KABATA KNIK ARM CROSSING DOT&PF

KNIK ARM CROSSING GEOTECHNICAL MEMORANDUM

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ABBREVIATIONS & ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ADOT&PF	Alaska Department of Transportation and Public Facilities
AEIC	Alaska Earthquake Information Center
API	American Petroleum Institute
BMP	best management practices
bpf	blows per foot
CMP	corrugated metal pipe
CPT	Cone Penetrometer Test
ft/sec	foot per second
HLA	Harding Lawson Associates
IBC	International Building Code
KAC	Knik Arm Crossing
km	kilometers
ksf	kips per square foot
lb	pound
LRFD	load and resistance factor
MCE	maximum considered earthquake
MHW	Mean High Water Mark
MLLW	Mean Lower Low Water
mm/yr	millimeters per year
MOA	Municipality of Anchorage
MSB	Matanuska-Susitna Borough
NEHRP	National Earthquake Hazard Reduction Program
pcf	pounds per cubic foot
PDA	Pile Dynamic Analysis
POA	Port of Anchorage
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
RP2A	American Petroleum Institute Recommended Practices 2A
SPT	Standard Penetration Test
tsf	tons per square foot
UBC	Uniform Building Code
UHS	uniform hazard spectrum
USGS	U.S. Geological Survey
WCC	Woodward-Clyde Consultants

EXECUTIVE SUMMARY

This technical memorandum presents the results of a review and evaluation of available geotechnical data in support of engineering studies for the proposed Knik Arm Crossing (KAC) project, near Anchorage, Alaska. The purpose of this review was to provide baseline foundation recommendations along the main possible bridge approach corridors on the Anchorage side and the two on the Matanuska-Susitna Borough side to aid the team designers in evaluating the environmental impacts associated with constructing the main proposed KAC bridge, the adjoining highways, and other support structures. For the purpose of this technical memorandum, data were compiled from many Shannon & Wilson, Inc., studies and a few studies by others in areas that the corridors pass through. The boring location maps, site plans, and representative boring logs near the corridor alignments that were selected from the study reviews are presented in Appendix A.

Shannon & Wilson's geotechnical review effort is broken into two major components: 1) the proposed bridge, and 2) the proposed onshore roads and other support structures needed to interconnect the bridge to the existing road system. The portion of the memorandum addressing the crossing borrowed heavily from the 2004 "Preliminary Geotechnical Report, Knik Arm Bridge Project, Anchorage, Alaska" (Shannon & Wilson 2004). Analyses for the proposed bridge indicated that driving 8- to 10-foot-diameter, high-capacity pipe piles into the hard or dense glacial deposits is one of the better ways of developing deep foundations for the widely spaced piers. Although world-class derrick barges and hydraulic hammers would be needed to handle and drive the piles, the number of piles per pier would be one-third to one-quarter of the number needed to be installed if using medium-size piles (4-foot diameter). Additionally, the field construction time to install the fewer piles would be greatly compressed. The pile cap for the fewer piles could also be much smaller, which should result in smaller lateral forces on the piers in the intertidal zone.

To achieve allowable pile capacities in the range of 15,000 to 20,000 kips per pile, 8-foot-diameter piles approaching 200-300 feet long with a wall thickness in the 2.5-inch range would be needed. In addition to the specialty driving equipment; these construction challenges characterize the bridge foundation work: splicing the long piles with thick walls, achieving suitable penetration (hard driving) in the glacial soils, penetrating possible boulders, and drilling out soil in the piles. Such considerations are unique in this harsh environment of large tides, strong currents and winds, sea ice, cold winters, and water with poor visibility, and would exist even if smaller pile diameters were to be selected. A test pile program is thus recommended as part of the design process. This program would allow contractors to gain confidence concerning the labor and difficulties associated with installing the piles.

A weak link in the understanding of the conditions at the proposed bridge crossing site is the lack of deep boring data beneath the east half of the channel. The results of 2004 explorations indicated that the borings were not deep enough to determine

whether the soils at depth are sands and gravels (till-like soils) or clays (Bootlegger Cove Formation clays). This lack of understanding greatly affects a reliable determination of pile lengths that would be needed in this area. The pile lengths over this east area could be 50 or 100 feet shorter if the granular till soils that dominate this region are found to be present and the clays are thin or absent. As described in this report, additional geotechnical studies would be needed in both the over-water areas and onshore for final design.

To connect the bridge to the existing road systems, a significant component of the project would be the land/shoreline construction work that would involve a possible cut-and-cover or bored tunnel through Government Hill; four-lane roads through former landslide debris as mapped by United State Geologic Survey (USGS); and shoreline embankments with toe buttresses, riprap faces, and special features for slope drainage control. Additionally, the proposed bridge abutments would penetrate or merge with high steep bluffs on the east and west shorelines and must be treated to address the natural erosion process. The locations of the various features and recommended soil parameters for sizing each structure are presented in the text and figures of this Technical Report.

The analyses of geotechnical issues presented in this document incorporate elements of baseline conditions, environmental impacts, and mitigation measures in the form of appropriate investigation and design recommendations. A summary of the environmental consequences of the proposed project on geology and soils as a resource, as well as the impacts of geotechnical issues and seismic hazards on the project, is provided at the end of this report.

1.0 Introduction

This Technical Report provides documentation of the geotechnical conditions in the Matanuska-Susitna Borough (Mat-Su) and the Municipality of Anchorage (Anchorage) that would be affected by the proposed Knik Arm Crossing (KAC) project. The Study Area and the alternatives forwarded in the Draft Environmental Impact Statement (EIS) are described below.

The Study Area covered by this document extends primarily along the proposed bridge and road corridors for the purpose of analyzing the direct effects of geotechnical issues on the proposed project. Where pertinent to the analysis of indirect effects, the Study Area was expanded to include areas of the Mat-Su Valley bounded roughly by the Little Susitna River, Parks Highway, Knik Arm, and Cook Inlet.

This Technical Report summarizes the subsurface conditions along a proposed hybrid cross section of Knik Arm and provides guideline foundation recommendations for understanding the difficulties associated with constructing a proposed highway bridge across the approximately 12,500-foot width of the Knik Arm waterway. The work also encompasses the proposed approaches, new highway, and other structures to connect with the existing road system. The main bridge alignment crosses the Knik Arm in an east-west direction about 7,400 feet north of the Port MacKenzie Dock.

This document is organized as follows:

- Section 3.0 contains a description of the overall methodology used in compiling this technical document.
- Section 4.0 contains general information on the affected environment of geology, soils, and seismic hazards.
- Sections 5.0 through 7.0 contain site-specific geotechnical information on each segment of the proposed project, as well as discussions of the effects of soil properties on project design. These analyses incorporate elements of both the affected environment and the projected impacts, which would lead to mitigation measures in the form of design recommendations.
- Section 8.0 summarizes the projected environmental effects of the proposed project on geology and soils, as well as the potential impacts of geotechnical issues and seismic hazards on the proposed project.

2.0 Project Description

More than 80 years of transportation, land use, and economic plans and studies for the Upper Cook Inlet region of Alaska have addressed the need for a Knik Arm crossing project to connect Anchorage with the Mat-Su.

In 2003, the Alaska State Legislature established the Knik Arm Bridge and Toll Authority (KABATA) as a public corporation and an instrumentality of the State of Alaska within the Alaska Department of Transportation and Public Facilities

(ADOT&PF). The specific mission of KABATA is to "… develop, stimulate, and advance the economic welfare of the state and further the development of public transportation systems in the vicinity of the Upper Cook Inlet with construction of a bridge to span Knik Arm and connect the Municipality of Anchorage and the Matanuska-Susitna Borough" (Alaska Statutes chapter 19.75).

In accordance with this mission, the purpose of the proposed KAC project would be to provide improved access and connectivity between Anchorage and the Mat-Su through an efficient and financially feasible crossing of Knik Arm, including adequate connections to the committed roadway network on both sides of Knik Arm. A Knik Arm crossing would:

- improve regional transportation infrastructure to meet existing and projected population growth in Upper Cook Inlet
- enhance the movement of people, freight, and goods between Anchorage, the Mat-Su, and Interior Alaska
- offer safe, alternative connections between regional airports, ports, hospitals, and fire, police, and disaster relief services for emergency response and evacuation

The length of the proposed bridge crossing of Knik Arm would be approximately 2.5 miles and located approximately 1.25 miles north of Cairn Point (see Figure 1.2). The roadway connection on the Mat-Su side of Knik Arm would be Point MacKenzie Road near the Port MacKenzie District. The roadway connections on the Anchorage side of Knik Arm would be the A-C and Ingra-Gambell Couplets, generally in the Port of Anchorage (POA)/Government Hill/Ship Creek area. The total length of the project from the intersection of Point MacKenzie and Burma Roads to the intersections of the A-C and Ingra-Gambell Couplets with Third Avenue would be approximately 19 miles.

The proposed project would be a controlled access toll facility with a toll plaza located in the Mat-Su near the western bluff of Knik Arm. The proposed project would be classified as a rural principal arterial in the Mat-Su and across Knik Arm, transitioning to an urban principal arterial in Anchorage in the vicinity of the POA. The proposed project would be phase-constructed as travel demand warrants and would be anticipated to generally be an initial two-lane facility with expansion to a four-lane facility by 2030, the design year. Initial construction would include a connection to the existing A-C Couplet

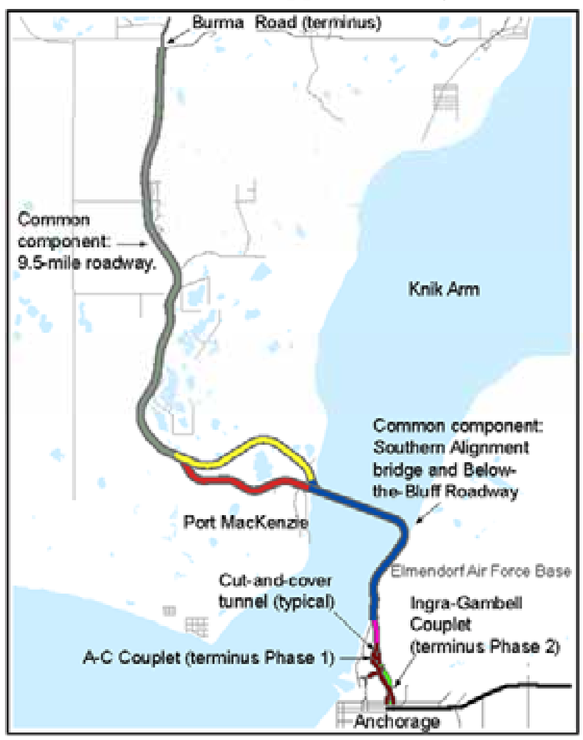

Figure 1. shows that the proposed project begins at Burma Road and ends in Downtown Anchorage. Components common to all routes being considered are also identified.

and, by approximately 2022 to 2025, a connection to a new viaduct (elevated bridge) across the Ship Creek rail yard. The viaduct would be constructed to connect with the Ingra-Gambell Couplet.

Right-of-way (ROW) widths for the project would vary by specific design element. The proposed project right-of-way in the Mat-Su would be approximately 400 to 450 feet in width. In the Anchorage portion of the proposed project, the right-of-way would be approximately 260 feet along the east shore of Knik Arm down to the future expansion of the POA, then vary from 200 to 350 feet as it passed behind the port. Then, as it climbed Government Hill, the right-of-way would expand to 985 or 585 feet wide to accommodate a cut-and-cover tunnel and access points along either a Degan Street- or Erickson Street-area alignment, respectively. Continuing southward it would cross the Ship Creek rail yard along an approximately 80-foot-wide, pier-supported viaduct ending at Third Avenue, the proposed project terminus.

The Federal Highway Administration (FHWA) is preparing an EIS as part of the National Environmental Policy Act (NEPA) process to evaluate a Knik Arm crossing sponsored by the Knik Arm Bridge and Toll Authority (KABATA).

2.1 Description of the Proposed KAC Project Study Area

The Study Area for the proposed KAC project is located within the boundaries of Anchorage and the Mat-Su in the Upper Cook Inlet region of Southcentral Alaska (Figure 1.2). The Study Area has a combined population of nearly 350,000, which represents over 50 percent of Alaska's total population. The Anchorage and Mat-Su portions of the Study Area are separated from one another by Knik Arm, a 30-mile-long waterway, which varies in width from 2 to 6 miles. Anchorage is located approximately 3 miles from Port MacKenzie and the adjacent Mat-Industrial district.

