GEOTECHNICAL INVESTIGATION
GOLD CREEK RECLAMATION PROJECT
JUNEAU, ALASKA
FOR THE
CITY AND BOROUGH OF JUNEAU

APRIL 1, 1982
06842-003-20

Dames & Moore
April 1, 1982

EMPS
P.O. Box 2317
Juneau, Alaska 99803

Attention: Mr. George Davidson, P.E.

Gentlemen:

We are pleased to transmit herewith our final report entitled "Geotechnical Investigation, Gold Creek Reclamation Project, Juneau, Alaska," for the City and Borough of Juneau.

We provided you our preliminary conclusions and recommendations for review and comment in our February 23, 1982 draft report. Your comments and questions provided during our February 26, 1982 meeting have been incorporated in this report.

We appreciate the opportunity to provide these services and look forward to assisting you in the future. Should you have any questions, please call.

Yours very truly,

DAMES & MOORE

By

J. Michael Blackwell
Partner

JMB mb
10 copies submitted
cc: Sverdrup & Parcel (2)
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*Our preliminary recommendations for foundation support are provided in a supplementary letter report.
GEOTECHNICAL INVESTIGATION
GOLD CREEK RECLAMATION PROJECT
JUNEAU, ALASKA
for the
CITY AND BOROUGH OF JUNEAU

INTRODUCTION

This report presents the results of our geotechnical investigation at the site of the proposed Gold Creek Reclamation Project in Juneau, Alaska. The project area is illustrated relative to the City of Juneau on the Vicinity Map, Plate 1.

The site, which encompasses about 24 acres of tidelands, is bounded by Egan Drive on the north, the extension to West 8th Street on the east, and the subport pier on the west. Current site grades vary from about Elevation 10* to about Elevation -2. The project will involve filling portions of the site to about Elevation 25 using soils dredged from the Gastineau Channel. At the time of our investigation specific land use plans had not been identified and location/type of structures to be included in the development were not planned.

The potential sources of fill that have been considered during this investigation are located on the Project Area and Potential Borrow Source Location Map, Plate 2. The project site is illustrated on Plate 3, Gold Creek Reclamation Project Area.

SCOPE

The purpose of this geotechnical investigation is to provide recommendations for design and construction planning of containment dikes and the site fill. Our study has not addressed issues related to land use or specific foundation support requirements for future structures. Our investigation has included the following elements:

*Elevations in the text and appendix of this report refer to Mean Lower Low Water (MLLW).
- Gathering and review of existing geotechnical and environmental information pertaining to the site and adjacent regions. The results of this review were presented in our progress report of November 9, 1981.

- Field investigation
  -- Subsurface conditions at the site were explored by means of 11 borings.
  -- Several potential borrow sources were evaluated by data review and exploration; 10 borings, each about 20 feet deep were completed during this investigation.
  -- A bathymetric survey of the site and the area immediately offshore of the site was completed.

- Laboratory testing
  -- Conventional testing to evaluate the character of the soil at the site.
  -- Elutriate and turbidity tests on selected samples from potential borrow sources.

- Engineering analysis, with the objective of supporting conclusions and recommendations on the following topics:
  -- Slope stability
  -- Soil liquefaction potential
  -- Site dynamic response (expressed as site period)
  -- Relative suitability of candidate borrow sources
  -- Site settlement upon filling
  -- Site filling procedures
  -- Containment dike design and construction requirements
  -- Wave defense requirements
  -- Pile capacity for support of waterfront structures

Dames & Moore is also providing support to this project by undertaking an environmental evaluation. Fieldwork for this element of our services was begun during March 1982.
DESIGN AND CONSTRUCTION CONSIDERATIONS

The primary geotechnical and environmental issues that have been identified and that must be considered during design and construction are as follows:

1. Slope Stability. Inclination of submerged slopes, which extend from the project area into Gastineau Channel average about 2-1/2:1 (horizontal to vertical) but are steeper than 1:1 in some areas. The results of our study indicate that these slopes will be unstable during the design seismic event. Setback of the site fill from the top of existing slopes will be required in order to reduce the potential for loss of portions of the reclaimed area during an earthquake.

2. Liquefaction. Temporary loss of soil strength during an earthquake resulting in general surface settlement and differential building settlement may be caused by liquefaction. The soils underlying the project area are characterized by moderate density and strength and may be susceptible to this phenomenon.

3. Fill Placement. Construction of containment dikes will be required prior to filling in order to reduce turbidity and to retain dredge soils at the project area. Dredging and filling must be scheduled to avoid work during in- and outmigration of salmonids, especially if soils are dredged near active spawning streams. A short-term variance of Alaska water quality requirements may be solicited with the permit application to the Corps of Engineers. The long-term impact of dredging/fill placement on water quality is expected to be negligible.

Balancing the costs and risks related to the principal geotechnical issues (1 and 2 above) will be complex, requiring a reflection of the City's land use planning policies and economic factors. As plans for this project progress, we should be kept informed of geotechnically related decisions, and be given the opportunity to review and advise on
these topics. The risks associated with site performance during earth-
quakes may be better defined by accomplishing a seismic evaluation and
companion geologic study. We recommend these studies be implemented
prior to final design in order to assess the issues of cost and risk.

