# **GEOTECHNICAL INVESTIGATION SUMMERLIN HOV FLYOVER BRIDGE U.S. 95 WIDENING PROJECT**

LAS VEGAS, NEVADA

**MAY 2008** 

Prepared for:





Black Eagle Consulting, Inc. - Geotechnical & Construction Services



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May 30, 2008 Project No.: 0324-01-7

#### **Geotechnical Investigation**  $RE:$ **Summerlin HOV Flyover Bridge U.S. 95 Widening Project** Las Vegas, Nevada

Dear Mr. Cotton:

Attached please find five copies of the geotechnical report and an electronic copy on CD-ROM for the referenced project. Three of the copies are for PBS&J. Parviz Noori of the Nevada Department of Transportation (NDOT) Materials Division requests that one copy be forwarded to the project manager, John Terry, and one to the Assistant District 2 Engineer, Mohamed Rouas. We are forwarding 6 copies directly to NDOT Materials Division in Carson City, with distribution as listed below.

We appreciate being of service to you on the project. If you have any questions, or require any additional information, please contact us.

Sincerely,

**Black Eagle Consulting, Inc.** 

Larry J. Senior Consultant

JWP:LJJ:lmk

Copies to:



Addressee (5 copies, 1 CD-ROM) Parviz Noori, NDOT Materials Division (6 copies) (NDOT Materials Division, 2 copies plus CD-ROM) (Nancy Kennedy, NDOT Bridge Division, 1 copy) (Sharon Foershler, NDOT Construction Division, 1 copy) (Natalie Caffaratti, NDOT Roadway Design, 1 copy) (Terry Philbin FHWA, 1 copy)

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### **GEOTECHNICAL INVESTIGATION**

## **SUMMERLIN HOV FLYOVER BRIDGE U.S. 95 WIDENING PROJECT**

### **LAS VEGAS, NEVADA**

## **1.0 INTRODUCTION**

Presented herein is the Black Eagle Consulting, Inc. (BEC) geotechnical investigation for the proposed Summerlin Parkway High-Occupancy-Vehicle (HOV) Flyover bridge (Structure I-2744) that is proposed as part of the U.S. 95 / Rainbow Road / Summerlin Parkway interchange in Las Vegas, Nevada. The investigation was performed for the Post, Buckley, Schuh, and Jurnigan (PBS&J) design team, which is preparing the project design for the Nevada Department of Transportation (NDOT). The objectives of this study were to:

- 1. Determine general soil conditions pertaining to the 100 percent design and construction of the proposed bridge and associated retaining walls.
- 2. Provide recommendations for design and construction of the proposed bridge and approaches, as related to these geotechnical conditions.

Our investigation included field exploration, laboratory testing, and engineering analysis to determine the physical and mechanical properties of the various on-site materials. Results of our field exploration and testing programs are included in this report and form the basis for all conclusions and recommendations. This report includes pertinent borings from previous BEC investigations on this site (BEC, 2002a, 2002b, 2002c, 2003). The services described herein were conducted in accordance with the Subcontract Addendum to PBS&J Sub-consultant Contract for *PBS&J Project 511300.01 with Black Eagle Consulting, Inc., with the short title, "U.S. 95 HOV Flyover."*

## **2.0 PROJECT DESCRIPTION**

### **2.1 Site Conditions**

The Summerlin HOV Flyover bridge will be located within the U.S. 95/Summerlin Parkway/Rainbow Road interchange in the City of Las Vegas. The interchange is contained in Sections 26 and 27, Township 20 South, Range 60 East, Mount Diablo Meridian.

The east half of the HOV Flyover bridge will be located in the future centerline of U.S. 95 freeway, which is being reconstructed for widening. The U.S. 95 freeway in the Summerlin/Rainbow Road interchange is generally depressed as much as 2 to 3 m below original natural grade. The footprint of the south half of the bridge at the time of our exploration was approximately level at the adjacent freeway grade with gravel surfacing and no improvements. Existing traffic on U.S. 95 was moved from the northeast side of the proposed bridge to the southwest side of the proposed bridge shortly before our exploration.

The HOV Flyover bridge will cross over the Rainbow Boulevard bridge and Ramp R7 bridge. North of Rainbow Boulevard, the bridge will cross above U.S. 95 southbound, Ramp R6, and various landscaped areas within the interchange. Landscaping is minimal, consisting of scattered trees or shrubs and gravel hardscape. The west end of the bridge approaches the west end of the Ramp 11 overpass, which connects from U.S. 95 northbound to Summerlin Parkway westbound. The west abutment and Piers 9 through 11 will be built over an existing 2H:1V (Horizontal:Vertical) fill slope for Ramp 11 which is approximately 11 m high.

Various utilities including traffic control loops, overhead lighting wiring, water and storm drain are present in the area. A 1,060-mm-diameter water supply pipeline and a storm drain box culvert cross beneath the flyover bridge footprint south of Rainbow Road.

