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Final Report

Geotechnical Overview
Point Thomson Development
Alaska

Prepared for

Exxon Company, U.S.A.
1800 Avenue of the Stars
Los Angeles, California 90067

February 1983

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Project No. 41557B

Exxon Company, U.S.A.
1800 Avenue of the Stars
Los Angeles, California 90067

Attention: Mr. Roger W. Walls

Subject: Final Report
Geotechnical Overview
Point Thomson Development, Alaska

Gentlemen:

With this letter we are transmitting thirty copies of our final report on the geotechnical engineering investigation. This is one of the three reports that have been prepared to present the results of our studies performed under Agreement Number PTD-8203. The other reports address the onshore geophysical survey and the shallow marine geophysical survey.

We thank you for the opportunity to have worked on this interesting ~~projects~~ project. Please call if you have any questions.

Sincerely,

Ulrich Luscher, Sc.D.
Principal

O. S. Ghuman
Task Manager

adb

Attachment

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SUMMARY

Major facilities are planned to be constructed near Point Thomson on the Alaskan Beaufort Sea Coast in connection with projected crude oil production. The area poses major challenges as a result of the extreme arctic climate, offshore as well as onshore locations of the proposed facilities, and presence of terrestrial and subsea permafrost and shifting pack ice. The development will be costly but costs can be minimized by knowledgeable interpretation of available subsurface information in relation to the planned development.

Because the offshore permafrost is generally deep, buried pipelines without thermal protection appear feasible over much of the offshore area; other areas may require thermal protection. Depth of burial will mostly be controlled by potential ice gouging and will need to be quite deep (up to 12 ft). Onshore pipelines will probably need to be elevated on piles.

Gravel islands are feasible throughout the study area. However, use of the contemplated ice-rich gravel could lead to large settlements over the first several years after construction.

Both shallow and deep foundations have applicability on artificial gravel islands. Shallow foundations can be used to support non-critical structures, for summer-constructed gravel islands, and in island areas outside the central core. Pile foundations used in all other cases may need to be designed for downdrag forces which develop as the island continues to settle.

Causeways will be able to be constructed either as a continuous gravel berm or in an elevated mode on large-diameter piles. The shifting ice pack imposes severe design requirements on these structures. The berm causeway will increase in strength as it freezes gradually over the first few years to resist the full applied ice loads.

Pipeline transitions to shore can be made either in a buried or berm mode. Because of their critical nature, these will require careful, site-specific design attention.

The report concludes with several recommendations for future studies.

ACKNOWLEDGMENTS

This study was completed for the Western Division of Exxon Company, U.S.A., with overall project direction from Mr. R. Ashley Erwin of Exxon. Mr. Roger W. Walls was the Point Thomson Unit Manager for Exxon and Messrs. Andrie Chen, Chris Heuer, and Kenneth Gram of EPR provided technical review and direction.

Key Woodward-Clyde participants in the study were Mr. Opjit Ghuman, Dr. Ulrich Luscher, Messrs. Howard Thomas, Rupert Tart and Stanton Clarke. Dr. Ken Vaudrey was consulted on the telephone about ice effects in the Beaufort Sea area.

1.0
INTRODUCTION

1.1 PROJECT BACKGROUND

Exxon Company, U.S.A. (Exxon), Western Division, is planning facilities for the development of potentially-commercial quantities of hydrocarbons in the area of Flaxman Island in the Alaskan Beaufort Sea. This is termed the first phase of Exxon's possible development of the Point Thomson area. Exxon contracted with Woodward-Clyde Consultants (WCC) for reconnaissance geophysical investigations of the offshore and onshore portions of the Point Thomson Development Area. The geophysics work was to obtain ground-proofing data from soil investigation borings previously drilled by Exxon (Harding-Lawson Associates (HLA) report dated June 1982) and by the US Geological Survey (HLA, 1979). WCC was retained to provide a geotechnical overview of the area.

The Point Thomson Area is located about 50 miles east of Prudhoe Bay on the arctic seacoast in Alaska and lies between Bullen Point and Brownlow Point. The area covered is about 23 miles east-west along the coast; it extends about 5 miles offshore and 3 miles onshore. About 3 miles offshore and generally trending WNW-ESE are a series of low barrier islands including Flaxman, North Star, Duchess, Alaska and Challenge Islands.

In meetings with Exxon and Exxon Production Research (EPR) representatives, facilities likely to be developed in the Point Thomson area were discussed. For the purpose of this review these were summarized to include the following:

- Offshore Facilities (pipelines, gravel islands, foundations for structures on gravel islands, and causeways)

- Onshore Facilities (pipelines, gravel pads and roads, and foundations for structures)

- Pipelines in the transition zones

A listing of these facilities and the pertinent evaluation considerations and design parameters are presented in Table 1.1. This listing has served as the main background for our considerations described in this report.

1.2 ASSIGNMENT OBJECTIVES

The objective of the geotechnical engineering services assignment was to consolidate the geotechnical data previously available, to incorporate geotechnical information developed during the geophysical investigations, and to provide our opinions on geotechnical conditions and design considerations in the project area. This was viewed as being a synthesis task to pull together the pertinent geotechnical information and to prepare a report which would serve as a useful summary of geotechnical considerations in the project area.

1.3 SCOPE OF WORK

To achieve the objectives, a detailed work plan, dated 29 October 1982, was prepared describing our approach. As agreed with Exxon (Exxon letter dated 4 November 1982) our work scope was defined to address the subjects identified in our work plan in a generic manner. The specific scope of work included the following:

- planning the assignment and preparation of a detailed work plan in consultation with Exxon personnel;

TABLE 1.1

LIST AND DETAIL OF FACILITIES

POINT THOMSON DEVELOPMENT

1.0 OFFSHORE FACILITIES

1.1 Pipelines

Evaluate siting (generally), construction modes (trench or anchored, depth, cover material), potential design constraints.

- Offshore of barrier islands
- Inside of barrier islands (in lagoon)

1.2 Gravel Islands

Evaluate siting, stability considerations (e.g., sliding), settlement considerations, potential design constraints. Assume Point Thomson C-1 gravel for fill.

- Offshore of barrier islands, water depths 16 to 40 ft, freeboard 16 to 26 ft
- In lagoon, freeboard 15 ft
- In lagoon built partially on barrier island, freeboard 10 to 12 ft

1.3 Foundations for Structures on Gravel Islands

Evaluate preferred types, potential design constraints. Foundations located outside area affected by any wellbore thaw settlement. Consider summer or winter island construction. Consider three types of islands (offshore, in lagoon, partly on barrier island).

- Modules up to 2000 tons, skid-mounted, area 60 by 120 ft up to 100 by 200 ft, with some vibratory loads.
- Storage tanks 50 to 60 ft in diameter, 20,000 bbl capacity.
- Light lodging and similar facilities.

1.4 Causeways

Evaluate siting (generally), construction modes (berm, elevated on supports), potential design constraints. Assume freeboard 15 ft, ice load 270 k/ft. Supports for elevated causeway may be piles, caissons, or small gravel islands.

TABLE 1.1 (Continued)

LIST AND DETAIL OF FACILITIES

POINT THOMSON DEVELOPMENT

2.0 ONSHORE FACILITIES

2.1 Pipelines

Evaluate siting, construction modes (likely elevated), potential design constraints.

2.2 Gravel Pads and Roads

Evaluate siting, design parameters, potential design constraints. Consider massive ice, terrain units, and terrain constraints.

2.3 Foundations for Structures

Evaluate preferred types, potential design constraints.

- Modules up to 4000 tons, 100 by 200 ft maximum.
- Tanks and other facilities like on islands.

3.0 PIPELINES IN TRANSITION ZONES

Evaluate pipeline design in transition zones, considering soil/permafrost conditions, topography, erosion. Evaluate situations minimizing problems. Consider use of causeways or aboveground pipelines.

- Gravel island to subsea.
- Barrier island to subsea (offshore)
- Barrier island to lagoon
- Lagoon to onshore

- review of geotechnical data made available by Exxon and geophysical data collected by WCC to develop a summary understanding of the study area's geotechnical conditions;

- conducting of office analyses and evaluations to present preliminary geotechnical design considerations for the proposed facilities described in Table 1.1;

- recommendation of geotechnical items for future study; and

- preparation of a report summarizing our assessments.

The following sections of the report summarize site subsurface conditions, outline geotechnical design considerations for proposed facilities, and present our recommendations for future studies. A list of pertinent references is presented at the end of the report.

2.0
GENERAL GEOTECHNICAL CONDITIONS

Our understanding of the general geotechnical conditions in the Point Thomson Development area is derived from a review of the soil investigation reports by HLA dated June 1980 and June 1982 and the data collected during the geophysical investigations by WCC in the summer of 1982. The geophysical data interpretations completed during this program have been reported in final reports dated 15 January 1983. Our review has incorporated the results of the work done to date but we have not attempted to do further data interpretations during the course of this assignment.

The purpose of this section of the report is to identify the subsurface soil conditions in the study area and to evaluate the pertinent properties of soil strata that may underlie the proposed facilities. These interpreted conditions and properties have been utilized for the analyses and design considerations presented in subsequent sections of the report.

2.1 DATA BASE

2.1.1 Harding-Lawson Associates Report Dated 4 June 1980

This report presented the results of soil investigation studies to evaluate gravel sources in the Point Thomson area and in the Sagavanirktok Delta region. A total of 118 borings were drilled to depths ranging from 20 to 100 ft. The borings pertaining to the Point Thomson area are summarized below:

Site	Number of Borings	Location
T1 to T10 and TC	22	Onshore between Point Gordon and Point Thomson
T3	30	Inland from Point Hopson
T9	24	Inland from Point Thomson

A total of 76 borings were drilled in the Point Thomson area and impulse radar survey lines were run concurrently with the drilling program to correlate stratigraphic data between boreholes.

The main purpose of this investigation was to evaluate gravel sources and a large number of the borings were concentrated in the area of Sites T3 and T9. The data from the borings and the radar surveys indicate a typical onshore soil profile consisting of the following: a thin, surficial layer of organics, three to six ft of silts and silty sands underlain by gravelly sand or sandy gravel. It was noted that the ice content in the subsurface soils decreased markedly below 15-ft depth. In the fine-grained silts, as much as 50 percent of the soil volume was typically found to be ice and massive ice; these conditions were encountered in about 30 percent of the borings.

2.1.2 Harding-Lawson Associates Report Dated June 1982

A preliminary geotechnical investigation program was undertaken to study the onshore and offshore areas near Point Thomson. A total of 23 test borings were drilled during this program as follows: 5 borings onshore, 14 borings offshore over the ice and 4 borings on the barrier islands. Ground temperature monitoring instrumentation was installed and temperature readings were taken. Laboratory testing was done on soil samples to evaluate the engineering properties.

The data presented regarding the geotechnical conditions in the Point Thomson area form an important data base utilized in our evaluations. These conditions and our interpretations are summarized in Section 2.2 of this report.

2.1.3 Woodward-Clyde Consultants Reports Dated 15 January 1983

WCC conducted onshore and marine geophysical investigation surveys during the summer of 1982. The results of these surveys were reported in our 15 January 1983 reports. For purposes of the geotechnical interpretations contained in this report, the following items of geophysical data were used:

1. Onshore seismic refraction data allowing interpretation of acoustic permafrost and massive ice occurrence in the shore transition zones;

2. Onshore conductivity and soil probing resistivity data providing information on active layer thicknesses and massive ice occurrences;

3. Offshore data on seafloor features providing information on water depths, mudline soils and sea floor features caused by ice movement;

4. Offshore geologic cross-sections along 5 lines constructed from 3.5-kHz and UNIBOOM subbottom profile data providing information on the shallow stratigraphy and structure, depth to a Pleistocene gravel horizon and to acoustic permafrost;

5. Temperature measurements in three wells providing a comparison of summer and winter temperature vs. depth relationships;

6. Maps of shallow gas, potential locations of non-ice-bonded offshore gravel deposits, and generalized models of acoustic permafrost.

2.2 SUBSURFACE CONDITIONS

The purpose of this section is to provide a succinct review of subsurface information provided by HLA and our geophysical program and to describe the pertinent engineering characteristics of those formations which we feel will be important in the Point Thomson development. For this review we have, as did HLA, divided the area into four zones in which subsurface conditions are categorized: onshore, lagoon, barrier islands, and offshore. These zones are shown in Figure 2.1. Based on the geology and borings provided by HLA and confirmations of our geophysical program, we have outlined the geotechnical characteristics which may affect conceptual and preliminary design planning. A conceptual north-south geotechnical profile through the Point Thomson Development area is presented in Figure 2.2. Table 2.1 presents selected soil properties for the geologic strata of interest.

2.2.1 Onshore

Onshore borings, geology and geophysical data indicate that the Pleistocene deposits start at depths of 5 to 15 ft and generally consist of gravels and sands with varying amounts of ice and silt. Overlying these gravels are Holocene deposits, which are generally silts to silty sands with organics and contain little, if any, gravel. Both deposits within the upper hundred feet appear to be completely frozen except for a thin active layer, 2 to 3 ft in thickness. Gravel is generally available onshore and is covered by 5 to 15 ft of silty sandy overburden. This gravel may have high silt and ice contents in some locations and is clean with low ice contents in other locations. Preferred sources of gravel would come from the cleaner, more ice-free materials.