PROJECT AREA AND POTENTIAL BORROW SOURCE CONDITIONS

PROJECT AREA

General

Conditions at the site of the Gold Creek Reclamation Project were
evaluated by review of existing information, a site bathymetric survey,
and subsurface exploration. Our findings follow. Results of the
bathymetric survey and location of site borings are shown on Plate 3. A
description of the field equipment and procedures used for the bathy-
metric survey are presented in Appendix A. Equipment used for site
exploration and laboratory procedures, including logs of the site borings
designated as "SB," are presented in Appendix B.

Surface Conditions

The site of the planned reclamation project is located on about
24 acres of tidelands along the outer margin of the Gold Creek delta.
Beyond the landward margins of the site on the northwest, north, and east
the delta has been filled previously, and the developed land lies about
10 to 15 feet above the presently-exposed delta. The seaward slopes of
the existing fill are protected by riprap.

A fuel transfer dock extends about 650 feet offshore from Egan
Drive near the east margin of the site. We understand that sand and
gravel was occasionally extracted from near the mouth of Gold Creek as
late as the early 1950s. Other than the existing dock and the reported
gravel extraction, the site is essentially undeveloped and in its natural
condition.
About half of the site is covered by a brown algal mat with mussels, barnacles, and other invertebrates. Surficial soils are fine to coarse sand with a variable gravel and cobble content. The coarser soils appear to be more prevalent near the north margin of the site.

Subsurface Conditions

Our knowledge of soil conditions at the site is based on the results of the 11 borings drilled during this investigation, supplemented by logs of borings drilled by the Alaska Department of Transportation and Public Facilities (DOTPFL) along Egan Drive and at the new Gastineau Channel Bridge. The soils encountered in the borings consist generally of very loose to medium dense very fine to coarse sand with a variable silt, gravel and shell content. A stratum of soft to medium stiff silt was encountered in Boring SB-2-81 at the surface and at the location of SB-3-81 and SB-6-81 at depths on the order of 110 to 128 feet.

The soils identified during this investigation are typical of a delta for a fast-moving stream. The dominant soil type identified by the explorations at or near the project area consists of slightly silty to silty fine to medium sand. A greater percentage of coarse soil deposits including gravel, cobbles, and boulders appear to underlie the area near the site's north margin. This particle size variation is primarily the result of the decreasing energy gradient of Gold Creek where the stream empties into Gastineau Channel.

Based on subsurface conditions inferred from State of Alaska DOTPF explorations along the alignment of the new Gastineau Channel Bridge, we expect that the sand soils underlying the project area will extend to depths on the order of 150 to 170 feet below current site grade. A dense to very dense strata of glacial soil, typically referred to as glacial till, is anticipated to underlie the sand soils. For the purposes of this study, we have assumed that bedrock is present beneath the site at depths ranging from about 170 to 210 feet. These estimates are based on extrapolation of data developed by the DOTPF at the new Gastineau Channel Bridge site.
POTENTIAL BORROW SOURCES

General

Soil conditions at 10 potential borrow sources including submerged slopes adjacent to the site were evaluated by review of existing information and by 10 borings drilled at four of the sites. The approximate locations of explorations completed during this and previous investigations by others are shown on Plate 2. A description of each of the areas evaluated is presented in a following section. Logs of the borings completed during our site investigation are illustrated in the appendix and are designated "BB."

Soils which underlie the potential borrow sites can be categorized into three distinctly different units as follows:

Unit 1: Slightly silty to silty fine to coarse sand with some gravel and cobble layers. Unit 1 soils typically occur as deltaic deposits where streams discharge into Gastineau Channel.

Unit 2: Sandy silt, silty gravelly very fine to medium sand, and sandy gravel. Unit 2 deposits are represented by soils present within the limits of the intertidal zone and shallow reaches of Gastineau Channel. These soils are the result of stream deposition in the channel and subsequent transport and redeposition by tidal action.

Unit 3: Angular rock fragments and fine to coarse sand with angular gravel. Unit 3 soils are tailings from the Alaska-Juneau (AJ) and Alaska Gastineau mine sites.
CONCLUSIONS AND RECOMMENDATIONS

SLOPE STABILITY ANALYSES

General

The south periphery of the project area is bordered by submerged slopes with average inclinations of about 2-1/2:1; some areas are, however, steeper than 1:1. We have evaluated the stability of these slopes under dynamic (earthquake) conditions. The conventional pseudo-static method of computer analysis based on Bishop's simplified method was used to represent earthquake loading of the slopes by simulating the horizontal acceleration with horizontal static forces. Seismic criteria developed for design of the Gastineau Channel Bridge by the DOTPF were adopted for this study. A discussion of the seismic parameters and soil properties used during our analyses, are presented in Appendix C.

Safety Factors

The adequacy of a factor of safety obtained from slope stability analyses must be examined considering the following:

1. Potential effect of failure on loss of life, liability, facility damage, and operations.

2. Replacement/repair of damaged facilities and loss of use.

3. Sensitivity of the factor of safety with respect to the representativeness of parameters used in the analysis, including:
   (a) soil parameters
   (b) subsurface profiles
   (c) ground acceleration

4. The uncertainty of definition of seismic criteria based on seismic history, geologic structure, site response characteristics, attenuation of earthquake events, etc.
5. The method of analysis used, especially for the dynamic loading conditions where the ground motion and soil behavior are simulated by pseudostatic analyses.

