Phase I Geotechnical Report

Gravina Access Project

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EXECUTIVE SUMMARY

This preliminary geotechnical report presents the results of our review of existing literature and limited geotechnical engineering analysis for the Gravina Access Project, Ketchikan, Alaska. The purpose of the literature review was to identify the likely subsurface conditions at various potential water crossing locations. Preliminary geotechnical engineering studies were then conducted to develop conceptual foundation design recommendations and assist the design team in narrowing the crossing alternatives to the more promising alignments. Because most existing data is near the shoreline and there is essentially no drilling data in the middle or deep parts of the Tongass Narrows, the recommendations we have provided should be recognized as being conceptual for use in planning and feasibility studies for the project.

For this preliminary study, four subsurface cross-sections were developed to represent our rough estimate of the anticipated depths and possible subsurface conditions that could be encountered in the various water crossing corridors. The sections are situated to the North, directly across from downtown Ketchikan and to the south crossing Pennock Island where two water crossing are required to complete the hard link. These profiles are conceptual at this stage and will have to be refined with follow-on exploratory studies. The profiles generally show one of three conditions: 1) a thin veneer of sediments over bedrock, 2) a thick package of glacial drift material overlying bedrock, or 3) loose sands overlying glacial drift and bedrock. The loose sands that occur near Carlanna Creek near downtown Ketchikan would present the most challenge in designing and constructing foundations for the bridge from a geotechnical view. A fault may exist within the Tongass Narrows and in the channels on either side of Pennock Island.

While others are considering tunnel or other crossing methods, the focus of our preliminary analyses is on a bridge crossing. Preliminary design capacities for bridge pier foundations followed by a brief discussion of construction considerations are therefore presented to aid the designers in establishing rough estimates of pier diameters, lengths, and numbers at the various crossings. Conceptual design recommendations are also presented for the abutments, for the approach fills, and retaining structures. Seismic considerations are discussed at the end of our recommendations.

In the offshore areas of the narrows, foundations to support the bridge columns would penetrate up to 200 feet (60 m) of water, and various thicknesses of loose and dense soils, and derive foundation support in bedrock or the dense glacial soils. Generally bridge foundations are tentatively envisioned to be drilled piers with diameters in the order of 4 to 12 feet (1.2 to 3.6 m). Because much of the construction work must be carried out on floating or jacked-up platforms and in deep water, construction of the piers will likely be carried out in the wet or without

dewatering using "wet" construction methods. As the bridge extends out from the shorelines, foundations for abutments can vary depending upon local topography and subsurface conditions. If competent soils or rock are present, either the large diameter piers or embankments and spread footings can be used.

Once the more feasible three or four corridors are identified from this Phase 1 study, we recommend that exploratory borings be conducted in each area to refine our interpretations of the preliminary subsurface conditions we have described herein. At each of the three or four crossings, exploratory drilling work should consist of a minimum of two offshore borings on either side of the inferred fault and two onshore borings (one on each side) in areas where bedrock is not exposed on the surface. After a preferred alignment is chosen, for final design, we recommend additional explorations at each anticipated bridge pier location and in areas where special structures are planned.

The preliminary engineering recommendations contained in this report are based on site conditions as they were extrapolated and interpreted from limited existing literature and are assumed to be typical of the subsurface conditions throughout the site. It is possible (and likely) that some of our interpretations of offshore sediment thickness and material properties may have to be adjusted to accommodate different conditions. These changes may also lead to some changes in our preliminary recommendations.

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ABBREVIATIONS & ACRONYMS

Cm	Centimeters
DOT&PF	Alaska Department of Transportation & Public Facilities
G	Acceleration of gravity
H	Horizontal
Kg	Kilogram
Kg/m^3	Kilograms per cubic meter
Km	Kilometers
KN	Kilonewtons
Kpa	Kilopascals
M	Meters
NFS	Non-frost susceptible
NOAA	National Oceanic Atmospheric Administration
Pcf	Pounds per cubic foot
PGA	Peak ground acceleration
Psi	Pounds per square inch
RQD	Rock Quality Designator
TEA 21	Transportation Equity Act for the 21st Century
Tsf	Tons per square foot
USCG	United States Coast Guard
USGS	United States Geological Survey
V	Vertical

PHASE 1 GEOTECHNICAL REPORT
GRAVINA ACCESS PROJECT
KETCHIKAN, ALASKA
DOT&PF Project No: 67698
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1.0 INTRODUCTION

1.1 Purpose & Scope

The Gravina Access Project is intended to provide improved access between the City of Ketchikan on Revillagigedo Island, and Gravina Island. Gravina Island is separated from Revillagigedo Island by the Tongass Narrows, a 1,500 to 6,500 feet (450 to 2,000 meters (m)) wide waterway with maximum water depths of about 200 feet (60 m). The first phase of this project is to complete permitting and an engineering evaluation of alternatives to crossing the Tongass Narrows. The information generated during the Phase 1 study will be used to identify the more promising three of four locations for crossing the Narrows and determine feasible alternative bridge types. As part of other Phase 1 work, tunnel alignments and a continued ferry service are also being considered in addition to bridge crossings.

This geotechnical report presents the results of our review of existing literature and available geotechnical engineering studies. The purpose of the literature review was to identify the likely subsurface conditions at various potential bridge crossing locations. Preliminary geotechnical engineering studies were then conducted to develop conceptual foundation design recommendations and assist the design team in narrowing the crossing alternatives to the more promising alignments. It was our intent and per your request to extrapolate the existing data into offshore areas and to develop conceptual recommendations recognizing that site-specific information is limited. The recommendations we have provided should therefore be recognized as being conceptual for use in planning and feasibility studies for the project. Included in this report are a description of the site and project, an outline of literature reviewed, interpretation of the geology and subsurface conditions, and conceptual foundation design recommendations developed from the preliminary engineering studies.

1.2 Authorization

This work was performed in general accordance with our Subconsultant Agreement dated July 16, 1999. Authorization to proceed with this preliminary study was provided in HDR Engineering Inc.'s letter dated December 8, 1999. Mr. Larry Kyle, HDR project manager for the design, approved the scope of our work.

2.0 SITE AND PROJECT DESCRIPTION

The Gravina Access Project is a special project funded under the Transportation Equity Act for the 21st Century (TEA 21) to improve transportation access from the City of Ketchikan on Revillagigedo Island to Gravina Island. The intent is to provide access to both the Ketchikan International Airport on Gravina Island as well as other parts of the island. Access between the two islands is currently provided via a regular ferry service.

2.1 Site Conditions

Ketchikan is located in Southeastern Alaska at the extreme southern tip of the Panhandle. This coastal region of Alaska is accessible only by air and water and is characterized as a typical wet marine environment with one of the highest annual rainfalls in Alaska. This part of Alaska is often referred to as the Alexander Archipelago, which reflects a group or chain of islands. Figure 1 presents vicinity maps of Alaska and the Panhandle area. Tongass Narrows borders the City on the west and separates it from Gravina Island. A site map is presented in Figure 2.

The Tongass Narrows is a long narrow waterbody that is orientated approximately northwest by southeast and is approximately 11 miles (19 kilometers (km)) long in the study area. As shown in Figure 2, channel width varies from about 1,500 feet (500 m) in the vicinity of the airport to 6,500 feet (2,000 m) near Refuge Cove and at the north end of Pennock Island. Flow within the Narrows is generally from the southeast to the northwest during flood tides and some weak ebb tides and reverse during strong ebb tides. The velocity ranges from less than 0.3 miles/hour (0.5 km/hour) to about 1.6 miles/hour (2.7 km/hour).