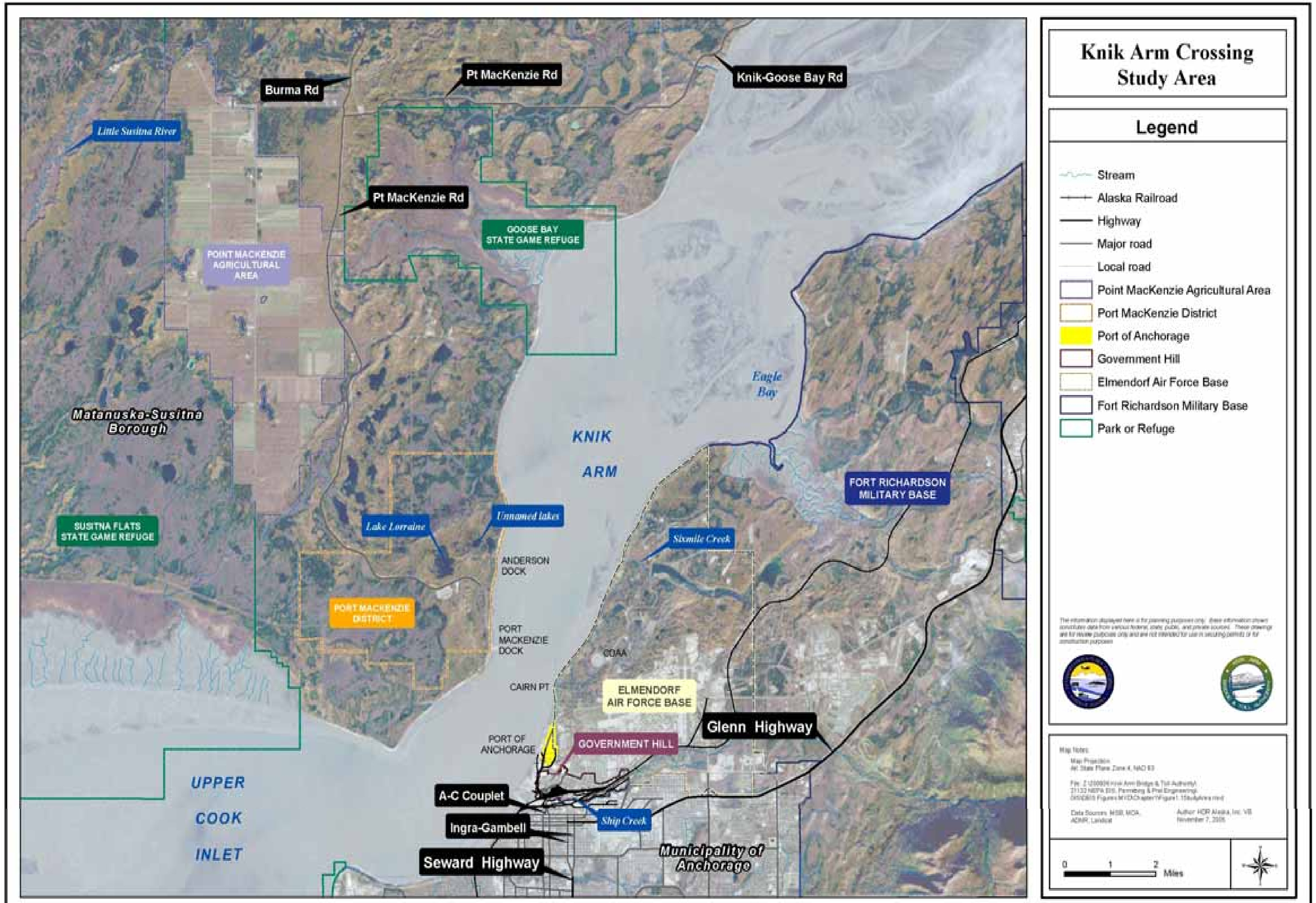

Figure 2. KAC Draft EIS Study Area. The Study Area covered by this document extends primarily along the proposed bridge and road corridors for the purpose of analyzing the direct effects of geotechnical issues on the proposed project. Where pertinent to the analysis of indirect effects, the Study Area was expanded to include areas of the Mat-Su Valley bounded roughly by the Little Susitna River, Parks Highway, Knik Arm, and Cook Inlet.

Although the physical separation between these two areas consists of a short span of waterway, the only current surface transportation access between Anchorage and the Port MacKenzie District (port district) is by 80 miles of existing roadway around the head of Knik Arm.

Located along the eastern shore of Knik Arm, Anchorage encompasses 1,961 square miles, 84 percent of which is occupied by National Forest, State Parklands, and tidelands. With an additional 6 percent occupied by military reservations, only about 10 percent of the entire municipality is inhabited and available to accommodate existing and future growth. Most residents of Anchorage live in the Anchorage Bowl, the most urbanized portion of the municipality. The Anchorage Bowl occupies approximately 112 square miles and is bounded by Chugach State Park, Knik and Turnagain Arms, Elmendorf, and Fort Richardson Military Base (Fort Richardson). Anchorage residents outside the Anchorage Bowl live either further north in the suburban communities of Chugiak-Eagle River or in small residential areas along the Glenn Highway and Turnagain Arm. Also located within this portion of the Study Area are the POA—a vital intermodal facility—and the adjacent Ship Creek industrial area.

On the western shore of Knik Arm, the Mat-Su consists of an area of 24,683 square miles, which encompasses approximately 23 percent of all private land in the state of Alaska. Because there is a substantial amount of undeveloped land available in the Mat-Su, the area provides an alternative to more costly and limited residential, commercial, and industrial lands within Anchorage. This availability has resulted in numerous changes that have recently occurred or will be occurring in the Mat-Su, including construction of Port MacKenzie in the late 1990s, existing and planned expansion of the connecting transportation network to and from Port MacKenzie, and planned development of the 9,000-acre port district. The Mat-Su Borough is also developing a ferry link between Port MacKenzie and the POA; the ferry is projected to begin operation in 2008.

2.2 Alternatives

The proposed KAC project would begin at the intersection of Point MacKenzie and Burma Roads and follow the existing roadway alignment south to the western boundary of the port district. From here, there would be two alternative routes for getting to the proposed bridge crossing. The proposed Point MacKenzie Road Alternative would use the existing Point MacKenzie Road most of the way through the port district before deviating from the established road and heading toward the bridge crossing near the western bluff. The proposed Northern Access Alternative would skirt the core port area on the north side on a new alignment. With either proposed alternative, there would be a toll plaza and intersection/access road to allow access to and from Port MacKenzie.

The proposed crossing itself would measure approximately 2.5 miles, bluff-to-bluff, across Knik Arm. The proposed bridge would begin approximately 1,500 feet south of

Anderson Dock on the Mat-Su side and end 1.25 miles north of Cairn Point on the Anchorage side.

From the eastern bridge abutment, the proposed Anchorage approach road would travel southwest on fill along the tidelands and below the bluff, toward Cairn Point, then turn southward, closely following the natural curve of the shoreline, from this point the proposed roadway would climb to and parallel the eastern boundary of the POA where the route would connect to the A-C Viaduct and the proposed Ingra-Gambell Viaduct by way of either of two routes: the Erickson Alternative or the Degan Alternative.

The proposed Degan Alternative would follow the alignment of Degan Street through a cut-and-cover tunnel that would initially connect to East Loop Road with an at-grade, T-intersection (Phase 1). As travel demand would warrant, the route would continue on the proposed new Ingra-Gambell Viaduct over the Ship Creek rail yard before tying into the Ingra-Gambell Couplet at 3rd Avenue. At that time, Loop Road would be elevated over the proposed KAC route to provide access to Government Hill and Elmendorf. The proposed Erickson Alternative would be similar, but the cut-and-cover tunnel would align with Erickson Street and connect directly into Loop Road in Phase 1 (ramps would continue to provide access to Government Hill and Elmendorf). When travel demand would warrant, or Phase 2, the route would continue in a parallel cut-and-cover tunnel under Erickson Street onto the proposed Ingra-Gambell Viaduct, tying into the Ingra-Gambell Couplet at 3rd Avenue.

2.3 Preferred Alternative

FHWA screened the range of alternatives against criteria for purpose and need and technical criteria to identify reasonable alternatives for detailed study in the Draft EIS. Based on these screening criteria and subsequent detailed evaluations, FHWA has identified a Preferred Alternative.

The preferred approach route to the proposed Knik Arm Bridge on the Mat-Su side is Point MacKenzie Road from the intersection with Burma Road south to the Port MacKenzie District and connecting to the Northern Access Alternative through the port district. FHWA chose this route because it would avoid wetlands, would not impact Port MacKenzie operations, and is favored by Mat-Su Borough and Port MacKenzie officials.

The proposed Southern Alignment is the preferred route for the bridge to cross Knik Arm. The Southern Alignment, with its accompanying Below-the-Bluff Roadway on the Anchorage approach, would be the most technically feasible and practical alignment that would avoid the Cairn Point Trench (a submarine trough), would not impact military mission and operations at Elmendorf, and would minimize potential impacts to beluga whales that congregate in areas of Knik Arm further to the north.

An 8,200-foot-long pier-supported bridge is preferred over a 14,000-foot-long, pier-supported bridge because a shorter bridge would require fewer piers, result in less construction noise and pile driving impacts that might adversely affect beluga whales and marine fishes, would require shorter in-water construction time, and would have substantially lower construction costs.

The preferred Anchorage approach to the proposed bridge would be a cut-and-cover tunnel under Government Hill, along either of the proposed Degan or Erickson Street alignments, to connect initially to the A-C Couplet, and ultimately to the Ingra-Gambell Couplet.

All reasonable alternatives evaluated in the Draft EIS are under consideration and have been developed to a comparable level of detail. Final identification of a Recommended Alternative will not occur until the alternatives, impacts, written comments on the Draft EIS, and comments received at the public hearings have been fully evaluated and considered. The Recommended Alternative will be provided in the Final EIS.

3.0 Methodology

Data compilation for this technical memorandum included a review of 1984 and 2004 project information and available new data from more recently completed geotechnical and seismic studies for design of nearby projects (see references at end of this document). From this new information, more recent construction experience on adjacent projects, and changes in local codes, key preliminary design parameters are to be developed. The information is intended not to determine what different foundations are feasible, but rather to focus on what is believed to be the most likely future foundation system and seismic design criteria for the proposed alignments. This information will assist the team in developing preliminary design sections that can support a rational construction cost estimate. This study also presents results of new field exploratory work to supplement existing information.

4.0 Regional Geology

4.1 Physiographic Setting

4.1.1 Onshore Physiography

Knik Arm lies in the Cook Inlet-Susitna Lowland physiographic province. Numerous lakes, ponds, and wetlands associated with glacial tills and outwash deposits are found throughout this gently sloping lowland area. Knik Arm was buried by ice and flooded by proglacial lakes several times during the Pleistocene. The lowlands are fed by multiple drainages that originate in the Alaska Range and the Talkeetna and Chugach mountains. Several of these drainages, including the Susitna, Matanuska, and Knik Rivers, are large, glacially fed rivers with heavy sediment loads that braid across valley bottoms and coastal flats (Nowacki et al. 2002; Wahrhaftig 1965).

The shorelines of Knik Arm are characterized by large mud flats in the intertidal zones and 50- to 150-foot-high bluffs. These bluffs are subject to erosion from wind, runoff, slope failure, and exposure to tides and waves, creating a secondary source of sediment into Cook Inlet. The prominence of Cairn Point on the eastern shore marks the southwestern extent of Elmendorf Moraine, which is an end moraine of the combined Matanuska and Knik glaciers that advanced during the Naptowne Ice Age, beginning about 30,000 years ago. This major geomorphic feature extends across Knik Arm from Cairn Point to the Susitna Lowlands. The Elmendorf Moraine was breached by rapid downcutting of outwash streams during a period of lowered sea level that marked the end of the Pleistocene. The waters of Cook Inlet then rose during the Holocene in response to a worldwide sea level increase, as melting glaciers flooded the valley of the Knik-Matanuska River system, creating modern-day Knik Arm. During this period, isostatic rebound from melting glaciers raised overall land levels, adding relief to the Knik Arm bluffs (Reger et al. 1995).

4.1.2 Submerged Landforms

Landforms beneath Knik Arm near the proposed KAC bridge consist of shallow tidal flats and gently sloping, hummocky benches that extend to about 3,000 to 5,000 feet from shore, and to depths of about 20 to 30 feet below mean lower low water (MLLW) (Figure 3). The deepest part of the channel in the middle of Knik Arm is about 4,000 feet wide and extends to depths of about 60 to 65 feet MLLW. The channel is steep-sided to the west and more gently sloping to the east.

Figure 3. Knik Arm crossing bathymetry

Between about 0.3 to 3 miles south of the proposed Knik Arm Crossing Alignment, near Cairn Point, the Knik Arm channel deepens to about -180 feet MLLW along an isolated northeast-trending steep-sided depression (Harding-Lawson Associates 1983; HDR and URS 2005; Smith 2004; URS 2005), a feature often referred to as the "big hole" or the Cairn Point Trench. This feature may be an erosional remnant from a stream canyon during the late Pleistocene-early Holocene lowered sea level stand. At its lowest point during the last glacial maximum, sea level reached a depth of about 300 feet below its current level, at which point the coastline was near the mouth of lower Cook Inlet.

4.2 Geologic Units

Cook Inlet is located in a structural trough overlying Tertiary rock formations and surrounded by Quaternary deposits of varying densities. These rocks consist of interbedded shale and sandstones with coal beds. They are exposed closest to the Study Area along Eagle River, west of the Border Ranges fault, and more extensively in the Matanuska Valley and on the Kenai Peninsula (Magoon et al. 1976; Winkler 1992). Geophysical surveys across Knik Arm and oil exploration wells drilled in the area indicate that bedrock is likely deeper than 600 to 1,000 feet in the project vicinity (HLA 1983; Golder 2003; Shannon & Wilson 2004).

The existing topography of the surrounding area is the result of numerous glacial periods. Knik Arm has endured at least five glacial events in the last 2 to 3 million years (Karlstrom 1964). The most recent events include the Knik Glaciation and the Naptowne Glaciation, both of which occurred within the past 75,000 years.

During the Knik Glaciation (30,000 to 75,000 years ago), thick sequences of sediment, known as the Knik Ground Moraine, were deposited as glaciers retreated. Within the Study Area, these deposits extend from Eagle River valley to Point MacKenzie and Point Woronzof, and lie mostly below sea level. The deposits generally consist of poorly sorted till sediment deposited directly by glacial ice (Karlstrom 1964; Mat-Su Borough 1995).

The Naptowne Glaciation (11,000 to 30,000 years ago) is responsible for the majority of glacial deposits currently encountered in the Anchorage area (Figure 4) and across the Knik Arm near Port MacKenzie. At its maximum, the Naptowne Glaciation extended across the Anchorage Bowl area from the north and terminated at Point Woronzof and Point Campbell (Reger and Pinney 1997). The Bootlegger Cove Formation was formed during this time in ice-free areas of the Susitna River valley, lower Knik Arm, and Upper Cook Inlet (Reger et al. 1995). Bootlegger Cove sediments generally consist of estuarine, marine, and lacustrine clays and silts with lesser amounts of sand and scattered pebbles and cobbles (Schmoll et al. 1984; Updike and Carpenter 1986). Prior to and concurrent with the deposition of the Bootlegger Cove clays, material was being shed out of the uplifting Chugach Mountains through alluvial processes (Hamilton 1994), causing wedges of sand and gravel to interfinger with and underlie the clay in many areas of the Anchorage Bowl. A sensitive silty clay unit within the upper half of the Bootlegger Cove Formation was responsible for major translational landsliding that occurred in Anchorage during the 1964 earthquake. The northern extent of the sensitive clay unit is poorly known (Updike et al. 1988).

Figure 4. Quaternary geologic map, Anchorage area

Overlying the Bootlegger Cove Clay Formation are sand and gravel glacial deposits, including the Naptowne Outwash and the Elmendorf Moraine. Approximately 14,000 years ago, the Elmendorf Moraine was formed at the terminus of the Knik-Matanuska glacial lobe and is now a prominent topographic feature on both sides of Knik Arm. The Elmendorf Moraine consists of a wide variety of poorly sorted sediments and can be seen stretching across Elmendorf Air Force Base from about Cairn Point northeast to the town of Eagle River. On the west side of Knik Arm, the Elmendorf Moraine can be seen from Port MacKenzie arcing north towards Wasilla and Big Lake. The retreat of ice following deposition of Elmendorf Moraine left behind ground moraine, kame fields, kame terraces, and abandoned channels on both sides of Knik Arm (Karlstrom 1964; Reger et al. 1995). As described above, a portion of the moraine has been eroded by the tidal influxes of the Knik Arm, which formed in response to a worldwide rise in sea level because of retreating glaciers.