The most important consideration in the material characterization of onshore subsurface materials is the fact that these materials are frozen and are variable in their ice and fines contents. For this reason, any facilities, including pipelines, structures and pads, which have a tendency to thaw these permafrost strata, must consider thaw settlement.

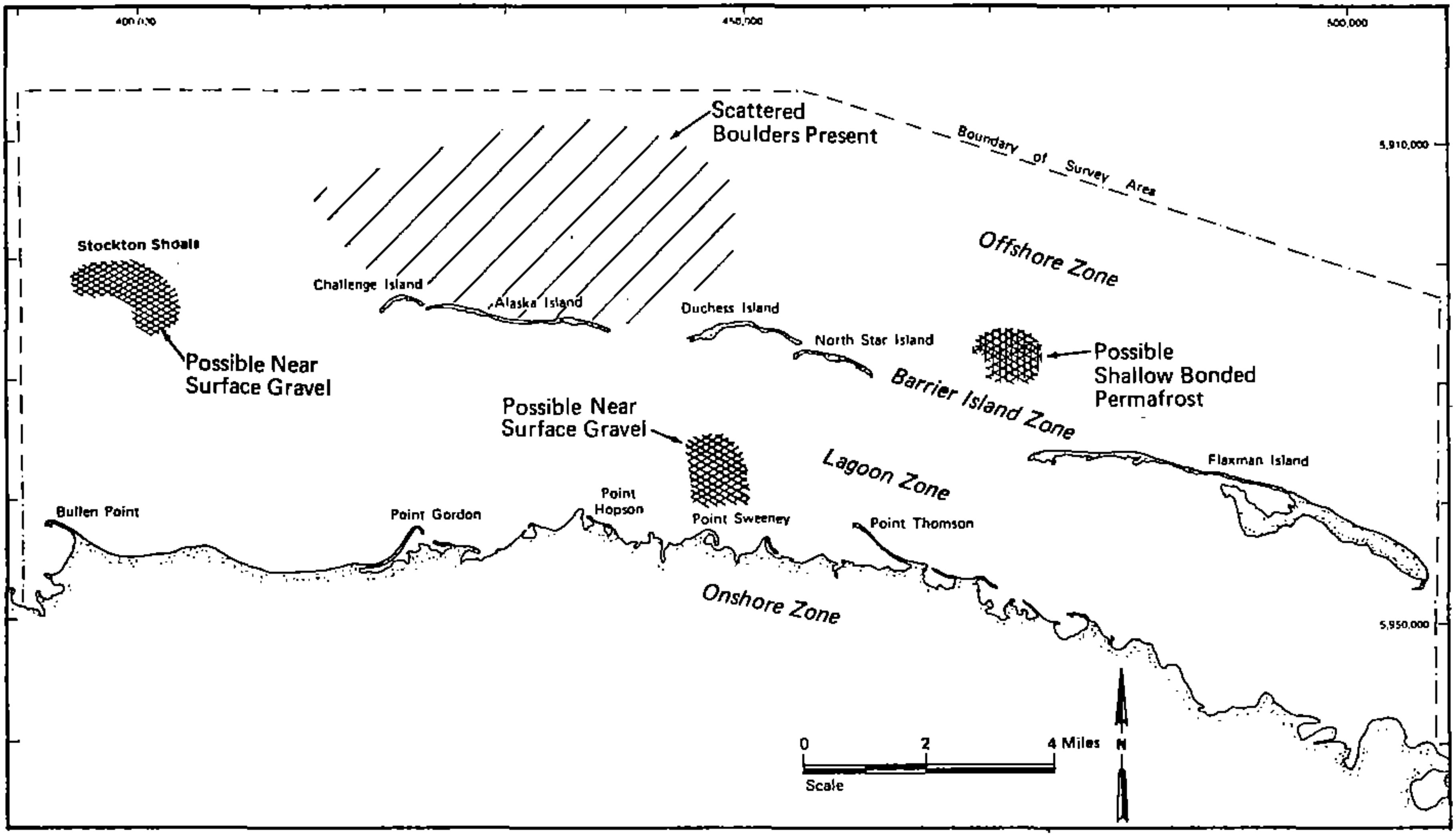


Figure 2.1 – GEOTECHNICAL ZONES

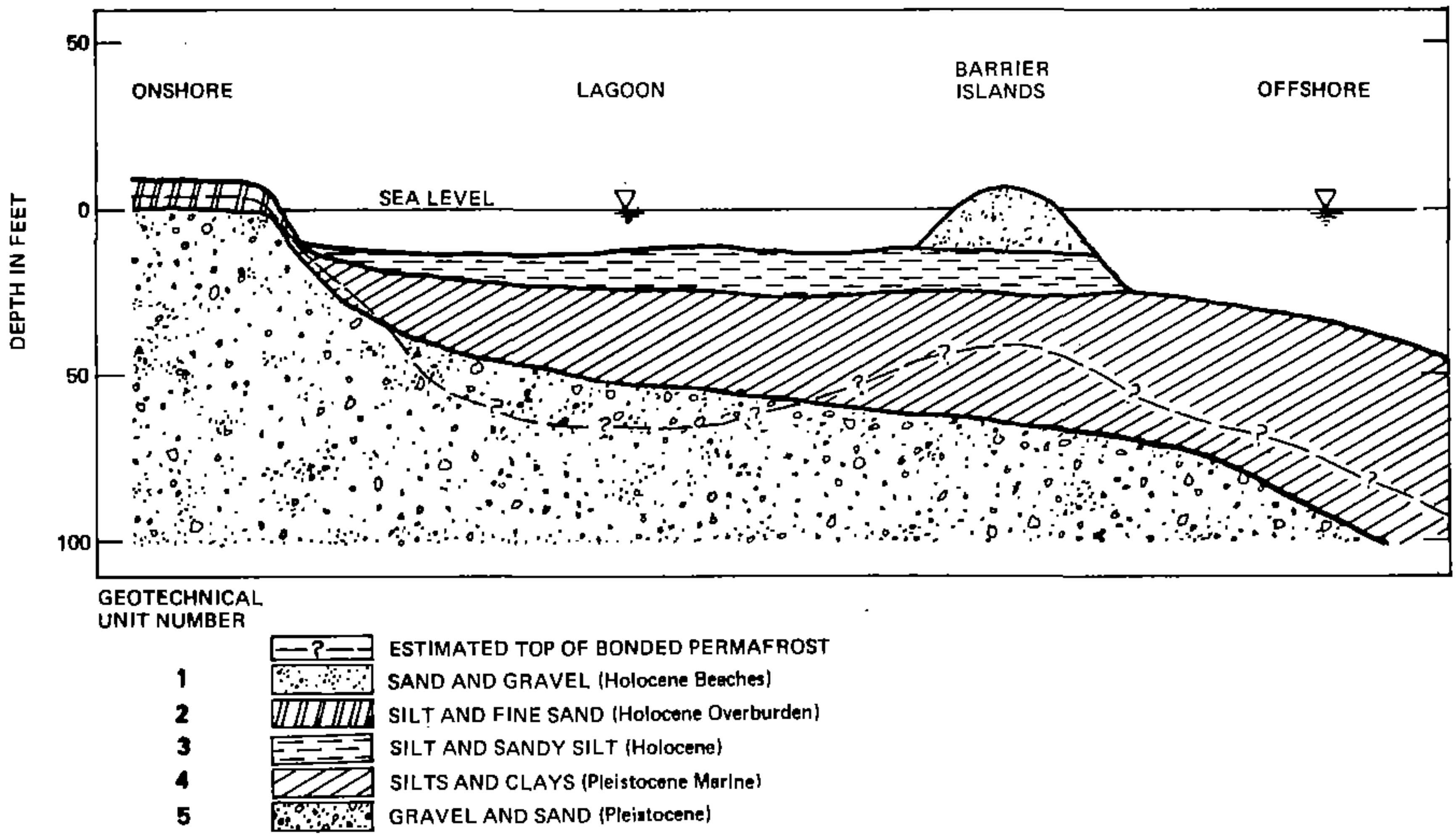


Figure 2.2 - CONCEPTUAL GEOTECHNICAL PROFILE

Table 2.1

SUMMARY OF ESTIMATED SOIL PROPERTIES

FOR CONCEPTUAL DESIGN

Geotechnical Unit*	Material Type	Thermal State	Moisture Content (%)	Dry Density (PCF)
1	Sand and Gravel	a	15	110
2	Silt and Fine Sand	b	25	90
3	Silt	Thawed	30	80
4	Silt and Clay	c	25	100
5	Gravel and Sand	Mostly Frozen	10	130

Notes

a Mostly thawed with seasonal frost

b Mostly frozen with seasonal thawing

c Mostly thawed with some frozen in lower portions of stratum

*See Figure 2.2

Although some of the gravel strata may be thaw stable, it is believed that extensive boring and testing programs would be necessary to identify those areas.

2.2.2 Lagoon Area

An east-west trending lagoon is situated between the shoreline on the south and the barrier islands and associated shoals to the north. The lagoon is two to three miles wide and extends for over 20 miles across the project area. The lagoon is underlain by a Pleistocene alluvium layer of sandy gravels and gravelly sands. This layer is found at depths of 20 to 60 feet below mudline in the lagoon and dips gently to the north. In general, these gravels are overlain by Pleistocene marine deposits which are more fine-grained, consisting primarily of silts and clays. These deposits are in turn overlain by the more recent (Holocene) deposits, which are generally more coarse-grained and consist primarily of silts and silty sands. With the exception of the zones that are very close to the shoreline, the deposits in the lagoon were found to be thawed (i.e., non-ice-bonded) during the HLA boring program. Our geophysical surveys were unable to detect any strata in this area which could be identified as "acoustic permafrost", i.e., material that is sufficiently bonded to act as a seismic reflector or to have a high, 8000 fps plus, seismic velocity, except near the shorelines. Near the shorelines, the permafrost dips steeply going as deep as 25 to 35 ft below the mudline within 100 ft of the shoreline, as shown in our refraction seismic lines near Point Gordon and by HLA's borings near Point Gordon and Point Hopson.

2.2.3 Barrier Islands

Most of the barrier islands are low, ephemeral sand bars. The locations and shapes of the islands are continually changing with the exception of the eastern end of Flaxman Island, which is apparently a relic of a formerly more extensive mainland. This shifting is important in understanding the engineering characteristics of the barrier islands. With the exception of the eastern end of Flaxman, there are indications (low seismic velocities, low blow counts and logged thawed zones) of the possible presence of taliks, i.e., zones of non-ice-bonded or partially bonded sediments bounded by frozen zones, on all of the barrier islands. The significant feature of these taliks is that they represent a transition zone that may have engineering properties more similar to thawed zones than to frozen zones. The islands are underlain by materials that are mostly non-bonded to a depth of 30 ft and weakly bonded to as deep as 60 ft, with a cap of frozen soil near the surface which is related to the depth of annual frost penetration. Foundations on the barrier islands would have to be designed to accommodate a frozen layer near the surface, a cold but still non-bonded or weakly bonded layer from some depth below the surface to about 30 to 60 ft depth, excepting the eastern end of Flaxman Island which is like an onshore area. The design of pipelines and other related structures will also need to take these features into account.

2.2.4 Offshore

Beyond the barrier islands, available information is more sketchy. In general, it appears the permafrost dips steeply in this area; the stiff, fine-grained Pleistocene marine deposits which are generally near the mudline (within the top 10 ft) and are 50 ft or so in thickness will provide support for most structures to be built in this area. It appears that these materials are generally thawed to depths which exceed the influence depths of most structures that may be planned, with the exception of localized areas as found in HLA Boring 16, which encountered bonded permafrost very near the seafloor. If conductor pipes, deep hot pipelines, and other structures capable of raising temperatures at depth are used in these areas, the design of these structures should consider the effects of thawing of deep frozen strata.

3.0
GEOTECHNICAL DESIGN CONSIDERATIONS

This section of the report addresses design considerations related to geotechnical aspects of the project. Addressed are pipelines, gravel islands, foundations for structures on gravel islands, causeways, and pipeline transitions from offshore to onshore.

3.1 PIPELINES

3.1.1 Introduction

This section addresses geotechnical design considerations for offshore and onshore pipelines in the Point Thomson area. As depicted in Fig. 3.1, offshore of Point Thomson is a series of low, barrier islands at a distance of roughly three miles from the coast. Water depths in the lagoon inside the barrier islands range from 6 to 20 ft. Water depths outside the barrier islands range up to 50 ft. Open water is limited to a few weeks during the late summer season each year. Ice gouging will be a design consideration for pipelines, but mainly in the region offshore from the barrier islands. Strudel scour is also a consideration but is quite localized, probably limited to the mouth of the Canning River.