To the south, Pennock Island is approximately ½ to 1 mile (1 to 2 km) wide by 3 miles (5 km) long and separates Tongass Narrows into an east channel and a west channel. Access to Pennock Island is by private boat or floatplanes. As shown in Figure 2, the west channel to Pennock Island varies from 1,000 to 2,000 feet (300 to 600 m) in width while the east channel to Gravina Island varies from 1,500 to 2,600 feet (450 to 800 m) in width. Typical water depths in the channel at mean lower low water range from 80 to 200 feet (25 to 60 m) between Refuge Cove and Gravina Island and 50 to 200 feet (15 to 60 m) in the West Channel and 65 to 150 feet (20 to 45 m) in the East Channel.

Gravina Island is largely undeveloped and covered with lush vegetation except for the Ketchikan Airport, which is located on the eastern side directly across from the City of Ketchikan. Ferry service presently provides access to the airport. Gravina Island protects Ketchikan from the Clarence Strait.

2.2 Geography

The localized topography consists of steep mountains plunging into Tongass Narrows. Due to the steepness of the mountains near the shoreline, much of the city is restricted to the about 3 miles (5.5 km) wide corridor along the coast. There is additional development to the south in the town of Saxman and to the north in Ward Cove. Altitudes reach about 1,000 feet (300 m) within 2,600 feet (800 m) of the Narrows and near-vertical cliffs are located along much of the coast. Figure 3 presents pertinent geologic and geographic site features.

Tongass Narrows in part is a glacially scoured fjord. Bathymetric contours indicate a relatively flat floor with water depths ranging from about 100 to 200 feet (30 to 60 m). The deeper water areas along the Narrows are generally reflected by the 130 feet (40-m) contour lines shown in Figure 3. The diurnal tidal range is about 15.4 feet (4.7 m). The highest tide on record is 20.7 feet (6.3 m) and the lowest tide is -5.2 feet (-1.6 m). The shoreline varies from beach type deposits of mud and sand to steep rocky areas as illustrated by the gray and green bands in Figure 3. Much of the coastline on Revillagigedo and Pennock Islands is rocky. Gravina Islands eastern side has several mud and sand coastal areas. Within Tongass Narrows, the bottom conditions range from muddy substrate to rocky pinnacles.

The climate is predominantly cool maritime. There are mild winters, cool summers and very heavy precipitation. Average annual precipitation is about 152 inches (386 cm). Strong winds are common especially in winter and cloud cover is persistent. Average annual temperature is about 46 degrees Fahrenheit with a mean January temperature of about 35 degrees and a mean August temperature of almost 59 degrees. The area is considered a cool rainforest. Vegetation is heavy and dense consisting of western hemlock, Sitka spruce and Alaska red cedar. Treeline is about 1,500 to 2,000 feet (450 to 600 m) above sea level with sedges, mosses, and alpine forbs and shrubs directly above. Many areas on lower slopes are subject to rapid surface runoff or spring seepage and in valley bottoms the surfaces are covered with mosses, sedges and other plants typical of muskegs.

2.3 Project Description

The purpose of the Gravina Access project is to construct a hard link such as a bridge or bridges or tunnel between Gravina Island and Revillagigedo Island in Ketchikan, Alaska. Previous studies have identified and discussed possible crossing corridors along the Tongass Narrows. Figure 4 presents locations of eight previously proposed bridge crossings. For the overall Phase 1 study, the eight proposed alignments and newly defined crossings will be studied with the objective of identifying the most promising single corridor. For this report, the eight previously proposed crossings were studied as part of three general crossing regions: a north area near Ward Cove, a central region near the town of Ketchikan and the Airport, and a southern zone that involves crossing the east channel of Tongass Narrows to Pennock Island and the west channel to Gravina Island.

As part of this Phase 1 work, the design team will also evaluate several types of bridge structures including concrete bridge types such as an arch bridge or cable-stayed bridge, or steel bridge types such as box girder, tied arch, plate girder or truss structures. The bridge must be designed to accommodate cruise ships with a minimum 200-feet (60-m) clearance above the high water line and satisfy air space clearance, if near the airport. For conceptual design purposes, based on discussions with the design engineers, we have assumed that maximum pier loads will be roughly 30,000 to 50,000 kips (133,000 to 222,000 KN).

3.0 PRIOR EXPLORATIONS AND TESTING

Various groups have studied the idea of constructing a link between the two islands since the early 1970's. In addition to the studies directly related to constructing this link are various engineering projects located within Ketchikan and Gravina Island prepared by Shannon & Wilson and other engineering consultants. Additional information was obtained from the United States Geological Survey (USGS) and the National Oceanic Atmospheric Administration (NOAA). A detailed reference list used for this study is presented at the end of this text.

Some of the more significant "project specific" reports that were reviewed include the October 1994 document prepared by Alaska Department of Transportation and Public Facilities (DOT&PF) titled "Ketchikan Alaska Tongass Narrows Crossing, Preliminary Draft, Environmental Impact Statement" and the 1981 and 1982 Tongass Narrows Study prepared by Emps Sverdrup. Figure 4 is a referenced map of the study area showing the locations where subsurface data existed in prior reports or where bottom/slope conditions could be inferred. As shown in this figure, information regarding the northern section of the project is scarce. Many of the prior studies are focused in the central region with a few addressing general conditions in the southern and northern sections of the project. There is essentially no drilling data in the middle or deep parts of the channel. Appendix A presents previous boring logs and associated site maps. Seismicity studies were conducted by Shannon & Wilson for the Ketchikan Federal Building, and the findings were used in our preliminary studies.

The conceptual foundation recommendations presented are based on our understanding of local geologic processes and our interpretation of the subsurface conditions given the limited data obtained. These recommendations are provided for the conceptual evaluation of the bridge foundation support. They will be refined once site-specific subsurface data becomes available.

4.0 GEOLOGICAL CONDITIONS

The regional geology and tectonic elements are presented as a basis for extrapolating shoreline conditions offshore and for developing conceptual foundation recommendations. With limited existing subsurface information, the geology of the region, as reflected by surface exposures and published maps, provides preliminary information as to the likely conditions that could be encountered. In addition, seismicity maps and tectonic elements provide the background for understanding the regional seismicity and structure.

4.1 Regional Geology

Southeastern Alaska, like much of the southern portion of Alaska and the Pacific Coast of North America is composed of a series of microplates or terranes. These terranes are large crustal blocks that have their own internal structural and stratigraphic history. Deciphering the geology within and between the terranes has been a major focus of the science in the past 30 years. The structure between terranes tends to be major lineaments or faults. In the region of the project there is the Alexander Terrane and the Taku Terrane. Figure 5 shows that the Alexander Terrane lies to the west (or outboard) of the Taku Terrane. These terranes are separated by a package of rocks termed the Gravina Belt. To the east of the Taku Terrane is the Coast Mountains, which form the border between southeastern Alaska and Canada. Figure 5 also shows that the relationship between the various terranes and the Gravina Belt is still controversial and the exact forms of deposition and tectonic histories still have many questions.

The Alexander Terrane is composed of stratified metamorphic and plutonic rocks of Paleozoic through Middle Jurassic age (580 to about 170 million years old). The most widespread rocks within this terrane are submarine debris flow deposits, shallow-marine carbonate rocks and conglomerates (Brew, 1996). The Taku Terrane is a poorly understood group of rocks composed of deformed and metamorphosed strata. Rocks range from phyllites, basalt flows, slates, and marble (Berg, et al, 1988). The Gravina Belt is a series of marine argillite, greywackes, volcanics, and conglomerates. By the end of the Cretaceous (about 65 million years ago) the terranes and rocks were in roughly their present day locations. The Coast Mountains rapidly uplifted after this time. The rocks within the project area belong mainly to the Gravina Belt.