#### **2.2 Structure/Development Information**

The HOV flyover bridge will provide a route for HOV traffic from the median of U.S. 95 northbound to the median of Summerlin Parkway westbound, and from the median of Summerlin Parkway eastbound to the median of U.S. 95 southbound. The bridge will carry one travel lane in each direction, with wide shoulders in each direction and a central barrier rail. The bridge will be 18.3 m wide and 740 m long, with 12 spans between 55 and 69 m in length. Abutment 1 will be located in a Mechanically Stabilized Earth (MSE) fill approximately 5 m high in the median of U.S. 95. Mechanically Stabilized Earth walls H1 and H2 will be as high as 4 and 5 m high

extending 71 to 111 m to the southeast, on the northeast and southwest sides of the HOV lanes, respectively. Abutment 2 will be located on an existing 2H:1V fill for the adjacent Ramp 11 overpass. Mechanically Stabilized Earth walls H3 and H4 will be as high as 4 and 11 m high, extending 33 and 75 m to the west, on the north and south sides of the HOV lanes, respectively.

Two concrete cantilever walls, H-5 and H-6, are planned to support the Ramp 11 embankment adjacent to new bridge piers. Wall H-5, which will be up to 3.9 m high and 17 m long, will support the edge of the Ramp 11 adjacent to Pier 11. Wall H-5 will be founded on a 4H:1V slope immediately uphill from Pier 11. Wall H-6, which will be up to 2.65 m high and 4.5 m long, replaces the south end of an existing tieback wall which must be excavated to construct Pier 10. Wall H-6 has level ground at its base and supports a 2H:1V slope below an existing bridge abutment.

The bridge will consist of steel plate girders with a concrete deck supported on cast-in-place concrete piers. Piers will have a single rectangular column, typically 1.8 by 5 m in cross section, and columns will vary in height between 2.5 and 14.1 m above final grade. Abutment 1 will be a closed abutment. Abutment 1 and Piers 1 to 8 will be supported on spread footings that will be least 2 m below the ground surface. The footings for Piers 9 through 11 are proposed to be 1 to 4 m above adjacent interchange grade within existing 2H:1V embankment slopes. The footing for Abutment 2 is proposed to be between 1 to 8 m above adjacent interchange grade. If built at lower elevation, the Abutment 2 footing would be entirely in existing embankment fill. If Abutment 2 is built at the highest elevation, the existing embankment would be widened by the addition of new borrow fill. The north side of the abutment footing would be on existing embankment but the south edge of the footing would be on as much as 3 m of new embankment fill overlying existing embankment fill.

The bridges will be designed using Load and Resistance Factor Design (AASHTO, 2007). Vertical service loads (consisting of dead plus live loads, load factor of 1.0) were 27,000 kN for typical pier foundations. Abutments will be subject to soil and bridge deck longitudinal loading.

## **3.0 GEOLOGIC CONDITIONS AND SEISMICITY**

#### **3.1 Regional Geologic Conditions**

The project lies within the Las Vegas Valley. The Las Vegas Valley occupies a topographic and structural basin transitional between the "younger" Basin and Range topography of the Great Basin of Nevada and Utah, and the "older" Basin and Range topography characteristic of the

Mojave and Gila Deserts of Arizona and California. Extensional or normal faulting started in mid- to late-Tertiary time resulting in the basin-and-range topography currently observed in the Las Vegas area. Following the peak of tectonic activity in Miocene time, and continuing through Quaternary time, a thick, semi-continuous sequence of terrestrial sediments accumulated in deep structural basins, including the structural basin that forms the Las Vegas Valley.

The center of the Las Vegas structural basin is characterized by a gradual alluvial plain sloping from the west and northwest to the east and southeast, that is crossed by a series of generally north-south-trending normal faults that extend discontinuously across the center of the valley. The majority of the faults show movement down to the east and extend to depths that approach 150 m.

## **3.2 Surficial Geology**

The HOV flyover bridge alignment is on soils that have been mapped by the Nevada Bureau of Mines and Geology (NBMG) (Matti, et al., 1987) as intermittently active alluvium (Holocene) overlying Older Alluvium of the Red Rock Fan (Pleistocene). According to the NBMG, the Holocene alluvium consists of slightly to moderately consolidated sand and pebble to cobble gravel. The Older alluvium of the Red Rock Fan ("older fan deposits") consist of mostly moderately well-consolidated and cemented, pebble to small cobble gravel with pebble-bearing sand.

## **3.3 Seismicity**

The Las Vegas area is relatively quiet seismically compared to the northern portions of the state of Nevada. The mountain ranges and deep alluvial basin in the Las Vegas area were formed primarily by Tertiary tectonic movements, i.e., activity greater than 1.6 million years before present. The strongest historic seismic activity in the project vicinity has been related seismic response to filling of Lake Mead (M 5.0; several earthquakes between 1938 and 1952) or energy released from underground atomic testing north of Las Vegas in Yucca Flats (maximum magnitude M 5.8).