The Naptowne Outwash is a flat sprawling apron of glaciofluvial sediment that overlies much of the Bootlegger Cove Formation on both the east and west sides of Knik Arm. This material was deposited by large, braided stream channels that contained sand and gravel and flowed from the Knik-Matanuska glacier. These sediments were subject to constant reworking by the glacial runoff and consist of a variety of sorted sediment that has been deposited in front of the Elmendorf Moraine. The distribution of coarse-grained alluvium of the Naptowne Outwash across the Anchorage area is shown on Figure 4. Locally, this outwash has been named the Mountain View Fan, and underlies parts of Government Hill, Mountain View, and Downtown Anchorage.

Numerous landslide and colluvial deposits occur along the Knik Arm and Ship Creek bluffs (Figure 4). These have been mapped and described on the basis of relative age by Updike and Carpenter (1986). Older slides are characterized by hummocky surfaces, obscure slide toes grading to tidal deposits, heavy vegetation, and continuous soil cover. Younger slides exhibit ridge-and-trough topography, ponded surface water, recent vegetation, and discontinuous soils; they generally occur directly downslope from identifiable scarps and overlie older slide material. Several prominent slides occurred during the 1964 earthquake in the Study Area as a result of translational failures along sensitive clays of the Bootlegger Cove Formation. Other types of Quaternary deposits in the Study Area include Holocene alluvium and alluvial terraces in modern stream drainages, and artificial fill.

4.3 Knik Arm Soils

Knik Arm contains several types of surficial sediment distinguished by transport processes and elevation. These include upper and lower active tidelands, dry mud flats, and small drainage channels (Bartsch-Winkler 1982; Colonell and Jones 1990). Dry mud flats are often covered with moss or grass and crossed by more permanent drainage channels. A surficial deposit of gravel and boulders lies along the eastern shore of the proposed Knik Arm Crossing, apparently associated with active erosion of morainal till in the eastern bluffs (Golder Associates 2003; Shannon & Wilson 2004). Lower tidelands consist primarily of sand with occasional gravel deposited under fast tidal current conditions, while upper tidelands or mud flats consist of finer sediment such as sandy silt and clay deposited by settlement and flocculation during high tides. In the vicinity of the proposed Knik Arm Crossing and the northern part of the proposed Below-the-Bluff Roadway, there is little to no settlement of fines occurring, because of currents and turbulence that keep finer sediment in suspension in this narrow part of Knik Arm (Smith 2004). Recent tidelands deposits are also thin to absent south of Cairn Point, where geotechnical borings have encountered mostly stiff to hard clays of the Bootlegger Cove Formation at or near the surface (Shannon & Wilson 2004). At Port MacKenzie dock, however, suspended sediment is being deposited on both the north and south sides of the structure, as both ebb and flood tidal currents slow around the dock (Aeromap 2001, 2002).

The subsurface soils across the channel and in the bluffs are of glacial or marine origin and, except for near-surface deposits in the main channel bottom, are generally dense to very dense or very stiff to hard. The glacial geology in this area appears to be complex and has developed as a result of a number of ice advances and retreats during the Knik and Naptowne Glaciations, resulting in scouring and redeposition as tills in both glacial lake and marine environments, and consolidation of deposits from glacier overriding. Surface exposures indicate that these depositional characteristics are not only present below the waters of Knik Arm and the mud flats, but also exist in the steep bluffs on both sides of the channel.

In general, the material beneath and on the sides of the channel consists of four basic geologic units; recent channel marine deposits, glacial tills or moraine deposits, glacial lake clays or marine/alluvial sands, and possible Knik tills (Section 5.2). The recent channel marine deposits consist of silty to clean fine sands as thick as 40 feet. This unit is somewhat mobile and probably tends to shift over time with the changing currents and tides. The glacial till or moraine deposits are exposed on both sides of the channel in the bluffs, but appear to have been eroded away in the center of the channel. This glacial till or moraine unit is characterized as both a gravelly clay and sand because of its variability in particle sizes. Its lack of apparent bedding or well-defined structure is consistent with expected till deposits, and its thickness is variable up and down the channel. The next unit consists of glacial lake clays and marine/alluvial sands and is probably the most dominant geologic unit beneath the channel. Beneath this unit lies the possible Knik till material, which is classified as gravelly, sandy clay with gravelly and silty clay zones. This unit probably grades to sandy and gravelly soils and lies atop bedrock, which is considered to be at depths of greater than 600 feet below the channel (Golder 2003).

4.4 Surface Soils

Surface soil types mapped by the Natural Resources Conservation Service (NRCS) in the vicinity of the proposed project are depicted on Figure 5. Soils along the northern part of the Mat-Su approach alternatives consist primarily of Kashwitna series silt loam on glaciofluvial outwash plains and hills. These soils are typically well-drained and consist of up to 1.5 feet of silt loam over gravelly sand. Associated with the Point

MacKenzie Agricultural Area, current uses of these soils include cropland, hayland, pastureland, homesites, and wildlife habitat (NRCS 1998).

Figure 5. Surface soils map

Soils along the southern part of the Mat-Su approach alternatives consist of a mixture of well drained Chilligan, Estelle, and Kichatna-Delyndia series soils; and poorly drained Cryaquepts, Histosols, and Disappoint series soils, all of which were derived from the glaciofluvial and glaciolacustrine plains, hills, and depressions left behind by the Naptowne Glaciation. Current use of these areas is primarily for wildlife habitat, with minor homesite use in the more well-drained soil zones. In addition, Kichatna-Delyndia soils are locally used as a sand and gravel source. Chilligan soils are typically composed of about 2 feet of sandy silt loam over stratified sand, silt, and clay loam. Estelle soils typically consist of about 1.3 feet of sandy silt loam over gravelly loam, and Kichatna-Delyndia soils of 0.3 to 0.8 foot of sandy silt loam over sand and gravelly sand. Cryaquepts and Disappoint soils typically consist of about 0.3 foot of wet mucky silt loam over gravelly sandy to cobbly silt loam. Histosols consist of up to 5 feet of peat and mucky peat (NRCS 1998).

Soils along the face of the west bluff of Knik Arm are referred to as Cryods and Cryochrepts. These consist of very well-drained silt loam over gravelly sand and gravelly to cobbly loam. Soils mapped along the east bluffs of Knik Arm are composed of well drained Deception-Estelle-Kichatna and Kashwitna-Kichatna complex silt loams derived from gravelly till and outwash deposits (NRCS 1998, 2001).

The soils of Government Hill and Ship Creek terraces are classified as Cryorthents and urban land. Cryorthents typically consist of up to 5 feet of very well-drained gravelly sandy loam derived from glacial outwash. Soils along the steep slopes of Ship Creek are composed of Smithfha very fine sandy loam derived from eolian deposits, as well as the Kashwitna-Kichatna complex described above. Soils in Ship Creek bottom consist of poorly to moderately well-drained Moose River-Niklason complex derived from alluvium parent material. These are typically composed of silt loam and loamy sand to depths of 2 to 4 feet, over gravelly sand (NRCS 2001).

4.5 Tectonics and Seismic Hazards

The Upper Cook Inlet region is one of the most seismically active areas in the United States and has been historically subjected to large earthquakes. Alaska experiences approximately 24,000 earthquakes per year, which accounts for 52 percent of all the earthquakes in the United States (AEIC 2005). Alaska is by far the most seismically active state in the United States and exhibits more seismicity than any other region in North America.

The tectonics and seismicity of southern Alaska are the result of ongoing relative motion between two lithospheric plates; the Pacific plate moves about 5 to 6 centimeters per year northwestward relative to the North American Plate. The margin of convergence between the plates is the subduction zone and is marked on the surface by the Aleutian trench, southeast of Anchorage. Active seismicity in southcentral Alaska occurs as both deep earthquakes associated with the subduction zone, as well as shallow earthquakes associated with long linear transform faults and smaller fault-cored fold structures (Figure 6).

Figure 6. Potentially active near-surface faults and fault-cored folds

Earthquakes cause ground shaking that can result in ground failure and structural damage or loss. Further discussion of ground shaking and its potential effects on the proposed project are presented in a separate document, Knik Arm Crossing *Seismic Studies* Technical Memorandum by HDR and PND (2006).

4.5.1 Ground Failure and Liquifaction

Ground failure in the event of a major earthquake can take several forms, such as liquefaction, surface cracking, land spreading, landsliding, and subsidence. Winterhalder et al. (1979) and the Municipality of Anchorage (2001) mapped the relative potential for seismically induced ground failure in the Anchorage area based on a combination of the above causes and localized ground conditions, such as soil type and thickness, groundwater depth, distance to topographic lows, slope angle, and fault proximity (Figure 7). The proposed Below-the-Bluff Roadway and Anchorage alternatives cross areas of high to very high predicted ground failure, a result dominated by the effects of translational landsliding.

Figure 7. Earthquake-induced ground failure map, Anchorage area

Liquefaction generally occurs during earthquakes in loose, saturated sands and silty sands, because of the rapid buildup of pore water pressure and resulting significant loss of strength and bearing capacity. Clay deposits typically have little to no liquefaction potential. Section 5.7 describes the results of liquefaction analyses conducted on soil samples near the proposed Knik Arm Crossing alignment.

4.5.2 Surface Faults

No active surface faults are known to cross the proposed Knik Arm Crossing alternatives. The closest potentially active surface fault to the Study Area is the Little Susitna River Scarp fault, located about 7 miles west of the Mat-Su alignments (Figure 6). This fault is suspected of offsetting late Pleistocene deposits (Plafker et al. 1993).

Near-surface folding and reverse faulting of Tertiary and Quaternary strata are suspected to be actively occurring throughout the Cook Inlet basin as a result of the oblique compressional tectonics of the region (Haeussler et al. 2000). Many of these features form traps for oil and gas fields in the Cook Inlet basin. Several of the fault-cored folds along the west side of Upper Cook Inlet exhibit evidence of active deformation of the seafloor or ground surface in this area (for example, at Granite Point, North Cook Inlet, Ivan River, and Stump Lake fields, Figure 6). A similar fold was mapped along the west side of Knik Arm and crosses the proposed Mat-Su alternatives; it is referred to as the Lorraine-Alaska Gulf fold (Haeussler et al. 2000; Magoon et al. 1976). Little information is available on this structure. Based on its similarity to other well-studied features in the Cook Inlet basin, it is possible that it could represent the site of shallow, buried, active reverse faulting.

4.5.3 Transitional Landsliding

The bluffs along Knik Arm have been the sites of major catastrophic earthquake-induced landslides in the past. The ability of slopes to remain stable is continually altered by many geologic and man-made processes, such as direct erosion by downslope water drainage, loss of support by erosion at the base of bluffs by waves or currents, slow soil creep, debris or mud flows, oversteepening by construction activities, wind raveling of cohesionless soils, increase in groundwater content adding to slope mass or causing the material to lose strength, and ground shaking during earthquakes. Larger, more catastrophic slope failures are most likely to occur when triggered by earthquakes, particularly if increased precipitation and high groundwater conditions are present.

Both the west and east bluffs of Knik Arm are in a state of marginal stability, as erosion from tides and currents are slowly cutting away the toes and slopes at an estimated rate of about 1/2 foot per year (Shannon & Wilson 1971). Although the west bluff exhibits evidence of slumping and mud flows, it is not known whether the sensitive clay unit of Bootlegger Cove Formation extends to this area; thus, the likelihood of large-scale translational landsliding during an earthquake is unknown.

The steep bluffs bordering the east side of Knik Arm and Ship Creek have been mapped by Dobrovolny and Schmoll (1974) using stability categories based on soil type, slope angle, and history of slope movements. These bluffs are mostly mapped as having low or very low stability. In addition, the Knik Arm bluffs from about 0.5 mile south of the east abutment to Government Hill, as well as the Ship Creek bluffs, are mapped as having potentially large earthquake-triggered landslide risk because of the presence of the Bootlegger Cove sensitive clay unit in the lower part of the bluffs. Two landslides that occurred during the 1964 earthquake, the Elmendorf and Government Hill Elementary School slides, are mapped as having moderate stability because of the resulting lower angles of the failed slopes. Slope stability conditions for specific segments of the proposed project are further discussed in Section 5.0.

5.0 Subsurface Geotechnical Conditions

In this section, the anticipated soil conditions and design challenges along the main bridge crossing are briefly described and the expected conditions for the onshore alignments are discussed. The soils across the channel and in the bluffs are of glacial or marine origin and, except for near-surface deposits in the main channel bottom, are generally dense to very dense or very stiff to hard.

5.1 Mat-Su Soils

Helicopter reconnaissance studies by Shannon & Wilson in 1971 describe 125- to 150-foot-high steep slope exposures along the west bluff. These slopes consist of medium dense alluvium and dense, till-like sands and gravels in the top half and very stiff or hard clays or silts in the lower half with spring seepage at the interface. These deposits may be part of the Bootlegger Cove Formation. Materials below the base of the bluff, beginning at the mean high water mark (MHW), consist of clay and gravelly till of the Knik Glaciation (Mat-Su Borough 1995; Schmoll et al. 1984; Shannon & Wilson 2004). The bluff section is characterized by numerous shallow slump areas and mud flows in the fine-grained soils caused by the seepage over the face. To construct a highway in this bluff region, the seepage flow and toe erosion would need to be controlled and stabilized with a combination of slope-flattening, riprap protection in the intertidal areas, and cross-slope and/or finger drains.

Borings drilled west of the bluff on Elmendorf Moraine in the vicinity of the Mat-Su approach alternatives and potential material extraction sites are provided in Appendix A. Soils west of the bluff are predominantly glacially derived deposits overlain by organic material up to 10 feet thick. More typically, the organic layer should be less than 3 to 5 feet thick. The glacially derived deposits generally consist of dense to very dense, slightly silty, gravelly sand and sandy gravel; with lesser amounts of hard gravelly, sandy silt. These glacial soils have a low to moderate frost susceptibility, are generally free-draining, and should make a good foundation subgrade material for support of the proposed west side access highway. The alternatives would cut through several areas of moderate to steep side slopes where they cross Elmendorf Moraine. A large gravel pit is located near Port MacKenzie, and from gradation curves on material from this area, it is believed that suitable borrow should be available for road embankments and subbase materials. Groundwater is likely perched at shallow depths in the organic material and on local silt lenses and results in shallow ponds in low-lying areas.