4.2 Regional Tectonics

An intricate network of reverse, normal and strike-slip faults dissects southeastern Alaska. This in part is due to the number of terranes that comprise the region. On the west, the area is truncated at the North American continental margin by the Queen Charlotte-Fairweather fault system shown in Figure 5. This system is known to be an active right-lateral fault with large displacements. The location of this fault, which represents the plate boundary between North America and the Pacific Plate, is approximately 100 to 110 miles (160 to 175 km) southwest of Ketchikan, off the west coast of the Alexander Archipelago.

Another major fault system is the Chatham Strait fault, which was active in the Tertiary (2 to 65 million years ago) having offset rocks as much as 95 miles (150 km). This fault truncates in the south into the Queen Charlotte-Fairweather fault. A third major strike-slip fault, closer to Ketchikan, is the Clarence Strait fault. It has approximately 9 miles (15 km) of displacement.

Figure 6 presents the locations of the faults and major earthquakes in Alaska and the region. The upper figure of 2,400 epicenters reflects that Alaska is a highly seismic area, however, both figures show that the Ketchikan area is considerable distance from major seismic activity. The typically shallow depths of the epicenter in the region (the orange dots in Figure 6) and the large distances suggest small ground accelerations at the site.

Large earthquakes in the region include the August 21, 1949, Queen Charlotte Islands and July 30, 1972, Sitka events. Both appear to be located in and associated with the plate transform boundary. The 1949 magnitude 8.1 (Rogers, 1983) earthquake was located approximately 100 miles (160 km) southwest of the site. The 1972, magnitude 7.6 event was located approximately 110 miles (180 km) northwest of the site (Coffman and von Hake, 1974). While no information was found indicating the intensity of the ground shaking felt from the 1949 event in the area, reports from the 1972 event indicate that Ketchikan experienced Modified Mercalli Intensity V ground shaking. This is probably the strongest ground shaking event felt at the site in its short historic record and is generally consistent with peak ground accelerations on the order of 0.03g to 0.04g (Shannon & Wilson, 1995).

4.3 Local Bedrock Geology

The local geology is dominated by bedrock of the Gravina Belt and Quaternary glaciation. The bedrock throughout much of Ketchikan is composed of metamorphic rocks that are highly deformed phyllites and schists. Figure 7 presents a simplified geologic map for the region. The phyllites and schists are shown as MzPzms, MzPzmv, and KJgv. The MzPzms rocks are metamorphosed sedimentary rocks with a Mesozoic to Paleozoic (65 to 580 million years old) age. The MzPzmv rocks are metamorphosed volcanic rocks, generally basalt lavas from the same time frame as the MzPzma rocks. The KJgv rock packages, which cover most of the eastern portion of Gravina Island, are Cretaceous-Jurassic (65 to 170 million years old) andesites to basaltic volcanics that have been metamorphosed. These rock groups have been subjected to moderate pressures and temperatures during mountain building and terrane accretion processes. This range of pressures and temperatures defines the greenschist facies of metamorphism. This facies typically give the rocks a greenish-grey appearance (due to the minerals that are created) and foliations or thin weaken planes are produced due to the pressures. The foliations generally trend northwest with moderate to steep dips to the northeast with local variations as shown in Figure 3. Although these rocks are basically the same in composition, degree of weathering and hence competency are affected by the foliation orientation and quantity. Increased foliations tend to produce small platy fragments upon weathering and low competency.

In addition to the phyllites and schists are igneous rocks of several varieties. Most of the rocks are diorites, which are darker and have more iron and magnesium in their composition than granites, but have similar textures. The diorites are shown in Figure 7 to outcrop in the northwestern part of Revillagigedo Island. Kpg is the unit designation and indicates a quartz rich diorite of Late Cretaceous age about 82 to 97 million years old (Brew 1996). There is also a gabbro, a coarse grain equivalent to basalt magmas, mapped in places as Tgb. The age for this gabbro is Tertiary Epoch and is about 23 to 25 million years old (Berg et al, 1988). It is located near downtown Ketchikan and a few outcrops to the north. On Gravina Island is an outcrop across from Ward Cove. These igneous rocks tend to be hard, resistant to weathering, and a good source of aggregate for engineering structures. Larger and more durable blocks can also generally be created from both the diorite and the gabbro for riprap.

4.4 Local Unconsolidated Deposits

Glaciers advanced over the region during the Pleistocene (~10,000 to 2 million years ago). The last period of glaciation ended approximately 13,000 years ago. Ice thickness was on the order of 3,900 feet (1,200 m) causing depression of the land and subsequent rebounding (Lemke, 1975). The land emerged above sea level as evidence by elevated terraces, U-shaped valleys, and deeply scoured fjords. The glaciers laid down a deposit of undifferentiated drift. Elevated marine deposits are the result of deposition during glacial times and subsequent uplift of the land. In addition to these unconsolidated deposits, there are modern beach deposits, stream deposits including alluvium and fan-delta deposits, muskeg, and colluvium deposits.

Undifferentiated drift deposits will typically consist of till and other glacially deposited material. These materials are composed of mixtures of clay, silt, sand and gravel. Cobbles and boulders occur occasionally. The deposits range from bluish gray to brown and are compact (generally very dense). Dames and Moore, 1972, tested a sample from near the Ketchikan Airport. The grain size was reported to be 36 percent sand, 35 percent silt, 19 percent clay and 10 percent gravel. The shrinkage limit was 15, the liquid limit was 19, the plastic limit was 15 and the plasticity index was 4. Based on this information this material would be classified according to the Unified Soil Classification System as a slightly gravelly, sandy, clayey silt with low plasticity. Till deposits and other undifferentiated drift deposits typically have a wide range of grain size and may be classified as sands, gravels, silts or even clays depending upon the exact mechanism of deposition.

These glacial deposits are reported in the literature with varying thickness estimated from observations and limited exploratory drilling. Excluding infilling in fault areas, thicknesses are believed to range from about 10 feet (3 m) to about 50 feet (15 m) depending upon where in the Tongass Narrows and the islands the exposures or drilling was performed. As indicated by the references in Figure 4, this material is generally reported in the central part of the project. Where these till deposits are observed on land, it is directly underlain by bedrock.

Elevated marine deposits extend to an altitude of approximately 220 feet (68 m). These deposits by their nature are above sea level and may not affect the project except in placement of approach corridors and abutments. These deposits are thin on the order of less than 10 feet (3 m) within the central portion of the project but may reach about 20 to 40 feet (6 to 12 m) thick in the north and south areas. The deposits consist of poorly graded, fine-grained sand with some gravel and in a few places silt. The deposits that are observed overlie bedrock.

