GROUP DELTA CONSULTANTS

Certified DBE/MBE

Geotechnical Engineering
Geology
Hydrogeology
Coastal Engineering
Hydrology
Hydraulics
Environmental Engineering

Project No. I-181-03
January 28, 1999

BOYLE ENGINEERING CORPORATION
7807 Convoy Court, Suite 200
San Diego, California 92111

Attention: Mr. Clark Fernon, Project Manager

DRAFT TYPE SELECTION REPORT
McGONIGLE CREEK BRIDGE
MIDDLE SEGMENT, STATE ROUTE 56
SAN DIEGO, CALIFORNIA
11-SD-56-KP 3.3 TO 10.5, EA 172820

Gentlemen:

We are transmitting five copies of our preliminary geotechnical type selection report for the McGonigle Creek Bridge, on the Middle Segment of the proposed State Route 56 alignment in San Diego, California. Laboratory testing is currently underway. A final Type Selection Report will be issued at the completion of the laboratory testing. Based on our assessment of the site conditions, we do not anticipate significant changes in our conclusions.

We appreciate the opportunity to be a part of your design team for this project. If you have any questions or require additional information, please call.

Very truly yours,
GROUP DELTA CONSULTANTS, INC.

Kul

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GEOTECHNICAL
STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 Background

This Type Selection Report is based on a geotechnical investigation performed by Group Delta Consultants, Inc. (GDC) to provide recommendations for the foundation design of the McGonigle Creek Bridge. The bridge structure is part of the Middle Segment of the proposed State Route 56 (Ted Williams Freeway), extending from Rancho Penasquitos to Carmel Valley, in the City of San Diego, California (See Site Location Map, Figure 1).

The County and City of San Diego and California Department of Transportation (Caltrans) District 11, have authorized improvements of the Middle Segment of State Route 56. The development limits for the overall Route 56 improvement project extend from Interstate Highway 15 (Escondido Freeway) in Rancho Penasquitos to Interstate Highway 5 (San Diego Freeway) in Carmel Valley. The Middle Segment contains 7 proposed bridges, and extends from metric Station 45+13.527 on the west (in Carmel Valley) to metric Station 109+00 on the east (near Rancho Penasquitos).

Our understanding of the proposed project is based on the following drawings provided by Boyle Engineering: 1:2000 scale plan and profile entitled "SR-56 Selected Alignment," dated August 10, 1998, and "Planning Study" drawings for the proposed bridges dated 1-98 through 9-98.

1.2 Existing Facilities and Proposed Improvements

The site of McGonigle Creek Bridge is located where the proposed SR-56 alignment crosses McGonigle Creek. McGonigle Creek is a large, southwesterly flowing alluvial drainage which crosses the centerline of the proposed SR-56 alignment between approximate metric Stations 96+40 and 97+40. Several smaller tributary drainages feed McGonigle Creek in the vicinity of the proposed bridge. The area surrounding the bridge site remains in a generally natural condition. A 20-ft wide City of San Diego sewer easement runs along the east side of the creek at the proposed bridge location. An 18-inch trunk sewer is buried in the easement, and could be affected by placement of fill at the east abutment of the proposed bridge. Measures to protect-in-place or relocate this utility will be required prior to project construction.

1.2.1 Proposed Bridge

The proposed improvements consist of McGonigle Creek bridge, where Route 56 will pass over the creek (Figure 2, General Plan). The proposed creek crossing will consist of two separate bridges (a Left Bridge and Right Bridge) with a clear distance between bridges of 21.7 m. Both the east-bound (Right) and the west-bound (Left) bridges will carry two lanes of traffic, with the provision for the future construction of bridges for 1 additional lane in each direction. The beginning of the left bridge is at Station 96+25.283, and the beginning of the right bridge is at Station 96+5.057 of the proposed Route 56 alignment. The approximate elevations at the start of the bridges are El. 92.7 m and 92.0 m for the left and right bridges, respectively. End of bridge elevations are about 96.9 and 96.4 m, for the left and right bridges, respectively. The creek elevation is at about El. 68 m, or roughly 24 to 29 m below the proposed elevation of the bridge deck.

Each bridge will be a three-span, cast-in-place prestressed concrete box-girder structure supported by abutments on the northwest and southeast, and two bents in the bottom of the creek channel. The span from abutment to bent is 51.157 m on each side, and the central span is 60.5 m. The bridge deck measures 162.814 m along the centerline of SR 56, and each bridge is 12.77 m wide. Abutments will be founded on approach fills. Type N(9S) approach slabs are planned at all abutments. Slopes below the abutments are planned at a 1:1.5 inclination, with maximum height on the order of 28 meters. The bridge will be skewed with respect to the alignment of McGonigle Creek at about 38 degrees. Slope paving is not indicated on the current plans.