NRCS (1998) ratings related to soil suitability for building (Figure 8) illustrate the effects of surface soils on potential future development in Mat-Su Valley. These ratings incorporate problems related to wetness, ponding, low strength, frost action, and slopes into categories ranging from slightly to severely limiting for building. Soils on the west side of Burma and Upper Point MacKenzie Roads, including those associated with the Point MacKenzie Agricultural Area, exhibit some of the best conditions for construction of homesites and small commercial buildings. Soils along the east side of Knik-Goose Bay Road have similar building suitability ratings. Soils in the remaining parts of the Study Area are rated as having moderate to severely limiting building suitability characteristics, largely because of the presence of wet hydric soils and low-strength peats and mucks.

Figure 8. Soil suitability for building

5.2 Knik Arm Crossing Soils (including Below-the-Bluff roadway)

5.2.1 Channel Soils

The subsurface conditions near the proposed water crossing (Figure 9) are characterized by profile A-A', as located on Figure 9 and detailed in Figure 10. Soil units were extrapolated into areas where conditions are not well-defined on this profile and represent assumed conditions that were needed to develop construction costs in these areas. Except possibly for minor length differences, the channel crossing profile is, thus, assumed to reasonably represent subsurface conditions at the section, and furthermore is the best over-water subsurface information currently available for this vicinity.

As summarized in Figure 10, four basic geologic units were penetrated with the deep borings by Shannon & Wilson, Inc., in 2003/2004 and are summarized in descending order as follows: recent channel marine deposits, glacial till or moraine deposits, glacial lake clays and marine/alluvial sands, and possible Knik tills.

Figure 9. Preliminary corridor concepts and bridge crossing

Figure 10. Channel Crossing Profile A-A'

5.2.1.1 Recent Channel Marine Deposits

Up to 40 feet of loose to medium dense, silty to clean, fine sands are present in the center of the main channel, as shown in Figure 10. Locally, these loose sand deposits appear to thin on the east side to less than 10 or 15 feet and are absent on the west side as water depths diminish. It is believed that these sands are recent marine deposits that are somewhat mobile and tend to shift over time as sand dunes with the changing currents and tides. These deposits likely formed during the Holocene sea level rise as tidal and marine sediments settled on top of the eroded Bootlegger Cove Formation seaward of the Knik Arm bluffs. They are likely present on the east side because of slightly slower currents and flatter bottom slopes and absent on the west mud flats because of higher currents and steeper slopes. The marine deposits are similar to deposits described by Updike and Carpenter (1986) in the POA area, where up to three cycles of deposition are recorded within the Holocene section above the Bootlegger Cove Formation. These cycles were laid down during fluctuating Holocene sea levels, each beginning with a gravel deposit and ending with silt and fine sand. With depth, each cycle is denser than the one above it because of overburden pressure and increasing age.

Measured uncorrected Standard Penetration Test (SPT) N values from the two borings (Borings A-2 and A-10) that penetrated deeply into this deposit near the proposed Knik Arm Crossing were between 5 and 10 blows per foot (bpf), with an average value of about 7 bpf. When corrected for the rod length, auto hammer, and confining pressure effects, the average corrected N value, or N(1)60, is about 10 or 11 bpf, which indicates that this material is on the border between loose and medium dense. The generally low N values indicate a high possibility of these recent deposits liquefying under strong earthquake shaking.

A shallow gravel cover has also been deposited on the mud flats near the eroding toe of the east bluff. This surficial unit is generally less than 10 feet thick and appears to be remnant particles eroded from the east bluff till-like soils and too coarse to be transported out of the area. The general lack of noticeable thicknesses of similar gravelly soils on the west-side mud flats, but the presence of boulders and coarse gravel on the surface, suggest that these mud flat slopes are steeper and subject to stronger erosive forces than slopes on the east-side.

5.2.1.2 Glacial Till or Moraine Deposits

The glacial till or moraine deposits that mantle much of the channel bottom side areas, extend up into and are exposed in both bluffs, but appear to have been eroded away in the center of the channel. Based on correlations to work by others (e.g., Karlstrom 1964; Mat-Su Borough 1995; Schmoll et al. 1984; Updike and Carpenter 1986), these deposits appear to be the equivalent of Knik Glaciation material on the west side of Knik Arm and the lower Bootlegger Cove Formation on the east side. Figure 10 and upgradient Borings A-5 and HLA-5 show that this unit is quite variable in thickness up and down the channel. This unit has estimated maximum thicknesses of more than 100 feet, particularly near both proposed abutments, and extends to elevations ranging from about -70 to +5 feet MLLW along the proposed Below-the-Bluff Roadway (Figures 10 and 11). Its general lack of apparent bedding or well-defined structure suggests that it is a glacial till. In addition to its lack of structure, it is characterized as both a gravelly clay and sand because of its changing mixture of particle sizes. This unit is locally classified as gravelly, silty clay—particularly on the east sid—and silty, gravelly sand with thick gravelly clay zones or layers on the west side. Gravel is generally present in this material, although in small quantities compared with the matrix materials, which include finer grain sizes. This unit is also consistently very dense or hard, with SPT values generally in excess of 50 bpf and frequently in excess of 100 bpf.

The till mantle is believed to be one of the stronger support soils at the site; however, where thin, its load-carrying capacity would be limited, requiring that foundation piles penetrate through it to deeper soil layers to achieve the required design capacities. Pile tip damage also could occur while attempting to penetrate this very dense unit or, where thick, its high density/hard consistency might make achieving a suitable minimum embedment difficult. Both these factors need to be considered in determining pile sizes, lengths, and wall thicknesses to handle the possible high-driving stresses in these soils. Although the limited drilling program indicated the occurrences are rare, the possibility exists for encountering boulders and causing the piles to stop short of intended tip depths or to be damaged.

5.2.1.3 Glacial Lake Clays and Marine/Alluvial Sands

Once the upper till-like unit or the loose marine deposits are penetrated, the borings typically encountered thick clay and sand deposits, probably the most dominant geologic unit beneath the channel. This unit is distinguished from the till-like soils by its general lack of gravel particles, with the exception of a few gravelly zones. As shown on the channel crossing profile in Figure 10, the sand is thin (or absent) near the west end, but thickens to more than 160 feet to the east, and then changes into a 200- to 250-foot massive silty clay stratum over the eastern one-third of the channel.

The continuity of the marine/alluvial sand and clay stratum across the site in Figure 10 was determined by placing the boundary results from the geophysical survey (Golder 2003) on the profile and performing minor adjustments in the sub-bottom data to match conditions in each boring on the alignment. This shows that normal straight-line interpolation methods between borings may not be an accurate representation of actual conditions in this case. Based on the boring data alone, the transition from sand to clay near the east side appears to be one of interfingering facies, whereas the Golder (2003) geophysical data suggest that the sand unit postdates the clay unit. The latter interpretation is plausible in light of the preferred interpretation of the submerged canyon (“big hole”) to the south (Section 4.1.2), which lies at nearly the same elevation as the base of the sand unit. If the canyon is an erosional remnant from the late Pleistocene lower sea level stand, the sand unit may represent late Pleistocene and Holocene material filling the canyon as sea level rose, and the interfingering part to the east may belong to the older Bootlegger Cove Formation. The base of the sand unit contains a 7- to 15-foot-thick section of gravel that corresponds to the cyclic Holocene depositional scenario described by Updike and Carpenter (1986) in the POA area. It is also possible that the submerged bench at the head of the "big hole" canyon (the only location where interfingering layers have been encountered, Boring A-5), represents an early Holocene landslide deposit of Bootlegger Cove material that slumped along a former east side bluff during a lower sea level.

The glacial lake clay beneath the eastern part of the channel is classified as a stiff to hard, gray, silty clay with generally low plasticity characteristics. Laboratory shear strength results, including unconfined compression tests, triaxial tests, and pocket penetrometer measurements, generally show consistent strengths with depth. Most values in this unit fall in the range of 2 to 5 kips per square foot (ksf) ("very stiff" to "hard") with slightly lower strengths at about Elevation -250 MLLW feet.

Mohr Circle analyses from numerous unconsolidated, undrained triaxial and unconfined compression tests were conducted with a confining pressure close to the in-situ effective confining pressure. The results show average shear strengths of 1.75 to 2 tsf (3.5 to 4 ksf), but also reflect local hard zones or layers with strengths several times the average values.

The index properties of the clay portion of this unit are also reflected in the laboratory tests on clay samples in Borings A-1 and A-6. These data show an average water content of about 23 percent, which is only slightly above the average plastic limit (about 19 percent) and well below the liquid limit (about 37 percent). The Atterberg Limit test results consistently plot above the A-Line, indicating this clay has low-plasticity characteristics. Measurements made on numerous undisturbed test specimens indicated that the average wet-unit weight of the clay is about 138 pounds per cubic foot (pcf).

Limited cone data (Cone Penetrometer Test (CPT)) were taken adjacent to Boring A-1 to check the strength and uniformity of the glacial lake clays. Measured CPT tip resistances in the 23- to 107-foot depth range were generally between 40 and 50 tsf and had friction properties that are typical of a competent cohesive soil as opposed to a granular unit. Using Nkt values of 12 to convert the CPT measurements to strength resulted in calculated undrained shear strength in the 5.5 to 6 ksf range. Similarly, using Nkt values of 15 resulted in calculated undrained shear strength in the 4.5 to 5 ksf range. This indicates that an Nkt value of 15 is probably appropriate. More important, the tip results also show very uniform strengths with depth even though both hard and less stiff zones were found to exist in the borings at other depths.

Low calculated friction ratios of between 2 percent and 2.5 percent and an inferred soil behavior classification suggests that, based on CPT (A-1) data in the 23- to 107-foot-depth range, the glacial lake clays may have silt, sandy silt and silt mixture properties.

The marine/alluvial sand is classified as a dense to very dense, gray, clean to silty, fine sand generally grading into a silty sand or sandy silt to the east. From gradation and Atterberg limit results, the Unified Soil Classification symbol for this fine sand is largely an SP or SP-SM and the silty sand to sandy silt is an SM or ML. Locally, at depth, the fine sand appears to be deposited as a glacial rock flour and, except in the sand/clay interbedded zone noted in Boring A-5, seldom exceeds 20 percent fines, has little apparent cohesion, and is nonplastic in many cases.

Cone data were recorded adjacent to Boring A-5 to check the density and uniformity of the sands. Measured CPT tip resistances in the upper 100-foot-depth range were relatively uniform and generally between 40 and 50 tons per square foot (tsf) increasing to 70 tsf below. Friction ratio values are about 1 percent (not normalized) and 2 percent or slightly more (normalized), which is typical of a granular soil. The inferred soil behavior classification, based on the CPT data, is silty sand and sand, using nonnormalized data, and silt mixtures, using normalized data.

The average density properties of this granular unit are best taken from Borings A-2, A-5, and A-10 because each penetrates a thick part of this unit.

The CPT N_{60} data and the Boring A-5 N values are believed to reflect lower values for the sands because of higher real transfer energy than used in the CPT calculations and both are in the more silty or interbedded sand and clay deposits in Boring A-5 area where lower N values should be expected. A consistent increase in density with depth and with corrections applied would probably still show that most of the fine sands below about Elevation -130 MLLW, or roughly 70 feet below mud line, are near the borderline of dense to very dense, becoming very dense with depth.

The average shear velocities were about 1,135 feet per second (ft/sec) in the more silty sands with clay interbeds at Boring A-5 and are probably several hundred ft/sec higher in the more massive sand unit found in the center of the channel.

The friction ratios for the sands in CPT A-5 had slightly lower friction ratios of 1 percent (not normalized) and 2 percent (normalized) and inferred soil behavior classification of silty sand and sand using nonnormalized data and silt mixtures using normalized data. This suggests that the behavior differences between the glacial lake clays and alluvial sands are small and reflective of a larger unit deposited under a similar geologic environment.

The above sand and clay properties depict a competent soil unit that will provide substantial skin friction support for the piles. They also provide significant end bearing assuming reasonable plug development; the clays offer lower end bearing support and lower total pile capacities compared with the sands.

5.2.1.4 Possible Knik Tills

The possible Knik till unit is the deepest unit encountered in the borings. It lies at a depth of about 10 to 80 feet below MLLW on the west side of Knik Arm and about 200 to 300 feet below MLLW on the east side (Shannon & Wilson 2004). Instead of being the typical very dense sands and gravels found in deep borings throughout the Port of Anchorage (POA) and Downtown Anchorage, it is classified as hard, gray, gravelly, sandy clay with gravelly and silty clay zones. Average SPT N values were generally over 50 bpf and often in excess of 100 bpf. Triaxial and unconfined compressive strength tests report 2.5 to 4.7 tsf. The hard consistency, solid physical appearance in undisturbed samples, and high pocket penetration values (often greater than 4.5 tsf), however, indicate that a cylindrical test specimen may be failing prematurely, and often as a brittle specimen where the in-situ value is probably higher.

This Knik till unit, like the shallower tills, is an excellent soil for pile support. However, the largely clay matrix theoretically makes end bearing lower than if it were a granular soil. Based on a strong reflector from the geophysical survey (Golder 2003), Figure 10 shows that a deeper basement layer lies below the borings at Elevation -190 MLLW feet and deeper near the west side. This is interpreted to be sand and gravel and also likely a part of the underlying Knik tills.

The data that were recovered lead to the conclusion that the geology is complex and not well defined in this channel. The borings, summarized in Figure 10, support the following conclusions: 1) the shallow tills overlie both the deep alluvial sands and glacial lake clays in the channel, 2) the alluvial sands are more extensive and (from Boring A-5 results) may form a wider and deeper channel than suggested by the geophysics, and 3) the Knik tills are much deeper over the eastern one-third of the channel. An alternative explanation suggested by the geophysical data implies that the deep alluvial sands may be early Holocene and completely postdate the adjacent interbedded sequence and shallow tills.

5.2.2 East Shoreline and Bluff Soils

The soils along the east shoreline of Knik Arm between the proposed east bridge abutment and the POA will support a highway embankment and pavement section to provide access to the proposed bridge. East Shoreline Profile B-B', presented in Figure 11, illustrates an interpretation of the soils across this section of the shoreline at the base of the bluff. Figure 9 shows the location of this profile.