Fan-delta deposits are present along the coastline and offshore. They may directly affect the bridge supports for the crossing. These deposits have been laid down at the mouths of Ketchikan, Carlanna and Hoadley creeks as well as other smaller streams. Figure 3 shows these deposits in purple. The outer edges extend into Tongass Narrows and at the mouths of the larger creeks form benches within the narrows. The deposits consist mostly of sand, gravel and boulders. They are expected to become finer-grained seaward. Based on limited data, the deposits are believed to range from 15 feet (4.5 m) thick to about 50 feet (15 m) thick. The fan-delta deposits at Carlanna Creek in the central portion of the project have been reported to be between 20 to 100 feet (6 to 30 m) thick (Emps Sverdrup, 1981) and extend out into the Tongass Narrows approximately 1,800 feet (550 m) (Dames & Moore, 1972). These sands and gravels are generally loose to medium dense and saturated. They appear to lie on seaward-sloping bedrock surfaces and are overlain by fill in the town. On Gravina Island, Figure 3 reflects that there are delta deposits associated with streams that flow into Tongass Narrows as well. These deposits appear to be smaller in size than those associated with the larger streams on Revillagigedo Island.

Other shoreline surficial deposits consist of modern beach sediment, stream alluvium, muskeg, and colluvium. These deposits in general may only impact the project at the approaches/abutment areas. Most of these deposits are thin typically less than 10 feet (3 m) thick. Modern beach sediment consists of sands and gravels that are usually saturated at least part of the time. Stream alluvium will probably not impact the project except where major streams have created the deltas as discussed above and shown in Figure 3. Muskeg or peat deposits are highly compressible and are generally wasted if encountered during construction and if they are not too thick. Colluvium, material that has moved downslope, consists of fragments of decomposing bedrock, but may also include unconsolidated sediments.

Fill deposits along the waterfront in the city are common deposits that may affect the project at abutments and approaches. The fills along the waterfront consist of a variety of materials including sand, gravel and shot rock. In addition, there may be deleterious materials such as concrete slabs, muskeg, and other organic debris. The fill is generally believe to be loose to medium dense and consists of at least in part hydraulic fill material derived from dredging operations.

5.0 SUBSURFACE CONDITIONS

Offshore exploratory data is limited and therefore the composition of sediments is difficult to determine. NOAA charts indicate anticipated bottom conditions. Figure 3 presents our interpretation of the bottom conditions based on the NOAA charts and additional information from the references in Figure 4. Tongass Narrows channel bottom is generally believed to consist of varying depths of soil overlying bedrock. Bedrock is at or near the surface in many places and several projections cause navigational hazards. Near shore for about 1,000 feet (300 m) are a variety of soils including the undifferentiated glacial drift, fan-delta deposits, and silts and sands of beach deposits. From the Dames & Moore Pipeline Crossing report, 1972, a bottom "pavement" or "erosion protector" of the coarser sediments has formed in some areas. This pavement is estimated to range from about 5 feet (1.5 m) thick to about 20 feet (6 m) thick. It consists of gravelly/cobbly bedrock fragments from the phyllite and platy schists. In some places, a thin unit of softer sediment may lie between the pavement layer and glacial drift. It is believed that the underlying glacial drift material is highly compacted soil, which predominately overlies bedrock within parts of the channel.

For the Phase 1 alternative alignment studies, four subsurface cross-sections have been developed and are presented in Figures 8 and 9 to represent our rough estimate of the anticipated depths and possible subsurface conditions that could be encountered in the various water crossing corridors. Figure 2 shows the locations of these profiles. Based on the limited deep water exploratory information within the Tongass Narrows itself, it should be realized that these profiles are conceptual at this stage and will have to be refined with follow-on exploratory studies.

5.1 Soils

A few of the previous studies advanced borings within a specific project area. These included, referring to Figure 4, the Century/Quadra report (A), Shannon & Wilson 1998 report (F), and Shannon & Wilson 1995 report (H). The Dames and Moore reports (B and C in Figure 4) conducted sampling by using divers to retrieve samples from the floor of Tongass Narrows. Appendix A provides the borings logs and site maps for some of these projects.

The approximate profiles (Figures 8 and 9) reveal gentle and flat lying slopes, although in shallow rock areas, outcrops and pinnacles of rock may reflect a more irregular structure locally. The profiles also indicate that at most crossings, one to three types of soils overlying bedrock may be encountered. An inferred fault is shown in the bottom of the Tongass Narrows and in the channels on either side of Pennock Island. While this fault zone is shown on Figure 3 and the profiles (Figures 8 and 9), the exact location and extent of gouging can only be illustrated as an interpretative band of ground disturbance. As indicated in Section 5.3 the fault is believed to be inactive and covered with dense or hard glacial drift and possibly other sediments.

In Profile A-A' in the East Channel to Pennock Island and Profile D-D' to the north near Ward Cove (Figure 8), bottom soundings and limited explorations suggest a thin veneer of sediment overlying bedrock. The layer of sediment is probably less than 10 feet (3 m) and could consist of loose to medium dense sands, silts, soft mud and shells. Soils encountered in offshore borings drilled at the US Coast Guard Station (F and G in Figure 4) on the north side of the East Channel consisted of medium dense to very dense, gravelly silty sand to sandy silt. This unit extended from the surface to approximately 8 to 13 feet (2.5 to 4 m) below the mudline. A green schist was encountered below the sediments.

Profile D-D’ was developed from geophysical profiles. The presence of rocky islands and pinnacles within Tongass Narrows along this alignment indicates that bedrock is shallow with a thin layer of sediments either overlying the bedrock over the entire bottom or only in depressions within the bedrock. A cobbly layer a few meters thick could be encountered between the soft sediments and the bedrock. These cobbles would likely consist of weathered and broken pieces of the schist bedrock. A cobbly, irregular weathered bedrock surface of a meter or more is common with schist due to the spacing and degree of foliations producing fragments.

In the West Channel of Profile A-A’ (Figure 8) and in Profile C-C’ in Figure 9, our review of conditions suggests that these sites contain a thicker layer of soil sediment overlying bedrock. The sediments could be subdivided into a less than 10 feet (3 m) layer of loose, surficial sediments overlying denser glacial drift deposits about 55 feet (17 m) thick. The surficial deposits consist of the sand, broken shells and mud typical of bottom sediment conditions. The glacial drift deposits are inferred from the local geology. Glacial drift deposits typically exhibit a wide range of grain sizes from silts and sands to gravels and cobbles and as indicated previously are very dense or hard.

In Profile B-B’, Figure 9, near Carlanna Creek, our review indicates that a thick layer of fan-delta deposits may be present near the northeast shore of Tongass Narrows. The fan-delta deposits are generally loose to medium dense, sands and silts. Near the shoreline, the gravel or coarser particles in the deposit may increase in quantity. In our profile, the fan-delta deposits are shown overlying denser glacial drift but could also be in direct contact with bedrock. This thick deposit of sediments, is roughly 80 to 100 feet (25 to 30 m) thick. The soils from the borings reported in Century/Quadra report (A in Figure 4) just southeast of Carlanna Creek, were medium dense to dense, gravelly sands encountered to about Elevation –55 feet (–17 m) based on mean lower low water (MLLW).

5.2 Bedrock

Bedrock that underlies the soil in each of the profiles would likely consist of a moderately hard to hard, highly foliated phyllite to schist. At the US Coast Guard (USCG) Breakwater site, the bedrock consisted of green, moderately hard, schist with moderate to steeply bedded fractures and thin quartz inclusions. Dip on the foliations was about 67 degrees.

The Rock Quality Designator (RQD) is a measurement of the percent of recovery of rock core greater than 4 inches (10 centimeters). An RQD of 100 percent generally indicates that the rock is fairly massive and without fractures or weak bedding planes. An RQD of zero typically indicates that the rock may be soft or thinly bedded/foliated and large intact samples can not readily be recovered by coring. Low values thus reflect mass behavior that is only a fraction of the engineering properties of that measured when testing a more intact core piece. The rock RQD for the schist at the USCG facility ranged from about 6 percent to 75 percent and did not necessarily increase with depth. In two of the borings, it was difficult to determine if bedrock or very dense soils were encountered since the sampler encountered refusal when driven, but the drilling was able to be advanced. For planning purposes, we have developed our conceptual foundation recommendations assuming that the rock at the various bridge sites would posses an average RQD value of about 25 percent.