We anticipate that the abutments may be supported on spread footings founded in the approach fills, or alternatively on drilled or driven piles. Abutment footing / pile cap elevations will be about El. 89 m and 93 m at Abutments 1 and 4, respectively.

The foundation at Bent 2 as shown on the Planning Study drawings will be at about El. 65.5 m, and if deepened by 1 to 2 m could be supported on spread footings founded in formational material. Alternatively Bent 2 could be supported on drilled (or driven) piles penetrating 3 to 5 m into formational soils. The Bent 3 foundation will be at about El. 66 m. Due to depth of alluvium, Bent 3 must be supported on either drilled or driven piles.

1.2.2 Design Foundation Sizes and Loads

No data is available on foundation loads at this time.

1.2.3 Existing and Proposed Cut/Fill Slopes

No existing cut or fill slopes are present at the site of the proposed bridge.

The fill slopes to be constructed below the abutments are proposed at a 1:1.5 (Vertical : Horizontal) gradient, and will be about 24 to 28 m in height. The slopes below the abutments are not currently planned to be paved. The north and south slopes of the approach fills will be unpaved, and are currently proposed at a gradient of 1:2 (Vertical : Horizontal).

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 Field Exploration Program

To investigate the subsurface conditions at the bridge site, four 20.3 cm diameter hollow-stem auger borings were drilled on December 24 and December 30, 1998, to depths between 9.5 and 18.5 m below existing grade. In addition, two 0.76 m diameter bucket auger borings were drilled northeast of the west abutment (as part of the investigation for the Middle Segment Geotechnical Design Report) to investigate a large landslide in this area. The location of these borings are presented in Figure 3, and boring logs from the bridge investigation are in Appendix A.

Bulk and drive samples were taken during the drilling operation at selected depths for identification and laboratory testing. All drive samples were advanced with a 63.5 kg hammer dropped from a height of 76.2 cm. The sampler penetration resistance, or number of blows, to advance the sampler 30 cm was measured and recorded on the boring logs, and was used to assess the in-place density or consistency of the site soils.

Intact samples were obtained with a 6.15 cm I.D., 7.62 cm O.D., California Ring Drive Sampler. Representative samples were obtained from a Standard Penetration Test (SPT) drive sampler. Bulk samples of the cuttings from the auger were also collected. Samples were visually identified and classified in the field in accordance with the Unified Soil Classification System (USCS), placed in moisture tight containers, labeled, and taken to the laboratory for further inspection and testing. Pocket penetrometer tests were performed on cohesive ring samples.

2.2 Laboratory Testing Program

Selected samples were tested in the laboratory to measure relevant engineering properties. Testing was performed in general accordance with applicable Caltrans testing methods, where appropriate. The following types of tests were performed:

- Moisture Content and Dry Density
- Grain-Size Distribution
- Liquid and Plastic Limits
- Direct Shear
- Corrosivity (pH, minimum resistivity, Sulfates, Chlorides)
- Pocket Penetrometer

The results of the laboratory tests are presented in Appendix B. (TO BE COMPLETED)

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Climatic Conditions

The project is located west of the Rancho Penasquitos area of the City of San Diego, California. Elevations in the vicinity of the bridge site range from approximately 68 to 108 m above mean sea level (MSL). The annual rainfall ranges from approximately 30 to 38 cm with over 95% of all precipitation occurring between October and May. The area has a semi-arid climate with average high temperatures during the year ranging from 15 to 21 degree C during the winter months to 27 to 32 degree C during the summer months. Average lows are generally 0 to 5 degree C during the winter months, to 10 to 15 degree C in the summer. Soil freeze/thaw conditions are not known to exist within the project alignment.

3.2 Subsurface Conditions

3.2.1 Geology and Soil Conditions

The project site lies within the Peninsular Ranges Geomorphic Province of California, in the coastal plain area of San Diego. The mesa topography of the coastal plain is characterized by low hills and ridges dissected by intervening alluvial canyon drainages. This area is generally underlain by terraced coastal sedimentary formations of Quaternary to Tertiary age. These formations are overlain locally by Holocene (recent) overburden deposits such as alluvium, slopewash, and man-placed fill soils.

The proposed bridge will span a fairly large southwesterly flowing alluvial drainage , as shown in Figure 3. Test borings indicate that the bridge site is underlain by about 2.3 to 5.6 m of overburden deposits, which in turn are underlain by Eocene sedimentary formational material of the Torrey Sandstone (Tt). The geologic units encountered are described below.