Figure 11. East Shoreline Profile B-B'

As shown on Figure 11, soils similar to the above four units, except for the marine sands, are also present along the east shoreline, although there is a tendency of encountering slightly weaker soils south of Cairn Point as the POA is approached. The dominant soils, however, are the glacial lake or marine sands and clays, which are probably part of the Bootlegger Cove Formation. In addition, zones of dense, gravelly, till-like deposits are locally present north of Cairn Point and loose to medium dense, silty sands and stiff, silty clays exist near the POA. During the 1964 earthquake, there were numerous examples of ground fissuring, slumping, and lateral spreading in the tidal flats near the POA, but little evidence of liquefaction features such as sand boils or sand-filled fissures (Updike and Carpenter 1986).

The 70-foot-high bluff at the eastern end of the proposed bridge abutment contains a 20-foot-thick section of interbedded sand, gravel, and peat that is likely morainal and glacial lake material associated with the Elmendorf Moraine; these deposits are underlain by interbedded sand and clay and till-like, sandy, gravelly clay, interpreted to be part of the Bootlegger Cove Formation. The bluffs south of Cairn Point contain 20 to 85 feet of outwash sands and gravels from the Naptowne Glaciation, underlain by the Bootlegger Cove Formation. The east Knik Arm bluffs exhibit multiple ravines, slough material, and localized slump deposits. The sensitive clay unit of the Bootlegger Cove Formation occurs between about mean sea level and 50 feet in elevation along the Knik Arm and Ship Creek bluffs, and extends laterally from about 0.8 mile north of Cairn Point to south of Ship Creek (Dobrovolny and Schmoll 1974; Shannon & Wilson 2004; Updike and Carpenter 1986).

A series of landfill deposits are located between Cairn Point and the POA. Elmendorf AFB operated a surface dump at the top of the bluff in this area in the 1940s and 1950s, and debris from the landfill slumped downslope onto the beach. There are multiple locations along the base of the bluff where debris is visible and being eroded by tidal action (Shannon & Wilson 2004).

5.3 Anchorage Soils

5.3.1 East of Port of Anchorage

Directly inland (east) of the Port of Anchorage (POA) property are steep slopes that failed because of water erosion and historical earthquakes. The proposed alignment would cross continuous younger landslide deposits that are superimposed on top of older landslide material in some places. It also would cross the toe of a 1964 earthquake-triggered deposit known as the Elmendorf landslide. Located about 0.3 mile north of Government Hill in a prominent gully; this deposit consists partly of an old landslide and partly of a newly activated block, resulting in compression ridges parallel to the coastline (Updike and Carpenter 1986).

The POA facility was largely developed on granular fill of varying qualities and densities placed over the silt-covered mud flats. The fill typically ranges from 3 to 10 feet thick, and has been introduced into the area over a period of more than 70 years.

The fill is generally composed of aggregate from nearby borrow pits in glaciofluvial or floodplain deposits. Following the 1964 earthquake, some landslide debris was hauled to the tidal flats as part of reconstruction. Much of this material consisted of slide debris from the Bootlegger Cove Formation, and was typically laid down in 1.5- to 3-foot-thick layers, graded, compacted, and covered with aggregate, paving, or foundation material (Shannon & Wilson 2004, Updike and Carpenter 1986).

The main structures at the POA have been constructed on 20- to 36-inch driven steel pipe piles. At the toe of the slope, along Terminal Road, east of the POA, the soils consist of shallow, soft slope sediments underlain by medium to very stiff, silty clays to depths of more than 200 feet. Existing corrugated metal pipe (CMP) perforated subdrains beneath the fill would have to be incorporated into the road design. Ditches would be needed along the toe to intercept slope seepage and carry excess water below the new highway. Much of the developed or paved areas of the POA are underlain with Municipality of Anchorage Type 2 granular fill (Golder & Associates 1990).

5.3.2 Government Hill Soils

Concept studies have identified Degan and Erickson Streets as proposed alternative alignments for the road to transition through Government Hill.

The proposed Degan Alternative would cross mostly older landslide material along the north bluff of Ship Creek leading up to Government Hill (Updike and Carpenter 1986). As the proposed Erickson Alternative would approach the north bluff of Ship Creek and Government Hill, along the 60-foot bluff and through the site of the old Government Hill Landslide (Shannon & Wilson 1964), it would traverse a segment of slope that failed in the 1964 earthquake. Slope failure occurred through the upper 40 feet of sands and gravels and rotated in the weaker and locally sensitive clay soils found in the lower parts of the slope. Shannon & Wilson visualizes that multilane roads traversing this slide would require slope flattening, terracing, and possibly toe buttresses in this area.

The soils forming Government Hill consist of 25 to 60 feet of medium to dense, clean outwash gravels and sands to roughly Elevation +60 feet MLLW overlying medium to stiff clays with varying depths of groundwater perched on the clays and in the clean granular soils. The potential for liquefaction may exist in the granular soils where saturated (Updike and Carpenter 1986). The granular soils are from the Naptowne Outwash Formation, while the clays are part of the Bootlegger Cove Formation. In the above conditions, the anticipated elevation of a tunnel crown would likely occur mostly in the granular outwash materials. Running ground and piping conditions have been experienced when tunneling in similar outwash soils in Anchorage. A tunnel constructed in a braced or tied-back anchor trench could work well for penetrating this hill.

At the south-facing part of Government Hill, where a tunnel entrance portal is proposed to be located, the slope is steep in some areas. This area was generally stable during the

1964 earthquake. In this region the overlying granular soils are similar to most of Government Hill and underlain by the previously described clays.

Beyond the northwest edge of this area, where the north end of the tunnel portal or egress would be constructed, alignments may pass through the former Defense Fuel tank farm area, where the slopes are relatively gentle and it is believed that stability would not present a concern. Depending on the road location, some petroleum-contaminated soils from prior spill and leaks from this former tank site might be encountered. Therefore, it may be necessary to deal with soil remediation during construction as the ground is regraded to accommodate the traffic lanes that would traverse this property. This site has been well studied (Shannon & Wilson 1997). Areas of contamination, soil, and groundwater conditions are well defined from more than 100 borings.

5.3.3 Ship Creek Soils

It is understood that a possible highway corridor might traverse the south bluff of the Ship Creek drainage. At this bluff, the roadway would pass through the old First Avenue Slide, a portion of the slope that failed during the 1964 earthquake. This slope and road segment has been well studied, with prior Municipality of Anchorage-funded designs developed by Lounsbury & Associates that entail lowering the crest of the failed bluff at the former Alaska Native Hospital site to improve the stability of the slope and use the excavated, clean granular soils from the upper part of the slope as embankment fill elsewhere. The soil conditions in this bluff area are described in prior reports by Shannon & Wilson 1964, 1994, and 2001.

To reflect conditions in the large Ship Creek drainage, deep borings were drilled by the Alaska Department of Transportation and Public Facilities, ADOT&PF, in 1966 for the A Street Bridge over Ship Creek. The soils encountered in these borings largely consisted of about 15 to 25 feet of loose to dense gravelly sands overlying medium to very stiff, silty clay (mostly stiff) with very dense sands and gravels roughly 160 feet (about Elevation –145 feet) below the valley bottom. At the north and south bank of the drainage way, the deeper bearing stratum occurs at about Elevation –80 and -60 feet respectively (Shannon & Wilson 1964), and the clays appear to range from soft to stiff (Shannon & Wilson 2001).

6.0 Geotechnical Analyses and Recommendations

The key geotechnical components of this project include the foundations for the main proposed bridge crossing of Knik Arm, possible causeways at each proposed bridge abutment and the proposed approach roads to tie the project into the existing road systems. A closer evaluation should be completed in final design studies once the prism width of the roadway, the embankment heights, and original ground elevations were to become better defined.

Additionally, a site-specific ground response analysis was performed as part of this initial effort to compare with the American Association of State Highway and Transportation Official (AASHTO) code-based spectrum to further guide the planners in assessing the likely seismic design loads on different types of bridge structures. A cursory evaluation of liquefaction was also included because strength loss could reduce the support capabilities of the proposed piles, particularly lateral resistance.

It is understood that causeways are tentatively planned at the proposed bridge approaches, but their lengths and water depths/embankment heights have not yet been established. In general, the borings indicate favorable foundation soils for support of sizeable embankments. However, in selecting causeway lengths, a hydrology study should be done to evaluate future scour of the soils at the proposed causeway ends and in the bridge section. The till is thin or absent in this area, and the surficial fine sands in the center of the channel are highly erodable.

6.1 Pier Description

The final design concept has not been chosen, but it has been proposed in previous studies that 1) the proposed bridge concept currently under consideration is a two-, three- or four-lane concrete or steel bridge, 2) the proposed bridge piers would be designed with approximately four to six driven, large-diameter pipe piles, 3) pier spacing would be anticipated to be on the order of 500 feet, and 4) the pier footings would act as pile caps. In this concept, the pile caps would likely be located in the intertidal zone and protected from sea ice by a sloping jacket or hood.

It is likely that the desired ultimate axial pile capacities are on the order of 15,000 kips or more. Preliminary calculations suggest that to obtain those capacities, piles on the order of 8 feet in diameter could be required. An average wall thickness of at least 2 inches should be expected because of the anticipated hard driving to achieve enough embedment into the hard till-like gravelly soils or to penetrate this dense cap and achieve deep embedment into the underlying sands and clays. In reality, a variable or thinner section might be appropriate in some areas for final design. Four-foot-diameter piles were also evaluated in the previous studies to provide a range in capacities such that smaller sizes (or intermediate sizes by extrapolation) could also be assessed. For the smaller piles, assumed pile ultimate capacities were in the range of about 3,000 to 5,000 kips each and wall thickness in the 1- to 1.5-inch range were considered. The piles sizes that were considered in previous studies, and reported in this document, are in no way intended to suggest a limit to the possible pile solution for the proposed bridge foundation. These were merely the sizes suggested by the previous understanding. These data are given for informational purposes only.

6.2 Pile Analysis Methodology

As described above, four major soil units were encountered in borings along the channel crossing alignment. Within many of these units, strengths and density changes with depth were common and in some cases significant where use of average strength properties for each unit was not considered appropriate for a proper analysis. Therefore for pile capacity analyses, the conditions in each of the nine deep boring logs were modeled as nine idealized profiles along the over-water portion of the bridge. Changes in contacts and material types generally were interpreted to reflect those conditions noted on each boring log, while soil parameters were estimated from the strength and density results noted from field and laboratory measurements.

The above soil properties were then converted to pile parameters and analyzed by following guidelines presented by the American Petroleum Institute (API) in its manual on recommended practice (RP2A). Since the API procedures in RP2A were developed for large-diameter offshore platform piles, this procedure was selected as the preferred analysis, with use of APILE-plus as the computer program for carrying out the analysis.

As noted in the API analysis, the procedures for clays are based on the use of undrained shear strength and are largely empirical. Correspondingly, for sands or cohesionless soils the API procedure is also empirical, but effective stress techniques are employed because, for long-term performance, no excess pore water pressures are assumed.

In the analysis, both side friction and end bearing are assumed to contribute to the total capacity. It is also assumed that to achieve suitable embedment, the piles would have to be driven as a nondisplacement pile with an open tip. In calculating tip resistance, two approaches are possible in assessing the extent of plug development. The shear strength (or a remolded strength) on the inside of the pile sidewalls could be assumed to build up tip resistance gradually. Otherwise, the pile could be assumed to be plugged totally, recognizing that the dominant soils are compact and that a substantial fraction of the pile would be cleaned out and replaced with structural concrete. The latter assumption of a full plug was assumed in the analysis, recognizing that, during driving, it would not likely develop for these large piles, particularly the 8-foot-diameter piles. If plug development were not provided for, and the first approach were assumed, the calculated total capacity in the analysis might be as much as 20 percent too high.

6.3 Compression Capacity

The results of the calculated ultimate axial pile capacity versus embedment depths for each of the conditions at the nine borings are presented in Appendix B for both 4- and 8-foot-diameter piles. Also shown in each plot is the ultimate side friction (or uplift support) and bearing support or tip resistance, the sum of which is the ultimate axial capacity. The 8-foot-diameter capacities versus embedment depth curves for each of the nine borings are shown in Appendix B as Figures G-1 through G-9, while values for the 4-foot-diameter piles are presented in Figures G-10 through 18 (also in Appendix B).

In most cases, the total calculated capacities increase with embedment depth. However, in several instances the curves take a reverse or saw-toothed shape. This drop in capacity with increased embedment occurs when the pile passes from a dense granular soil with high bearing values to a cohesive soil where much lower tip capacities must be used. This drop also occurs if the pile passes through a weaker soil unit with reduced bearing and/or side friction values.

Using these analyses the theoretical embedment depths (or pile lengths) below the mud line can be determined at each boring location for a given pile capacity. As examples, if 3,000- to 5,000- and 10,000- to 15,000-kip-ultimate capacities are desired for 4- and 8-foot-diameter piles, respectively, the required pile embedment depths for the various piers proposed across Knik Arm can be estimated from the pile tip contour lines noted by the blue and green lines in the Figure 7 profile. The top and bottom values reflect capacities for the upper and lower example capacities noted above. Figure 7 may also be used to extrapolate approximate embedment depths for other capacities or even intermediate pile diameters.

Other approaches can involve taking the capacity/embedment depths from the plots in Shannon & Wilson's 2004 geotechnical report directly and grouping or assigning a number of piers to each boring area. With this approach, the Figure 10 Profile A-A' section and Figure 9 should probably be used to estimate the number of piers closest to each boring and total pile embedment lengths for each group of piers. Assuming that the actual pile butt would be situated near the mid-point in the tide fluctuation zone (about Elevation +14 feet), actual pile lengths can be estimated as the embedded length plus the water depth to MLLW Elevation 0' datum (both in Figure 12) plus 14 feet to reach the mid-point elevation. Allowing space for a proposed causeway on each end, they show that total pile lengths for the 15,000-kip ultimate capacity would range from about 75 to 150 feet on the west side and up to 255 feet on the east side in the clays (at Boring A-1). The smaller piles to reach 5,000-kip capacities would have about the same lengths or slightly more on the west side and 20 to 70 feet less on the east side.

Figure 12. Summary of pile embedment depths

For load and resistance factor design (LRFD), AASHTO's 2003 Standard Specification for Highway Bridges recommends pile resistance factors of 0.7 for operating loads with pile driving analyzer tests or 0.8 for an actual load test.