Figure 3 presents the bedrock that is observed at the surface within the project area. Where strike and dip measurements were obtained from surface exposures, they are summarized on Figure 3. In general the phyllite to schist strikes to the north and has foliations that dip between 35 to 90 degrees. As indicated previously, the RQD's are generally low in these types of rocks.

At the Swan Lake Hydroelectric Project located on Revillagigedo Island, unweathered bedrock similar to the phyllite and schist was tested. Unconfined compressive strengths of 7,000 to 13,400 pounds per square inch (psi) (48,300 to 92,400 kilopascals (kPa)) were obtained on intact specimens. A modulus of elasticity of 5.2 million psi (36 million kPa) and Poisson's ratio of 0.2 (likely intact values) were used in the design. The RQD for this rock ranged from 40 to 90 percent, with an average of 71 percent. Based on drilling explorations nearer the site and the close proximity of the site to an inferred fault, the bedrock within the project area appears more foliated and thus is probably not as competent as rock at Swan Lake. Intact unconfined compressive strength for our conceptual recommendations presented in this Phase 1 report is taken as 5,000 to 7,000 psi (34,500 to 48,300 kPa).

The reviewed literature indicates that quartz diorite bedrock outcrops between Carlanna Creek and Peninsula Point. This rock would typically be massive and not have foliations. RQD's in this rock would generally be much higher than in phyllites and schists (generally greater than 50 percent). The granitic rock itself is also believed to be much harder than the schists and phyllites with compressive strengths of 10,000 psi (69,000 kPa) or greater.

5.3 Local Faulting

Lemke (1975) indicates that no evidence of faulting during Pleistocene of Holocene (the last 2 million years) time has been found in the Ketchikan area and that the high-angle faults within the region are of middle Tertiary age (less than about 35 million years old). Berg and others (1988) suggest that the strike slip faults in the area have been covered by Tertiary and Quaternary Volcanics. As such, these structures would seem to be old and inactive.

As indicated in Figure 3 and the profiles Figures 8 and 9, there is an inferred, 30-mile (48-km) long strike-slip fault located within and striking the same direction (northwest) as the Tongass Narrows. The channel itself may have been partially formed as a result of this faulting. Berg and others indicate that this may be the youngest strike-slip fault in the immediate region. It appears to have offset an Oligocene or Miocene (about 25 million years old) gabbro pluton to the right by about 10 miles (6.5 km) (Figure 7). There is little indication of whether the structure is currently active or when movement last occurred on it. Based on the lack of historically recorded seismicity in the Ketchikan area (Figure 6) and the inferred inactivity of similar structures in the area, it is our opinion that this fault may be considered relatively inactive.

6.0 FOUNDATION CONSIDERATIONS

6.1 Foundation Types

A highway bridge structure that must span roughly 3,500 feet (1,000 m) of water and provide 200 feet (60 m) of clearance for ships will require a number of offshore piers as well as two shoreline abutment structures and/or approach fills. In the offshore areas of the narrows, foundations to support the bridge columns would penetrate up to 200 feet (60 m) of water, and various thicknesses of loose and dense soils, and derive foundation support in bedrock or the dense glacial soils. As summarized in Section 5, the expected thicknesses of soil sediments is generally thin enough where in most instances, rock could be reached and must be penetrated to develop the needed vertical and lateral foundation support.

Since driven piles will not readily penetrate the moderately hard rock at this site or derive the needed lateral or uplift support in some areas, most bridge piers will likely have to be supported on one or more large diameter piers with sufficient penetration into the rock. Generally they are envisioned to have diameters in the order of 4 to 12 feet (1.2 to 3.6 m). Because much of the construction work must be carried out on floating or jacked-up platforms and in deep water, construction of the piers will likely be carried out in the wet or without dewatering using one of the two "wet" construction methods in common use. These methods are described in Section 6.2.5.

As the bridge extends out from the shorelines, foundations for abutments can vary depending upon local topography and subsurface conditions. If competent soils or rock are present, either the large diameter piers or spread footings can be used. In either case approach fill embankments are suitable, however, for footings the seaward face and fill side slopes should be protected from toe scour and frost heave with riprap or abutment faces and wing walls.

If the soils underlying the abutments are soft/loose and compressible as in the delta areas, shown in Figure 3, only the piers should be considered and possibly without large approach fills. Approach fills will induce embankment settlements and downdrag forces on the piers as the fill weight consolidates the underlying soils. These settlements and downdrag forces can be minimized by placing the embankment fill prior to the pier construction. Footings on compressible foundation soils would also settle under fill and bridge loads.

The following sections present first the conceptual evaluation of design capacities for the piers followed by a brief discussion of construction considerations. Conceptual design recommendations are then presented for the abutments, for the approach fills, and retaining structures. Seismic considerations are discussed at the end of our recommendations.

6.2 Drilled Piers

In evaluating the cost of various water crossing routes, key factors that must be assessed are the approximate size and number of piers at each bridge column bent and their lengths (or embedment depths). This requires both a knowledge of the general soil thicknesses and depth to rock in each crossing area as well as the general bridge design loads. Our preliminary estimate of depth and nature of the various soils are generally outlined in the profiles shown in Figures 8 and 9 and summarized in Section 5.1. With this information and the preliminary evaluation of design capacities provided below, the pier diameters, approximate embedment lengths and number of piers per bent can be assessed at each selected crossing alternative.

6.2.1 Pier Bearing Capacities

Depending upon the crossing location being evaluated, the two likely bearing strata for the 4 to 12 feet (1.2 to 3.6 m) diameter piers are the bedrock and the glacial drift. Glacial drift areas could include the inferred fault zones noted in Figures 8 and 9 and a crossing near Profile C-C' (Figure 2 and 9). Based on our preliminary study of the geology and available subsurface information we recommend that drilled piers be extended through any soft materials (soft sediments and loose fan-delta soils) and into the underlying bearing strata. Piers could be sized to carry column compression loads using a recommended allowable soil bearing pressure of 40 tons per square foot (tsf) (3,830 kPa) if founded in the rock and 20 tsf (1,900 kPa) if founded in the glacial drift. The recommended bearing values may be increased by 1/3 for short term seismic and wind loading. These preliminary bearing pressures assume that the piers will extend a minimum of 6 feet (1.8 m) into the rock or at least 15 feet (4.6 m) into the glacial drift. Figure 10 presents graphs that can be used to estimate the depth of embedment based on the amount of load each pier needs to carry. Three soil conditions were considered in the development of the graphs; embedment in bedrock only, embedment in the glacial drift only, and embedment through the glacial drift into the bedrock.

As indicated in Section 5, we assume that the rock would likely be highly foliated, moderately hard, unweathered phyllite or schist with an average RQD of 25 percent or greater. The glacial drift is assumed to consist of a very dense silty sand and gravel.