Factors of safety against slope failure for the dynamic case were developed during our study. The safety factor for slope stability was computed using the modified Bishop method of stability analysis, which computes the safety factor by comparing the driving forces with the available soil shear resistance along a postulated failure plane. The computerized analysis program arrives at minimum safety factors for given input parameters which include soil unit weight, soil shear strength, soil profile, and slope geometry.

For most slope stability projects, safety factors on the order of 1.0 to 1.2 for pseudostatic analysis of temporary dynamic loading conditions are usually appropriate but are dependent to a large degree on the probability of occurrence of the design seismic event. It is our opinion that a minimum safety factor of about 1.05 for the dynamic case is appropriate to evaluate the effect of slope failure on the planned development.

Results of Slope Stability Analyses

Safety factors for several slope inclinations were evaluated using a ground acceleration based on the "operating earthquake" (Magnitude 8.5 at a distance of 90 miles which results in a horizontal ground acceleration "a" of 0.15 g) which has been described in a study by the DOTPF (Appendix C) as the highest magnitude earthquake which could occur during the design life of the new Gastineau Channel Bridge. The results of the DOTPF study indicate this event has a high probability of occurrence. A range of soil friction angles was input for each slope inclination for comparative analysis and to evaluate the change in safety factor as a function of soil strength. One slope inclination was analyzed using a ground acceleration based on the contingency earthquake (Magnitude 6.5 at a distance of 25 miles which results in a = 0.18 g). The DOTPF study concluded this event has a low probability of occurrence. The results of the analyses are presented as a family of curves, which are a function of
friction angle and safety factor, on Plate 4, Results of Pseudo-Static Slope Stability Analyses.

It should be noted that conventional seismic risk evaluations define the operating earthquake as less severe but more probable than the contingency earthquake. Some confusion can occur when the operating earthquake is of higher magnitude but at further distance from a site than the contingency earthquake. The confusion is compounded when there is a very low probability of exceeding the magnitude of the operating earthquake. Because of its high magnitude and low probability of exceedance, the DOTPF operating earthquake chosen for design of the new bridge may be considered by some to represent a contingency level event. The implication is that the horizontal ground acceleration corresponding to the DOTPF operating earthquake and used for our slope stability analyses may be somewhat high. We expect, however, that other factors not introduced during the stability analyses, such as pore pressure increases and liquefaction potential of the soils, balance the apparent conservative values of ground acceleration.

It is our conclusion that the safety factor of slopes with inclinations greater than about 4:1 will be less than unity during the operating earthquake. This is based also on a range of soil strengths which we have evaluated from the results of laboratory tests and blow count data. A comparative analysis using the ground acceleration for the contingency earthquake indicates that safety factors for slopes inclined at about 5:1 should be close to unity for the same range of soil strength. Use of the higher ground acceleration is somewhat conservative, however, since the recurrence interval for the contingency earthquake is longer than that of the operating earthquake as defined by the DOTPF.

The stability analyses have been used to establish a fill boundary in order to limit the risk of damage should slope failure occur. We have selected a fill set back limit as illustrated on Plate 5, Recommended Site Fill and Construction Boundaries, based on failure of existing slopes to a stable 4:1 configuration (approximate) during the operating earthquake. We recommend that this fill limit be included in the
design to reduce loss of and damage to facilities located near the south side of the site. We further recommend that a construction set back, as illustrated on Plate 5, be located at least 75 feet north of the preliminary fill limit. The intent of the construction set back is to establish a zone where no construction will be planned, with the possible exception of marine structures.

Slope failure will affect to some degree marine structures located near the top of the failure plane. Failure of slopes can be expected to exert lateral and vertical forces on piles and pier abutments which may exceed the structural capacity of the supporting elements. The magnitude of these loads and risk of structural failure will be dependent on the locations of the structures and the degree of earth movement which may accompany a seismic event.

Accordingly, we recommend that the slope inclination in the vicinity of future marine structures be reduced to less than 4:1 by means of dredging. We expect that dredging will be limited by equipment capabilities to water depths on the order of 40 feet. Soils dredged from the existing slopes may be utilized as general site fill to establish planned grades behind containment dikes which would be an initial part of the development. An illustration depicting envisioned dredge limits relative to existing slopes and required containment dikes is presented on Plate 6, Typical Section.

It is our opinion that the fill and construction set backs outlined above are appropriate considering the seismicity of the Juneau area and the character of the soils which underlie the site. It is important to note, and we emphasize that the methods of analyses, expected variation of soil parameters, and assumed behavior of the soil mass, introduce uncertainties regarding the performance of the site during a seismic event. Furthermore, our results and conclusions are based on the assumption that liquefaction of the site soils does not influence the mechanics of slope failure. If this assumption is not valid, then propagation of slope failure towards the center of the site could result in slopes much flatter than 5:1. A further discussion of the uncertainty that this phenomenon introduces is presented in a subsequent section.
The set back limits may be altered by accepting lower safety factors. However, a higher level of risk is inherent if the recommended boundaries are moved further south.

LIQUEFACTION POTENTIAL OF SITE SOILS

Liquefaction of sand deposits is the transformation of the soil mass from a solid state to a liquified condition as a result of increased pore water pressure. This transformation may be accompanied by loss of soil strength and horizontal and vertical movement of the soil mass, which could range from negligible amounts to many feet. The impact of liquefaction on structures may include minor settlement, lateral deflection, tilting or failure of structural elements as a result of reduced bearing capacity of the soils underlying the structure.