6.4 Uplift Capacity

Calculated ultimate uplift capacity changes with pile embedment below the mud line are also summarized in Appendix B as the skin friction on each plot. In general, the skin friction component of the total capacity typically represents greater than 50 percent (and as much as 85 percent) of the total capacity in clay soils and in granular soils where the pile embedment is greater than 100 feet. In the upper 100 feet of granular soils, where penetration to achieve high total capacities is limited (like in the till soils at Borings A-6, A-7, and A-9), the uplift resistance could be small and might control over compression capacity in estimating the pile lengths in these areas unless these shallow water areas are covered with a causeway.

6.5 Pile Settlements

The total pier settlements that could be permitted for this proposed bridge structure depend on many factors, including the actual loads that are to be applied and/or the span and pier loads, the permissible amount of pile load transfer in skin friction and end bearing for each of the main soil types (clays, sands, and tills), possible group action or battering, and seismic considerations. Because the channel bottom soils are generally very stiff to hard or dense to very dense, excluding the recent marine sediments, it is believed that total and differential settlements could be kept relatively small and within tolerable limits for suitable long-term performance of a pile-supported bridge at this location. Refined settlements analyses will need to be performed in the future once a proposed bridge width and design concept for a typical span and pier cap become better defined.

6.6 Pile Drivability

The static pile calculations above and in Appendix B do not consider drivability and assume that the stated capacities and pile embedments could be achieved if sufficiently large hammers were available to drive the piles to the prescribed depths without encountering refusal or obstructions such as boulders. Boulders were reported on the boring logs and, while rare, are known to be present at the site. If very large boulders were to be encountered, refusal to pile driving could occur, which would require pile cleanout and/or the need to core through or breakup the boulder. A suitably equipped pile top-drive drilling rig (likely required for pile cleanout in order to place structural concrete) should be available during this effort.

The following table summarizes the pile sizes, pile sections, and hammers considered in a previous drivability analyses (PB/HDR 2003):

Pile diameter (feet)	**Pile wall thickness** (inches)	**Hydraulic hammer – rated energy** (kip-feet)
8	2	1,180
8	3	1,180
10	3 1/8	1,180
4	1	148
4	1 ½	369

Because the soil's density and consistency characteristics considered in this earlier analysis are similar to what was actually encountered, the results of the preliminary pile drivability evaluations are still considered applicable. These concluded that

- Deeper penetrations (and, therefore, higher capacity) could be achieved for a given pile size with a larger pile wall thickness (i.e., driving stresses will be less with piles with thicker sidewalls).
- The larger-diameter (8-foot-diameter) piles would provide pile capacities that would be three to four times greater than the intermediate (4-foot-diameter) piles.
- Wall thicknesses in the 2- to 3-inch range would be appropriate for the 8-foot-diameter piles, while 1- to 1.5-inch-thick walls should work for the 4-foot piles.
- If the shallow tills could be penetrated, average embedments for 8-foot-diameter piles of 273 to 290 feet were predicted in hard clay soils, while 123 to 173 feet of embedment was calculated in very dense granular soils, the larger embedments occurring with the thicker sidewalls. For the 4-foot piles, maximum embedments were 138 to 205 feet in the clay soils and 80 to 143 feet in the very dense granular soils.

A comparison of these above maximum embedment depths with those in Figure 12 support that the depths in Figure 7 are theoretically achievable, especially with the use of thicker-walled piles.

Case histories exist from the offshore experience in Cook Inlet (with somewhat similar soils conditions) for 1) piles on the order of 34 to 84 inches in diameter, 2) piles driven to penetrations ranging from 60 to 125 feet, and 3) use of air-steam hammers with rated energies on the order of 870 kip-ft. In some instances, pile cleanout was required to achieve design penetration. Similarly, 42-inch-diameter by ¾-inch wall, high-strength piles were driven with a Delmag D 125-13 diesel hammer (350 kip-ft) to penetration depths of more than 220 feet including up to 30 feet into similar very dense till-like soils at the Glenn Parks Highway Interchange Project near the head of Knik Arm. This local experience also supports that long piles and reasonable penetration of dense soils are possible with large hammers.

Compressive driving stresses calculated from additional drivability studies using GRL WEAP, a 1,180-kip-ft hydraulic hammer, and the above 8-foot piles were in the 32 to 40 kips per square inch (ksi) range with the higher values occurring in the thinner pile sections or in the more granular soils, as noted above. This suggests that if thinner pile sections (2-inch or possibly 1- or 1.5-inch walls) were to be used for 8-foot-diameter piles, steel with yield strengths greater than 36 ksi would be needed. Comparable wall thicknesses for 4-foot-diameter piles would be 1 or 1.5 inches, as noted in the table above.

A further check of the drivability of shorter 8-foot-diameter piles into and/or through the very dense till-like soils with high end bearing resistance reveals even higher driving stresses (exceeding 60 ksi) for the thinner sections (i.e., if a 1- or 1.5-inch wall pile were selected to penetrate the high density tills and achieve high capacities). The use of thinner steel sections would increase the likelihood of possible pile damage. Thus, for piles penetrating the till-like soils, pending more refined analyses and a test pile program, 2-inch wall thicknesses with 56,000 pounds per square inch (psi) (or better)-strength steel should be assumed for these larger piles for future concept studies. A-56 steel was used on the 42-inch piles for the Glenn Parks Highway Interchange Project with a driving shoe to penetrate hard or gravelly layers.

In summary, the deep penetration of large-diameter pipe piles into these soils appears feasible. However, large hammers and piles with thick side walls and higher strength steels might be required to achieve the desired penetration and high capacities. Recognizing that the calculated drivability results in dense or hard soils are highly sensitive to the input assumptions and that boulders might be present, further studies and/or a test pile program with pile dynamic analysis (PDA) measurements and/or static load tests might prove valuable as part of future design studies to give contractors bidding the proposed construction work confidence that driven piles would be feasible and that suitable pile penetration could be achieved by driving alone.

6.7 Liquefaction Considerations

Liquefaction of the soils under future earthquake shaking could reduce the axial and lateral support for the piles. Liquefaction generally occurs in granular soils, typically loose saturated sand and silty sands, because of a rapid buildup of pore water pressure and subsequent decrease in effective stress and significant loss of strength. As discussed previously, the recent fine sands or marine deposits encountered at relatively shallow depths in the borings (above 30 to 40 feet below mud line) were loose to medium dense and appear to have the highest liquefaction potential. Conversely, much of the deeper alluvial sands encountered by the borings (with a few isolated exceptions) are largely medium dense to very dense and are, therefore, not likely to liquefy in a future earthquake.

To confirm the above statements, liquefaction analyses were performed on the soils encountered by Borings A-2, A-5, and A-10, or those borings containing mostly deep sandy soils. The liquefaction analyses generally followed the steps outlined in Youd and Idriss 2001.

In this assessment, the potential soil shear strength reductions in the noncohesive and low-plasticity soils considered residual soil shear strengths for soils with a factor of safety less than one under the design earthquake. For each boring, the liquefaction potential of the soils was evaluated using Seed's simplified empirical procedure and in accordance with National Center for Earthquake Engineering Research (NCEER) technical report NCEER-97-0022 (Youd and Idriss 1997). For the liquefaction calculations, and consistent with the above ground response analyses, a site peak ground acceleration of 0.36g was assumed. Reduced soil shear strengths were then estimated for the soils with a factor of safety less than 1.0 using the empirical relationships by Seed and Harder (1990), and assuming strengths approximately ¼ above the lower bound of this relationship.

The majority of the sandy soils evaluated for liquefaction potential have a factor of safety of greater than 1.0 against liquefaction. According to analyses, the loose to medium-dense sand in the upper 35 to 40 feet below mud line of Boring A-2 are liquefiable under the model conditions. Two deeper samples from this boring were determined to be liquefiable. However, these soils are isolated within liquefaction-stable soils and should have a negligible effect on pile foundations. The analyses of Boring A-5 revealed only one sample, at approximately 113 feet below mud line, that was liquefiable under the assumed conditions. In Boring A-10, it was found that the soils associated with the top layer of loose to medium-dense, slightly silty sand (from 0 to 25 feet below mud line) were generally liquefiable.

The above analysis of the sandy soils confirms that only the recent marine deposits in the upper approximately 40 feet may liquefy under strong earthquake shaking (see Figure 13). Therefore, Shannon & Wilson recommends that for this cost evaluation and for estimating pile lengths, it should be assumed that these marine deposits in this upper maximum 30- to 40-foot zone would contribute no axial or lateral support for the pier. Fortunately, this is not considered a serious limitation to the feasibility of placing a bridge at this location because the axial support provided from these weak soils would be small and extra lateral resistance could be achieved by increasing the stiffness of the piles in this zone and/or battering the piles. The risk of a change in pile lengths or foundation costs would be small and should be reduced even further by using large-diameter, high-capacity piles.

Figure 13. Sand density values

Liquefaction analyses were not performed on the clay soils because it was not considered necessary. Atterberg Limit results indicate that much of the glacial lake clay soils possess low-plasticity characteristics (a Liquid Limit below 33.5 percent) and may have some potential for liquefaction. Shannon & Wilson concludes, however, that these soils have a low or no liquefaction potential under strong earthquakes for two reasons. The undrained shear strengths of the clay soils in Figure 14 are mostly in the very stiff range and the sensitivity is low. Also, recent studies from cyclic tests at the POA for similar, but weaker clay soils, found that the clay "is not sensitive to cyclic loading and strength reduction, does not have the tendency to liquefy under seismic loading conditions and does not exhibit anisotropic strength behavior." Based on these findings, it is believed that under strong earthquake shaking, significant liquefaction or strength losses are not likely to occur in the clay part of this unit.

Figure 14. Typical clay strengths

As discussed in Section 8, till-like deposits occur in abutment bluffs, at shallow depths in the intertidal zone, and as the site's basement material and comprise a heterogeneous and varying mixture of sands, gravels, silts and clays. These tills are consistently very dense or hard with Standard Penetration Resistances generally in excess of 50 bpf and more frequently in excess of 100 bpf. Because of these high resistance values, these tills are considered to be very compact and stable and generally not susceptible to liquefaction. Thus, where they are present, these tills offer excellent lateral and axial support for piles penetrating and/or bearing in these materials.

6.8 Design Development Considerations

From a geotechnical/foundation engineering perspective, the design development phase of the project should, at a minimum, include the following:

- **Preliminary Site Characterization**. Prior to preliminary design, a preliminary site characterization program should be conducted to provide comprehensive geotechnical data along the entire alignment to the depths required for pile design and site response analyses. The program, therefore, should include deep borings (with detailed sampling, lab testing, and in-situ testing) integrated with a detailed geophysical exploration program.
- **Pile Installation Demonstration Project.** Prior to final design, a full-scale pile-installation demonstration project (PIDP) should be conducted to verify pile capacity and constructability. The PIDP should be based on typical pile sizes developed during preliminary design, and a sufficient number of piles should be installed to bracket the range of soil conditions defined in the preliminary site characterization.
- **Final Site Characterization.** The final site characterization should be conducted to provide pier-specific characterization for the final proposed bridge alignment. Borings should be drilled to depths in excess of the planned pile lengths.

Additional details and perspectives relative to geotechnical site characterization and pile installation demonstration projects will be provided in Section 8.

6.9 Construction Considerations

The preliminary evaluations reiterate the importance of adequate wall thickness for the installation of piles in hard driving conditions. For piles with thicker sections, more energy can be transferred to the pile tip, which generally will delay the occurrence of pile-driving refusal. In some instances, piles could encounter refusal in the shallow sandy gravel layers above the maximum penetration depths. Piles with a smaller pile section have a greater potential for encountering refusal in those layers than piles with a larger sectional area.

As indicated on Figure 10 and noted on some of the borings, there is a high potential for cobbles and boulders to be encountered during proposed construction. If very large boulders were to be encountered, refusal to pile driving would occur, which would require pile cleanout and the need to core through the boulder. A suitably equipped pile-top drilling rig (likely required for pile cleanout in order to place structural concrete) should be available during foundation construction.

Oversized materials also have a potential for damaging the pile during installation. In general, the use of a thicker-wall pile and a driving shoe should reduce the potential for damage to the pile. The driving shoe should also have adequate inside clearance to reduce inside skin friction during driving.

Because of the potential for cobbles, boulders, variable geology, and hard driving conditions, piles should be monitored using pile-driving analyzers to reduce the potential for overstressing and damaging the piles. When pile stresses are being monitored, it might be possible to safely advance the pile even though relatively high blow counts would be required.

The handling and driving of long, large-diameter piles, with large hammers, in areas of strong currents and large tidal variations would present significant challenges during construction. The uncertainties associated with working in such an environment should be taken into consideration during the preparation of cost estimates.

7.0 Highway Embankment Considerations

Proposed Anchorage alignments would make landfall on the beach south of Green Lake on Elmendorf Air Force Base (Elmendorf) and continue south along the east shoreline of Knik Arm through the Ship Creek delta into Downtown Anchorage. It is proposed that the alignment would tunnel through Government Hill by way of the Degan Alternative or the Erickson Alternative. It is understood that the alignment would then connect to the A-C Couplet across the Ship Creek delta and connect later, in Phase 2, with a proposed new viaduct incorporating Ingra and Gambell Streets. The proposed bridge would cross Knik Arm over about 60 to 70 feet of water to the 125-foot-high west bank and extend west and north to merge with the existing road system. The Mat-Su-side landfall would split into two alignments that would bypass Lake Lorraine to the north (the Northern Access Alternative) or the south (the Point Mackenzie Road Alternative) and connect to Point MacKenzie Road.

This section provides preliminary foundation parameters for sizing and evaluating the proposed approach corridors and the tunnel and overpass structures to tie the proposed Knik Arm bridge into the existing road system. The focus of this section is intended to develop preliminary recommendations to assist the Study Team in developing a rational construction cost estimate for the remaining onshore portions of the proposed project.

7.1 Government Hill Landslide

About 400 feet south and east of the potential south tunnel portal, the proposed Anchorage side alignments potentially pass across an 800-foot-wide toe section of the slope that failed during the 1964 earthquake. This over steepened slope at that time dropped up to 20 feet and moved laterally about 50 feet. To develop and accommodate highway lanes would require slope-flattening, terracing, and possibly a toe buttress. Prior stability studies of the bluff area to the east of this slide suggest that the overall average cut slopes should be kept to about 2H to 1 V or flatter. Deep permanent cuts in the toe or large fills at the higher elevations should both be avoided if dynamic stability conditions are to be maintained.