American Association of State Highway Transportation Officials (2007) shows horizontal rock acceleration potential to be 0.10g for a 10 percent probability of exceedance within 50 years in this area.

#### **3.4 Faults**

Area Quaternary faults have been mapped by the NBMG and are presented in the *Map of Faults and Earth Fissures in the Las Vegas Area* (dePolo and Bell, 2000). This map identifies traces of potential Quaternary age tectonic faults approximately four kilometers (km) west and two km east of the site, but there is no evidence of faulting in the site vicinity.

#### **3.5 Ground Subsidence**

Regional land subsidence in the Las Vegas Valley related to ground water withdrawal has been monitored since 1935 (Bell and Price, 1993; Bell et al., 2001, 2002). A map included in the 1993 open-file report, titled *Subsidence in Las Vegas Valley 1963 Through 1986/87,* identifies three major Las Vegas subsidence centers located about seven km north, seven km east, and eight km southeast of the site. The map also shows that subsidence of as much as 150 mm may have occurred in the site vicinity between the years 1963 and 1987. The more recent work (Bell, et al., 2001) indicates that subsidence within the project area has been arrested since 1987 and no additional settlement has occurred over the last two decades.

#### **3.6 Ground Fissures**

The nearest areas of ground fissures have been mapped approximately three to four km east of the site. No fissures have been mapped by others and no evidence of fissuring was observed in explorations at the project site.

#### **3.7 Liquefaction Potential**

There is no potential for liquefaction at the site because design earthquake motions and magnitude are low and the ground water is located at a depth below 30 meters.

#### **4.0 EXPLORATION**

#### **4.1 Drilling**

Portions of the bridge alignment were explored in 2001 to 2002 by drilling hollow-stem auger borings to a maximum depth of 30.5 meters. The borings were drilled with 152-millimeter (mm), outside-diameter (O.D.), 83-mm-inside-diameter (I.D.) augers using a truck-mounted Foremost B90 and a track-mounted Diedrich D-50 Turbo drill rig. The locations of closest previous test borings with sufficient depth (B-01, B-02, B-04, B-07 through B-11, B-13, B-18 and B-19) are shown on Plate 1 - Plot Plan. Numerous other borings were performed at greater distances from the HOV bridge alignment, including borings by others, which are shown in BEC (2002b).

Seven borings (FB-01 through FB-07) were drilled in April 2007. Three borings (FB-08 through FB-10) were drilled in October 2007 on the Ramp 11 embankment between Pier 9 and Abutment 2. The borings were drilled with 152-mm O.D., 83-mm I.D. augers to a maximum depth of 24.4 m using a track-mounted Diedrich D-50 Turbo drill rig.

Native soils were sampled in place every 600 to 1,500 mm by use of a standard, 51-mm, O.D., Split-Spoon Sampler or an 89-mm O.D. Split-Spoon Sampler (ASTM D 3550), both driven with a standard 63.6-kilogram (kg) drive hammer and a 760-mm stroke (Standard Penetration Test, AASHTO T 206). The number of blows in a Standard Penetration Test (SPT) is an indication of the density and consistency of the material. Also, at various locations, where the split spoon samplers were not able to retrieve an adequate sample in the cemented, coarse, granular soils, grab samples were acquired from the auger spoils to obtain a sample of sufficient size for determining the approximate maximum particle size and particle gradation.

Coring was attempted in two borings (FB-04 and FB-05) from 1.5 to 4.5 m depth using rock coring equipment. Recovery of intact core was poor, indicating that dense gravel soils are not highly cemented.

The horizontal and vertical locations of each boring were resurveyed by PBS&J after drilling. Logs of borings are included in Appendix B (Subsurface Exploration Data).

## **4.2 Material Classification**

A geological engineer examined and identified all soils in the field in accordance with ASTM D 2488. Additional soil classification was subsequently performed on soil samples in accordance with ASTM 2487 (Unified Soil Classification System [USCS]) upon completion of laboratory testing. Where soil tests are not listed in the appropriate column of the boring log, or soil gradation in the material description column is listed as "estimated," the USCS symbols and terminology are based on manual identification (ASTM D 2488) rather than laboratory classification. A classification and symbol key is provided as Plate B-2 in Appendix B.

Some unavoidable bias in the grain size distributions is present in the soil identification and classification due to drilling and sampling methods. A majority of samples were collected from driven samples which met refusal (greater than 50 blows per 15 cm driven interval). Sampler refusal frequently results in crushing of material at the sampler tip and a greater fraction of smaller particles and some non-plastic fines (rock flour). Samples were also obtained from auger cuttings, where the auger lifting operation also tends to fracture and degrade larger particles into smaller particles. One sample (FB-04 at 1.8 m) was collected by rock coring techniques. Coring can reduce crushing or breaking of larger-sized particles (up to the diameter of the core barrel) but can result instead in washing out of fine particles. The cored sample had only 2 percent fines, 7 percent sand, and 91 percent gravel-sized particles. For comparison, SPT and auger cutting samples at adjacent depths consistently had 5 to 20 percent fines, 30 to 70 percent sand, and 10 to 65 percent gravel.