6.2.2 Pier Skin Friction

Additional compressive and uplift capacities can be derived in skin friction between the shaft soil/concrete interface. We recommend that the skin friction be estimated assuming an allowable unit friction of 2 tsf (191 kPa) (in rock) and 0.5 tsf (48 kPa) (in glacial drift) acting along the entire outside circumference of the shaft. These values assume the concrete is allowed to bond directly to the soil/rock side walls (i.e. the hole in the denser soil and rock stratum will stand open unsupported or that any casing used will be withdrawn as concrete is tremied into the pier. Skin friction from any overlying softer sediments will be relatively small and therefore has been ignored. The graphs in Figure 10 were developed based on both the bearing capacity and the skin friction of the pier embedment material.

6.2.3 Pier Lateral Resistance

Piers will be required to resist lateral forces such as those that may result from wind or seismic loading or impacts from ships. The maximum horizontal load that can be applied at the top of the pier foundation is limited by the maximum horizontal reaction that can be mobilized in the soil or rock. The resistance to lateral loading in a pier depends on the size, stiffness, spacing of the individual piers, the degree of fixity at the pier top, and the amount of deflection the pier can tolerate. The lateral resistance analysis can be performed after preliminary design information and site-specific conditions become available.

The depth of embedment of the piers can be estimated based on the anticipated axial and lateral loads anticipated for each pier. The point of fixity is defined as the approximate depth below the mudline at which there is essentially no deflection of the pier. The following are approximate point of fixities for the assumed soil/rock conditions that could occur:

MODEL	POINT OF FIXITY	
Bedrock only	20 to 25 feet	(6 to 7.5 m)
Bedrock & Glacial drift	30 to 35 feet	(9 to 10.5 m)
Glacial drift only	25 to 30 feet	(7.5 to 9 m)

The embedded length of a pier to achieve lateral resistance can be estimated as the distance from the mudline to the point of fixity plus a reserve length of pier to achieve a reasonable factor of safety.

6.2.4 Pier Settlements

Based on the recommended allowable bearing and skin friction values presented in the previous sections and our past experience, the total settlements of the piers would be on the order of 1/2 inch (1.2 cm) to 1 inch (2.5 cm). In our opinion, the estimated settlements would occur elastically as the loads are applied such that long term settlements would be small.

6.2.5 Pier Construction

Most offshore piers or cassions are constructed in the wet or without dewatering using two "wet" methods; large hole rotary shaft drilling or drop caissons. In either case, work is generally accomplished from a floating barge or one supported by long legs (jackup barge) to the bottom of the channel. The latter is often preferable so that rough water and currents would not prevent work from being conducted.

For the first method (drilling), an oversize steel casing could first be set through the channel bottom of muds to firm support and to seal off channel water. The pier could then be drilled through this casing with mechanical boring equipment for the full depth. In general, the shaft is advanced by a rotating or chopping bit or series of bits. The final diameter may be attained in one pass of the equipment or by reaming with a series of drill units of increasing diameter. Drill cuttings are removed from the bottom by a reverse circulation procedure using either water or drilling mud as the circulating fluid. Under the right combination of equipment and geologic conditions, rotary drilling in both soil and rock has a generally high rate of penetration.

The glacial soils will have both caving and gravelly conditions, which will tend to slow and complicate drilling operations. With an elevated platform, it is possible to maintain a positive head or internal pressure inside the casing at all times, such that flow of drilling fluid at the face and sidewalls will always be outward. This, together with the use of drilling mud can be used to minimize caving. Special rotary equipment with large diameter hollow drill rods is available for removing fairly large gravel and cobbles. If boulders are encountered, they can most likely be removed using a clamshell. Follow-on exploratory work should better define the potential for hole caving, rock integrity and the presence of boulders.

The major disadvantage of drilling is problems with deviation of the boreholes. Borehole deviation can occur due to variations in soil such as boulders and steeply dipping bedding planes in rock. The use of drill collars and controlling the sequence of drilling and casting piers in a group can help minimize this condition. Other means of maintaining hole stability include advancing casing simultaneously as the hole is drilled.

Once the hole is advanced through the soil and rock medium to the desired base elevation, a steel reinforcement cage can be lowered into the hole. The void space can then be filled with concrete using tremied grout procedures. If a casing is not withdrawn during grouting, either the assumed uplift capacity should be lowered or attempts made to grout the annular space between the borehole wall and the outside of the casing.

The second "wet" method includes installing the pier using drop caisson procedures. The caisson is usually composed of a steel cutting shoe and a heavily reinforced concrete shell built on top of the shoe. The caisson is usually advanced by dredging with a clam bucket from inside; as the lower edge is undercut, the caisson would sink. Additional lifts of concrete shells are then added to the top of the advancing caisson as it sinks. The concrete forms are usually constructed prior to the start of advancing the caisson. The major problems are keeping the caisson vertical and providing enough weight to overcome skin friction and continue to drive the caisson downward. Since adding weight is a cumbersome procedure, especially working overwater, it is our opinion that the cost of construction by this procedure could prove too great when compared against large hole rotary drilling methods.

6.3 Abutment Support

Abutment foundations along both shorelines may consist of drilled piers as provided previously or as spread footings depending upon location and types of subsurface conditions encountered. The applicability of these conditions is discussed in Section 6.1. The following sections present recommendations for footings, if appropriate.

6.3.1 Footing Bearing Capacities

Abutment footings or other footings used for retaining structures in the bridge approaches may be placed on a variety of soils as well as shot rock fills and bedrock. The magnitude of the bearing value for use in design is largely dependent on the density or stiffness of the underlying soils/rock at the chosen crossing point. Site specific explorations in these areas are therefore required.

For the alternatives studies, approximate allowable bearing capacities are provided below for footings placed on the different soils and bedrock likely to be encountered in the bridge approach areas.

Soil Type	Allowable Bearing Capacity
Bedrock	6 tsf (575 kPa)
Dense Soils/Shotrock fills	2 tsf (190 kPa)
Loose to Medium Dense Soils*	1.5 tsf (144 kPa)
Loose to Medium Dense Soils**	1 tsf (95 kPa)

* For flexible retaining walls (gabion, bin, sheet pile or reinforced earth walls) with fill heights of less than 25 feet (7.6 m) where greater settlements can be tolerated.

** For small rigid retaining structures with adjacent fill heights of less than 10 feet (3 m) where fill induced settlements will not exceed 1 inch (2.5 centimeters).

Figure 3 shows likely shoreline subsurface conditions for determining where pier or footings are appropriate at selected crossings. For those areas with thick delta deposits, such as Carlanna Creek, shown in Figure 3, the compressible unit is thought to be between 20 to 100 feet (6 and 30 m) thick. In these areas we recommend that abutments and other connecting structures be tentatively supported on piers.

Minimum dimensions for all footings should generally be 2 feet (0.6 m). Footings should also be embedded at least 2.5 feet (0.76 m) below the exterior grade for frost penetration. The above bearing values may also be increased by 1/3 for short-term loads. Total settlements for the above conditions would be on the order of 1 inch (2.5 cm) and generally within tolerable limits.

6.3.2 Abutment Earth Pressures and Lateral Resistance

Abutment walls below ground which support earth fills should be designed to resist horizontal earth pressures. The magnitude of the pressures is dependent on the method of backfill placement, the type of backfill materials, drainage provisions, surcharge loads, and whether the wall is allowed to deflect after placement of the backfill. If walls are permitted to deflect laterally or rotate an amount equal to about 0.001 times the height of the wall, an active earth pressure condition under static loading would prevail. For these conditions, it is recommended that walls be designed based on a pressure derived from a fluid with a density of 40 pounds per cubic foot (pcf) (560 kilograms per cubic meter (kg/m^3)). This is commonly referred to as an "Equivalent Fluid Weight" of 35 pcf (560 kg/m^3). For rigid walls that are not allowed to deflect at the top, an at rest earth pressure condition would prevail and an equivalent fluid weight of 55 pcf (880 kg/m^3) is recommended. These pressures assume that hydrostatic forces cannot develop behind the walls and will require a drainage system.