The phenomenon of liquefaction is generally associated with loose and saturated sand deposits. The factors which may affect the liquefaction potential of the soil deposit are as follows:

1. Grain size distribution of the soil deposit.
2. Initial relative density ($D_r$)
3. Magnitude of ground vibration
4. Location of drainage and dimensions of deposit
5. Magnitude and nature of superimposed loads
6. Soil structure which is dependent upon depositional environment
7. Duration of ground vibration
8. Previous stress history
9. Entrapped air

Results of Liquefaction Study

The empirical and analytical procedures used during our liquefaction study in addition to the graphic results of our analyses are presented in Appendix C.

It is our opinion that the potential for liquefaction of the site soils to depths on the order of 40 to 60 feet is moderate to high. The most susceptible deposits will be those soils with relative densities
less than about 60 percent and with a mean effective grain size less than about 0.7 mm. Soils which underlie the project area near Egan Drive appear to be less susceptible to liquefaction due to the percentage of coarse sand and gravel. The results of our liquefaction study are based on state-of-the-art analytical procedures. However, these procedures do not include consideration of previous site stress history or a conclusion regarding the consequences and magnitude of settlements resulting from liquefaction.

Our conclusions regarding the depth of liquefaction have been evaluated using case history studies and the apparent increase in density (based on blow count data) between depths of about 40 to 60 feet. This increase in density could be influenced by many factors, including densification during previous earthquakes which may have preceded deposition of the overlying soils.

**IMPLICATIONS OF STABILITY ANALYSES AND LIQUEFACTION STUDY**

The issues which must be considered during an evaluation of these analyses include definition of the level of risk associated with a seismic event, limitation of the impact of that event on the development, and reduction of the potential for loss/damage as a result of earthquake loading. The level of risk can be associated with the design seismic event. Studies conducted by the DOTPF indicate that the operating earthquake has a high probability of occurrence for a 50-year project life.

A method for limiting the impact of a seismic event would include zoning of the project area for different levels of risk associated with liquefaction potential prior to siting facilities and structures. Although we have concluded that the potential for liquefaction is lower along the north side of the project area, field and laboratory data developed during this investigation are insufficient to delineate zones with respect to a specific degree of risk.

The combined effects of liquefaction and slope failure must be considered also. Current methods of analyses including finite element analysis cannot provide any degree of certainty of site performance for a
combination of slope failure and liquefaction of adjacent level or near level ground. It is our opinion that liquefaction could contribute significantly to reduced safety factors of submerged slopes which border the project area. This phenomenon could result in general site settlements accompanied by failure of the slopes and progressive movement of the south portion of the site toward the center of Gastineau Channel.

We have recommended fill and construction limits as described previously. These limits will reduce the potential for damage which may be the result of slope failure, but do not include consideration of soil liquefaction potential. We have indicated that site zoning using available data is not practical. Therefore, we recommend that three alternative development schemes and the corresponding relative risk for each be considered. The alternatives for site development as we envision them are as follows:

<table>
<thead>
<tr>
<th>Alternative Description</th>
<th>Relative Development Cost</th>
<th>Relative Risk</th>
</tr>
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<tbody>
<tr>
<td>1. Construct containment dikes, fill the site, and accept the risk of liquefaction and slope failure associated with the design seismic event.</td>
<td>Low</td>
<td>High</td>
</tr>
<tr>
<td>2. Stabilize the soils underlying the containment dike, construct dikes, fill the site, and accept the risk of liquefaction where stabilization is not accomplished.</td>
<td>Low to Moderate</td>
<td>Moderate to High</td>
</tr>
<tr>
<td>3. Construct dikes, fill the site, and stabilize all areas of potential liquefaction to 50-foot depths.</td>
<td>High</td>
<td>Low to Moderate</td>
</tr>
</tbody>
</table>

It is our opinion that the second alternative represents the best balance between cost and seismic hazard. This alternative, if implemented, should reduce the potential for catastrophic site failure resulting from the combined effects of slope failure and liquefaction. However, the risk that liquefaction of the remaining unstabilized soils will occur must be accepted. We expect that settlement of unstabilized
potentially liquefiable soils will occur but that the risk of translation of the site toward Gastineau Channel will be reduced.

SOIL IMPROVEMENT

Assuming partial stabilization is the selected alternative, we conclude that stabilization of the site soils by one or more of several proprietary methods currently in use should be implemented to reduce the potential for liquefaction beneath the recommended containment dikes along the south side of the site and to improve the performance of soils near the top of submerged slopes. Stabilization of soils underlying other portions of the project area will depend upon the purpose of the development and type of structure. In general, we expect that stabilization will be appropriate beneath major structures, such as multi-story buildings. Low-level wood frame structures may not require stabilization.

The soil stabilization procedure which may be most effective for a specific site is dependent upon characteristics of the soils, effect of the procedure on existing facilities, topographic features, and cost. Soils underlying the project area consist of relatively clean, fine to coarse sand and very fine to medium sand deposits. Several soil improvement procedures could be implemented for these conditions, including blasting, vibratory probe, and dynamic consolidation. The principal for each procedure is to initiate localized liquefaction by imparting energy into the soil mass. Blasting and dynamic consolidation represent the lowest cost procedures. However, quality control may be difficult. Blasting has the added disadvantage that near-surface soils cannot be densified.