#### **4.3 Drive Hammer Calibration**

Borings in the 2007 investigation (FB-01 through FB-10) were sampled with a calibrated automatic hammer with an efficiency of 69 percent (Foundation Tech, LLC, 2007). The field SPT values should be multiplied by 1.15 to obtain a standard 60 percent efficiency. Where borings were performed with a calibrated automatic hammer, the hammer efficiency is also listed in the "remarks" column on each boring log.

Previous explorations were performed with a down-hole hammer operated with a wire cable winch system. The wire cable is raised and lowered by a hydraulic winch. Hammer efficiency for this system is unknown but is generally low.

## **4.4 Shear-Wave Velocity Survey**

Redpath Geophysics conducted a shear-wave velocity survey during July 2001 in Boring B-01. The velocity study used conventional down-hole survey methods, where travel times of the shearand compression-wave arrivals are measured progressively from the surface to the bottom of the borehole. The velocities are shown on Table 1 – Shear-Wave Velocities in Boring B-1.



## **5.0 LABORATORY TESTING**

All soils testing performed in the BEC soils laboratory were conducted in accordance with the standards and methods described Nevada Department of Transportation (NDOT, 2001), American Association of State Highway Transportation Officials (AASHTO, 2004) and ASTM (2005).

## **5.1 Index Testing**

Samples of each significant soil type were analyzed to determine their in-situ moisture content (NDOT T 206F), grain size distribution, and plasticity index (NDOT 210E, 211E, and 212E). The results of these tests are in Appendix C.1 - Laboratory Test Results.

#### **5.2 Strength Tests**

Direct Shear Tests (AASHTO T236-92) were performed on representative samples of soil from the project alignment. Since even ring samples were disturbed and all had particles greater than the 2 mm (No. 10) sieve, all direct shear test samples were prepared by removing particles greater retained on the No. 10 sieve and recompacting near the optimum water content. Direct shear test samples in the present investigation were also split on the No. 200 sieve, and then recombined to provide the same fines content (percentage of material passing the No. 200 sieve) as was present in the original bulk soil sample. Results of these tests are shown in Appendix C-2 – Strength Test Results.

A Harvard Miniature compaction test (NDOT T101E) was performed on one of the screened samples for direct shear testing (FB-10 at  $9.1 - 10.7$  m) for comparison of tested and compacted densities. The test results showed the maximum dry density of 19.8 kN/ $m<sup>3</sup>$  (126 pcf) at an optimum moisture content of 9 percent by dry sample weight. The tested sample at that location had an average density equal to 97 percent of the maximum dry density by this test method.

#### **5.3 Corrosion Potential Tests**

Chemical testing was performed on representative samples of site foundation soils to provide data for corrosion potential evaluation. Chemical testing was subcontracted to Western Environmental Testing Laboratory of Sparks, Nevada. Testing for pH was performed in accordance with Environmental Projection Agency (EPA) Method 9045B (AASHTO T289-91). Testing for soil resistivity was performed in accordance with EPA Method 2510B (AASHTO T288-91). Testing for soluble chloride and soluble sulfate was performed in accordance with EPA 600/4-79-020- 300.0 (AASHTO T290-94 and T90-95). These test results are shown in Appendix C-3 – Chemical Test Results.

## **6.0 DISCUSSION OF INVESTIGATION RESULTS**

#### **6.1 Geologic and Geotechnical Conditions**

The proposed HOV flyover alignment is mapped by the Nevada Bureau of Mines and Geology [NBMG] (Matti et al., 1987) as Pleistocene Older Alluvium of the Red Rock Fan, which is described as *mostly pebble to small cobble gravel, with pebble-bearing sand that is moderately well consolidation and cemented*. The materials encountered in our borings were consistent with the description of the Older Alluvium by the NBMG. Because the interchange is cut 2 to 3 m below original grade, recent (Holocene) deposits are not present at foundation levels. Soils were uniform in consistency across the site and at all depths, and consisted of clayey and silty sands with gravel, clayey and silty gravel, poorly-graded and well-graded sands or gravels with silt or clay, and rarely poorly graded sands or gravels. Samples ranged in proportion from 5 to 25 percent low- to medium-plasticity fines, and 20 to 65 percent gravel content. Some cobbles are likely present but were not extensively observed due to the relatively-small-diameter sampling methods.