Lateral forces may be resisted by passive earth pressures against the sides of footings, exterior walls below grade and grade beams. For compact, saturated, granular soils, these resisting pressures can be estimated using an equivalent fluid weight of 200 pcf (3,200 kg/m^3). This value includes a factor of safety of 1.5 on the full passive earth pressure. Along with this value, the backfill around the footings and walls should be compacted to a density of at least 95 percent of the Modified Proctor maximum dry density.

Lateral resistance may also be developed in friction against sliding along the base of foundations placed on grade. As shown in Figure 11, these forces may be computed using a coefficient of 0.4 between concrete and soil.

The above earth pressures are increased whenever surcharge loads such as wheel loads are transmitted to the fill immediately behind the wall. For preliminary design, wheel loads may be assumed to be equivalent to a uniformly distributed load often taken as 2 feet (0.6 m) of equivalent fill weight or as shown in Figure 11 about 240 lb/sq ft (11.5 kPa).

6.4 Fill Considerations

6.4.1 Material Types

Approach fills leading to or part of an abutment structure should consist of granular fill material with a free-draining zone near the abutment face to minimize hydrostatic and seasonal frost pressures. Fill material forming the road's structural prism should be a free draining, well graded, non-frost susceptible (NFS) granular material within the top 3 feet (1 m) of the section. Within deep fills, soils with a higher percentage of fines may be used in the deeper sections of the fills, provided the soil can be worked and compacted to the specified density. Shot-rock fill may be used provided the rock does not degrade over time. Local fills using schists and phyllites are common along the waterfront. A better quality rock such as the gabbros usually is more resistant to wear and weathering forces; however its use as fill may not prove practical.

In order to limit frost problems with the abutments, we recommend that abutment and wing wall foundations be embedded at least 2.5 feet (0.76 m) below the lowest exterior grade and that backfill within 4 feet (1.3 m) of the abutment wall consist of a free-draining, NFS granular soil. A backfill material such as Select Material, Type A, according to the Alaska DOT&PF Standard Specifications for Highway Construction would meet this requirement.

The ground around the abutments should also be graded and sloped to drain so that water pressures cannot build up behind or below the walls. Weep holes should be placed on 12 feet (3.6 m) centers across the vertical wall faces to drain any excess water that may collect behind abutment or wing walls.

6.4.2 Fill Placement

Above the groundwater table or higher tidal fluctuation zones, fill construction can be accomplished using conventional dry earthwork placement and compaction procedures. In the lower tidal fluctuation zones, similar placement procedures and similar specified materials can be used, however, the contractor must be prepared to work at low tides so that suitable compaction can be achieved in the dry. When working near water areas, a coarser well-graded more select granular fill or shotrock material should be used as the preferred material as it simplifies placement and compaction, resists erosion by the tide moving in and out, and provides better slope drainage to relieve the slopes and abutment structures from extra hydrostatic pressures.

For the initial fill above the native soils, a two-foot thick lift of fill should be placed and compacted by passes with tracked equipment. The remainder of the fill should be placed in lifts no thicker than 1-foot (0.3 m) and compacted to 95 percent of the Modified Proctor maximum dry density. Filling should be carried out on thawed ground and the fill materials should be placed in a thawed condition.

Care should be exercised when compacting backfill against the abutment wall. To reduce temporary construction loads on the walls, heavy construction equipment should not be used for placing and compacting fill within five feet of the wall. Within this specified zone, small hand-operated equipment should be used. With this small equipment, it may be necessary to reduce lift thicknesses in this area to 6 inches (15 cm) or less to obtain the required compaction.

6.4.3 Embankment Slopes

Embankment fills will likely be constructed on each end of the bridge. Additional fills may be needed for the approaches. The stability of the fills is largely dependent upon the strength of the embankment and foundation materials, and the angle of the embankment slope.

If right-of-way is available to provide for these slopes, we recommend that where possible they be made 2 horizontal (H): 1 vertical (V). Locally steeper slopes of 1.5H:1V are feasible and can be used in most cases where space is limited. However, in this wet marine environment, slopes, if not properly compacted to the edges, can ravel and erode locally due to runoff until suitable vegetation can be developed or quarry spalls are placed on the face of the slope.

Where poor foundation soils exist and support deep fills, the stability of slopes (both statically and seismically) needs to be addressed further. Consolidation of these compressible sediments may also pose an additional design issue especially where fills must meet the bridge structure.

6.4.4 Slope Protection

Riprap covers the shoreline in developed areas both along Ketchikan's waterfront and in front of the airport on Gravina Island. It generally consists of the better diorite and gabbro, and typically is about 3 to 4 feet (1 to 1.2 m) in diameter on the surface. Riprap may also consist of the schists and phyllites that are more massive and have few foliations. Studies for the airport by Dames & Moore indicate that armor rock with a density of 165 pcf (2,640 kg/m^3), about 3 feet (1 m) in diameter and weighing about 2 tons (20,000 kg) would be suitable for shore protection with a design wave height of 10 feet (3 m). Quarries are located throughout Revillagigedo Island as shown in Figure 3. These quarries have produced the material used in many of Ketchikan's fills. Since both types of rock appear to be working well as riprap in these areas, we expect that similar material types and rock sizes will be specified to protect bridge abutments and approach fills that may be constructed in intertidal zones.

6.5 Retaining Structures

Some vertical retaining walls may be needed along the approaches to the bridge to accommodate a limited right-of-way. The portions of these walls supporting earth loads should also be designed to resist lateral earth pressures. The magnitude of the pressures is dependent on the same factors discussed for abutment earth pressures (Section 6.3.2). For most gravity walls, those pressures presented in Figure 11 would apply.

Several types of retaining structures are available depending upon site conditions and height requirements. Rigid retaining structures such as concrete retaining walls work well when the wall heights are generally small (less than 10 to 15 feet (3 to 4.6 m)) and foundation support soils are compact and can support the rigid structures with tolerable amounts of settlement. As the foundation soils become more compressible where larger settlements are likely, flexible walls are usually selected that can tolerate these deflections. Such walls are sheet pile walls, and gabion or bin/crib walls. Other similar walls also include internal stabilizing systems such as reinforced earth walls, anchored bulkheads, tiebacks, geotextiles or soil nailing. The most economical support system is usually established based on the height of the wall, the availability or cost of local materials (concrete, steel, and granular backfill), the subsurface conditions, the construction procedures, and space that is available to construct the wall.

For developing a preliminary wall design for most of the aforementioned gravity type structures, the earth pressures in Figure 11 would generally apply. These horizontal pressures may also used for preliminary design of cantilever or bulkhead structures. Generally the greater embedment depths would be required for cantilever structures while anchor bulkheads require the lesser depths of embedment. In considering sheet piles for this project, it should be recognized that bedrock is often at shallow depths and shotrock fills typically contains large rock fragments that are not easily penetrated with sheet piles.

6.6 Seismic Design Criteria

Regional probabilistic ground motion studies by Algermissen and others (1990) indicate that a peak ground acceleration (PGA) of 0.025g at the site would have a 90 percent chance of non-exceedence in a 50-year time interval (i.e., about a 500-year recurrence interval). By contrast, the 1994 Uniform Building Code seismic design provisions, which are also implicitly based on a 500-year event, indicates 0.20g for design in most of southeast Alaska. Regional probabilistic studies performed for Canada (Basham, 1983) indicate that PGA's of about 0.08g to 0.16g would have a 90 percent chance of non-exceedence in 50 years for neighboring sections of British Columbia.