The proprietary methods developed by Terra-Probe and Vibroflotation are approaches which also may be appropriate for use at the Gold Creek reclamation site. Each procedure involves insertion of a pipe pile section or probe by means of vibration and water jetting at various intervals across the site to increase the relative density of the soils. Backfilling during this procedure is required as a result of settlements.
which occur at each probe location. Spacing of probes is typically on the order of 6 to 8 feet, depending on the required improvement and specified final soil relative density. The following tabulation adapted from Mitchell (1981) summarizes the methods and relative cost of each procedure.

<table>
<thead>
<tr>
<th>Method</th>
<th>Effective Treatment</th>
<th>Advantages, Limitations</th>
<th>Relative Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blasting</td>
<td>More than 100</td>
<td>Rapid, inexpensive; final density varies, no near-surface improvement, dangerous</td>
<td>Low</td>
</tr>
<tr>
<td>Dynamic Consolidation</td>
<td>100+</td>
<td>Good improvement, reasonable uniformity</td>
<td>Low</td>
</tr>
<tr>
<td>Terra-Probe</td>
<td>10 to 60</td>
<td>Relative densities up to 80 percent, ineffective in some sands</td>
<td>Moderate</td>
</tr>
<tr>
<td>Vibroflotation</td>
<td>100*</td>
<td>High relative densities, good uniformity</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

*Equipment for improvement to 60-foot depths available in the United States only.

The improvement procedure selected will require additional fill to raise site grades during and after stabilization due to surface subsidence related to soil densification. The amount of additional soil required will depend upon the method implemented. We estimate that about 0.2 to 0.4 cubic yards of additional fill will be required for each square foot of stabilized area. This estimate will vary depending on the method used, effectiveness of the procedure, and initial soil densities.

\[(0.4)(2.7) = 10.8\text{ cubic yards}\]

SITE FILLING AND DEVELOPMENT

General

We have identified four potential borrow source areas which appear to be viable from the standpoint of proximity to the site and cost in place. Prior to filling, construction of containment dikes using select
borrow from onshore sources will be necessary in order to retain the
dredge soils and to reduce turbidity of the effluent before return into
Gastineau Channel.

A phased site filling program may be required depending on available
funding. This will require division of the project area into two or more
sections, each of which will comprise a major cell to be filled during
successive phases. The number of major cells will depend, again, on
budget allocations for the project.

Compaction of the dredged soils may be accomplished after filling
and drainage of the fill mass. However, segregation of fine-grained
particles from the coarse fraction of the dredge fill may create
zones of weak, compressible soil deposits depending on cell geometry and
method of fill placement. Those areas are expected to drain and gain
strength more slowly than other portions of the dredge fill, and may
require preparation procedures, such as preloading, different than for
the coarse fraction dredge spoil.

Phased Containment Dike Construction

We recommend that the project area be divided into at least two
cells (that may be divided into smaller cells to accommodate various
phases of site filling). The two main cells would be separated by a
channel which will be required for Gold Creek.

The main cells should be formed by construction of a containment
dike constructed within the limits of the fill setback, as noted on
Plate 5, and parallel to the new channel formed for Gold Creek. These
peripheral containment dikes should be constructed with select granular
fill imported from on-shore sources. The fill should consist of a sand
and gravel or fine angular rock mixture with less than 5 percent passing
the No. 100 sieve to facilitate placement during periods of high tides.
The select fill will provide a dike section with sufficient strength to
retain the lower strength dredged soils. Soil or rock fill placed during
containment dike construction may be placed without compactive effort, if the materials conform to the recommended gradation.

If angular rock fill is used for dike construction, increased turbidity resulting from leaching of dredge effluent through the dike may result. Should the coarse rock fill be more economical than a sand/gravel mixture, the gradation of the rock should be carefully examined to evaluate the possible increase of turbidity and to assess alternative procedures to limit leaching of fine particles through the dike section.

The outer dike slopes should be inclined no steeper than 1.5:1 and 2:1 for angular rock fill and sand/gravel mixtures, respectively. Soils placed to achieve steeper inclinations will require compaction and may be susceptible to instability. The inner dike slopes may be constructed as steep as is practical. The dike crest width should be planned at about 10 feet to provide access for equipment.

Slope Protection Requirements

The outboard containment dike slopes will be subjected to wind- and vessel-generated wave forces which will erode the dike. The lower portion of the dike slopes may be exposed to the action of breaking waves depending upon embankment toe elevation, inclination of near-shore slopes, and tide levels. If dredging is accomplished in the vicinity of marine structures as recommended previously then the less critical condition of a nonbreaking wave would control design of slope protection in those areas. Where dredging is not accomplished, the breaking wave condition should be considered during design.