According to the available information, continuous permafrost is present throughout the development area. Onshore, the top of the permafrost is within 2 to 3 ft of the ground surface. On the barrier islands, the active layer is 6 to 10 ft thick with 10 to 25 ft of non-ice-bonded material in some areas. In the lagoon and offshore of the barrier islands, the top of the ice-bonded permafrost generally dips down to 50 ft or more

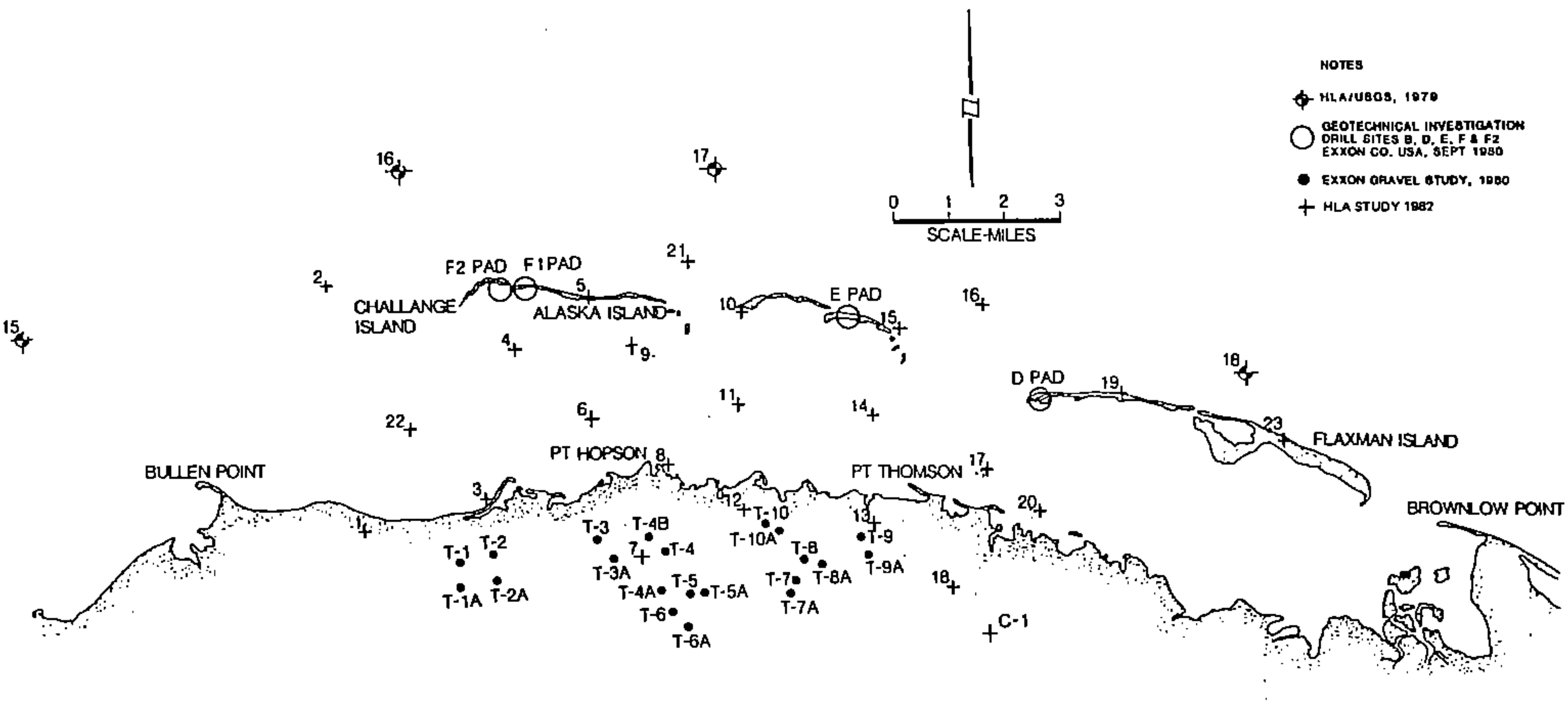


Figure 3.1 - MAP OF POINT THOMSON AREA

below the mudline. However, it is shallower north of Flaxman Island and in localized areas of shallow relict permafrost. The soil profile typically consists of a variable thickness of Holocene overburden overlying Pleistocene deposits. The overburden consists typically of silts and silty sands and is less consolidated than the Pleistocene materials, which are typically silts and clays overlying dense sands and gravels. The overburden is 5 to 15 ft thick onshore but is up to 30 ft thick in the lagoon, and is again less than 5 ft thick outside the barrier islands.

3.1.2 Routing Considerations

Onshore pipelines should preferably avoid thawed areas such as in and around thaw lakes. Alternately, pile supports will need to be considerably deeper in these areas. Also, major floodplain crossings, such as at the Canning River, will need to be designed for conditions of heavy overland flow and ice movement at breakup if they cannot be avoided. Offshore pipelines should avoid shallow permafrost where possible; most of the shallow permafrost in the offshore areas is along the barrier islands. The stiffer (Pleistocene) bottom sediments may be preferable from an excavation standpoint; because of the flatter ditch side slopes needed, excavation quantities will be significantly greater in the loose Holocene silts and sands which are the typical overburden in the lagoon area.

Our bathymetric survey (WCC, 1982a) confirmed presence of significant bottom roughness offshore of the islands. The deepest ice gouge mapped was 8 ft. Because of ice-gouging potential, pipelines in the ice-gouge areas offshore of the barrier islands may need to be deeply buried. For the same reason, the entrances between the islands should be avoided if possible. It appears that pipeline routings passing through or near the Point Hopson area could take advantage of near-surface stiff and/or gravelly soils there. Similarly, routings across the lagoon in the vicinity of HLA Boring 11 might be advantageous because of the shallow depth to gravel there. Bundling of several lines into the same trench (if they are compatible with each other) would probably be optimal from a cost standpoint. The area immediately north of Challenge and Alaska Islands contains numerous ice-rafted glacial boulders up to 2-1/2 ft in size (HLA, 1982); these could pose a hindrance to seabottom excavation there.

3.1.3 Construction Methods

Experience to date with offshore, arctic pipelines is limited to the Canadian Arctic. In the U.S. Beaufort Sea, plans are underway to bring crude oil ashore from Endicott Field in a gravel causeway. This causeway will be in relatively shallow water and will be underlain by fine-grained permafrost. To prevent thawing of the permafrost by the heat from the warm (up to 200°F) oil, the gathering lines will be insulated.

Offshore of Point Thomson, it appears that different construction methods would be optimal inside and outside the barrier islands. Inside the barrier islands, the most promising pipeline construction methods are (1) burial in a causeway as is planned for part of the Endicott development, (2) winter laying through the ice as depicted in Figure 3.2, or (3) pulling into place either in summer or winter. Feasibility of using lay barges at Point Thomson is limited by (1) shallow water depths, and (2) the very short summer season, during which the pack ice does not always retreat from the barrier islands. Laying of pipelines offshore of the barrier islands will be challenging and may require the development of innovative construction methods.

3.1.4 Design of Offshore Pipelines

Several possible construction modes have been considered for offshore arctic pipelines. Three of these are depicted in Figure 3.3. Causeway burial (3.3a) provides good protection for and access to the pipelines but would be costly in deeper water (>10 ft deep) because of gravel quantities required. Stability of the causeway under ice forces is also in question; see Section 3.4. Also, a pipeline buried in a causeway may be subjected to heavy superimposed traffic loads. Burial below the mudline (see Fig. 3.3b) is probably more economical. Based mainly on considerations of ice gouging, Timmermans (1982) recommends minimum cover depths as

Based on the above criteria and a 50-kip design load, the following table presents design pile lengths for 12-inch-diameter pipe piles slurried back in 18-inch-diameter predrilled holes. Since jacking controls this particular design, consideration of end bearing would not shorten the pile lengths required for this case.

EMBEDMENT LENGTH FOR PIPE PILES

Type of Cold Permafrost	Design Embedment Length (ft)
Ice-Rich (γ_d<70 pcf)	17
Ice-Poor (γ_d>70 pcf)	10

These lengths are similar to those for slurried piles obtained by HLA (1982, Plate VII-1). For driven piles, HLA recommend greater penetration depths. For a particular site, field load tests (see Black and Thomas, 1979) will be the best way to determine the pile embedment needed to support a particular design load.

3.1.6 Conclusions

Because permafrost is quite deep, buried pipelines without thermal protection are expected to be feasible over much of the offshore area. Depth of burial is mostly controlled by potential ice gouging. Required cover depths are about 12 ft offshore of the barrier islands but less in the lagoon. Onshore pipelines are conventionally elevated on piles.

3.2 GRAVEL ISLANDS

3.2.1 Situation Considered

The fundamental condition to be considered is described in Table 1.1. That information was obtained at the project meeting in October. To restate:

"Evaluate siting, stability considerations (e.g., sliding), settlement considerations, potential design constraints. Assume Point Thomson C-1 gravel for fill.

a) Offshore of barrier islands, water depths 16 to 40 ft, freeboard 16 to 26 ft.

b) In lagoon, freeboard 15 ft.

c) In lagoon, built partially on barrier island, freeboard 10 to 12 ft."

At the meeting it was also decided to use an ice load of 270 kips per foot of exposed surface on causeways. This same load was presumed to act on gravel islands. Further, not discussed at the meeting, we presumed the shape of the island to be circular, with 600-ft diameter at the shoulder, and 3 to 1 side slopes. (A larger island would be more stable.)

The term "C-1 gravel" refers to a gravel-ice mixture removed from Exxon's C-1 pit in the onshore portion of the Point Thomson area. In addition to the specified gravel, we also considered a less ice-rich gravel similar to the one used for the Beechey Point Island, with about 10 percent ice by weight. This was considered where use of the C-1 material with 25 percent of ice by weight indicated potentially unsatisfactory performance.

Erosion protection is not discussed herein. It is understood that, in all cases, suitable erosion protection should be provided.

3.2.2 Gravel Islands in Lagoon

The offshore area inside the barrier islands has a maximum water depth of about 20 ft. Unconsolidated silt sediments with maximum 20-ft thickness overlie consolidated clay and deeper granular Pleistocene strata. Bonded permafrost lies deeper than 50 ft.

Evaluations - The evaluations made included stability, settlement, and siting and design constraints. Four modes of potential instability were considered: horizontal sliding in the gravel (cone truncation), base sliding in the foundation, local edge failure under ice loads, and edge slumping during construction. These modes are illustrated in Figure 3.7. Contributions to settlement may come from several sources, including settlement due to thawing of the active layer, compression of the remaining above-water fill, compression of the below-water fill, consolidation of the weak silt overburden, and compression in the deeper clay and gravel. Siting or design constraints considered included principally identification of unusual or exceptional subsurface conditions which would place limitations on use of gravel islands.

Results - For gravel islands in the lagoon, typical results were as follows:

- Horizontal sliding in the gravel (cone truncation) at 10-ft depth below the water table (the depth of frost penetration below the water table in the first full winter) gave a factor of safety exceeding 2, for a friction angle of 35 degrees. Base sliding indicated a factor of safety exceeding 2 for either a drained strength with 32 degrees friction angle or an undrained strength of 0.4 ksf + $0.5\sigma_c$; for the case of a loose, unconsolidated, non-dilatant silt with an undrained strength of $0.45\sigma_c$ existing at the sea bottom, the factor of safety for base sliding in 10 ft of water was only 1.6, but increased to a factor of 2.0 for 20 ft of water depth. Potential downward edge slumping during construction and upward edge failure under ice loads are items typically corrected during island maintenance.