3.2.1.1 Alluvium/Slopewash/Slide Debris (Qal/Qsw/Qls)

Our borings encountered alluvial/slopewash (overburden soil) deposits to depths of 2.3 to 7.6 m below existing grade at the bridge site. In addition, up to 19.5 m of landslide debris was encountered in a large landslide mapped to the northeast of the east abutment of the proposed bridge (Figure 3). The toe of the proposed approach fill at Abutment 4 daylights on this landslide.

About 5.2 m of alluvium was encountered in Boring MCB-HSA-2, however, deeper alluvium was encountered in McGonigle Creek in our Borings MGD-HSA-6, 7, and 8 (performed for the Middle Segment Geotechnical Design Report) where about 7.6 m of alluvium overlies the formation. On the west side of the channel, only 3.8 m of alluvium was encountered in Boring MCB-HSA-3. Overburden thickness on the slopes to the west and east of the channel, was measured at 2.3 and 5.6 m, in Borings MCB-HSA-4 and MCB-HSA-1, respectively.

The alluvial / slopewash deposits are generally characterized as moist to wet, very loose to very dense, light brown to dark brown with orange staining, fine to coarse sands and silty sands (SP, SM), with some gravels to 50 mm size and cobbles, and occasional layers of firm to hard, moist to wet, brown to rust orange sandy lean to fat clays (CL, CH). Equivalent Standard Penetration Test (SPT) blowcounts measured in the alluvial / slopewash soils range from 4 to in excess of 100, with an average value of 48. High blowcounts in the alluvium are often a result of gravels and cobbles.

3.2.1.2 Torrey Sandstone (Tt)

Torrey Sandstone was encountered in our four bridge borings at depths between 2.3 and 5.6 m below existing grade, corresponding to El. 77.2 m at Abutment 4 of the Right Bridge, El. 65.0 m at Bent 3 of the Left Bridge, El. 64.8 m at Bent 2 of the Right Bridge, and El. 72.2 m at Abutment 1 of the Left Bridge. This unit is generally characterized as very dense, moist, yellow brown to light gray to gray with orange mottles, silty and clayey medium to fine sands (SM, SC), locally with weak to moderate cementation, interbedded with hard, moist, gray to dark gray silts and clays (ML, CL). Occasional layers of gravel were also encountered within this unit. Equivalent SPT blowcounts measured within the Torrey Sandstone were generally greater than 50 blows per 0.3 meters.

3.2.2 Existing Landslides

The formational materials contain clay interbeds which, under adverse bedding conditions, can be susceptible to landslides. Based on a review of aerial photographs, we have identified a potential landslide (LS-1, Figure 3) on the northeast side of the proposed bridge. This landslide is confirmed by MGD-BA-6 and MGD-BA-43, which identified the presence of landslide debris to depths of up to about 20 m. This landslide could affect the approach fill at Abutment 4 on the northeast side of the bridge.

Another potential landslide (LS-2, Figure 3) may be present underneath Abutment 4. Boring MCB-HSA-1 shows potential landslide material to a depth of 5 to 6 m. If confirmed during the final structure foundation investigation, this slide should be removed before placement of additional abutment fill.

A third potential landslide (LS-3, Figure 3) is based on a review of the aerial photographs and is located south of the toe of approach fill on the southwest side of the bridge. The effects of the two potential slides (LS-1 and LS-3) should be investigated during the development of the GDR and the final bridge structure foundation report.

3.2.3 Groundwater

At the location of our bridge borings, groundwater was encountered between El. 64 and 69 m, primarily within the alluvium. Groundwater should be expected year-round at, or within several meters of, the elevation of the creek bed. Water levels should be expected to fluctuate seasonally due to variations in precipitation, runoff, and surface infiltration.

In Borings MGD-BA-6 and MGD-BA-43, groundwater was encountered near the base of the landslide deposits between El. 69 and 74 m. Strong groundwater seepage and caving were observed within the landslide deposits in these borings. Groundwater levels may typically be higher in the slopes surrounding the creek channel, where laterally migrating groundwater perches on less pervious soil horizons. Construction excavations, depending on location and depth, are likely to encounter groundwater. Excavations for landslide removal, if required, are likely to encounter groundwater seepage.

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Proposed Foundations

Based on our field exploration, the subsurface conditions at the bridge site consist of alluvium, slopewash, and landslide debris, underlain by dense to very dense silty to clayey sands with interbeds of very hard overconsolidated clays. From a foundation standpoint, the formational soils will provide good bearing support. Due to alluvium depth and groundwater conditions, it is our opinion that driven pile foundations are the preferred option for foundation support at Bent 2 and Bent 3. It is our opinion that the abutment foundations can be supported on spread footings, or alternatively driven piles installed in predrilled holes through the compacted fill at the abutments depending on the allowable settlement criteria. However, since the estimated differential settlement between the abutments supported on spread footings and bents supported on piles is greater than 2.5 cm, we recommend that the abutments be supported on pile foundations installed in predrilled holes and driven to refusal in the dense formational soils.