The soils are very dense or slightly cemented, indicated by high penetration resistance and shearwave velocities in excess of 600 meters per second. Soils were not sufficiently cemented to allow successful intact core recovery. No hard caliche layers were encountered that resulted in refusal of the hollow-stem-auger drilling equipment.

Fills for the existing embankment of Summerlin Parkway westbound (Ramp 11) were found to be well-compacted, dense, granular soils similar in grain size characteristics (coarse silty, sandy gravel with occasional cobbles) and density to the underlying native soils.

Ground water was not encountered during the explorations to depths in excess of 30.5 m, and is at a depth that would not affect construction.

#### **6.2 Geologic Hazards**

A moderate potential for dust generation is present if grading is performed in dry weather. Regional subsidence has occurred at this site but is not occurring presently, due to more-controlled ground water withdrawal from the aquifers under the site. Regional ground subsidence, if it recurs, is sufficiently gradual in magnitude and widespread in horizontal extent that it should not impact structural performance. No other geologic hazards were identified.

## **7.0 DISCUSSION AND RECOMMENDATIONS**

#### **7.1 General Information**

The proposed bridge alignment is on dense, slightly-cemented granular soils which provide good bearing conditions to support the proposed bridge on shallow foundations. Slight cementation may make excavation difficult with smaller equipment. Fills for the existing embankment of Summerlin Parkway westbound (Ramp 11) were found to be well-compacted, dense, granular soils similar to native materials. Gravel and likely cobbles are present that could make temporary excavation support with tiebacks or soil nails difficult, both due to difficult drilling and to possible caving or oversize excavation of drill holes.

#### **7.2 Seismic Design Requirements**

For the purposes of this project, we recommend a minimum design acceleration value of 0.15g to be used in accordance with NDOT design policy. Soil Profile Type II is appropriate given the considerable depth of the Las Vegas soil basin under the site.

#### **7.3 Structure Foundation Recommendations**

#### **7.3.1 Foundation Type Selection**

Shallow foundations are appropriate for the Summerlin Parkway HOV flyover bridge, due to the strong, very dense, granular subgrade soils. If there are tight space constraints or high uplift requirements, drilled shafts would be a suitable alternate foundation method. Driven piles would not be suitable due to very difficult conditions for pile driving.

For design, a friction angle of 37 degrees was used for native soils. The friction angle for native soils was considered to be at least 37 degrees based on high penetration resistance (e.g. FHWA 2007 Table 10.4.6.2.4-1). The selection of friction angle was further supported by the uniform, well-graded particle-size distribution, with sufficient gravel-size particles to provide interlocking of coarse-grained as well as sand-sized particles in the soil matrix. Geologic aging or other minor cementation also was considered in selecting the friction angle. Results of direct shear tests from the original investigation were reviewed. After removing particles larger than the No. 10 sieve, soils had a range of 25 to 48 percent fines content, but still had average friction angle of 37.5 degrees. Since the actual fines content of the original samples was only 10 to 17 percent, and coarse particles were present, lower strength values in the direct shear testing were discounted as not reflecting the actual material characteristics or strength.

A friction angle of 37 degrees was selected for existing embankment fill. Similar grain size distribution and similar to slightly higher penetration resistance was present as for the native soils; however, geologic aging and cementation would not be present. Direct shear testing of fill soils in this investigation used soils with the same fines content as the in situ material, with a resulting friction angle of 37 to 42 degrees; the lower bound was used for design.

#### **7.3.2 Shallow Footing Recommendations**

#### Level Ground Footings (Abutment 1, Piers 1 through 8)

Shallow footings should be designed using the lesser of the factored bearing resistance determined from nominal bearing capacities or the bearing pressure determined from settlement limitations. Nominal bearing resistances for strength design and service bearing resistances providing less than 1 inch of settlement are presented versus footing width on Plate 2. Bearing resistance limits are for footings 2 m or greater depth below adjacent grade in dense native soils (Abutment 1 and Piers 1 through 8). Assumptions regarding footing length and depth of footing are stated on the Plate. If the final footing dimensions (including the effects of eccentric loading), depth, or horizontal locations vary substantially from these assumptions, we should provide additional analysis for the actual design geometry.

Major utilities, including a water main and a box culvert, cross under the bridge alignment south of Rainbow Boulevard. Culverts, utilities, or other infrastructure should lie above a 1½H:1V plane projected downward from the edge of the footings. This separation is recommended to

avoid undue stresses on culverts or pipe, to avoid damage to the bridge if failure of the utility occurs, and to allow for repair or replacement of the utility at a future time without requiring complete removal or extensive underpinning or shoring of the bridge abutment. For footings on native soils, abandoned utility trenches which extend under the footing zone of influence should be excavated and backfilled with slurry cement.

