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Volume I

Point Thomson Development Project Winter 1982 Geotechnical Investigation

prepared for

EXXON COMPANY, U.S.A.
Production Department Western Divison

№ 012

VOLUME I

POINT THOMSON DEVELOPMENT PROJECT WINTER 1982 GEOTECHNICAL INVESTIGATION EXXON COMPANY, U.S.A

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A Report Prepared for

EXXON COMPANY, U.S.A.
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I SUMMARY

The Point Thomson Development area, located about 50 miles east of Prudhoe Bay, lies between Bullen Point and Brownlow Point, a distance of about 23 miles. The area extends approximately three miles onshore and five miles offshore, and encompasses several barrier islands. Development of the area will require numerous artificial islands for production and injection wells, processing facilities, and water intake for water-flooding. Pipelines will connect the drill pads, islands and onshore facilities. The sales oil and gas will be transported by pipeline to the Alyeska system and, when built, the Alaska Natural Gas Transportation System at the Prudhoe Bay unit. At this time specific locations, numbers or types of production facilities and other improvements have not been defined.

Harding Lawson Associates (HLA) drilled and sampled 23 test borings to depths ranging between 25 and 80 feet within the proposed development area to investigate the soil and permafrost conditions during the winter of 1982. Of the 23 borings, 5 test borings were drilled onshore and 18 test borings were drilled offshore, 14 of which were located over ice and 4 on barrier islands. Ground temperature instrumentation was installed in 3 of the offshore borings, 2 of the onshore borings and 2 of the barrier islands borings. The drilling data, along with the results of an extensive laboratory testing program, were used to analyze preliminary geotechnical aspects of: (a) fill materials, (b) island design and foundation support, (c) offshore pipeline burial, (d) causeway design, (e) onshore pipeline support and (f) roadway and camp design.

Soil samples from the borings were examined to identify their field classification and selected samples were tested to measure their pertinent engineering characteristics. The following classification tests were performed on cores obtained from each boring: moisture content, dry density, particle size distribution sieve analysis and mechanical analysis using a hydrometer, Atterberg limits, specific gravity, organic content, and pore water salinity and chemistry.

Additional tests included: triaxial shear, unconsolidated-undrained to measure undrained shear strength, triaxial shear, consolidated-undrained with pore pressure measurements and consolidated-drained to measure total and effective stress parameters, consolidation tests with time rates on fine-grained soils to evaluate the degree of consolidation and to compute the magnitude and rate of settlement under static loads, direct shear consolidated-drained tests on cohesionless materials to measure angles of internal friction, pore water conductivity tests to determine the freezing point depression of pore fluids, thermal conductivity tests of both frozen and thawed samples, using the thermal needle method, and thaw consolidation tests on frozen samples.

Climatic and subsurface conditions found in the Point Thomson area are similar to that observed in the Duck Island and Prudhoe Bay areas. Consequently, data acquired over the years from these areas can be directly applied to this study.

The onshore area is flat, treeless, and windswept. The Arctic cold has frozen the ground hundreds of feet deep; it remains frozen except during the hundred days of summer when one to three feet of soil thaws beneath the tundra mat. The permafrost terrain is exemplified by polygonal ground and shallow thaw lakes which drain to the north into the Beaufort Sea.

The boundary between the ocean and land has swept north and south across the project area during the glacial epochs of the last 100,000 years. During low sea level stands the sediments froze and permafrost was formed. The maximum depth of permafrost is estimated to be between a depth of 1,000 and 1,800 feet. Since the last sea level rise, about 4,000 years ago, saltwater has slowly penetrated and degraded the offshore permafrost forming an irregular frozen surface. In the offshore lagoon area between the shoreline and barrier islands, the water is typically 10 to 15 feet deep, while water depths outside the barrier islands are in excess of 30 feet and are ice covered most of the year.