The bent foundations can be supported on a single large diameter or multiple small diameter drilled piles or driven piles penetrating 3 to 5 m into the formational soils, with a pile cap supporting the bent column. Due to the year-round presence of groundwater in the creek bed, driven piles reaching refusal in the formational soils may be preferable to drilled piles at the bents.

4.2 Settlement

We estimated settlement of the abutment and bent footings using the program SETTL/G (GEOSOFT, 1988). For compacted fill, we used an average blow count of 25 and a modulus of 500 ksf. For the formational soils, we assumed a modulus of 2,500 ksf. If spread footings are used at the abutments, we estimate the settlement of the abutment footings supported on 18 to 20 m of compacted fill to range between 3 cm and 4 cm. The settlement of the bent foundations should be less than 1 cm. Differential settlements between the bents and abutments, if abutments are supported on shallow footings will likely be greater than 2.5 cm. If the abutments and the bents are supported on piles, differential settlement between the abutments and the bents are expected to be less than 1.3 cm.

4.3 Seismic Design Considerations

4.3.1 Ground Surface Rupture

The site is not located within the Alquist Priolo Fault zone. No faults were discovered on the site during our field investigation. Faults are not mapped as crossing the site or projecting towards the site in the geologic literature reviewed. As such, the possibility of ground rupture at the site is extremely remote.

4.3.2 Seismic Shaking

The site is located in a moderately-active seismic region of southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking due to nearby and distant earthquakes should be anticipated during the life of the facilities. The controlling fault for this project is the Rose Canyon Fault, located a distance of about 12 km from the site. The fault has a maximum credible earthquake magnitude of 7.0. Based on the Caltrans 1996 California Seismic Hazard Map, we recommend using a PGA of 0.3g for design. Depth to bedrock may be taken as 3 to 25 meters.

Response spectra at the bridge site should be selected in accordance with Applied Technology Council (ATC-32: Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, 1996) for soil profile Type C, with an applicable earthquake magnitude of 7.25 $\pm$0.25, and a PBA of 0.3 g (Figure R3-5 of ATC-32).

4.3.3 Secondary Seismic Effects

Secondary seismic effects for any site include liquefaction, seismic compaction, settlement, and slope instability.

Liquefaction involves a sudden loss in strength of a saturated, cohesionless soil (predominantly sand) caused by cyclic loading such as an earthquake. This results in temporary transformation of the soil to a fluid mass. Typically, liquefaction occurs in areas where groundwater is less than 9 m from the surface and where the soils are composed predominantly of poorly consolidated fine sands. Liquefaction could occur locally in the alluvium in the creek bed. However, all foundations are to be supported in the dense formational soils and should not be affected by liquefaction. Before construction of the abutment fills, all fill should be properly keyed into competent soils to minimize any potential for lateral spreading of the abutment slopes.

Settlement of dry sands can be caused by the cyclic loading of an earthquake. A procedure for estimating the probable settlement of dry sands was developed by

Seed and Silver (1972). This procedure was reviewed by Tokimatsu and Seed (1987). Based on this procedure and the relative density of the soils at the project site, the settlement of dry sands at the site are not expected to be significant.

Slope instability, in the form of landslides and mudslides, is a potential adverse impact associated with seismic shaking. The proposed 1:1.5 fill slopes at the abutments, if properly compacted, keyed at the toe, and benched into the formation materials, are anticipated to be stable under seismic shaking.

4.4 Excavation Characteristics

Based on drilling characteristics and our experience in the area, the formational soils underlying the site may be excavated with medium to heavy effort by conventional heavy-duty grading equipment. The planned excavations may encounter minor to moderate amounts of cemented concretions within the formational soils which may require localized heavy ripping effort, and generate significant amounts of oversize materials (requiring special handling).

4.5 Permanent Slopes

Fill slopes, about 24 to 28 m high, with a gradient of 1:1.5 are planned below the bridge abutments, while 1:2 unpaved slopes are planned for mainline Route 56 slopes. Fill slopes are anticipated to be grossly stable, if keyed at the toe and benched into competent soils. We recommend that all residual, alluvial, and colluvial soils in areas to receive fill be removed from the slopes and that the base of all fills be properly keyed and benched into competent soils. Unpaved fill slopes will be subject to surficial erosion, surficial pop-outs, and rilling if subjected to heavy rainfall. High unpaved slopes may be constructed with geogrid to minimize erosion and increase surficial stability.