#### Footings in Sloped Embankment Fills (Piers 9 to 11, Abutment 2)

Nominal bearing resistances and service bearing resistances providing less than 1 inch of settlement are presented versus footing width on Plate 3 for footings founded in existing Ramp 11 embankment soils (Piers 9 through 11 and Abutment 2). These conditions are appropriate for where the footings will be supported in existing embankment fill or are founded above the lowest adjacent grade (above the toe of a slope). Design values on Plate 3 are based on a measured soil friction angle for the existing embankment fill of 37 degrees or greater.

For Abutment 2, design options include either a tall abutment founded near the base of the existing fill, or a shallow abutment founded partly above the existing embankment fill prism. In the latter case, the embankment would be widened to provide sufficient fill to support and embed the abutment, so that the outer edge of the abutment could potentially be supported by an additional 3 m thickness of new select borrow. Since the properties of the new select borrow are not determined, a friction angle of 34 degrees is appropriate for design (per NDOT policy). Per NDOT policy, bearing resistances on new embankment fill are typically limited to prescribed values; however, given the size of the proposed footing (9 by 18 m in the present design iteration) we recommend values higher than the prescriptive limit for footings wider than 6 m. Nominal bearing resistances and service bearing resistances providing less than 1 inch of settlement are presented versus footing width on Plate 4 for the Abutment 2 footing founded partly on new embankment fill and partly on existing Ramp 11 embankment fill.

#### **7.4 Lateral Earth Pressures**

Earth-pressure coefficients are provided below for lateral load design of retaining walls, abutments, or structures with imbalanced earth loads. Fill slopes and retaining walls will generate active or at-rest pressures that will impose loads on structures. For shallow foundations, wind, seismic, or earth pressure loads may be resisted by passive soil pressure and friction on the bottom of the footings.

Table 2 shows the recommended static and dynamic soil pressure coefficients. The values are based on Mohr-Coulomb analyses, Mononobe-Okabe analyses, and log-spiral passive charts by Caquot and Kerisel as referenced in AASHTO (2007). Table 2 provides recommendations for approximately level ground at the top and bottom of the wall, and different values will be needed if sloped conditions are present. We recommend neglecting the passive pressure where the base of Abutment 2 is adjacent to a 2H:1V slope. Retaining walls can use bearing resistance recommendations provided in the previous section.

#### **7.5 MSE Wall Recommendations**

Mechanically Stabilized Earth walls should be designed based on the parameters on Table 3. Retaining walls can use bearing resistance recommendations provided in the previous section. For seismic design, the horizontal seismic coefficient should be one-half the design peak ground acceleration, or 0.075g. Mechanically Stabilized Earth wall external stability design (except for global stability) was performed by PBS&J and is not included in this report.



anchors or battered piles.

<sup>(3)</sup> Active pressures assume a vertical wall face and an interface friction angle of 17 degrees and the pressure resultant is oriented at the interface friction angle above horizontal.



 $(3)$  For "Rankine" conditions, earth pressure is parallel to the upper ground surface, or horizontal in this case. Rankine pressures determined using the Coulomb equation with interface angle  $\delta$ = slope angle β, wall back-face angle θ= 0.

#### **7.6 Slope Stability**

Global slope stability analyses were performed for the abutment fills, fills with MSE walls, and at Pier 9 where the bridge footing will be founded in the existing embankment above lowest adjacent grade. As requested by the NDOT Materials Division, for global stability purposes only, we have selected a friction angle of 32 degrees for the new and planned embankment fill. Mechanically Stabilized Earth walls with a minimum strap length of 70 percent of the wall height FHWA (2002) were found to provide adequate global slope stability. Mechanically Stabilized Earth wall internal and external stability (overturning, sliding, bearing resistance checks) was analyzed by PBS&J and is not included in this report.

Slope stability cross sections and resulting factors of safety are summarized on Table 4. These slopes have adequate factor of safety for seismic and static conditions. No seismic slope deformations are predicted to occur in the event of the design earthquake (peak ground acceleration of approximately 0.15g).



 $(1)$  Fill materials assigned zero cohesion and friction angle of 32 degrees for global stability ONLY, native material zero cohesion and friction angle of 37 degrees.

 $(2)$  MSE reinforcement length 70 percent of wall height.

<sup>(3)</sup> Factor of safety corresponds to  $\Phi$  x Nominal Resistance/Service I Load, where minimum factor of safety of 1.53 corresponds to Φ of 0.55, minimum factor of safety of 1.33 corresponds to Φ of 0.65 according to AASHTO (2007 11.6.2.3), and seismic factor of safety of 1.1 corresponds to Φ of 0.90.

## **7.7 Corrosion Potential**

Soils typically have concentrations of up to 1,400 parts per million (ppm) of soluble sulfate and up to 240 ppm of soluble chloride, pH between 7.5 and 10.6, and soil resistivity between 1,200 and 7,500 ohm-cm.

## **7.8 Earthwork and Grading Recommendations**

#### **7.8.1 Clearing, Grubbing, and Removals**

Clearing, grubbing, and removal of obstructions shall be performed in accordance with *NDOT Standard Specifications* Sections 201 and 202 (NDOT, 2001). Except for limited plantings, there is no vegetation on the site, a stripping depth of 0 to 10 cm is expected.