- Main contributions to settlement are thawing in the active layer to a depth approximating six feet below the surface, and compression of the underwater fill. Another significant contribution comes from compression of weak silt foundation material. In comparison, the compression of the remainder of the above-water fill and the

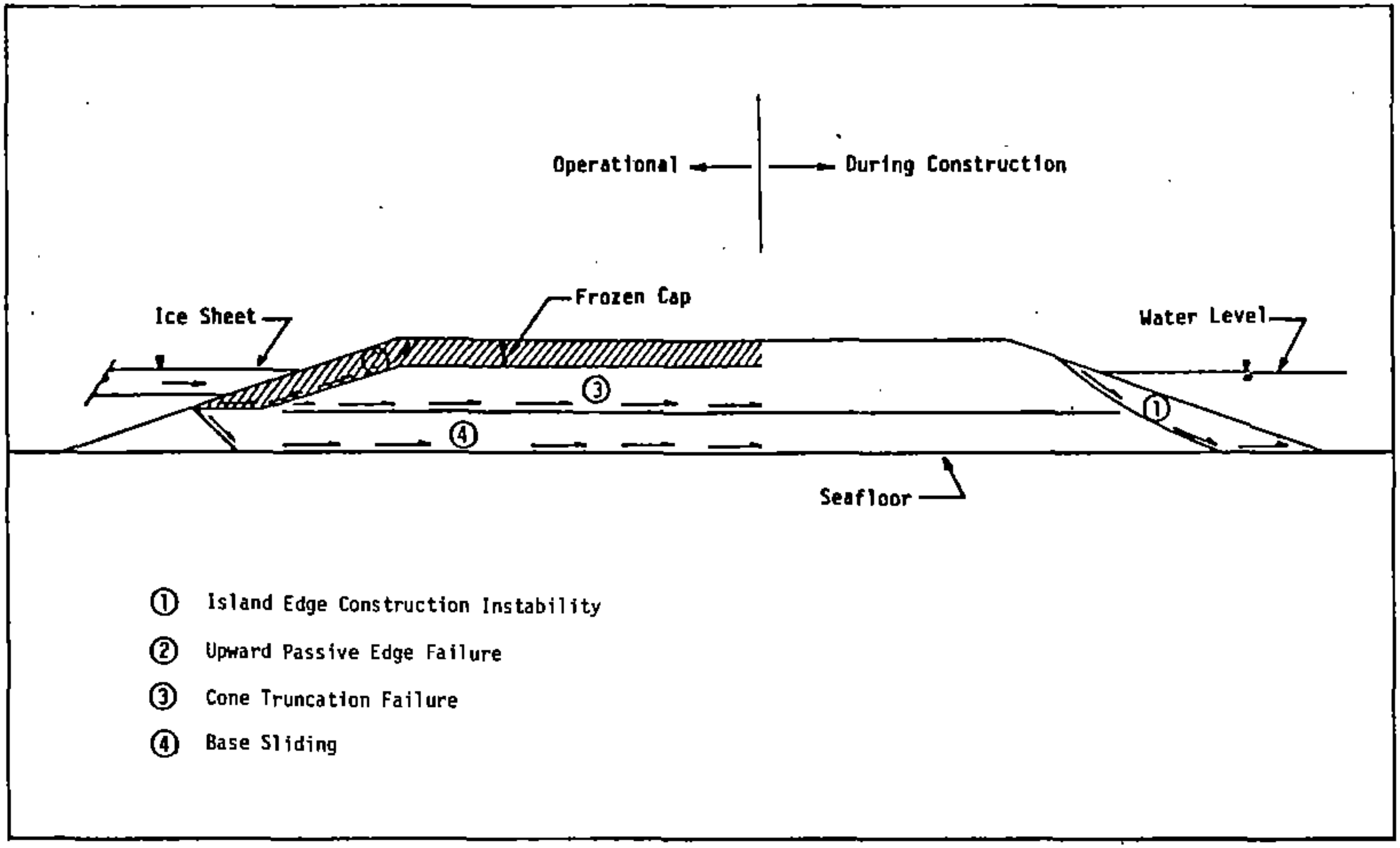


Figure 3.7 - POTENTIAL MODES OF ISLAND INSTABILITY

settlement in the deeper stiff clay and granular strata are small. The following representative settlements were estimated for the case of C-1 material throughout, for C-1 material with a 6-ft thick cap of gravel with lower ice content such as Beechey Point gravel, and for use of the drier gravel throughout the island:

Fill Material	Settlement for Island in 10 ft of water	Settlement for Island in 20 ft of water
C-1 Material	4.0 ft	5.5 ft
C-1 Material with 6-ft select fill cap	2.5 to 3 ft	4.0 ft
Select fill throughout	2.0 ft	3.0 ft

In all cases, 10 ft of compressible silt layer has been assumed. If the thickness of that layer is 20 ft, the settlement would be approximately 0.5 ft greater.

The calculated settlements are those expected after the end of construction. Significant settlement especially in the form of compression of the loose underwater fill is expected during construction, and this has not been included in the above estimates. It is expected that most of the settlement will take place over the first two to four years of the island's life. The active-layer settlement contribution will occur largely during the thawing season, while the deeper-seated settlement in the underwater fill will continue, though at a slower rate, in winter.

The following properties and behavior were utilized in making these estimates:

- o winter construction with winter-mined frozen gravel
- o active-layer depth 6 ft, thaw strain 33% (i.e., ultimate thaw depth below original surface is 9 ft)
- o compression of remainder of above-water fill, 6 in.
- o creep strain in below-water fill 20% (will remain frozen)

- o consolidation settlement in soft silt, 0.75 ft for 10-ft thickness, 1.25 ft for 20-ft thickness (based on $C_c = 0.075$)
- o consolidation settlement in stiff clay and underlying dense sand, 0.25 ft
- o settlement occurring during construction, 2 ft
- o all three main components of settlement--active layer thaw, creep in below-water fill, and consolidation in soft silt--will occur at the fastest rate initially (active-layer settlement in summers only) and will be substantially complete after 2 to 4 years, based mostly on judgment for each process.

- Main siting constraints relate to the occurrence of loose unconsolidated silt material at the mud line. If a gravel island is to be built on this material, the weak silts could be displaced during fill placement down to a depth of several feet. Also, localized sloughing of the perimeter slopes can occur during construction. This can be handled routinely during construction, but the builders should be aware of the need for additional fill material. Further, if the loose silt with an undrained shear strength of $0.45\sigma_c$ is encountered at shallow water depths (15 ft or less), the factor of safety against sliding could be below 2.

Conclusions - For a gravel island of the size contemplated, limiting the settlement appears to be the most difficult design condition to satisfy. Stability is of significant concern only where the near-surface material is loose silt. Settlements will be large if Point Thomson C-1 material is used throughout the island. Remedial measures include, alone or in combination:

(1) Placing fill to an elevation such that the after-settlement elevation would be approximately at the desired design elevation;

(2) Placing structures that are sensitive to differential settlement on appropriate foundations (see Section 3.3);

(3) Using drier gravel fill at least for the top 6 ft of the island cap;

(4) Using drier gravel for the entire island fill; or

(5) Using dry gravel in winter or thawed gravel in summer construction.

If loose silt is encountered at the location where an island is to be built and the island cannot be relocated, stability can be enhanced by removal of the silt or by constructing the island with a greater freeboard to increase, by consolidation strengthening, the shearing resistance on a horizontal sliding plane through the foundation.

3.2.3 Islands Offshore of Barrier Islands

Differences to Island in Lagoon - Water depths are deeper, approaching 40 to 50 ft. Near-surface Holocene materials are largely absent, improving foundation sliding resistance and slightly reducing expected settlements at the same water depth. Bonded permafrost is deep except near HLA Boring 16, which shows exceptional conditions that are addressed separately below under constraints.

Evaluations and Results - Because of greater freeboard, typically greater water depth and essential absence of the weak silt layer, stability is improved. Localized sloughing is still a possibility during construction, especially if some weak silt is encountered such as at USGS/HLA Boring 17.

With the deeper water, creep settlement in the below-water fill becomes dominant. The subgrade settlement becomes smaller because of the absence of weak silt. A matrix of typical settlement estimates has been developed as follows:

Fill Material	Settlement in 20 ft of water	Settlement in 40 ft of water
C-1 material throughout	5.0 ft	7.5 ft
Top 6 ft of island cap drier gravel, remainder C-1 gravel	3.5 ft	6.0 ft
Drier gravel used throughout	2.5 ft	4.0 ft

The soil properties and behavior used in making these settlement estimates are the same as those for the islands in the lagoon, Section 3.2.2.

Because of the predominance of creep settlement in the below-water fill and the lack of documented experience with it, the amount as well as the rate of these settlements cannot be well defined. Best estimates are for the rate to be similar to that for the island in the lagoon, i.e., substantial completion of settlements in 2 to 4 years.

A definite siting constraint is represented by the condition exhibited in HLA Boring 16, which shows permafrost at very shallow depth below the ocean bottom. This condition is so unusual and unexpected that it is believed to be localized, and it should be avoided in siting an island unless a detailed evaluation of the significance of this condition on island performance is made.

<u>Conclusions</u> - Stability presents no constraints. Settlement becomes more critical, and use of remedial measures discussed for the lagoon islands becomes more desirable. Localized subsurface conditions as exhibited by HLA Boring 16 may present significant problems with thaw settlement.

3.2.4 Gravel Island Partially on Natural Island and Partially in Lagoon

A gravel fill on natural islands has been used successfully by Exxon and others for exploratory drilling in the past. Exxon's drillsites were enclosed with sheet-pile walls.

Some of the natural islands are very narrow. Hence, it is visualized that perhaps 200 ft of the required 600-ft island dimension might be accommodated on the natural island, with the remaining 400 ft on the gently-sloping lagoon side of the island. A reasonable worst-case assumption is that, within this 400-ft distance from the island, the water depth might reach 10 ft and might be underlain by 10 ft of unconsolidated Holocene lagoon deposits.

Ice loads are not expected to present a stability problem because of the gentle underwater slope offshore. Use of structures to provide ice over-ride protection may be considered, as was done by Sohio at their drill site in the Point Thomson area.

Settlements will occur only due to active-layer thaw and some minor deeper compression on the natural island. On the farthest point on the lagoon side, the settlements might amount to the active-layer settlement plus two feet of post-construction settlement in the underwater fill and weak silt subgrade. Hence, a differential settlement of at least two feet across the north-south dimension of the island should be expected and accommodated. This differential settlement would develop in the first two to four years after construction.

In conclusion, a production island constructed partly on a natural island and partly on the lagoon side of a natural island represents a favorable condition. Stability does not appear to be a problem. Some differential settlements across the island are expected and should be designed for. Ice over-ride protection should be considered.

(Note: The calculated settlements reported above are significantly greater than those calculated by HLA. The reason for this is that they assumed the island gravel to be incompressible. This assumption is reasonable for summer-constructed islands or winter-constructed islands which do not thaw or experience creep. However, it underestimates settlements of winter-constructed islands which subsequently warm up, thaw partially, and experience creep of loosely-placed under-water fill.)

3.2.5 Conclusions

Gravel islands are feasible throughout the study area. Use of C-1 relatively ice-rich gravel will lead to large settlements over the first several years after construction; the settlements or their effects must be mitigated. Stability is only a potential problem where loose silt is encountered on the sea floor.

3.3 FOUNDATIONS ON GRAVEL ISLANDS

3.3.1 Introduction

To date, the only artificial gravel islands constructed in the Beaufort Sea have been exploration islands. These islands are designed for a short service life span (order of 3 years). Foundations for structures on these temporary islands have typically consisted of heavy timbers placed directly on the island's gravel working surface. Shimming of temporary structures on such supports is a relatively simple matter, but this may have to be done repeatedly. More stable permanent foundations will likely be needed for structures to be placed on production islands.

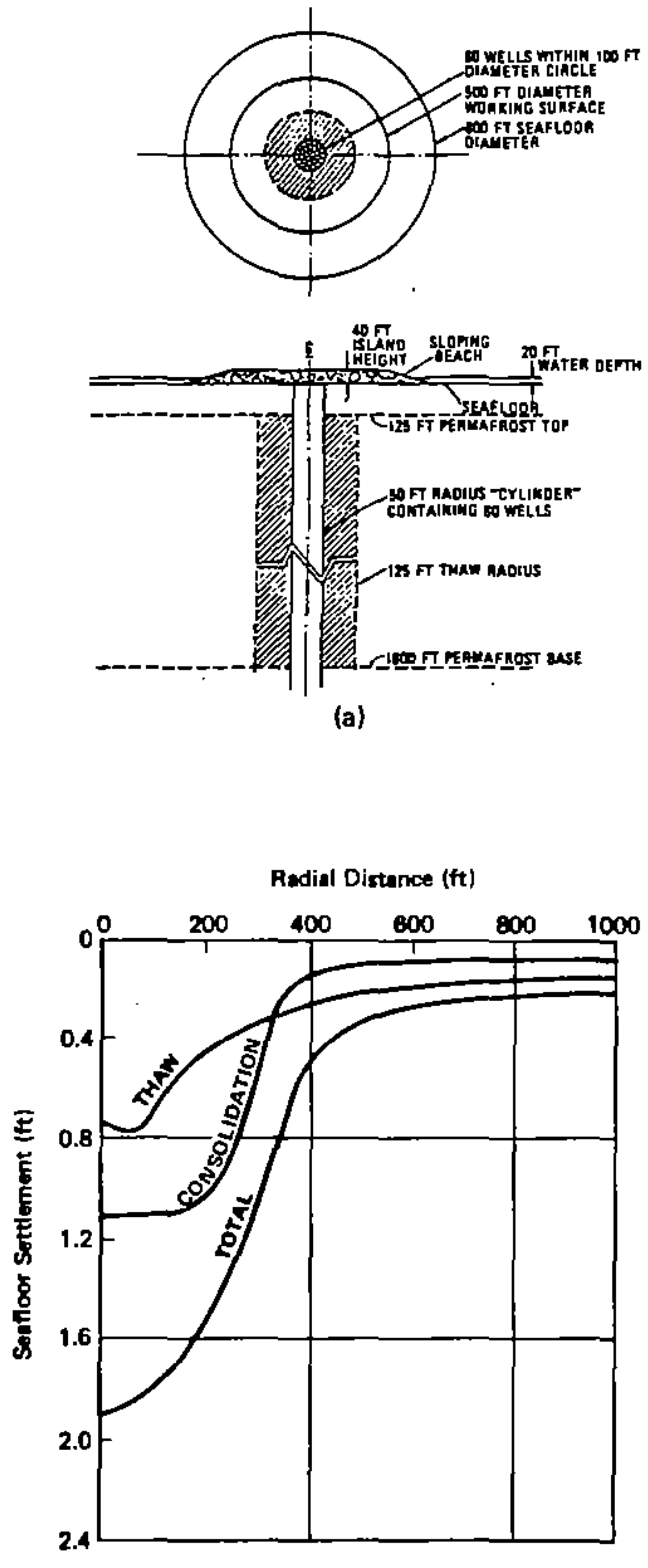
3.3.2 Structures

A number of types of structures are contemplated to be placed on production gravel islands. In addition to drill rigs and storage tanks, these include various kinds of one- and two-story modules associated with oil drilling and pumping which would be barged in, shop facilities, and portable living quarters for personnel. The loading applied by such structures is sudden compared with conventional buildings which are constructed in place over a period of months. Most of the structures would be heated and some buildings such as garages and shops may require slab-on-grade construction.

