case – Bent 8 only

OSF recommends full pile length temporary inner casing inside permanent casing [which is seated 0.61 m (2 ft) into rock] to construct pile due to possible caving and aquifer conditions in uncemented formational sand layer between mudstone layers.

If pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

Axial compression and tension values noted in the tables above are based on skin friction only within the rock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 in).

Bedrock topography (top of rock) is often quite variable across short lateral distances. Due to this fact, the pile data table above includes intended length of the rock socket at each support. The intended length of the rock socket should be measured from the bottom of the permanent casing down to the pile specified tip elevation. OSF feels that permanent casing should be seated into rock approximately 0.61 m (2 ft). If the bedrock slope is steeper than expected, the permanent casing may need to be seated slightly deeper to seal out water and potential caving soils.

Constructability

As mentioned above, OSF recommends installation of permanent casing to be placed into bedrock to prevent caving of loose soils and gravel/cobbles lenses into pile borings and help seal off ground water from entering the excavations once seated into rock. OSF and the Office of Geotechnical Earthquake Engineering feel that using a vibratory hammer to place steel casing down to a level close to casing specified tip elevation would facilitate pile construction and effectively reduce creation of voids along the pile length by undesireable caving of loose soils and subbrounded cobble/gravel material. Drilling ahead of the casing, especially within the upper loose/soft soil zones should be avoided to reduce caving and creation of voids, thus compromising lateral pile capacity. OSF anticipates center relief drilling to facilitate casing advancement. Hard slow drilling [through hard metavolcanic cobble zones (cobbles up to 215 mm diameter), cobble-size mudstone rock fragments, and bedrock] is anticipated during installation of permanent casing, temporary casing, and CIDH piles (rock sockets). Drilling ahead of casing may be required, in order to advance casing within the lower gravel/cobble lenses and within bedrock. Once casing is seated into bedrock, drilling for the rock sockets can be completed. As mentioned above, Bent 8 is a special case where full length temporary casing is also recommended to be placed inside permanent casing [seated 0.61 m (2 ft) into mudstone] due to the interfingering uncemented formational sands that exhibit aquifer characteristics. The pile at Bent 8 will penetrate the upper mudstone, underlying uncemented formational sand, and tip out within a lower mudstone layer.

The Caliper log within Boring 99-2 (proposed Bent 8) which was an uncased hole, shows that caving happens readily within shallow loose/soft often saturated alluvium and within the sand and gravel/cobble lenses overlying bedrock. The uncemented formational sand does not show much caving, however, drilling mud is circulated within the borehole just before measurement to prevent caving and loss of downhole geophysical tools.

Ground water should be anticipated at relatively shallow depths. Static ground water was measured at elevation +8.93 m (+29.3 ft) within Boring 99-6 (drilled near proposed Bent 2) and within Boring 99-3 (proposed Bent 7), the water table could be estimated as high as +7.2 m (+23.6 ft) elevation based on P-wave velocity, measured during the dry season. The wet method is advised for CIDH pile construction. The bottom of all excavations should be cleaned of loose debris before placing concrete.

Clay mineralogy within formational material appears sensitive to the introduction of fresh water, which could cause swelling of clays and slicking of borehole walls, resulting in reduced pile/soil skin friction capacity. OSF feels that a mud/polymer expert should be consulted and be available to the contractor to advise on proper drilling fluid/slurry chemistry in order to prevent clay swelling. OSF feels that seating permanent casing and using temporary casing seated into the formational mudstones/claystones/siltstones should help seal off ground water from reacting with the formational clays.

Corrosiveness

Corrosion samples were not taken for this bridge. Laboratory tests of composite soil samples [taken within Boring 99-1 for somewhat nearby Retaining Wall No. 524 indicate that fill and native material are corrosive. Corrosion tests on embankment fill show a pH of 7.48, minimum resistivity of 475 ohm-cm, sulfate and chloride content were measured at 5730 and 760 ppm, respectively. Corrosion tests on alluvial material show pH ranges from 7.48 to 7.98, minimum resistivity ranges from 475 to 746 ohm-cm, sulfate and chloride content were measured at 6000 to 360 ppm and 230 to 150 ppm, respectively.

Caltrans Corrosion Technology Branch has provided detailed corrosion review and corrosion recommendations for the N805/N5 Truck Connector, Br. No. 57-1070G (Tolin, June 22, 2000). The above memorandum should be consulted for corrosion recommendations regarding CIDH piles (cased and uncased), pile caps, footings, and walls.

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If you have any questions, please call Joe Pratt at (213) 620-2001 or Richard Fox at (916) 227-7085.

Report by:

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c: R.E. Pending File
DBarlow - Specs & Estimates
OAlcantara - Proj Mgmt
Dist. 11 (2)
ELeivas - OSF
RFox – OSF
JChai – OGS
DParks – METS-Corrosion Technology
LA File