#### **7.8.2 Excavations and Embankment**

Excavations and embankment should be prepared in accordance with *NDOT Standard Specifications* Sections 203 (NDOT, 2001). Assuming that embankment will be constructed of sound, gravelly fill similar to the native soil, settlement of the embankment and its foundation should be minor. Settlement of embankment materials should be completed as the fill is raised.

Excavated soils will be slightly-cemented gravelly soils. These soils were drilled without difficulty with hollow-stem augers to a depth of 30 m, indicating that the site materials can generally be excavated with standard excavation equipment, and blasting is not expected to be required. Subsidence and shrinkage of native ground or existing fills excavated and recompacted as embankment fill is expected to be negligible.

#### **7.8.3 Structure Excavation**

Structure excavations and backfill should be performed in accordance with *NDOT Standard Specifications* Sections 206, 207 and 208 (NDOT, 2001). All trenching should be performed and stabilized in accordance with OSHA standards. Regardless of excavation soil type or required trench slopes or shoring, pavement quantities should be determined per the *Standard Plans for Road and Bridge Construction* (NDOT, 2007).

As noted above, utilities which lie below a 1.5H:1V plane projected downward from the edge of footings should be completely removed. For footings designed for bearing resistance on native soils, backfill the excavation under the footing area of influence with cement slurry.

Significantly shored or sloped excavations will be required for footing excavations for Piers 9 through 11 and Abutment 2 due to the height of adjacent ramp 11.

The ground water under the project alignment is at a considerable depth and should have no impact on construction

#### **7.8.4 Settlement Monitoring**

We recommend that permanent settlement monuments be established and used during construction of the Summerlin HOV Flyover bridge. This will enable NDOT to measure the actual amount of ground settlement that occurs under piers and abutments during and after construction. This type of information has not been gathered by NDOT very often in the past but it is becoming more important as construction pace increased in major urban areas.

Temporary settlement points would be established after the footing is poured and before the columns are formed. The relative elevations would be transferred to a one permanently-accessible location on each column and two locations on the left and right side of the abutments after the forms are removed, and maintained until the end of construction. Settlements would be monitored approximately every 2 months, before and after major increases in bridge loading (column pour, superstructure erection, footing backfill, bridge opening) and once immediately before project completion, whichever is less frequent.

## **8.0 STANDARD LIMITATIONS CLAUSE**

This report has been prepared in accordance with generally accepted geotechnical practices. The analyses and recommendations submitted are based on field exploration performed at the locations shown on Plate 1 - Plot Plan and previous on-site investigations for existing structures. This report does not reflect soils variations that may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. This report has been prepared to provide information allowing the engineer to design the project. In the event of changes in the design or location of the project from the time of this report, recommendations should be reviewed and possibly modified by the geotechnical engineer. If the geotechnical engineer is not granted an opportunity to make this recommended review, he can assume no responsibility for misinterpretation or misapplication of these recommendations or their validity in the event changes have been made in the original design concept without his prior review. The geotechnical engineer makes no other warranties, expressed or implied, as to the professional recommendations provided under the terms of this agreement and included in this report.

#### **9.0 REFERENCES**

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## **APPENDIX A**

## **FIGURES**











## **APPENDIX B**

## **SUBSURFACE EXPLORATION DATA**

## **B-1**

## **BORING LOGS**

































































































NV\_DOT 0324017.GPJ NV\_DOT.GDT 5/30/2008 NV\_DOT 0324017.GPJ NV\_DOT.GDT 5/30/2008









 **B-2** 

## **KEY TO BORING LOGS**

### SOIL CLASSIFICATION CHART



NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL **CLASSIFICATIONS.** 



1- 10 low plasticity, 11 - 25 medium plasticity, 26 - 50 high plasticity, >50 very high plasticity



KEY TO LOGS (NDOT) CALICHE 0324017.GPJ US LAB.GDT 5/14/2008

Black Eagle Consulting, Inc. 1345 Capital Blvd., Suite A Reno, Nevada 89502-7140 Telephone: (775) 359-6600 Fax: (775) 359-7766

#### **EXPLORATION SAMPLE TERMINOLOGY**



unless stated otherwise in log remarks.

### **MOISTURE CONDITION**



#### **LABORATORY TEST ABBREVIATIONS**

Consol=Consolidation; DS = Direct Shear; E=Expansion; HYD = Hydrometer; MD= Moisture and Density PI= Atterberg Limits; R = R-value; Sv = Sieve Analysis; TXCU= Consolidated Undrained Triaxial: TXUU= Unconsolidated Undrained Triaxial: SpGr = Specific Gravity; Chem = Chemical Testing;<br> $M = \text{Moisture Content}$ 



#### RELATIVE DENSITY OF GRANULAR SOILS



Blowcounts on this site typically resulted in refusal before achieving all three 150-mm-increments of driving. The blowcount for each increment and the blowcount sum for the last 300 mm of driving is therefore recorded as 50[blows]/25mm typically. 100 11 Where the last increment of driving exceeds 100mm, there is not enough room between the log columns, and the "mm" label may be omitted (50[blows]/125[mm] being written as 50/125).