Planting of the unpaved slopes with appropriate, drought tolerant vegetation (using minimal irrigation) should be done as soon as possible after excavation to guard against surficial erosion. Care should be taken not to allow surface water to flow over the slope face in an uncontrolled manner.

4.6 Scour

The bridge will be constructed on an existing creek and the bent foundations and abutment slopes may be subject to scour from the creek. We recommend that a scour investigation be performed for the creek to assess the impact of potential scour on the slopes and the foundations.

4.7 Soil Corrosisvity

(TO BE COMPLETED)

5.0 REFERENCES

California Department of Conservation, Division of Mines and Geology, 1994, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, Special Publication 42.

Caltrans, 1995, Standard Specifications, Business, Transportation and Housing Agency.

GEOSOFT, "Settl/G, Settlement and Stress Distribution Analysis," a computer program for calculating stresses and settlements under groups of uniformly loaded rectangular areas, 1442 Lincoln Avenue, Ste. 146, Orange, CA 92667, 1988.

Mualchin, L., 1996, California Seismic Hazard Map Based on Maximum Credible Earthquakes (MCE).

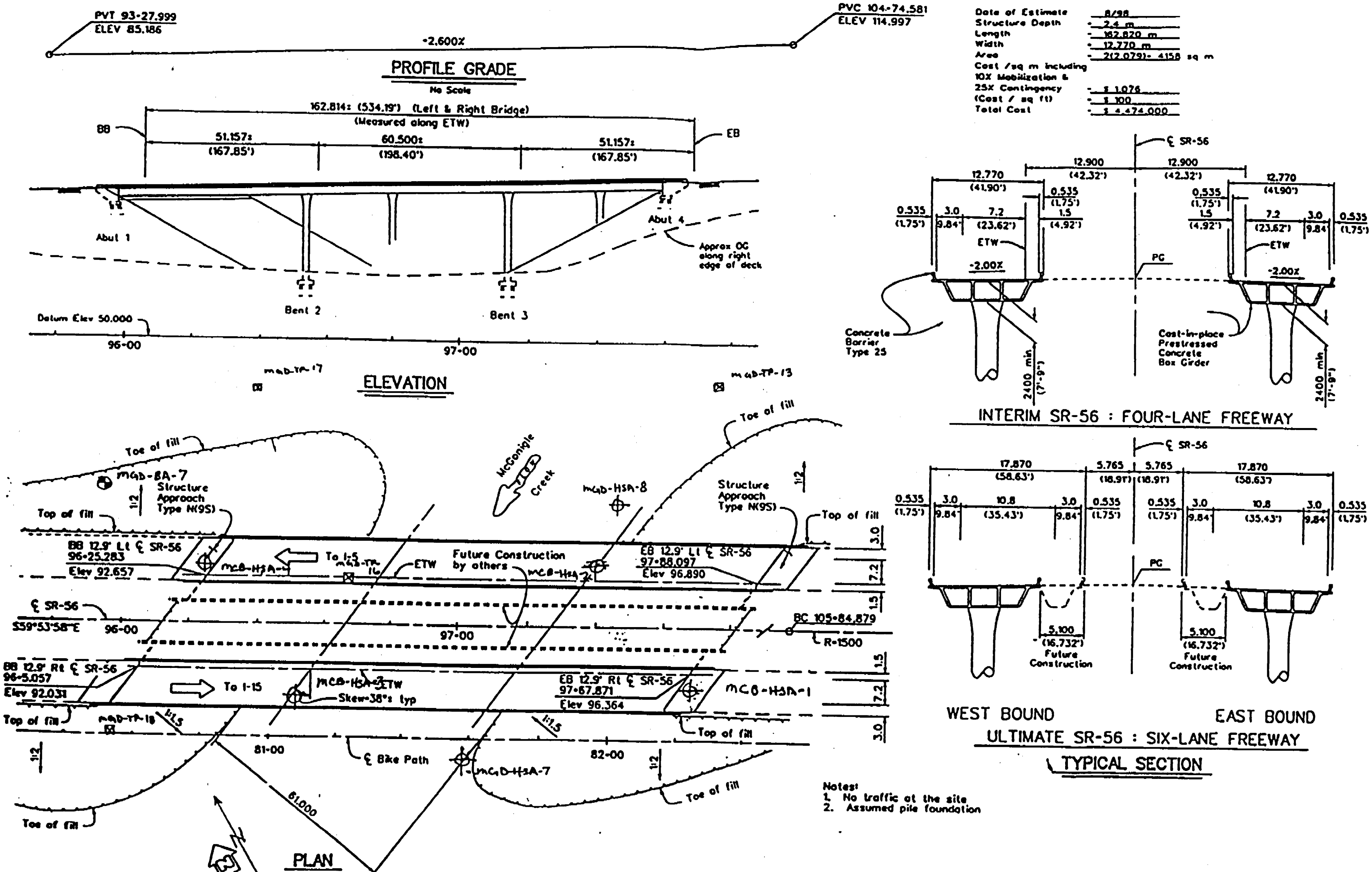
Seed, H.B. and Silver, M.L., 1972, Settlement of Dry Sands During Earthquakes, J. of Soil Mech. Found. Div., ASCE, Vol. 98, No. 4, pp. 381-397.

Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, J. of Geotech. Eng. Div., ASCE, Vol. 113, No. 8.

6.0 LIMITATIONS

The report, exploration logs, and other materials resulting from Group Delta's efforts were prepared exclusively for use in designing the proposed project. The report is not intended to be suitable for reuse on extensions, or modifications of the project, or for use on any other development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

Our recommendations and evaluations were performed using generally accepted engineering approaches and principles available at this time, and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is made.



McDANIEL ENGINEERING

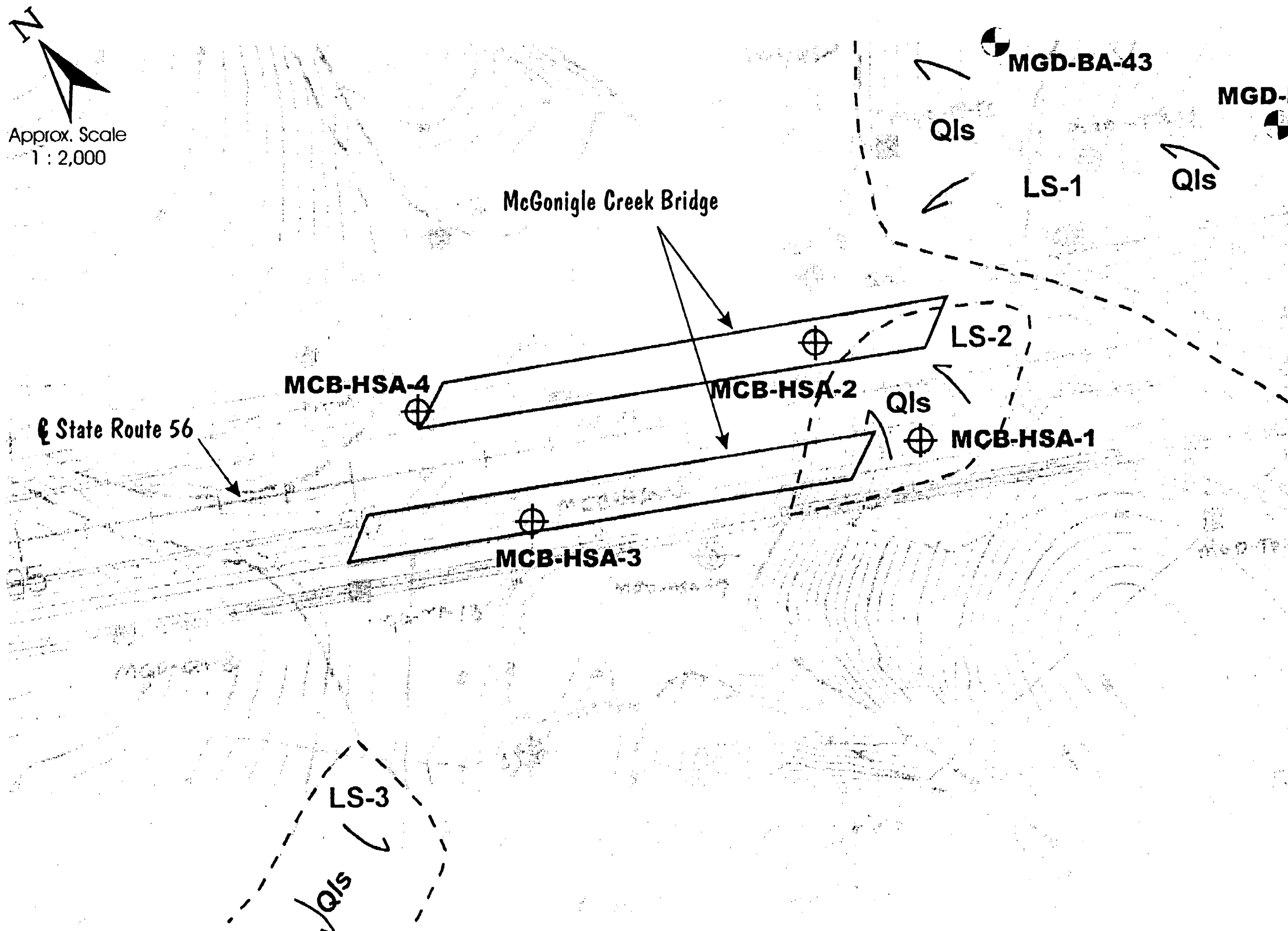
GROUP DELTA

GDC Project No. I-181

State Route 56
Middle Segment Bridges
McGonigle Creek Bridge

GENERAL PLAN

Figure 2



LEGEND

- ⊕ Hollow Stem Auger Boring
- Bucket Auger Boring
- ⊠ Backhoe Test Pit

Reference:
The base map is from Boyle Engineering
SR-56 Selected Alignment, 8-10-1998