The results of the seismic risk analyses conducted by Shannon & Wilson for the Ketchikan Federal Building suggest that a PGA of 0.10g would have approximately a 90 percent chance of non-exceedence in 50 years. Estimated ground motion is conservative because it exceeds our estimate of a maximum credible earthquake on the nearest, known active structure of 8 ¼ occurring on the plate transform boundary approximately 100 miles (160 km) from the site. This event would result in a PGA of about 0.06g at the site. The probabilistically determined 0.10g PGA would correspond to about a magnitude 6 to 6 ½ event located about 11 to 20 miles (19 to 30 km) from the site or a magnitude 7 about 20 to 25 miles (32 to 40 km) from the site. As previously discussed, there is the presence of an inferred fault in the Tongass Narrows, which is shown on Figure 3. If movement occurred on this fault, PGA's on the order of 0.5g to 0.6g may be expected. However, due to its apparent inactivity and the general inactivity in the entire region shown in Figure 6, it is our opinion that movement of this fault would not be the basis for determining design ground motion

6.6.1 Seismically-induced Geologic Hazards

Seismically-induced geologic hazards include liquefaction, differential compaction, slope stability, ground rupture due to fault movement, and flooding (i.e., tsunami and sieche). The potential for each of the hazards needs to be evaluated depending upon the final location of the structure. The fan-delta deposits and the loose fills are most susceptible to liquefaction, differential compaction and slope instability.

In general most of bridge corridors under consideration would not be susceptible to liquefaction under earthquake shaking because bedrock and the very dense glacial drifts are not susceptible to liquefaction. Liquefaction can, however, occur in loose granular soils, typically saturated sand and silty sand deposits due to a rapid buildup of porewater pressure and subsequent loss of strength. The area of primary concern for liquefaction would be the fan-delta deposits near Carlanna Creek and at the mouth of the other major creeks. Ground failure associated with liquefaction may include lateral spreading, landslides with limited displacement, and other flow type failures. If bridge structures or large approach fills are planned in these loose deposits, site specific explorations and studies will need to develop mitigation for these seismically induced failures.

Differential settlement may be caused when loose, cohesionless soils become compact upon shaking. This would most likely occur in the thicker, non-engineered fills such as along Charcoal Point, Spruce Mills and Bar Point. The fan-delta deposits, and some of the beach deposits may also experience excessive settlements.

Ground rupture due to faulting would not likely affect the bridge structure. The nearest inferred fault is within the Tongass Narrows, but this considered to be inactive.

Tsunamis generally are only produced from earthquakes with magnitudes of 7 or greater. The closest fault that is considered capable of generating this magnitude earthquake is the Queen Charlotte-Fairweather fault system, which is about 100 miles (160 km) to the southwest. There would be a natural attenuation of a seismically induced sea wave as it traveled through the straits and inlets before reaching Ketchikan. A tsunami from the 1949 Queen Charlotte Island earthquake produced a wave height of 3.5 inches (9 centimeters) in Ketchikan. The 1964 Alaskan earthquake produced a tsunami with a wave height of 2 feet (0.6 m) in the Ketchikan area. Based on this information, tsunami's with wave heights of less than 5 feet (1.5 m) may be expected during a large magnitude earthquake event. Additionally, destructive waves generated from submarine landslides are considered unlikely as well (Lemke, 1975).

6.7 Landslides

Typical slope failures in Ketchikan can occur due to a combination of several causes. They include: rapid runoff erosion or excess water pressures developing at the fractured rock/soil interface from extended periods of rainfall, or from construction disturbances which remove the support muskeg root mats from the slope or toe support resulting in failures of already steep slopes. With the exception of local glacial drift, most of the steep slopes are usually covered with a thin veneer of wet, weak (decomposed schist rock), and organic soils clinging to the shallow bedrock irregularities. During periods of high rainfall these surficial soils become saturated and have the potential to create debris flows. Known slides have occurred near Stedman Street/Nefco Road area and above Water Street, where a 700 foot segment contains unusually difficult ground, likely a ravine and/or steep slopes in an area that shows surface evidence of debris flows or signs of instability.

Once the preferred corridor alignments are established, we recommend a reconnaissance level geology survey of the alignment to evaluate potential areas for landslides. Due to the lush vegetation and steep nature of the topography, it is sometimes difficult to evaluate areas for previous landslides or the potential for landslides by aerial photography alone. If potential areas are identified during the reconnaissance, additional studies may be required for the preferred alignment.

7.0 ADDITIONAL EXPLORATIONS AND STUDIES

Figure 4 presents the locations where prior studies were preformed. It can be seen that for the most part there is some shoreline information, however, offshore drilling data in the middle of the Narrows is essentially non-existent. Offshore interpretations in deep water were estimated based on NOAA soundings and geophysical surveys.

Once the more feasible three or four corridors are identified from this Phase 1 study, we recommend that exploratory borings be conducted in each area to identify the subsurface conditions that we have assumed based on our interpretations provided herein. At each of the three or four crossings, exploratory drilling work should consist of two offshore borings on either side of the inferred fault in Figure 3 and two onshore borings (one on each side) in areas where bedrock is not exposed on the surface. Borings should generally be advanced at least 50 feet (15 m) into glacial drift or 40 feet (12 m) into bedrock. Split spoon drive samples should be taken at regular intervals in soils and continuous coring of the rock is recommended. Additionally, a geophysical survey using a variable frequency floating boomer source and acoustical sounding equipment should be conducted at each site being explored to allow extrapolation of muds, drift and rock thicknesses between borings. Geological reconnaissance along shorelines and some video dive surveys in offshore areas should be conducted on these alignments as well. Quarries identified on Figure 3, should also be visited to look for local aggregate for concrete and structural fills and riprap for roads and support structures in intertidal areas.

After a preferred alignment is chosen, for final design we recommend additional explorations at each anticipated bridge pier location and in areas where special structures are planned. Borings should be advanced into the dense glacial drift and into competent bedrock as required to define materials to be penetrated during construction and to evaluate the quality and engineering properties of the various support materials. Laboratory testing on the soil and bedrock should also be conducted to evaluate the index and engineering properties of these materials for use on this project. This includes not only the foundation materials, but also may apply to riprap for shore protection and available aggregate and borrow for concrete and asphalt as well as soil fills.

8.0 LIMITATIONS

The preliminary engineering studies, and conceptual recommendations contained in this report are based on site conditions as they were extrapolated and interpreted from limited existing literature and are assumed to be typical of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the literature. It is possible (and likely) that some of our assumptions of sediment thickness and material properties may have to be adjusted to accommodate different conditions. These changes can also impact our discussion of the foundation types. For example, if deeper drift material is encountered in the channel bottom, driven pile foundations may become a suitable support method in lieu of piers.

Please note that it was our intent and per your request to extrapolate the existing data into offshore areas and to then develop conceptual recommendations recognizing that site-specific information is limited. The recommendations we have provided should therefore be recognized as being conceptual for use in planning and feasibility studies for the project.

Unanticipated soil and rock conditions are commonly encountered and cannot fully be determined by merely reviewing existing literature. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Conversely, funds may be less, if more favorable conditions are encountered at the selected crossings. Therefore, some contingency fund is recommended to accommodate such potential variations in costs.

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