3.3.3 Design Conditions

Foundations for these structures need to be able to support the applied loads for the life of the structure without excessive settlement or heave or, alternately, provision for ready and periodic shimming adjustment of the foundations needs to be made. Design considerations are considerably different depending on whether the island is constructed in winter or summer. Because gravel placed in winter can contain up to 30 percent ice, subsequent settlement upon thawing of the active layer and due to creep can be substantial. On the other hand, unfrozen gravel placed and compacted in summer may experience little subsequent compression. These two cases are characterized in the following paragraphs and foundation design concepts for them are discussed in Section 3.3.4.

Summer-Constructed Islands - Settlements of an artificial island's surface result from (1) compression of the island fill, (2) consolidation of the frozen and unfrozen sub-bottom sediments, and (3) settlement due to radial thaw around a central casing cluster. Figure 3.8 illustrates the settlement patterns due to causes (2) and (3) as estimated by Goodman et al (1982) for artificial gravel islands in the Beaufort Sea. Figure 3.8a shows the typical island configuration and subsurface conditions considered. Figure 3.8b shows estimated settlements due to consolidation and thaw settlement (causes (2) and (3)) as a function of radial distance from the island center. The consolidation settlements presume that permafrost is encountered at a depth of 125 ft below the sea floor. The settlements associated with thawing correspond to a thaw radius of 125 ft. Total seafloor settlement is the sum of the consolidation and the thaw settlement. For the published case, this amounts to 1.9 ft at the center of the island and 1.4 ft at the edge of the working



From Goodman, Fischer & Garrett (1982)

Figure 3.8 – CALCULATED GRAVEL ISLAND SETTLEMENT PATTERNS

surface (250-ft radius), for a differential settlement of 0.5 ft in 250 ft (ratio 0.002). This is not a severe differential settlement; it would probably be more severe if permafrost were shallower. There may also be significant lateral displacement, as generally addressed in Goodman's paper. To this sea floor settlement must be added the settlement associated with compression of the island fill. For summer-constructed islands in relatively shallow water (less than 40 ft deep), this additional amount should be relatively small (order of 1 ft).

The summer-constructed island may experience some heave superimposed on the ongoing settlements, as the freeze front progresses down through the island fill and eventually into the sea floor soil. The heave is not expected to be large and will, it is believed, only retard the overall settlement.

Winter-Constructed Islands - Artificial islands built by Exxon to date have been constructed of frozen gravel during the winter season, initially through a hole cut into the ice, then built up in the dry to full height. Settlement due to consolidation and thaw is expected to occur as described above. In addition, the winter-constructed island will experience significant compression of the island fill. This is described in Section 3.2. The settlement estimates given there include both fill compression and sea floor soil consolidation. Observations of a completed island constructed of relatively ice-poor gravel indicated three distinct zones or layers; see Fig. 3.9 (from Tart, 1982). The upper, compacted zone extended down to sea level and had a dry density of about 105 pcf, a winter temperature of about 14°F, and was dry in appearance. Zone B had a dry density of about 85 to 90 pcf, a late-winter temperature of about 20°F, and was damp in appearance. Zone C started about 9 ft below sea level, had a dry density similar to that of Zone B, and consisted of slushy gravel with a temperature of about 28°F; borings had to be cased in Zone C to keep the hole open. The slushy nature of Zone C soon after construction of the island was apparently due to the salinity of the pore water. A boring drilled through the same layer about a year later showed the material to be unsaturated. The apparent reduction in degree of saturation with time is unexplained.

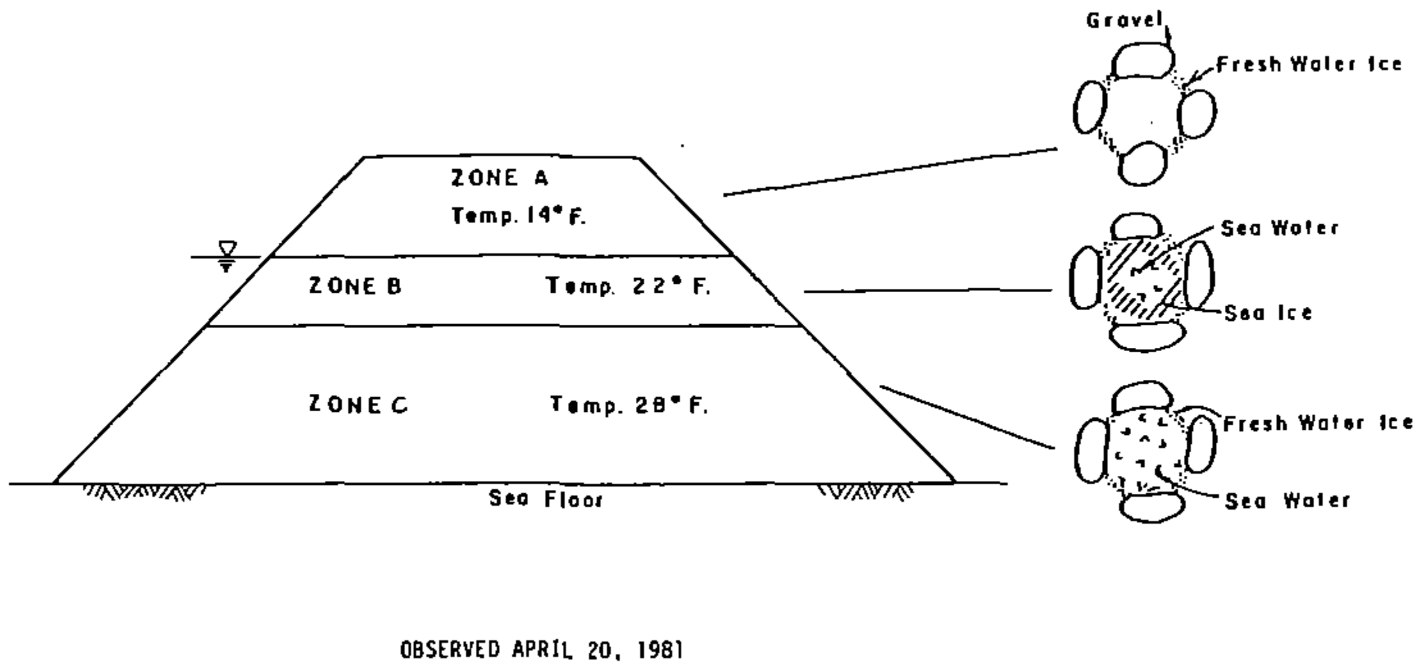


Figure 3.9 - THREE OBSERVED ZONES OF GRAVEL FILL
BEAUFORT SEA GRAVEL ISLAND

Seasonal temperature variation with depth observed at a Beaufort Sea gravel island is shown in Fig. 3.10 attached. The active-layer depth was about 6 ft and ground temperatures remained essentially constant throughout the year below about elevation -10 ft. Based on Sondex and survey data, the island settled about 1 ft during the first summer season and a total of about 2 ft. Most of the compression is believed to have occurred in Zones A and B. Rough estimates of island settlements made for the present study are shown in Section 3.2.

3.3.4 Foundation Design

Structures on the gravel islands may be supported either on shallow footings bearing on the fill material or on pile foundations gaining support from the materials below the island fill. It appears that, with the possible exception of the central core portion of the island, conventional spread footings will be adequate for summer-constructed islands. Similarly, if it is possible to "season" the island by waiting through the first one or two summers (one in case of select gravel cap, two in case of C-1 gravel) for the island to stabilize before erecting the structures, spread footings may also be satisfactory for winter-constructed islands. However, in the central-core portion of the islands and where structures must be erected during the first season, deep foundations may be needed. Furthermore, settlement of the island fill will likely exert downdrag loads on deep foundations and these additional loads will need to be considered in design.

Shallow Foundations - Shallow foundations are generally more economical than deep foundations. However, in the arctic, they face problems associated with freezing and thawing of the subsoils. If soils are initially frozen and can be kept frozen, even if relatively ice-rich, they seldom cause problems. Likewise, if soils are initially thawed and can be kept thawed, settlements are typically small. Bearing pressures and creep settlements are normally secondary issues for all but the most ice-rich materials or settlement-sensitive structures.

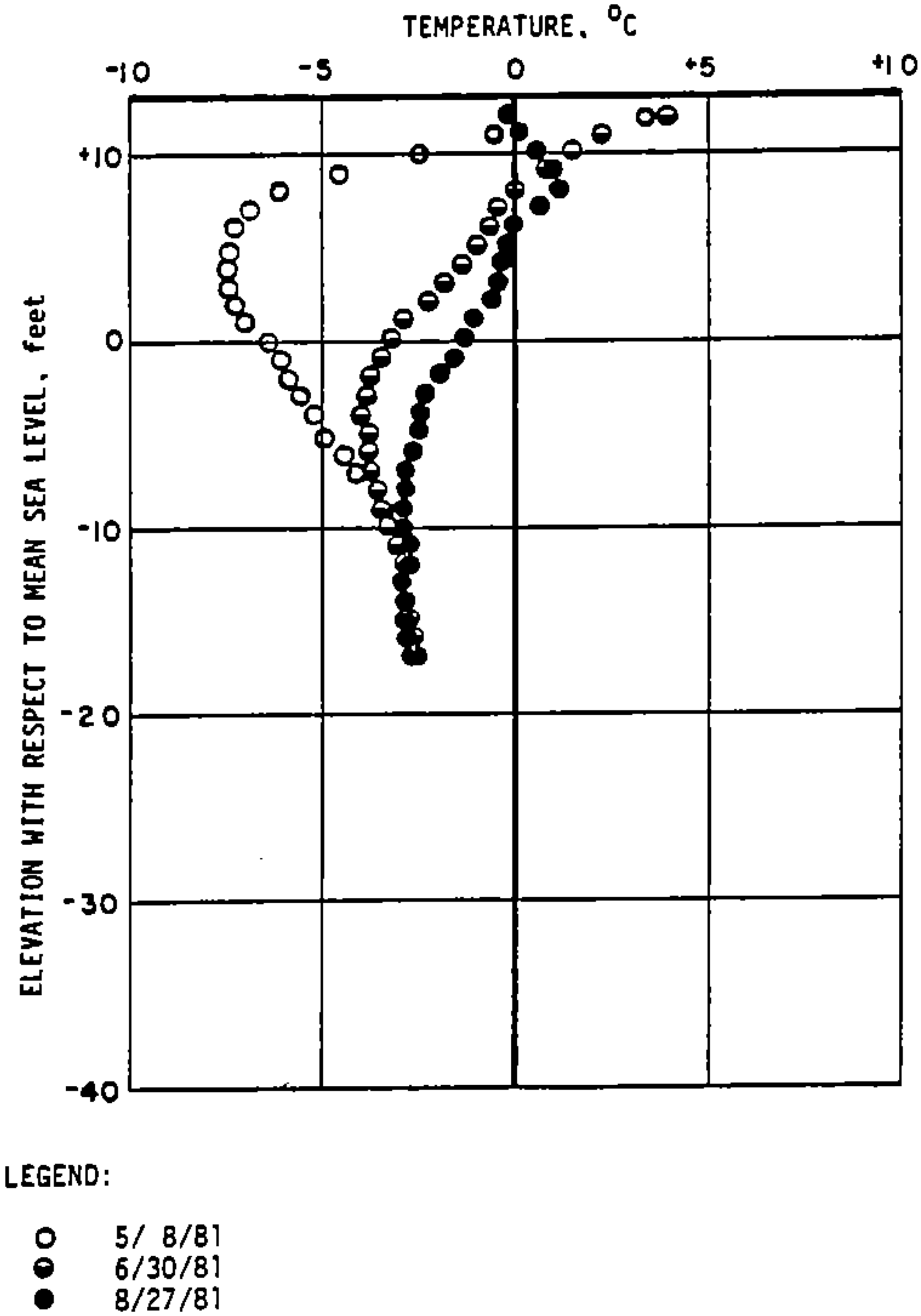


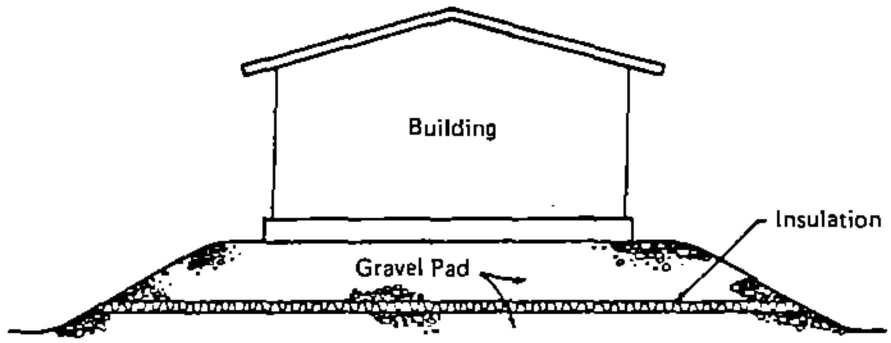
Figure 3.10 - THERMISTOR TEMPERATURE DATA
BEAUFORT SEA GRAVEL ISLAND

In an artificial gravel island, two unique phenomena, both of which tend to increase creep settlements, need to be considered. One of these is pore water salinity and the other is undersaturation. Salinity depresses the freezing point of the pore water and causes difficult-to-define relationships between temperature and ice bonding in soils. The degree of saturation of gravel materials above (and even to some depth below) sea level in a winter-constructed island is typically only about 50 percent with the remainder of the voids being filled with air.