GROUP DELTA CONSULTANTS

GDC Project No. I-181
State Route 56
Middle Segment Bridges
McGonigle Creek Bridge
EXPLORATION LOCATION MAP
Figure 3

BORING LOG

LOGGED BY: J. Brown	DATE DRILLED: 12-24-98	BORING ELEVATION: 82.8 M	BORING NO.: MCB-HSA-1 B -
DRILL RIG: CME 85 (Western Hazmat)	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	

Depth	Sample #	Type	Blow Ct.	Recovery	DESCRIPTION
					Cuttings very mixed - unable to recover representative bulk samples
					Approx 18" of Dark Gray sandy fat clay Residual Clay Removed During Road Construction
					stiff, moist, brn sandy lean clay (CL) (Road Fill)
	1	SP	8/6" 12/6" 14/6"	100%	M. dense, moist brn very clayey med to fine sand (SC) Possibly Landslide Deposits
					↓ increased moisture
5	2	CA	17/6" 30/6" 50/6"	100% (1st Ret)	w/ few gravels; High Blow Ct due to gravels
	3	SAH			
10	4	SP	10/6" 10/6" 10/6"		
					↓ increased clay content change to dark brn color & V. dense
15	5	CA	30/6" 60/6"	100% (1st Ret)	
					V. dense, moist, yellowish brn very clayey med to fine sand (SC) w/ local gravels and zones of sandy lean clay (CL); Also w/ local zones of white lime deposits ... Formational
20	6	SP	20/6" 30/6"	100%	

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1524-70	SR-56 McGonigle Creek Bridge	FIGURE NO.:

ROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

LOGGED BY: J. Brown	DATE DRILLED: 12-24-98	BORING ELEVATION: 82.8M	BORING NO.: MCB-HSA-1
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140lb DROP: 30in	B -

Depth	Sample #	Type	Blow Ct	Recovery	DESCRIPTION
	6	SP	20/6" 30/6" 50/6"		
25	7	CA	34/6" 62/6"	100% (1st Ret)	
					- - - - - - - - - - - - - - - -
30	8	SP	22/6" 24/6" 32/6"	100%	V. dense, moist. Lt brn clayey fine sand (SC) w/ mixed small clasts of reddish brn and Lt gray siltstone & sandstone and rare cemented zones and trace gravels
35	9	CA	25/6" 60/6"	100% (1st Ret)	
40	10	SP	18/6" 20/6" 30/6"	100%	

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1524-70	SR-56 MCB	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

LOGGED BY: J. Brown	DATE DRILLED: 12-24-98	BORING ELEVATION: 82.8 M	BORING NO.: MCB-HSA-1 B-
DRILL RIG: CME 85	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 lb DROP: 30 in	

Depth	Sample #	Type	Blow Ct	Recovery	DESCRIPTION
	10	SP	10/6" 20/6" 30/6"	100%	
45	11	CA	20/6" 18/6" 26/6"	25% (1 set Ret)	
					V. dense, moist, Lt gray w/ Lt yellowish brn mottles silty med to fine sand (SC)
50	12	SP	40/6" 60/6"	100%	← generally Lt yellowish brn color
55	13	CA	60/6" 60/4"	100% (1 set Ret)	
60	14	SP	80/6" 40/2"	100%	Increased moisture content, change to gray color (near groundwater table)
					BH GD.? Ft

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 1524-70 SR 56 MCB FIGURE NO.:

ROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

B O R I N G   L O G

LOGGED BY: GAS	DATE DRILLED: 12/30/98	BORING ELEVATION: 70.2 m	BORING NO.: MCB HSA B-2
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 # DROP: 30"	

Depth	Sample	Type	Blows		DESCRIPTION
	1	Ca	39/31		HARD, DAMP, DK. GRAY, SANDY CLAY (CL) ~~Qsw~~ W/TRACE OF GRAVEL & COBBLE
5	2	SP	7/10/12	∇	
10	3	Ca	8/13/20		DENSE, WET, GRAY-BROWN, CLAYEY SAND (SC) ~~Qat~~
15	4	SP	10/22/30		GRAVEL IN TIP.
					V. DENSE, MOIST TO WET, LT. GRAY CLAYEY SAND (SC) ~~Tf~~/TD FORMATION
20	5	Ca	8/60 3"		