7.2 Cut and Cover Tunnel/Government Hill

It is proposed that to accommodate four lanes of traffic, the two lanes in each direction would be stacked, forcing the tunnel to pass mostly through the overlying clean, granular soils and to a lesser extent the medium to stiff clays in the deeper parts of the hill (below Elevation +60 feet MLLW). Because of potential running ground conditions, the granular soils would be more favorable for cut-and-cover construction than would use of conventional tunneling methods. To develop a typical tunnel section and estimate the cost of this tunnel, the Study team assumed that the wide trench cut would contain vertical walls to the invert of the lower road section. Presumably, the walls would be retained by slurry wall, conventional steel soldier piles, and wood lagging or a similar bracing system, with tied-back anchors to carry the lateral earth pressures. For sizing this temporary support system, the following preliminary design criteria are recommended:

Assumptions

1. wall height: 50–70 feet
2. perched water will drain (i.e., no excess hydrostatic pressures)

Preliminary criteria

Lateral earth pressure

- rectangular pressure diagram with pressure of 25H pounds per square foot (psf), where H = wall height in feet

Soldier pile

- allowable Tip Bearing = 3,000 (psf) in stiff clay below Elevation -20 feet MLLW
- allowable Skin Friction = 450 psf
- Ultimate Passive Earth Pressure to check for toe kick out: $65\ H^2 + 3{,}000\ H$ in pounds (lbs)

Tie-Back Anchors

- No Load Zone: Horizontal line at toe for H/2 and then inclined at 60 degrees with horizontal to surface
- anchors inclined 10 to 20 degrees with horizontal
- Allowable Skin Friction for anchors behind no-load zone: 1,500 psf in medium dense sands, and 500 psf in medium to stiff clays (higher with staged pressure grouting)

Clay strengths vary considerably with depth at this site and are soft or medium stiff in some depth zones. Therefore, the above criteria would need to be modified during final design to reflect actual conditions, based on results of additional geotechnical studies.

The finished tunnel structure would need to accommodate the same above-earth pressures, the traffic loads, and the soil weights applied on the tunnel crown. For estimating crown loads, the unit weight of soil should be taken as 140 pcf. The finished structure should also contain subdrains to reduce future hydrostatic pressures or be designed to resist these water loads. The perched water level should be taken as Elevation + 65 feet or about 5 feet above the sand clay interface.

7.3 Port of Anchorage

In developing cargo storage pavements over the POA property, the land to most of the property edge is paved and generally contains about 3.5 to 4.5 feet of free-draining granular fill materials. It is understood that there is a 400-foot right-of-way proposed between the POA property and the toe of the steep slope to accommodate the proposed highway; this area is largely undeveloped. Where the land is undeveloped, the ground is often moist to wet, and contains interbedded or intermixed soft or loose silts, clays, and sands (slope colluviums and poor surface runoff) at shallow depths underlain by stiff clays. These poor soils would, thus, need to be excavated and replaced with nonfrost-susceptible fill as a future subgrade and pavement section. To accommodate these conditions, the structural section would need to be several feet greater than the 42-inch pavement section discussed in Section 7.6.

Because land is at a premium in this area near the POA, it may be suggested that the proposed highway be set into the toe of the slope. In considering this, it should be noted that much of the slope face contains landslide debris; designing and constructing a retaining wall would add to the cost of construction through this area. Also, because the slopes are very steep in this region and have failed in prior earthquakes and from continuing water erosion, permanent toe cutting should be minimized.

7.4 Shoreline Embankment

From the POA area to the proposed east bridge approach, the bluff slopes are steep and locally slumping and are subject to periodic failures from toe undercutting, bank face erosion, and occasional strong earthquakes. The proposed highway would thus have to be situated on the toe, which, in this case, would require covering the mud flats with embankment fill. For design, the minimum finished elevation of the road would have to be about Elevation +23 feet MLLW or higher, at least 6 feet above the highest observed tide level. This elevation would provide for a design wave of 5 feet, which was the wave used for design of many of the paved areas in the POA development.

For embankment design consistent with other fills on the mud flats, the fill supporting a pavement section should be carried to and buttress the toe of the existing natural slopes, and have maximum shoreline embankment slopes of 2H on 1V. Fill slopes within, above, and below the intertidal zone should contain riprap. At the toe of the fills at the POA, a below-grade gravel toe buttress up to 15 feet deep is keyed into the soft silts (often present on the mud flats) to contain fill and maintain stability of the embankment slope under earthquake loading conditions. However, recent studies on the mud flats north of the POA encountered stiffer shallow soils where a buttress in this local area would not be needed (Shannon & Wilson 1997). For estimating purposes, it is suggested that allowance for a toe buttress be included along about 30 percent of the shoreline between the north edge of the POA and the proposed east approach. Consistent with the buttress designed by Woodward-Clyde (1983), this proposed buttress should be assumed to be 40 feet wide along the toe, embedded about 15 feet below the mud flats and contain 2H on 1V slopes on the down slope sides of the below grade buttress. The 2H on 1V slopes form the outside of the embankment and should be covered with riprap.

The embankment section supporting the shoreline roadway should consist of granular fill that can be placed and compacted to support the pavement section. Generally a Select Material Type A, B or C as described in Paragraphs 1 through 3 of Section 703-2.07 of the Alaska Department of Transportation and Public Facilities (ADOT&PF) Standard Specification for Highway Construction would meet this requirement. Generally, for compaction and drainage considerations, the Type A material should be used in the intertidal or wet construction areas of the embankment.

Consistent with most of the banks along Knik Arm, spring seepage occurs at various elevations on the slope. This water is generally perched on clay layers resulting in subsurface seepage below the elevation to be filled with embankment materials as well as direct downslope runoff above the roadway fill. Similar to the POA area, both surface ditching and subdrains are needed to collect and carry this water using cross culverts through these embankment fills. Similar drainage systems should, thus, be planned for the proposed road segment north of the POA. The subdrains east of the POA and along the toe of this bank consists of a 10-inch perforated pipe buried about 10 feet below the ground surface to prevent seasonal freezing.

7.5 Abutment Bluffs

On the east approach, Shannon & Wilson visualizes that the proposed bridge would transition to a fill embankment along the toe of similar, but smaller, bluff heights. To avoid deep fills in water, and large blankets of riprap to accommodate the 40 foot tides, it is recommended that seaward filling of the mud flats be limited to water depths of 30 feet or less. In Shannon & Wilson's opinion, the proposed piers and bridge structure should, thus, be curved to closely merge with the embankment paralleling the shoreline. At the proposed abutment edge, the bridge loads could be carried on piles with a riprap slope for erosion protection or a vertical sheet pile cofferdam face.

As the proposed bridge grade would ascend to the high west side bluffs from the last bridge pier, bank seepage, slope erosion conditions, and toe erosion in the west abutment region need to be stabilized, as discussed for the east shoreline. The severe gullying and minor sloughing observed on the west bluff indicate that surface runoff and subsurface seepage would combine to create maintenance problems. Design of the slope modifications would depend on the elevation grades of the proposed highway relative to the bluff. It is recommended that for a receding toe and slope, at an estimated average rate of about 0.5 foot per year (Shannon & Wilson 1971), slope-flattening and drainage control of the bluff should be planned with riprap protection in the general intertidal zone.

7.6 Pavement Sections

Design of pavement sections along highway segments must consider the structural support capabilities of the subgrade soils and their frost behavior. Generally, with the possible exception of some of the proposed north approach highway alignments on the Mat-Su side, and a small segment of highway on the bluff between 2nd and 3rd Avenues, the subgrade soils are largely frost susceptible. Frost will, therefore, control design of the section and should largely consist of a full 42-inch pavement section with added drainage control to direct downslope surface and subsurface water away from the pavement section. Shannon & Wilson recommends for planning purposes, the pavement section, in descending order, consist of 3 or 4 inches of asphalt, 6 inches of base and 32 or 33 inches of subbase. Generally, the base course should consist of D-1 material as specified in Section 703-2.03, Table 703-2 of the ADOT&PF Standard Specification for Highway Construction. For the subbase, a Selected Material Type A, as described in Paragraph 1 of Section 703-2.07 of the above specification, is recommended.

For the proposed Mat-Su side approach and along the bluffs south of Second Avenue on the Anchorage side, better-quality granular soils are generally present below the surface organics and silt layers as indicated above. Shannon & Wilson recommends that, on the Mat-Su side of Knik Arm, the local peats should be removed and the proposed pavement section constructed on cuts or fill embankments of varying heights or depths as necessary for proper vertical and horizontal alignment. It should also be elevated as necessary to maintain positive surface drainage. Because the road grade is not yet established, a reasonable assumption for cost estimating would be that a full, 42-inch pavement section as recommended above would be needed in this area.

On the bluff south of Second Avenue, prior studies for the Ingra Street extension (Shannon & Wilson 1994), recommended that silty soils or suitable fills with excess fines or organics be removed and the site be brought to grade with the above 42-inch pavement section. Assuming the soils at the subgrade consist of mostly nonfrost-susceptible soils, the thickness of subbase could be reduced if the finished street grade would be less than 42 inches above the subgrade level.

7.7 Borrow Sources

Local project granular borrow sources for proposed embankment construction and the pavement section materials are available in the Mat-Su area near the proposed west bridge approach, in the upper parts of Government Hill, and, to a limited extent, in the First Avenue/Ingra Street bluff. Otherwise, granular aggregate as well as riprap and ballast would have to be imported from local Anchorage and Mat-Su area suppliers. Much of Anchorage's aggregate is imported by train from the Palmer/Wasilla area.

8.0 Additional Explorations and Data Limitations

As discussed above, additional explorations would be needed along the chosen channel crossing to confirm the conditions described by these adjacent explorations, and along the approaches and highway extensions to tie into the existing road system. The data along the water crossing are particularly weak as prior studies have focused on different alignments and even these data are limited. The borings shown in the attached profile do not penetrate deep enough to confirm the nature of the pile support units, particularly beneath the eastern half of the proposed water crossing.

A key factor in designing the anticipated pile foundations for the proposed bridge would be the ability to achieve suitable penetration of the piles into the very dense or hard glacial sediments. The hard driving that would be anticipated could result in pile damage and the inability to achieve adequate penetration and fixity or adequate lateral and vertical support. Boulders could also pose a concern during construction. The combination of these factors could force the contractor to use other foundation shaft sinking methods, an issue that could lead to changed condition claims and/or significantly increased construction bids/costs. A test pile program should be added to the exploratory program to confirm that the piles could achieve the desired penetration by driving, and that the design would be constructible. Actual test piles driven in advance of the bid would give the contractors confidence in the ability to accomplish the over-water work, thus reducing risk and securing lower construction bids.

8.1 Use of Design Information

The design information provided is extremely preliminary in nature and should not be used for purposes beyond that stated in this Technical Report. Although information contained in this memorandum may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the proposed project, the information in this Technical Report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this memorandum are modified or validated in writing by the geotechnical engineers on the Team.

8.2 Potential Variation in Subsurface Conditions

It is important to note that very little information is available at this time to define either the stratigraphy or the material properties along the proposed corridor options, including the bridge. Consequently, the proposed bridge design and cost estimates should take into consideration a high potential for significant variations from the idealized profiles and preliminary estimates obtained in these analyses. The available geotechnical data extend to limited depths and, therefore, do not "ground-truth" the geophysical data below Elevation -100 to Elevation -150 feet MLLW. Limited samples, in addition to limited laboratory and in-situ data were used to estimate soil properties. Along the eastern half of the over-water alignment, the HLA report suggests that even the geophysical data may be tenuous. Additionally, glacial soils in general are extremely variable. Consequently, there is a high potential for variations in soil conditions at the site and, therefore, variations in pile design data beyond the values presented in this Technical Report.

9.0 Summary of Environmental Consequences and Mitigation

9.1 Direct Impacts and Mitigation Measures

9.1.1 No-Action Alternative

The No-Action Alternative would have no project-related effects on topography, geology, or soils in the Study Area. No project-related excavation or fill would be required; therefore, there would be no project-related changes to existing landforms, soils, or sediment. New development that would occur in the Study Area without the proposed build alternatives would affect geology and soils through soil removal, alteration of landforms, and sand and gravel extraction.

As no project-related structures would be built under the No-Action Alternative, there would be no geotechnical or seismic impacts to the project. Geotechnical conditions and seismic hazards would continue to impact existing and new development that would occur in the Study Area without the proposed build alternatives.

9.1.2 Mat-Su Approach Alternatives

Widening of Point MacKenzie Road and construction of a new road along the Northern Access Alternative would entail removal of surface soils to accommodate roadway construction and grading. Extraction of sand and gravel at sites along the roadway for construction material would reduce remaining reserves of these resources. Material extraction and road construction would alter topography and landforms and could cause soil erosion and siltation. Use of best management practices (BMPs) for erosion control during construction would mitigate these effects.

Geotechnical issues and seismic hazards are expected to have minimal impact on the proposed Mat-Su approach alternatives. No proposed structures would require special foundation design. The use of nonfrost-susceptible foundation material and proper drainage design for the road would mitigate the potential impacts of wet, fine-grained soils on road stability and maintenance. The proposed alternatives would cut through several areas of moderate to steep slopes. Road cuts in these areas would be benched where necessary to mitigate the potential effects of sideslope instability on the roadway.

9.1.3 Knik Arm Crossing Alignment (Including Below-the-Bluff Roadway)

Construction of proposed armored fill approaches, abutments, and the Below-the-Bluff Roadway for the proposed 8,200-foot bridge alternative would have a combined footprint of about 40 acres on top of Knik Arm tidal sediment. The 14,000-foot bridge alternative would have a footprint of approximately 5 acres on tidal sediment. Over time, tidal sediment would be expected to build up on both sides of the proposed approaches, as suspended sediment during both ebb and flood tides would become partially restricted from moving through Knik Arm. Siltation impacts would be expected to cover about 280 acres for the 8,200-foot bridge, but would not be expected to extend further south than Cairn Point. Sediment buildup would not be expected to occur for the 14,000-foot bridge.

Current modeling studies suggest localized erosion of tidal sediment might occur near Cairn Point under the 8,200-foot bridge alternative, because of enlargement of a preexisting eddy in this area (HDR and URS 2005). In addition, about 5 feet of scour erosion would be expected at the seabed for the 8,200-foot bridge due to channel constriction and increased current velocities. This effect, however, would not be expected to have an impact on the location of the submerged canyon (“big hole”) south of the alignment; physical restrictions of Knik Arm might change the rate of sedimentation currently occurring in the hole, but would not be expected to cause hole migration.