HLA recommend use of timber or steel grillages sized on the basis of allowable bearing pressures of 3 to 8 ksf, depending on ice content of the gravel and the width (2 to 15 ft) and depth (0 to 8 ft) of the footing. We agree that this type of footing would probably be more practical than concrete, even precast concrete. However, thermal aspects need to be fully considered in the design recommendations.

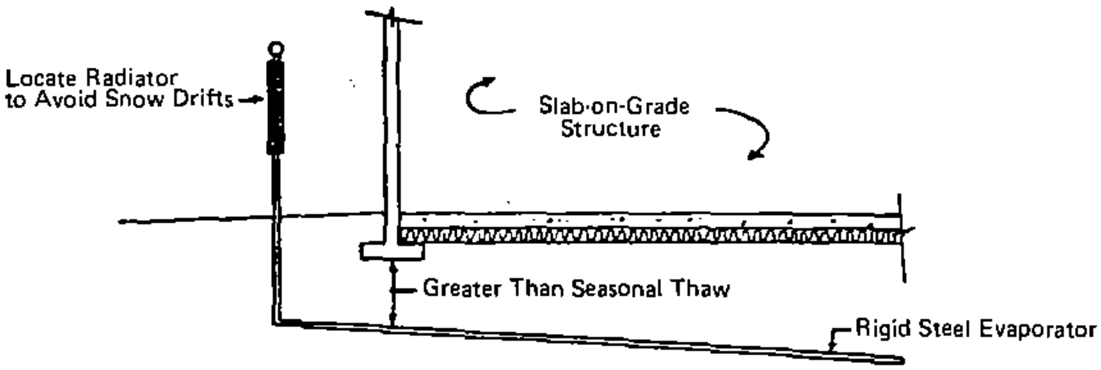
If heated structures are placed in contact with the ground, a thaw bulb will develop and grow beneath them. Depending on the ice content of the gravel, settlements will occur as the ice melts. If the fines content of the gravel exceeds a limiting amount (about 10 percent), frost heaving of the footings may occur during winter freeze-up. Provision of an unheated air space beneath heated structures to mitigate these effects is thermally a very effective and proven solution. If this cannot be done, thaw bulb growth can be limited by provision beneath the floor slab of (1) insulation (see Fig. 3.11a), (2) air ducts (see Fig. 3.12), or (3) convective heat pipes (see Fig. 3.11b). These are also proven techniques (see Long et al, 1982). Solutions to mitigate frost heave potential include local overexcavation and replacement with clean gravel, and placement of insulation around the footings to limit winter frost penetration. Mechanical refrigeration of foundations has been successfully used in the arctic but this solution appears to have long-term cost and maintenance drawbacks for this project.

In conclusion, several types of shallow foundations may be considered. Shallow foundations are most practical in island areas outside the



From Phukan et al (1978)

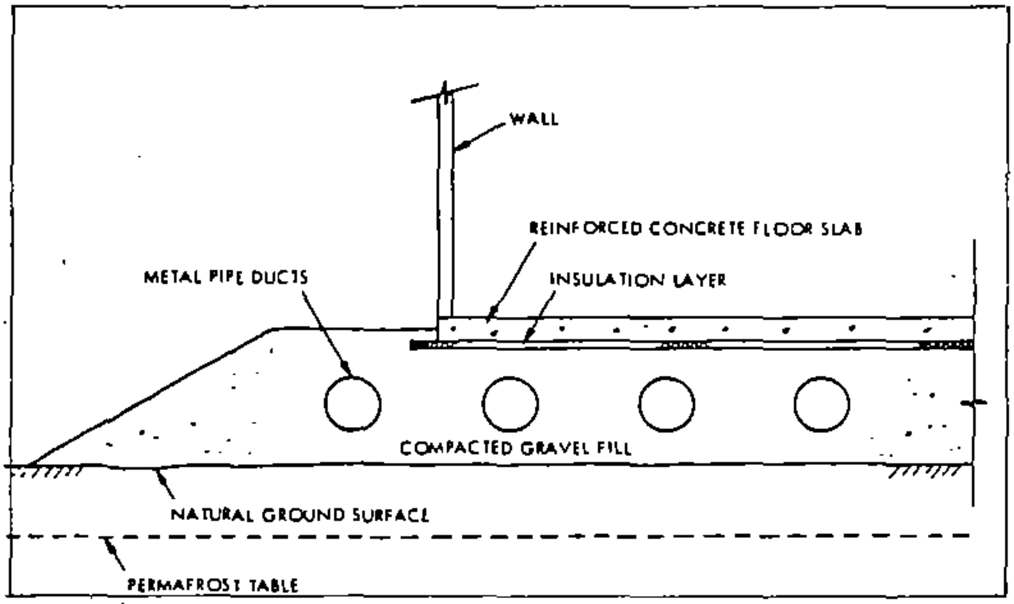
(a) Building Founded on Gravel Pad with Insulation



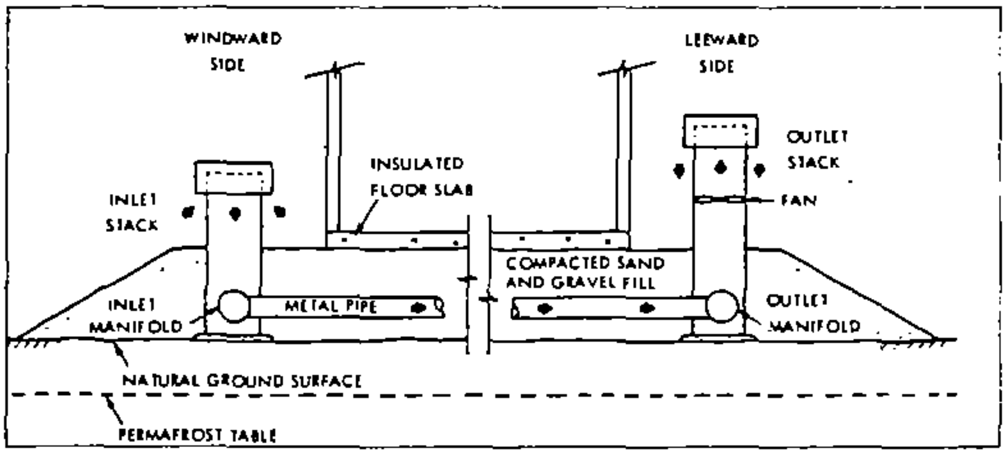
From Long & Yarmak (1982)

(b) Typical Thermo-Probe Installation

Figure 3.11 - TYPICAL ARCTIC FOUNDATIONS



(a) Sketch of insulated concrete floor slab on duct-ventilated compacted fill foundation.



(b) Sketch of insulated concrete floor slab on forced air, duct-ventilated fill foundation.

From Johnston, 1981

Figure 3.12 - DUCTED FOUNDATIONS

central core, for summer-constructed or "seasoned" islands, and for non-critical structures where periodic shimming of the foundations can be done.

Deep Foundations - Piles will likely be needed at Point Thomson to support settlement-susceptible structures to be placed (1) in the central core zone of the island, and (2) on "unseasoned" winter-constructed gravel islands. Typical offshore conditions for pile design are (1) gravel fill from the island top to the mudline (water depth up to 40 ft), and (2) primary load-bearing layers identified as a stiff, fine-grained (CL and ML) layer overlying dense sands and gravels. In the lagoon area, the stiff clays are overlain by a layer of loose silts and sands and, at the barrier islands, these layers are further overlain by a beach sand layer.

HLA proposed use of driven H-piles and steel pipe piles designed on the basis of skin friction. Based on assumed thawed soil skin friction values (which they apply to frozen as well as thawed soil layers), HLA (1982, Plate VI-8) calculated pile lengths of up to 120 ft required to support a range of design loads. According to the referenced plate, addition of downdrag loads may require installation of even longer piles.

We concur with use of driven piles, especially in view of the thawed, saturated, granular (caving) layers which will have to be penetrated. However, it appears to us that pile lengths could be shortened considerably by appropriate consideration of (1) end bearing, (2) use of frozen soil strengths in ice-bonded soil layers which will remain frozen, and (3) possible use of thermal devices in the piles. For example, 50 kips of allowable end bearing in the dense Pleistocene gravels will shorten the pile length by as much as 20 ft.

3.3.5 Conclusions

Where possible, provision of an unheated air space beneath heated structures is a desirable design feature. If this cannot be done, shallow foundations can still be used to support non-critical structures, for summer-constructed gravel islands, and in island areas outside the central core. Piles will likely be needed to support critical structures, structures on unseasoned winter-constructed gravel islands, and inside the central core area. Non-thermal piles may need to be designed for downdrag forces in addition to the structural load. Passively-refrigerated or, less likely, mechanically refrigerated shallow foundations may provide an economic alternative to deep foundations in some cases.

3.4 CAUSEWAYS

3.4.1 Situations Considered

Table 1.1 states:

"evaluate siting (generally), construction modes (berm, elevated on supports), potential design constraints. Assume freeboard 15 feet, ice load 270 kips per foot. Support for elevated causeway may be piles, caissons, or small gravel islands."

3.4.2 Berm Causeway

As is depicted in Figure 3.13a, the geometry considered for a causeway on a berm was an elevation of 15 ft, a shoulder-to-shoulder width of 40 ft, and side slopes of 3 (horizontal) to 1 (vertical). Evaluated were settlement, stability, and any design constraints. The potential requirement to allow water flow across the causeway at least at discrete locations was also considered.

Settlements are likely to be similar to those determined for gravel islands at similar locations; see Section 3.2. Hence settlements of the order indicated there should be expected and accommodated for in the design and utilization of the causeway. Drier fill material may be considered for use to reduce settlements; use of drier cap material is also of interest, and may be required for the top 2 to 3 ft for trafficability.