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I181-3	SR 56	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

LOGGED BY: GG	DATE DRILLED: 12/30/98	BORING ELEVATION: 70.2 m	BORING NO.: MCB HSA-B-2
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 # DROP: 30"	

DESCRIPTION

Depth	Sample	Type	Blows	Description
20	5	Ca	8/60/3"	V. DENSE, MOIST TO WET, LT GRAY, CLAYEY SAND (SC) TR/KD FORMATION
25	6	SP	60/6"	
29				HARD, DAMP, DK. GRAY, SANDY CLAY. (CL) KD FORMATION
30	7	Ca	120/6"	
34				FOSSILIFEROUS
35	8	SP	40/108	
36				BOH @ 36' GW 5½ @ TIME OF DRILLING
40				

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: I181-3	SR 56.	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
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BORING LOG

LOGGED BY: GAS	DATE DRILLED: 12/30/98	BORING ELEVATION: 68.6M	BORING NO.: MCB-HSA B-3
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140# DROP: 30"	

Depth	Sample	Type	Blows		DESCRIPTION
	1	Ca	27/61		HARD, DAMP, BROWN, SANDY CLAY (CL) ~~QSW~~ Alluvium/Slopewash
5	2	SP	8/12/18		BECOMES, MOIST & GRAY-BROWN IN COLOR
				▽	DENSE TO V. DENSE, WET, BROWN, CLAYEY-SAND W/TRACE GRAVEL Qot (SM)
10	3	Ca	10/37/50	3"	
					V-DENSE GRAVELS HARD DRILLING Qot
					V. DENSE, MOIST TO WET, MOTTLED GRAY/ORANGE CLAYEY SAND (SM) TD FORMATION
15	4	SP	30/60		
20	5	Ca	29/60	4"	

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PROJECT NO.: I181-3	SRSC	FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
Engineers and Geologists

BORING LOG

LOGGED BY: GAS	DATE DRILLED: 12/30/98	BORING ELEVATION: 68.6m	BORING NO.: MCB-ASA B3
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140# DROP: 30"	

Depth				DESCRIPTION
	5	CA	29/60 14"	SEE ABOVE
				HARD, DAMP, DK. GRAY, SILTY CLAY (CL)
				TD FM.
25	6	SP	50/10	V. DENSE, DAMP, GRAY & DK. GRAY, INTERBEDDED SILT, SAND & CLAY...
				TD
30	7	CA	22/60 14"	
				BOH @ 31' GW 7'3" @ TIME OF DRILLING

Descriptions on this boring log apply only at the specific boring location and at the time the boring was made. The descriptions on this log are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO.: 2181-3 SR56 FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
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B O R I N G L O G

LOGGED BY: GAS	DATE DRILLED: 12/30/98	BORING ELEVATION: 74.5m	BORING NO.: MCB-HSA B-4
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140 # DROP: 30"	

D E S C R I P T I O N

Depth	Sample	Type	Blows	Description
	1	Ca	13/20/38	HARD, DAMP, BROWN, SANDY CLAY, (CL) W/ TRACE GRAVELS. Qsw
5	2	SP	85	GRAVELS & COBBLES - HARD DRILLING
	3	Ca	10/60	V. DENSE, DAMP, MOTTLED, LT. GRAY / ORANGE CLAYEY SAND (SC) W/ OCCASIONAL OLIVE, CLAY, CHUNKS. Ty/Td Fm.
15	4	SP	60	
20	5	Ca	10/70	

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PROJECT NO.: I 181-3 SR56 FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
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B O R I N G L O G

LOGGED BY: GAS	DATE DRILLED: 12/30/98	BORING ELEVATION: 74.5m	BORING NO.: MCB+HSA B-4
DRILL RIG: CME 75	BORING DIAMETER: 8" HSA	HAMMER WT.: 140# DROP: 30"	

D E S C R I P T I O N

Depth	Sample	Type	Blows	Description
20	5	CA	18/70	V. DENSE, DAMP, MOTTLED, LT GRAY/ORANGE/BROWN CLAYEY & SILTY SAND (SC/SM)
25	6	SP	48/60/4"	
30	7	CA	65/6"	CEMENTED ZONES V. HARD DRILLING
35	8	SP	100/2"	CEMENTED BOH @ 35' NO GW

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PROJECT NO.: J181-3 SR56 FIGURE NO.:

GROUP DELTA CONSULTANTS, INC.
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