### **Key to Boring Logs** Project: Summerlin HOV Flyover Location: Las Vegas, Nevada Project Number: 0324-01-7 Plate Number:  $B-2$

## **APPENDIX C**

## **LABORATORY TEST RESULTS**

# **C-1**

# **INDEX TEST RESULTS**



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLDIC** 



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT ട്ട -<br>201 していてい **NGLDIC** 



LAB.GDT S -<br>201 していてい **NGLDIC** 급스



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT ട്ട -<br>201 していてい **NGLDIC** 급스





LAB.GDT S -<br>201 していてい **NGLDIC** 



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLDIC** 













US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLDIC** 



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLE** 







LAB.GDT ട്ട -<br>201 していてい **NGLE** 



LAB.GDT ട്ട -<br>201 していてい **NGLE** 




US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLDIC** 



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT ട്ട -<br>201 していてい **NGLDIC** SIZE



US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT S -<br>201 していてい **NGLDIC** 





LAB.GDT S -<br>201 していてい **NGLE** 







US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT ട്ട -<br>201 していてい **NGLE** 



 $\frac{2}{1}$ GDT  $\overline{AB}$ ട്ട -<br>201 していてい **NGLE** 



LAB.GDT S -<br>201 していてい **NGLE** 





US\_GRAIN\_SIZE2\_METRIC 0324017.GPJ US\_LAB.GDT 12/12/2007LAB.GDT  $\frac{a}{2}$ 0324017.GPJ VETRIC **GI7E7** 

















## **C-2**

### **STRENGTH TEST RESULTS**



 $M =$ Modified California Sampler

























## **C-3**

### **CHEMICAL TEST RESULTS**

# **Laboratory Report**





Comments: Sulfate & Chloride run from a 1:10 extract.

Lance Bell, Lab Manager
## **Laboratory Report**

Black Eagle Consulting<br>1345 Capital Blvd, Suite A Reno, NV 89502 Attn: Ron Weber

EPA Lab ID: **NV004** Received: 11/20/01 Lab Sample ID: 5-111-124 12/04/01 Reported:

Phone: 359-6600 Fax: 359-7766





Comments:

Sulfate & Chloride run from a 1:10 extract.

Lance Bell, Lab Manager

### **Laboratory Report**

Black Eagle Consulting<br>1345 Capital Blvd, Suite A Reno, NV 89502 Ron Weber Attn:

EPA Lab ID: Received: Lab Sample ID: Reported:

**NV004** 11/20/01 5-111-124 12/04/01

Phone: 359-6600 Fax: 359-7766





#### US 95/RAINBOW BORE H-A @ 3.5-5 Boring B-12 (not included in this report)



Comments:

Sulfate & Chloride run from a 1:10 extract.

Lance Bell, Lab Manager

**Calculations** Appendix III

# **Laboratory Report**







**Calculations** 

Comments:

Sulfate & Chloride run from a 1:10 extract.

Lance Bell, Lab Manager

#### **Western Environmental Testing Laboratory Analytical Report**

**Black Eagle Consulting** 1345 Capital Boulevard, Suite A Reno, NV 89502-7140 Attn: Gary Bomberger Phone: (775) 359-6600 Fax: (775) 359-7766 PO\Project: Summerlin HDV / 0324-01-7

5/29/2007 Date Printed: 0705087 OrderID:

Collect Date/Time: 4/6/2007

Receive Date: 5/8/2007 16:40

**Customer Sample ID:** FB-03 A 5 **WETLAB Sample ID:** 0705087-001



**Customer Sample ID:** FB-06 B 10 **WETLAB** Sample ID: 0705087-002 Collect Date/Time: 4/10/2007 Receive Date: 5/8/2007 16:40



### **Western Environmental Testing Laboratory Analytical Report**

**Black Eagle Consulting** 1345 Capital Boulevard, Suite A Reno, NV 89502-7140 Attn: Gary Bomberger **Phone:** (775) 359-6600 Fax:  $(775)$  359-7766 PO\Project: Summerlin/0324-01-7

10/31/2007 Date Printed: 0710235 OrderID:



FB-09 Bulk 30-35' **Customer Sample ID:** 

WETLAB Sample ID: 0710235-002

Receive Date: 10/24/2007 15:15



**Customer Sample ID:** 0710235-003 **WETLAB** Sample ID:

FB-10 Bulk 10-15'

Collect Date/Time: 10/19/2007 Receive Date: 10/24/2007 15:15