Knik Arm subsurface sediment might have an effect on the stability of the proposed bridge and abutment because of loading of unstable soils and because of high-density soils preventing adequate pile penetration. Geotechnical investigations to date provide preliminary recommendations for depths of penetration in the various soil types; there are, however, many complexities and unknowns in the vertical and lateral extent of sediment types. In addition, boring data are limited along the actual proposed alignment and do not penetrate deep enough to confirm the nature of proposed pile support units, particularly in the eastern half of the crossing. Additional geotechnical investigation would be needed during final design to further assess subsurface sediment conditions and to provide final design recommendations to mitigate these effects on the proposed bridge and abutments.

Seismic hazards that could affect the proposed bridge and Below-the-Bluff Roadway include ground shaking, slope instability, liquefaction, and other types of ground R (HDR and PND 2006), provide preliminary predictions of ground shaking for two bridge design levels: the bridge would be designed to incur no damage and be immediately operational following a 100-year return period earthquake; and to sustain significant but repairable damage following a major 1,000-year earthquake. Peak ground accelerations predicted by the probabilistic hazard models for these earthquakes range from 0.205 to 0.330g for the 100-year event, and from 0.468 to 0.725g for the 1,000-year event. Geotechnical analyses of liquefaction have been conducted on subsurface soil samples collected to date, assuming a peak ground acceleration of 0.36g. These results have been incorporated into preliminary design recommendations of pile depths so that they extend below liquefiable soils and are not impacted by this phenomenon. Ground fissuring and land spreading could cause cracking and buckling of the proposed Below-the-Bluff Roadway. Additional geotechnical investigation during final design would further assess subsurface sediment conditions along the alignment, refine seismic and liquefaction analyses, and provide final design criteria to mitigate the potential damaging effects of ground shaking and ground failure on the proposed bridge and abutments.

Slope stability effects on the proposed project could take the form of large-scale, earthquake-triggered translational landsliding, or of more gradual slumping and erosion that could occur in the absence of earthquakes. Translational landsliding could damage the proposed west bridge abutment and cover the proposed Below-the-Bluff Roadway. Geotechnical investigations to date provide preliminary recommendations for the control and stabilization of seepage flow and erosion at the proposed west bridge abutment, embankment stability and toe buttressing along the proposed Below-the-Bluff Roadway, and drainage control along the base of slopes east of the POA. The raised embankment and toe buttressing of the proposed Below-the-Bluff roadway would provide the beneficial effect of reducing future current and wave erosion at the base of these unstable slopes. Additional geotechnical investigation during final design would further assess localized slope stability problems and provide final design recommendations to mitigate the potential effects of unstable slopes on the proposed project.

9.1.4 Anchorage Alternatives

Construction of the proposed Anchorage alternatives using cut-and-cover tunnel methods would entail removal of surface soils at Government Hill to accommodate proposed roadway construction and grading. These activities would temporarily alter the topography, which, following construction, would presumably be restored close to original form. Translational landsliding could damage the proposed Government Hill tunnel and Ship Creek slope approaches. Proposed construction in these areas, which contain 1964 and older landslide material, might entail permanent modification of topography through slope-flattening and terracing, removal of soft soils, drainage control, and possible buttressing. Additional geotechnical investigation during final design would further assess localized slope stability problems and provide final design recommendations to mitigate these effects.

9.2 Indirect Impacts

Increases in population and southward shifts in population that are projected to occur in the Mat-Su Valley if the project were built would have an impact on soils through expansion of development into previously undeveloped areas of the Mat-Su Valley. Such development would likely occur in areas of soils that are most suitable for building along the Point MacKenzie and Knik-Goose Bay Roads. Indirect effects might include an increase in soil erosion in areas of new building and road construction, timber harvest, and sand and gravel extraction. Development within the Point MacKenzie Agricultural Area would reduce the availability of soils suitable for agricultural uses and reduce remaining sand and gravel reserves.

Increases in population in Mat-Su would also subject a larger number of homesites and commercial buildings to potentially damaging ground shaking and ground failure in the event of a major earthquake. In addition, new structures built in the northwest corner of the Study Area, if situated within the Castle Mountain fault zone (Figure 6), risk the potential for damage due to permanent surface displacement in the event of an earthquake on this historically active fault. The indirect effects of seismic activity are further discussed in the *Seismic Studies* Technical Memorandum (HDR and PND 2006).

REFERENCES

Abrahamson, N., 1994, RSPMATCH computer program.

Aeromap, U.S. 2001. Color aerial photograph: West Cook Inlet, Roll #2001-8C, Exposure #5-26. June 12. Scale: 1 inch = 600 feet.

Aeromap, U.S. 2002. Color aerial photograph: Anchorage, Roll #2002-16C, Exposure #2-12. August 28. Scale: 1 inch = 600 feet.

Alaska Department of Highways Materials Section, 1968, Foundation Report, Anchorage Port Access Via Duct, Bridge No. 455, Anchorage, Alaska, April, F-042-1(23).

Alaska Earthquake Information Center (AEIC). No Date. <http://www.aeic.alaska.edu>

Golder Associates, 2003, Knik Arm Geophysical Investigation, November, Draft.

Bartsch-Winkler, S. 1982. Physiography, Texture, and Bedforms in Knik Arm, Upper Cook Inlet, Alaska, During June and July 1980. U.S. Geological Survey Open File Report 82-464.

Colonell, J., and D. Jones. 1990. *An Investigation of Sedimentation Processes in the Vicinity of the Ship Creek Waterfront Development.* Woodward-Clyde Consultants and Coastline Engineering, Anchorage.

Dobrovolny, E., and H. R. Schmoll. 1974. *Slope-Stability Map of Anchorage and Vicinity, Alaska*. U.S. Geological Survey Misc. Geo. Inv. Map I-787-E. Scale 1:24,000.

Haeussler, P. J., R. L. Bruhn, and T. Pratt. 2000. Potential Seismic Hazards and Tectonics of the Upper Cook Inlet Basin, Alaska, Based on Analysis of Pliocene and Younger Deformation. Geological Society of America Bulletin, 112:9, pp. 1414–29.

Hamilton, T.D., 1994, Late Cenozoic Glaciation of Alaska, The Geology of Alaska, Geological Society of America, Boulder, Colorado.

Harding Lawson Associates (HLA). 1983. *Marine Seismic Reflection Survey, Knik Arm Crossing, Anchorage, Alaska.* Report prepared for EMPS-Sverdrup, J. V.

HDR Alaska, Inc. and PND Inc. 2006. Knik Arm Crossing *Seismic Studies* Technical Memorandum. Draft report prepared for Knik Arm Bridge and Toll Authority, ADOT&PF, and FHWA.

HDR Alaska, inc. and URS Corporation. 2006. *Hydrologic Environment of Knik Arm*. Draft Report prepared for Knik Arm Bridge and Toll Authority, ADOT&PF, and FHWA.

Karlstrom, T. N. V. 1964. Quaternary Geology of the Kenai Lowland and Glacial History of the Cook Inlet Region, Alaska. U.S. Geological Survey Professional Paper 443.

Magoon, L. B., W. L. Adkison, and R. M. Egbert. 1976. Map Showing Geology, Wildcat Wells, Tertiary Plant Fossil Localities, K-Ar Age Dates, and Petroleum Operations, Cook Inlet Area, Alaska. U.S. Geological Survey Misc. Investigation Series Map I-1019. Scale 1:250,000.

Matanuska-Susitna Borough. 1995. *Point MacKenzie Area Which Merits Special Attention (AMSA) Plan.* Effective date May 23, 1993, printed November 1995.

Municipality of Anchorage. 2001. Seismic Dataset. GIS shapefile and metadata, last modified 1998. Digitized from Winterhalder et al. (1979), original scale 1:63,360.

National Resources Conservation Service (NRCS). 1998. Soil Survey of the Matanuska-Susitna Valley Area, Alaska.

National Resources Conservation Service (NRCS). 2001. *Soil Survey of Anchorage Area, Alaska*.

Nowacki, G., P. Spencer, M. Fleming, T. Brock, and T. Jorgenson. 2002. *Unified Ecoregions of Alaska.* U.S. Geological Survey Open-File Report 02-297. Scale 1:4,000,000.

Plafker, G., L. M. Gilpin, and J. C. Lahr. 1993. "Neotectonic Map of Alaska," in G. Plafker and H. C. Berg, eds., *The Geology of Alaska, Geology of North America, Volume G-1.* Geological Society of America. Boulder, Colorado. Scale 1:2,500,000.

Reger, R. D., R. A. Combellick, and J. Brigham-Grette. 1995. "Late-Wisconsin Events in the Upper Cook Inlet Region, Southcentral Alaska." in R. A. Combellick and F. Tannian, eds., *Short Notes on Alaska Geology.* Alaska Division of Geological and Geophysical Surveys Professional Report 117, Fairbanks, pp. 33–45.

Reger, R. D. and D. S. Pinney. 1997. Guide to the Geology of the Kenai Peninsula, Alaska. S. M. Karl, N.R. Vaughn, and T.J. Ryherd ed(s).

Schmoll, H. R. and E. Dobrovolny. 1972. *Generalized Geologic Map of Anchorage and Vicinity, Alaska.* U.S. Geological Survey Misc. Geo. Inv. Map I-787-A. Scale 1:24,000.

Schmoll, H., L. Yehle, C. Gardner, and J. Odum. 1984. *Guide to Surficial Geology and Glacial Stratigraphy in the Upper Cook Inlet Basin.* Alaska Geological Society, Anchorage.

Seed, H.B. and Harder, L.F., 1990, SPT based analysis of cyclic pore pressure generation and undrained residual strength, Proceedings of H. Bolton Seed Memorial Symposium, BiTech Publishers, Vancouver, BC, 2, pp 351-376.

Shannon and Wilson, Inc., 1964, Geotechnical Report, Anchorage Area Soils Studies, Anchorage.

Shannon & Wilson, Inc, 1971, State of Alaska, Department of Highways, Knik Arm Crossing, Anchorage, Synopsis of Studies for May 1971 Conference.

Shannon and Wilson, Inc., 1994, Geotechnical Report, Ingra Street Extension, Anchorage. (A-517-2)

Shannon and Wilson, Inc., 1997, Site Characterization Defense Fuel Support Point, Anchorage. (Y-5693)

Shannon and Wilson, Inc., 2003, Sand and Gravel Assessment, Elmendorf Moraine, Port MacKenzie, Alaska, October. (32-1-01622)

Shannon and Wilson, Inc., 2004, Preliminary Geotechnical Report, Knik Arm Bridge Project, Anchorage. (32-1-01536)

Shannon and Wilson, Inc., 2005, Geotechnical Report, TOFC Road and Rail, Port of Anchorage. (32-1-01719)

Shannon and Wilson, Inc., 2005, Phase III Additional Explorations and Consulting, Knik Arm Crossing. (32-1-01536-003)

Smith, O. P. 2004. Knik Arm Bridge Conceptual Engineering – Knik Arm Current, Sediment Transport, and Ice Studies. Report prepared for PND Inc.

Updike, Randall G., 1986, Engineering Geologic Maps of the Government Hill Area, Anchorage, Alaska, Map I-1610.

Updike, Randall G. and Carpenter, Bruce A., 1986, Engineering Geology of the Government Hill Area, Anchorage, Alaska, U.S. Geological Survey Bulletin 1588.

Updike, R. G., H. W. Olsen, H. R. Schmoll, Y. K. Kharaka, and K. H. Stokoe II. 1988. *Geologic and Geotechnical Conditions Adjacent to the Turnagain Heights Landslide, Anchorage, Alaska.* U.S. Geological Survey Bulletin 1817.

URS Corporation. 2005. *Draft Technical Memorandum, Seabed Movement of Knik Arm, Anchorage, Alaska*, for Knik Arm Bridge and Toll Authority and ADOT&PF, Anchorage.

Wahrhaftig, C. 1965. *Physiographic Divisions of Alaska.* U.S. Geological Survey Professional Paper 482. Scale 1:2,500,000.

Wesson, R.L., Frankel, A.D., Mueller, C.S., and Harmsen, S.C., 1999, Probabilistic Seismic Hazard Maps of Alaska U.S. Geological Survey Open-File Report 99-36, <http://geohazards.cr.usgs.gov/eq/>.

Winkler, G. R. 1992. *Geologic Map and Summary Geochronology of the Anchorage 1^o x 3^o Quadrangle, Southern Alaska.* U.S. Geological Survey Misc. Investigation Series Map I-2283. Scale 1:250,000.

Winterhalder, E. C., T. L. Williams, and J. M. England. 1979. *Geotechnical Hazards Assessment, Municipality of Anchorage, Anchorage, Alaska.* Report prepared by Harding-Lawson Associates for the Municipality of Anchorage, H-LA Job. No. 5502,009.08.

Woodward-Clyde Consultants, 1982, Final Report on Seismic Exposure Software Application, Prepared for National Oceanic and Atmospheric Administration, Vols. I and II.

Youd, T. L. and Idriss, I. M. April, 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefied Resistance of Soils, ASCE Journal of Geotechnical and Environmental Engineering, 127(4) 297-313.

Appendix A
Boring Logs and Other Relevant Information

Reports by Shannon & Wilson, Inc., unless otherwise noted

Matanuska-Susitna Side

- 01536-003 Knik Arm Crossing, Borings B-1 through B-4
- 01622 Port MacKenzie Sand & Gravel Assessment, Borings B-3, B-4 & Test Pit TP-2

Knik Arm Crossing (including'Below the Bluff' roadway)

- 01536 Knik Arm Bridge Preliminary Geotechnical Report, Borings A-1 through A-17, Port Boring B-13, & HL A-4 through HL A-6, CPT Testing Results
- 01719 TOFC, Borings G03-03, G03-07, G03-08, G03-12 (Boring Logs by Golder & Associates)

Anchorage Side

- Y-5693 Defense Fuels Support Point Site Characterization, Borings 200, 221, and 227

A/C Couplet

- Foundation Report, Anchorage Port Access Viaduct, (Report by Alaska Department of Highways)

Ingra/Gambell Couplet

- A-517-2 Ingra Street Extension, Borings B-1 through B-8

1964 Anchorage Area Soils Studies

- Government Hill Slide
- 1st Avenue Slide

Matanuska-Susitna Side

Knik Arm Crossing (including ‘Below the Bluff’ roadway)

Anchorage Side
A/C Couplet

Anchorage Side
Ingra/Gambell Couplet

1964 Anchorage Area Soils Studies

Government Hill Slide
1st Avenue Slide

Appendix B
Axial Pile Capacities

Appendix C
Downhole Seismic Velocity Survey