We have evaluated slope protection requirements for the outboard dike slopes using wind and wave data developed by the DOTPF for the new Gastineau Channel Bridge. Four alternatives have been selected based on our analyses. The approach for slope protection which is chosen will depend upon economic considerations related to initial construction costs, long-term maintenance costs, and, possibly, land value. The alternatives are tabulated below:
<table>
<thead>
<tr>
<th>Slope Inclination</th>
<th>Riprap Slope Protection*</th>
<th>Estimated Level of Risk, Percent Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative 1: 2:1</td>
<td>Class II</td>
<td>Low to moderate; less than 10%</td>
</tr>
<tr>
<td>Alternative 2: 1.5:1</td>
<td>Class II</td>
<td>Moderate to very high greater than 20%</td>
</tr>
<tr>
<td>Alternative 3: 1.5:1</td>
<td>Class III,† bottom 4 feet Class II upper slope</td>
<td>Low; less than 5%</td>
</tr>
<tr>
<td>Alternative 4: 2:1 bottom 4 feet 1.5:1 upper slope</td>
<td>Class III† Class II</td>
<td>Low; less than 5%</td>
</tr>
</tbody>
</table>

*Based on State of Alaska DOTPF specification 611-2.01.

†Class III riprap not required if dredging adjacent to containment dikes is accomplished.

We do not advocate implementing Alternative 2. It is our opinion that the cost associated with maintenance/repair will be greater than the savings realized by using the lighter weight Class II rock.

We recommend that Class II and Class III riprap slope protection consist of a rock cover about 2-1/2 and 3-1/2 feet thick, respectively. Provision for protection of the rock cover at the end of the containment dikes should be included in the design to reduce the potential for outflanking of the slope protection.

The riprap should be constructed on a synthetic fabric (geotextile) in order to reduce the potential for infiltration of embankment soils through the riprap as a result of wave action and differential water pressures. A graded rock filter is not recommended since Class I riprap in addition to fabric would be required.

Toe protection should be included in the design of the selected alternative. We recommend that the riprap be keyed into existing soil at the toe of the containment dike in order to reduce the potential for undercutting. The key should extend at least 3 feet below the dike toe and have a minimum bottom dimension of 3 feet with side slopes inclined at 1:1 as illustrated on Plate 7, Suggested Toe Protection Detail.
We estimate that runup of nonbreaking waves for slopes inclined at 1.5:1 or 2:1 will be in the range of 2 to 2-1/2 feet assuming that slope protection consists of graded rock as recommended. The higher value of runup applies to the steeper slope inclination.

BORROW SOURCE EVALUATION

Dredge Fill

We have identified the anticipated soil conditions at 10 potential borrow sources for general site fill. Review of existing data obtained during our study, in addition to the results of our borings, have been used as the bases for our conclusions regarding the suitability of these soils. Approximate cost information pertaining to distance of source from the project area, type of soil available, and equipment required to dredge/transport soil has been provided by Manson Construction and Engineering Company. Elutriate testing of mine tailing deposits indicate that total volatile solids, chemical oxygen demand (COD), and heavy metals do not exceed limits established by the EPA. These results and other test data are presented in Appendix B.

The results of our evaluation indicate the soils generally available for use as fill consist of fine to coarse sand with a silt content in the range of 5 to 20 percent and a variable gravel content. The soils encountered have been classified into three units as described in a previous section titled "Site Conditions, Potential Borrow Sources."

We have considered soil gradation, distance from the project area, types of equipment required, and available quantity, although the latter issue has been essentially a judgmental determination. The Unit 1 and Unit 3 soils are the most suitable because of their granular characteristics and low silt content. These soils will drain more rapidly after placement than the Unit 2 soils.

The methods and equipment required to dredge and transport soils from the borrow sites to the project area are generally dependent on
the types of soils and distance from the fill site. A suction dredge with a pipeline to transport the dredge slurry would be one means of dredging and transport. The suction dredge is limited to about 1-1/4 miles of discharge line with the dredge pump, but is capable of pumping for distances up to about 2 miles with a booster pump. A submerged discharge line would be required with the suction dredge to permit passage of boat traffic. The use of a hopper dredge would be appropriate for areas further than about 2 miles from the site. The hopper dredge is loaded at the borrow source, then moved by tugboat to the fill area for offloading. During dredging at the borrow source, soils placed in the hopper tend to segregate, resulting in discharge of water with suspended fine soils over the side while the coarser sediments settle to the bottom of the hopper. The result is increased turbidity at the dredge site but a lower fines content in the dredge fill.

Our evaluation has resulted in a relative engineering ranking for each of the potential borrow sources in order to delineate the more suitable areas. The qualitative rankings are as follows:

<table>
<thead>
<tr>
<th>Rank*</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Most suitable</td>
</tr>
<tr>
<td>2</td>
<td>Some constraints</td>
</tr>
<tr>
<td>3</td>
<td>Least suitable</td>
</tr>
<tr>
<td>4</td>
<td>Impractical</td>
</tr>
</tbody>
</table>

*These rankings do not consider the implications of seasonal environmental constraints.

The results of our study including the ranking for each site are presented in Table 1.

**Potential Armor Stone Sources**

We made brief visits to several potential sources of armor stone, making an initial assessment of stone availability and suitability. From this reconnaissance, we have identified two existing quarries (inactive at the time of our visit) that merit consideration as sources. A final
decision and evaluation should be made after project stone requirements are established -- in terms of stone size and quantities. An assessment of ownership and possible royalty costs has not been done.

The estimates of produced stone size given below are to be considered preliminary. These estimates are based on limited exposures, and do not account for variations in blasting techniques.

**Fish Creek:** On a spur, about 1/2 miles west of Eagle Crest Road. Haul distance about 5.9 miles. Rock type is greenstone with irregular lenses of slate and argillite, 1 to 4 inches thick. The effective joint spacing is about 3 to 4 feet (maximum) and 1/2 to 1 foot average. Occasional shattered zones were noted. Ten to 40 percent of this rock could probably be produced in the 1- to 2-ton size range.