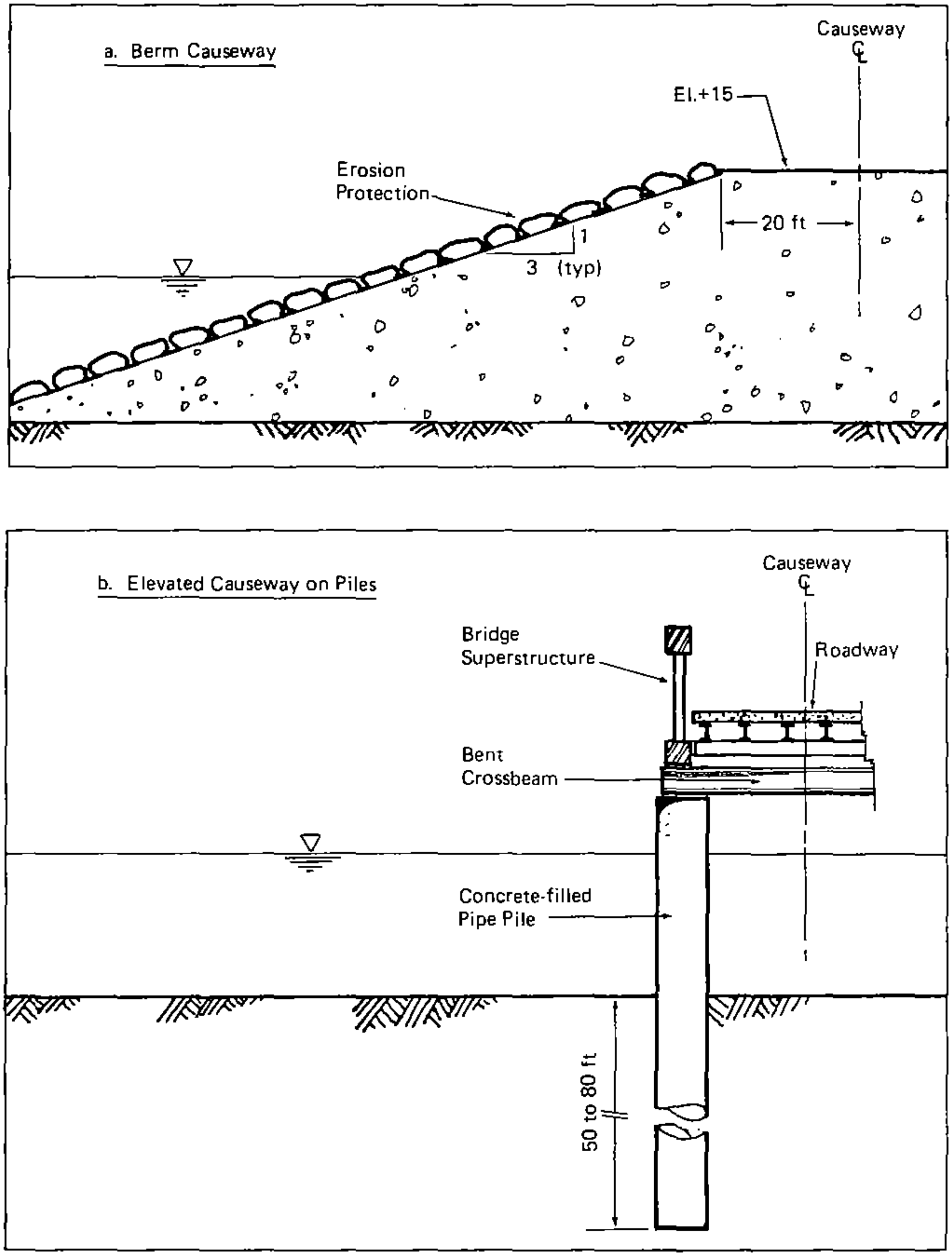


Figure 3.13 – CAUSEWAY MODES

Based on thawed soil shear strength (friction angle 35 degrees), mobilization of enough horizontal sliding resistance to resist the high specified ice force is difficult. For a plane located in the gravel fill at 10-foot depth below the water table, the resistance to truncation was calculated to be about 140 k/ft, which is only about half of the design ice force. Other stability cases, in particular bottom sliding, are less critical, except if unconsolidated silt is encountered at less than 20-ft depth and is not removed. Once the berm is frozen throughout, which is expected to be the case a few years after construction, adequate resistance to the relatively short-duration maximum ice force will likely be available. If the berm's size is increased to 50 ft shoulder width and 10 to 1 side slopes, the resistance mobilized to truncation exceeds 270 k/ft; in this case ice override must also be considered.

A non-geotechnical constraint may be represented by the need to allow flow of water across the causeway. This could be accommodated by provision of culverts or breaches with a clear-span bridge. Culverts could range from 12-inch-diameter pipes to large corrugated culverts, both placed mostly under water. Breaches could be sloped at longitudinal slopes of 3 (horizontal) to 1 (vertical) or steeper, and would have to be protected from erosion. For a 50-ft clear opening in ten feet of water, a bridge span of close to 200 ft would be required. Frequency or spacing of culverts or bridges could be selected as required, though due consideration should be given that the overall causeway structure is not weakened excessively.

Thus, the principal potential problem faced by the design is satisfaction of stability. While the stated numbers show inadequate stability, there are existing causeways in Prudhoe Bay, some of which have existed for many years. Water depth at the recently-completed causeway extension for the Waterflood Project is up to 14 feet, and the causeway has a dimension similar to that considered here.

Because the stipulated load of 270 k/ft was developed for refrozen rubble impinging on a circular island, it is probably too high for a causeway, especially in the protected lagoon. Also, the existing causeways are probably now largely frozen, and frost may have penetrated into the seafloor. The most critical period appears to be the first few years after construction. There is a risk of movement of the causeway in the early years if the applied ice force exceeds about one half of the specified 270 k/ft maximum ice force, and this risk is similar to that experienced in their first few years by existing causeway structures in the Beaufort Sea. We believe that a risk analysis for this situation would be highly revealing. This analysis should consider the likelihood of forces exceeding the resistance developed, the amount and time history of movement when the force exceeds the resistance, and the potential consequences in terms of access, oil flow in any pipelines carried by the causeway, and economics. This type of an analysis may well allow construction of bermed causeways to proceed if certain operational precautions are taken.

3.4.3 Elevated Causeway

Structural spans on the order of 200 ft were considered for the elevated causeway depicted in Figure 3.13b. Actually, the design of the superstructure can be optimized if it is considered that the design of the supports is mostly controlled by the ice forces and very little by the applied causeway loads, and hence the cost of one support is almost independent of the support spacing.

Superstructure design was not considered in detail. We visualize that the superstructure would be a two-girder modular bridge, designed for the appropriate environmental and surcharge loads. We expect that there would be a significant cost penalty if these bridges had to be designed to allow heavy modules to be transported on them.

Support types considered were small gravel islands, sheet-pile caissons, and piles. Of these, the piles appeared to be most promising, for the reasons given in the following.

Individual gravel islands of circular shape, with a top diameter of the order of 40 ft, a top elevation of about +5 ft, and a 3 to 1 side slope fell far short of providing adequate resistance to ice, because of their light weight. Strengthening the sliding resistance of these gravel islands by use of short spud piles or by use of heat pipes to freeze the soil in the contact zone was considered, but it was concluded that their use would be very expensive and of questionable ultimate benefit.

Sheet pile caissons with perhaps 40-ft diameter were considered and rejected because of inadequate internal stability to resist the large applied ice forces.

The pile support concept evaluated was that of a bent of two large-diameter piles supporting a cross-beam, which in turn supports the superstructure (see Fig. 3.13b). To better resist ice forces, telescoping piles and four-pile bents rigidly connected at the top were also considered for use. The specific pile types considered were concrete-filled steel pipe piles of 4- to 8-ft diameter, with wall thicknesses of 1-1/4 inch for the 4-ft diameter pile and 2-1/4 inch for the 8-ft diameter pile.

The ice forces, which may come from any direction, are the most important loads acting on the piles, with the loads imposed from the bridge having a secondary influence. Design of the pile bents and their cost are therefore essentially independent of the vertical applied load. Thus the spans between the bents may be selected to minimize the total cost of the system.

Preliminary conceptual P-Y analyses utilizing the 270 k/ft of ice force demonstrated that large-diameter piles are feasible as supports for an elevated causeway, provided the combined depth of water and weak silt sediments is limited. An 8-ft diameter, free-headed, concrete-filled pipe pile appears adequate for a 20-ft combined depth of water plus silt sediments. The pile had a wall thickness of 2-1/4 inches. The weak silt sediment was assigned a 50-pcf dry density, a 30-degree friction angle and 20 percent relative density. The bearing layer was assigned an 85-pcf dry density, a 40-degree friction angle and 95 percent relative density; the pile penetrated 80 ft into this layer. This case showed a waterline displacement of about 2 in. and a steel stress of 24 ksi. For a 40-ft combined depth of water plus weak silt sediments, the 8-ft pile either must be reduced to 4-ft diameter through the ice zone to halve the ice load, or the 8-ft pile must be fixed at the top, by use of four-pile square clusters of piles rigidly framed together at the top. In the former case, the estimated water line displacement was about 3 in. and the maximum steel stress 22 ksi; in the latter case the displacement was estimated at 1-1/2 inches and the maximum stress at 23 ksi. Other schemes are undoubtedly feasible; these may involve cross bracing below the ice level.

Four-foot diameter piles were used on ARCO's Kuparuk River pipeline bridge. To justify such smaller piles for a pile-supported causeway, design ice loads would have to be reduced to about half the 270 kips/ft used here (combined with a risk analysis as discussed for a berm causeway), and/or more steel would have to be used either by using a thicker pile section or by steel-reinforcing the concrete fill.

Thus it is seen that the pile diameter is essentially controlled by the lateral load imposed by the ice. The lateral load also requires a minimum depth of penetration. For the cases considered here, about 50 ft of penetration into the dense Pleistocene gravel or stiff clay is adequate to support the lateral loads. The vertical load-carrying capacity with this penetration is of the order of 1,500 kips for the 8-foot diameter pile, 750 kips for the 4-foot diameter pile. If greater vertical capacity is required (e.g., to support heavy modules), the piles may be lengthened; the carrying capacity increases approximately 30 kips for each foot of additional penetration of an 8-ft diameter pile, 15 kips for a 4-ft diameter pile. For piles bottoming out in gravel, end-bearing may also be considered.

3.4.4 Conclusions

A causeway constructed of a continuous gravel berm may not provide adequate resistance for the maximum specified ice load in the early years. In view of precedent, such a structure should still be given serious consideration. A risk analysis could quantify the risks and help in formulating operational precautions and constraints. Culverts or bridge spans could be provided in such a berm to allow water flow across the berm.

An elevated causeway could be constructed on heavy concrete-filled steel pipe piles. Such a structure can be designed to resist the specified ice loads without excessive stresses or displacements. However, the costs will likely be high, and a risk analysis may also allow lighter construction here at the price of operational constraints.

3.5 PIPELINE SEA-LAND TRANSITION

The point where an offshore pipeline comes on land is a particularly difficult location requiring careful attention. This situation includes the transition of the pipeline onto an offshore natural or man-made island. The challenge is to protect the pipeline from (1) permafrost thaw effects, and (2) ice forces and ice override.

It is presumed that far offshore the pipeline is buried at somewhere between 5- and 12-ft depth. Onshore the pipeline is visualized to be elevated on supports. At a significant distance from shore (say 1000 ft or more) permafrost is generally encountered at depths exceeding 40 ft below the mud line, and permafrost thaw effects are not expected to be severe. As the Alaskan shore is approached, permafrost rises to nearer the mudline and probably reaches the mudline where the water depth is about 5 ft (where ground-fast ice exists in winter). Onshore permafrost is continuous below an about 2-ft thick active layer. Effects of ice include potential ice override over the beach and ice pushing forces on any exposed structure such as an embankment. Erosion is also a factor as the shoreline is reported to be receding at rates of several feet per year (HLA, 1982, Vol. I, p III-47).

The two principal modes of construction for the transition are a buried insulated mode and a berm insulated mode. The principal features of these two modes are discussed below.

3.5.1 Buried Transition

The principal features of the buried transition mode are shown in the attached Figure 3.14. The pipeline is buried through the transition, and is partially thermally protected principally by insulation, although other means discussed below may be used.

On the water side the pipeline is buried in a trench with perhaps 4 to 8 feet of cover. The cover is deepened as deeper water is approached. The partial thermal protection is continued to the point where the offshore permafrost is about 40 ft below the mudline and is judged to have no severe effect (the exact depth at which thermal protection can be discontinued is still to be established). Near shore, at least out to the 5-ft depth, construction is probably best done in winter through the bottom-fast ice, however, construction can also be done in summer either underwater or by first placing an embankment to above sea level and then excavating the trench from that embankment. The winter construction method would provide better construction control in this critical zone, and would allow more flexibility in providing thermal protection, as discussed below. The zone of winter construction may be extended seaward by artificially thickening the ice.

On the land side, the buried thermally-protected mode is continued as far as is judged needed for reasons of ice override and erosion. However, because the thermal protection will likely be expensive and is only partial (i.e., there will be some thaw), this distance should be minimized, and may be as short as 100 ft. Beyond this distance is a transition to the elevated mode.

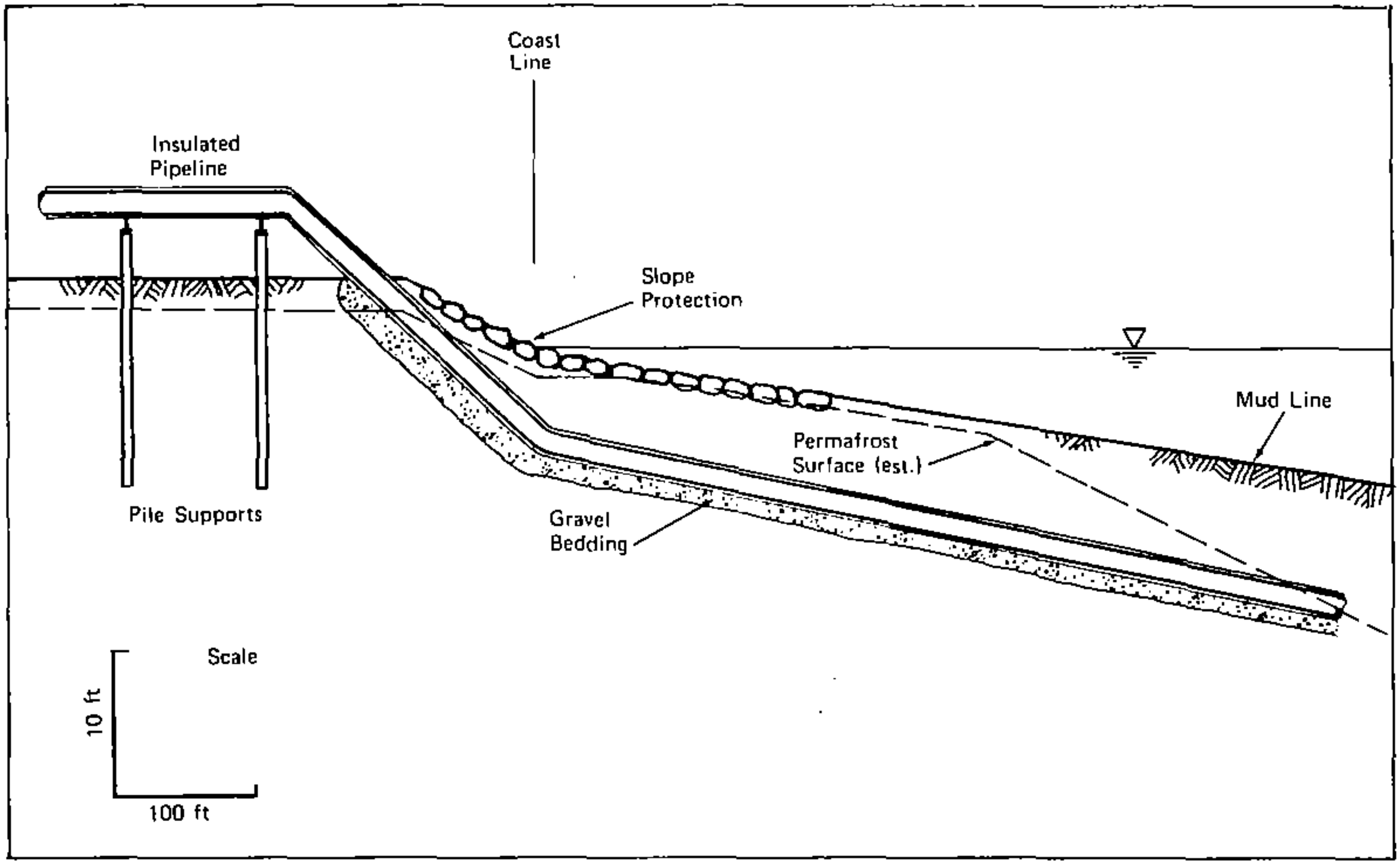


Figure 3.14 - BURIED SHORE TRANSITION (CONCEPTUAL)

The key issues for the buried scheme are the thermal protection and the permafrost thaw effects. The controlling thaw effect is likely to be thaw settlement. The thermal protection visualized is principally waterproof insulation applied to the pipe, plus a gravel bedding where feasible. Overexcavation and a thickened gravel bedding (perhaps up to 4 ft) may be utilized where possible.

Use of refrigeration may be considered as a last resort. This could consist of refrigeration lines placed in the trench beside the pipeline to some distance offshore through which refrigerated brine would be circulated. A refrigeration plant would be constructed on shore to maintain the refrigeration. Alternately, it may be feasible to utilize self-actuating heat pipe refrigeration with the radiators just on shore and the underground heat pipes extending on a gentle downward slope offshore to some distance, estimated to be perhaps 100 ft. Use of heat pipes farther offshore would require a protective berm, which is discussed under the berm scheme.

Thaw settlement and associated pipe bending have to be considered specifically and rigorously for this scheme. The factors to be considered are the thermal protection provided including insulation, gravel bedding, and any refrigeration; the nature of the subsurface permafrost; the expected amount of thaw and of consequent thaw settlement, including its variation along the line; and the susceptibility of the line to differential settlement. This evaluation would determine what detailed protection measures are needed to make this scheme feasible and safe.

Erosion protection would be provided in the vicinity of the transition to minimize erosion in this area. Nevertheless, in view of the ongoing shore migration in this area, consideration might be given to anticipate a certain amount of shore erosion in the design; otherwise, repairs may have to be made after a few years.

3.5.2 Berm Transition

In this mode the pipeline is brought to shore in a protective berm; see Figure 3.15. It is expected that the berm would extend about to the 8-ft water depth, or to the point where offshore permafrost is at least 40 ft deep and is judged to have no severe effect. At that point the pipe is brought out of the offshore trench and placed on a previously-placed gravel padding on the sea floor. Subsequently a protective berm is placed over the pipe. The berm will have just enough height over the pipe to provide adequate protection, expected to be perhaps 4 to 6 ft. The side slopes of the berm should be gentle enough that ice effects are adequately accommodated--this may require quite flat slopes and greater depth of cover. The design of the berm would also have to consider erosion and strudel scour. The entire berm, or at least the padding, should use such materials, construction seasons and construction methods to keep the berm's subsequent thaw settlement to acceptable levels; this is discussed in Section 3.2.2.

Thermal protection of the pipe is provided by heavy insulation, granular bedding, and possibly heat pipes. Use of better controlled, thicker bedding is believed more practical here than in the buried mode. Also, where the berm is significantly above the water surface, use of heat pipes may be considered to provide additional thermal protection. Nevertheless, mitigation of the thaw settlement caused by growth of a thaw bulb in the ground is still the most important design aspect of this scheme and requires specific, rigorous analysis. The berm transition is probably more vulnerable to ice action than the buried transition.

A variant on this scheme is support of the pipeline in a gravel causeway through the transition. The pipe would either be in the causeway for its entire length (for instance, from an offshore island to shore) or would enter the causeway from a buried mode at some distance offshore. Once in the causeway, the pipeline's performance is wedded to that of the causeway; design aspects of the causeway are discussed in Section 3.4. In addition, where permafrost is present at shallow depth below the sea

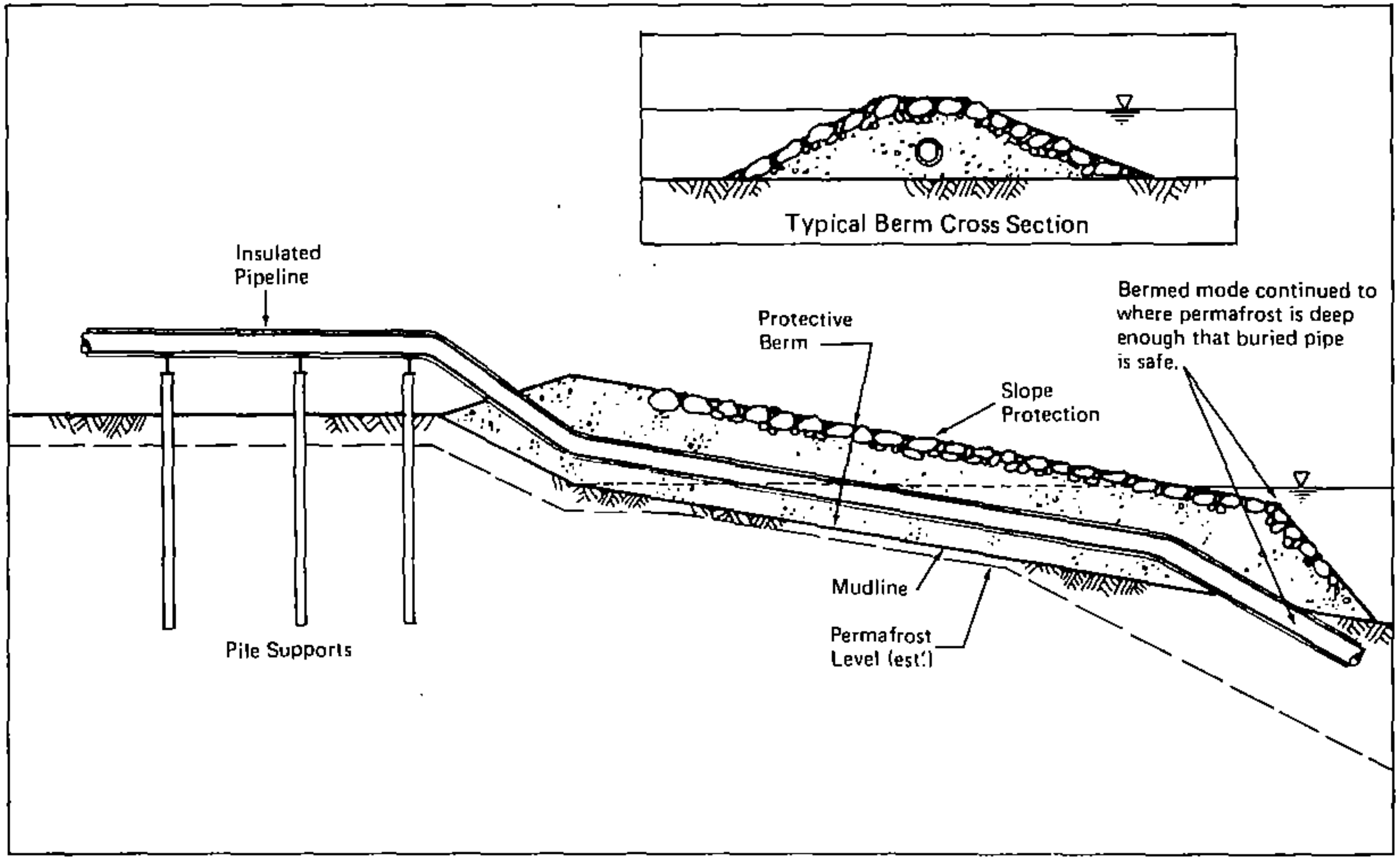


Figure 3.15 - BERMED SHORE TRANSITION (CONCEPTUAL)

floor, the heat from the pipeline may cause permafrost thaw and attendant additional settlement, so that the pipeline must be thermally protected to reduce thaw to acceptable amounts. Methods of thermal protection are basically the same as discussed earlier; however, board stock insulation may be considered to supplement the pipe insulation where the pipe is located above the water table, and conventional heat pipes appear practical in the causeway configuration. On land a conventional transition to the elevated mode can be made.

Support of the pipeline suspended from hangers below an elevated causeway, on cantilevers beside an elevated causeway (similar to Alyeska pipeline on Yukon River bridge), or on the deck of an elevated causeway, is also feasible through the shore transition zone, and would make the on-land transition to the elevated mode most simple. However, since an offshore transition from the buried mode to the elevated causeway mode does not appear practical, the entire offshore pipeline section would have to be constructed in this mode.

3.5.3 Transition to Islands

The transition from a man-made gravel island to an offshore buried pipeline mode is discussed in Section 3.1.

The transition from a natural barrier island to an offshore buried mode is similar to the shore transition, except possibly simpler, because permafrost appears to be less extensive on and near the island than it is near the shoreline. Because the islands migrate to the southwest, the lagoon side is generally free of shallow permafrost, but the north side may have relatively shallow permafrost. Hence, extensive thermal protection may be limited to the northern shore transition.

3.5.4 Conclusions

A pipeline transition to shore in either a buried or a berm mode appears feasible. Both modes will require heavy thermal protection through the zone of shallow (less than about 40 feet deep) permafrost, and careful evaluation and design for thermal effects are needed. While the buried mode is more difficult to construct and requires more thermal protection, it is less vulnerable to ice effects than the berm mode.

4.0
RECOMMENDATIONS FOR FUTURE STUDIES

The following paragraphs briefly outline our recommendations for future geotechnical studies in the Point Thomson Development Area.

4.1 SUBSURFACE CONDITIONS

Sufficient subsurface information has been obtained for conceptual design purposes. For site-specific route selection or structure location, site-specific borings will be needed and additional testing must be conducted. Data pertaining to permafrost depth, particularly in the lagoon area and including the taliks beneath the barrier islands, is lacking. New cone penetrometers have been developed which could provide quick, definitive information compared to conventional drilling or geophysics. The new system utilizes pumped mud and acoustic data transmission to allow penetration through frozen zones and simple operation. In addition to defining permafrost depths in the lagoon and offshore, nearshore permafrost profiles could be accurately defined. Also offshore gravel sources, such as the area near HLA Boring 11, could be quickly defined and more accurately quantified.

High-quality samples should be recovered from the sampled borings, and the temperature profile in the boring should be obtained. Laboratory tests should include tests to characterize the entire profile, plus shear strength tests to define resistance to sliding at shallow depth and thaw consolidation tests on bonded permafrost that may thaw during operation.

4.2 PIPELINES

Highest priority studies appear to be

- Evaluate minimum depth of cover needed to protect pipeline from ice gouging.

- Evaluate minimum depth of permafrost beyond which presence of permafrost does not significantly influence the buried warm pipeline.

- Identify areas where abnormally shallow bonded permafrost exists offshore (in view of thaw settlement of pipelines).

4.3 GRAVEL ISLANDS

Improve settlement estimates for gravel islands constructed of C-1 or drier gravel by laboratory model tests and by evaluation of the performance of existing gravel islands. Evaluate compatibility of various settlement mitigation measures with planned development of facilities on island.

Identify near-surface sediments on an area-wide basis to appraise stability problems. Conduct sufficient laboratory shear strength tests to define the shallow soils' resistance to sliding. Of particular importance are the expected strength properties of shallow weak silt, including the appropriate governing drainage conditions.

4.4 FOUNDATION DESIGN

Downdrag forces in a developing thaw bulb and their effect on pile bearing resistance are poorly understood. The effects of production from a cluster of wells on thaw settlement of the island surface need to be better explored.

4.5 CAUSEWAYS

Explore in more detail the rate of build-up of a berm causeway's resistance to ice forces with time, and make a risk analysis considering the chances of the ice force exceeding the resistance. If an elevated causeway is in serious contention, consider in more detail the design and constraints of support piles.

4.6 TRANSITIONS

Evaluate thermal aspects, thaw settlement and thermal protection of buried and berm transitions. Do coastal studies in area of highest interest to locate optimal transition location and its permafrost configuration.

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