The face is located on the side of a 30-foot high hill, oval in plan and about 100 feet wide and 300 feet long. No other development is in the area.

**Bonnie Brae:** About 1,000 feet uphill from North Douglas Highway, 3.3 miles north of the Douglas Bridge. A large face is not open, but based on existing exposures, the rock type is dominately schistose-greenstone. Effective joint spacing 2 to 3 feet (maximum), 1/2 foot (average). Five to 15 percent of this rock would probably be produced in the 1- to 2-ton size range.

There are no other developments in the area, but a residential subdivision is planned nearby.

**Dredge Slurry Turbidity Control**

Proper design and operation of overflow structures will be essential to avoid degrading water quality excessively. The State of Alaska water quality standards, which are administered by the Department of Environmental Conservation (DEC), are dependent on timing restrictions related to fish migration, the type of dredging equipment, and fill
<table>
<thead>
<tr>
<th>(A) Potential Source Name and Location</th>
<th>(B) Mile Distance, Center of Source</th>
<th>(C) Soil Characteristics and Unit</th>
<th>(D) Estimated Silt Content (percent)</th>
<th>(E) Estimated Quantity (cubic yards)</th>
<th>(F) Environmental and/or Seasonal Constraints</th>
<th>(G) Dredging, Transportation and Placement Methods</th>
<th>(H) Estimated Cost($) Per Cubic Yard</th>
<th>(I) Comments and Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Douglas Boat Harbor</td>
<td>1.8</td>
<td>Unit II silty fine to medium sand and sandy silt</td>
<td>less than 15 for sand soils</td>
<td>80,000 to 120,000</td>
<td>turbidity and salinity of adjacent benthic populations due to poor flushing (not certain of flushing characteristics)</td>
<td>Suction Dredge</td>
<td>$3.50 +</td>
<td>Competed area, submerged line with booster pump Rank = 4</td>
</tr>
<tr>
<td>2. Lawson Creek Delta</td>
<td>0.7 to 0.8</td>
<td>Unit I fine to coarse sand with variable silt and gravel content</td>
<td>less than 10</td>
<td>200,000 to 240,000</td>
<td>juvenile pink salmon outmigration in spring/early summer juvenile salmon feeding/refuge habitat waterfowl feeding/resting</td>
<td>Suction Dredge</td>
<td>$3.50</td>
<td>Submerged line; quantity based on 10-foot dredge depth Rank = 1</td>
</tr>
<tr>
<td>3. Eagle Creek Delta</td>
<td>1.6</td>
<td>Unit I silty fine to coarse sand and gravel</td>
<td>less than 10</td>
<td>200,000 to 240,000</td>
<td>juvenile salmon outmigration in spring/early summer juvenile salmon feeding/refuge habitat waterfowl feeding/resting</td>
<td>Suction Dredge</td>
<td>$3.50 +</td>
<td>Submerged line with booster pump works with high tides only; quantity based on 10-foot dredge depth Rank = 2</td>
</tr>
<tr>
<td>4. Gastineau Channel between Eagle and Falls Creeks on west side of channel</td>
<td>2.1</td>
<td>Unit II sandy silt and silty fine sand</td>
<td>10 to 60+</td>
<td>1.2 to 1.3 million</td>
<td>spring/summer feeding and refuge for outmigrating juvenile salmonida</td>
<td>Suction Dredge</td>
<td>$3.50 +</td>
<td>Hopper dredge, works with high tides only; high turbidity; quantity based on 10-foot dredge depth; includes intertidal zone Rank = 3</td>
</tr>
<tr>
<td>5. Falls Creek Delta</td>
<td>2.7</td>
<td>Unit I sandy fine to coarse gravel with variable silt content</td>
<td>less than 10</td>
<td>160,000 to 200,000</td>
<td>juvenile salmon outmigration in spring/early summer juvenile salmon feeding/refuge for outmigrating salmonida waterfowl feeding/resting</td>
<td>Hopper Dredge</td>
<td>$3.30</td>
<td>Hopper dredge works with high tides only; quantity based on 10-foot dredge depth; Rank = 2</td>
</tr>
<tr>
<td>6. Gastineau Channel Salmon Creek to Switzer Creek</td>
<td>2.9 to 4.0</td>
<td>Unit II silty fine sand with some medium sand and zones of sandy silt</td>
<td>10 to 15 percent for sand soils</td>
<td>360,000 to 400,000</td>
<td>outmigration pathway during spring/early summer for juvenile salmon</td>
<td>Hopper Dredge</td>
<td>$3.30</td>
<td>Hopper dredge, work with high tides only; high turbidity; quantity based on Plan 2 as described in COE feasibility report dated 1977; Rank = 3</td>
</tr>
</tbody>
</table>

(a) Cost could increase as a result of royalties for privately owned borrow.
<table>
<thead>
<tr>
<th>No.</th>
<th>Potential Source Name and Location</th>
<th>Bail Distance, Center of Source (miles)</th>
<th>Soil Characteristics and Unit</th>
<th>Estimated Silt Content (percent)</th>
<th>Estimated Quantity (cubic yards)</th>
<th>Environmental and/or Seasonal Constraints</th>
<th>Drifting, Transportation and Placement Methods</th>
<th>Estimated Cost ($/y)</th>
<th>Comments and Rank</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>Gastineau Channel, Switzer Creek to Fritz Cove</td>
<td>4.8 to 8.0</td>
<td>Unit II silty fine sand with some medium sand and zones of sandy silt</td>
<td>10 to 15 percent for sand soils</td>
<td>488,000 to 509,000</td>
<td>outmigration pathway during spring/early summer for juvenile salmon</td>
<td>No</td>
<td>Quantity based on 3-foot dredge depth; includes intertidal zone; Rank = 4</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>AJ Mine Tailings</td>
<td>1.5</td>
<td>Unit III slightly silty fine to coarse sand with some gravel</td>
<td>less than 10 percent</td>
<td>Unknown</td>
<td>turbidity effects during juvenile salmon outmigration</td>
<td>Suction Dredge</td>
<td>$3.25</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Alaska-Gastineau Mine, Tailings at Thane (Sheep Creek)</td>
<td>4.2</td>
<td>Unit III slightly silty fine to medium sand</td>
<td>less than 10 percent</td>
<td>Unknown</td>
<td>turbidity effects during juvenile salmon outmigration</td>
<td>Hopper Dredge</td>
<td>$3.30</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Submerged slopes at Project Site</td>
<td>--</td>
<td>Unit I fine to coarse sand with some gravel</td>
<td>10 to 15 percent</td>
<td>200,000 to 230,000</td>
<td>same as for filling at site</td>
<td>Suction Dredge</td>
<td>$3.00/y</td>
<td></td>
</tr>
</tbody>
</table>

*Note: The comments and rank values are provided for additional context and ranking purposes.*
placement procedures. The criteria which will most likely apply to the Gold Creek Reclamation Project dredge fill program is turbidity. The state regulations indicate that water quality cannot exceed 25 nephelometric turbidity units (NTU) above the natural background water turbidity level. This criterion is difficult, if not impossible, to meet for most projects. The state usually grants a short-term variance for projects which are accomplished in accordance with other stipulations. However, reduction of dredge slurry turbidity will be an essential element of the design and site filling phases.

Based on the results of laboratory analyses on soils obtained at potential borrow sources, we expect that water quality in the cells will be in the range of 80 to 120 NTU after about 60 minutes. Some variations should be expected due to mixing.

Location and design of the overflow structure will be important to reduce turbidity of the effluent prior to discharge from the cell. Various forms of adjustable weirs have been used successfully for control of discharge on other projects. Synthetic fabric curtains may be implemented also to limit the discharge of suspended fine particles. If weirs are used, we recommend a flow height of 2 inches.

The overflow structure should be designed assuming a dredge flow velocity in the range of 12 to 18 feet per second and 22 hours of dredge operation per day. The volume of dredge effluent, then, will be dependent on the diameter of the discharge line. Specifics of the overflow structure design may be determined by the contractor at the time of construction.

Some flow of water through the containment dikes will occur as filling begins. However, the granular containment dike should act as a filter and should effectively reduce the turbidity of the slurry water. Flow rate through the dikes will decrease as the voids within the dike structure become clogged with fines.
We recommend that the dredge line discharge be located so that the coarse fraction of the soils dredged are deposited near the peripheral containment dikes. As these coarse soils are deposited, they should be graded into place with a small cat behind the dikes in order to improve dike stability.

**FILL AND SUBGRADE SETTLEMENT**

Settlement of the dredge fill surface will result both from consolidation of the underlying natural soil and subsidence within the fill. We estimate that settlement due to consolidation of the natural soil below the dredge fill will be in the range of approximately 3 to 15 inches. Settlement beneath the containment dikes should be less than about 6 inches. Most of this settlement should occur fairly rapidly, probably within 1 to 3 weeks after completion of filling.

We estimate that soils of Unit 1 and 3 will have a bulking factor of about 5 to 10 percent. This implies that the average density of the soils as they exist will be reduced by a factor of approximately 1.05 to 1.1. This will result in a volume of in place dredge fill greater than the extracted volume of soil. The filled volume will decrease with time as the fill consolidates under its own weight. This volume change will result in an average areal settlement of the dredge fill in the range of 6 to 12 inches for 15 to 25 feet of dredge fill. Greater settlements, on the order of 1 to 2 feet, may occur where significant deposits of fine grain soils are present subsequent to filling.

The loads imposed by the fill mass may induce settlement of the pile foundations which support the Standard Oil dock. Filling around or near the existing structure should not be accomplished during off-loading of petroleum products. The structure and piping should be surveyed and inspected prior to, during, and after fill placement in order to assess the impact of filling and to identify any releveling of piping which may be required as a result of settlement.
The following plates and appendices are attached and complete this report.

Plate 1 - Vicinity Map
Plate 2 - Project Area and Potential Borrow Source Location Map
Plate 3 - Gold Creek Reclamation Project Area and Results of Bathymetric Survey
Plate 4 - Results of Pseudo-Static Slope Stability Analyses
Plate 5 - Recommended Site Fill and Construction Boundaries
Plate 6 - Suggested Toe Protection Detail
Plate 7 - Typical Section

Our preliminary recommendations for foundation support are provided in a supplementary letter report.

Respectfully submitted,

DAMES & MOORE

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Senior Engineer

And James B. Harakas, P.E.
Project Engineer

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