The onshore soil conditions generally consist of a thin layer of fine-grained Holocene silt and organic silt overlying Pleistocene alluvial silts, sands and gravel. The moisture content of the sand and gravel varies from 5 to 15 percent; this corresponds to an in-place dry density of 115 to 130 pcf. The soil was frozen throughout the depths explored, 50 feet, in all the onshore borings.

The offshore sediments consist of a complex sequence of Holocene and Pleistocene sands, silts and clays overlying Pleistocene fluvial and glacio-fluvial sands and gravels. The subsurface soil was unfrozen to the depths explored, 80 feet, in the lagoon area and frozen at depths varying from 0 feet to 39 feet at and outside the barrier islands. In addition, three of the barrier island borings encountered a zone of thawed soil sandwiched between the surface and deep frozen sediments. The Holocene and Pleistocene sands, silts and clays were found to have an effective angle of internal friction of 40 degrees. The Holocene silts and sands had moisture contents which varied between 25 and 35 percent and a dry density between 70 and 90 pcf. The Pleistocene silts and clays had a moisture content which varied between 15 and 25 percent and a dry density between 90 and 110 pcf. The overconsolidation ratio for all soils tested ranged between 1 and 3.

The test borings revealed subsurface conditions typical of the North Slope area currently being developed. Therefore, sound engineering design concepts currently be practiced are applicable to the Point Thomson area.

Oil field development facilities typically are constructed using gravel fill for roads and drill pads. Consequently, development of the Point Thomson area will require extensive fill material depending on final design configurations and the number of structures. The onshore Point Thomson development area has sufficient quantities of sands and gravels for the development of the entire project. In addition, gravel is present in the offshore area. Fine grained silty sand could be dredged from along the shoreline and placed hydraulically. For strength and compressibility, unfrozen onshore and offshore gravels are more desirable than frozen gravel or silty sand.

Subsea sediments will compress under the weight of island fill materials resulting in one to two feet of settlement. Most of the settlement will occur within three months of island construction and should have little effect on the structures installed after that time. The strength of the island fill will depend on whether gravel or silty sand material is used, whether construction is performed during the winter or summer season, whether the fill is frozen and on the brine content, ice content and fill temperature. In general, the silty sands have lower strength and higher compressibility than the sandy gravels. The estimated strengths used to analyze possible types of frozen fill were adjusted for the high ice content gravel observed in the Point Thomson C-1 material site using creep deformation theory.

Structures located on offshore islands, causeways, and barrier islands can be supported on either pile foundations or shallow spread footings. Pile foundations will develop vertical support by friction in the unfrozen soils below the sea floor. If piles penetrate frozen subsea soils, reduced friction values should be used in design. Shallow footings bottomed on gravel fill can be designed using relatively high bearing pressures. Sheet pile walls with pile-supported decks can be used as docks for unloading heavy modules.

In the zone of thaw caused by heat induced from the well drilling apparatus, large island and drill pad settlement will occur. Settlement will result from both thaw of gravel-ice fill and thaw of subsea permafrost. Consequently, structures sensitive to settlement should be located outside the area influenced by drilling or should be designed for releveling.

Subsea pipelines should be designed to withstand the sea floor scour that occurs from various ice processes during break up and summer months. Pipelines outside the barrier islands should be designed to withstand ice gouging and wave-induced ice pounding. Sea ice processes were not within the scope of this work and should be evaluated to determine pipeline burial depths.

It is anticipated that onshore pipelines will extend from the beachline of a causeway into pipeline corridors at Prudhoe Bay. Ice-poor and ice-rich frozen soils are present within the onshore Point Thomson area. Onshore structures and pipelines should be supported above grade on piles designed for both frost heave forces and long-term creep settlement in ice-rich soil. Pile capacity depends upon the bond between the pile surface and frozen soil; consequently, the pile system is referred to as a slurry adfreeze pile. High pore water salinity and ice will reduce the pile adfreeze strength. The minimum embedment depth is dependent upon the anticipated frost heave forces but in all cases should not be less than 11 feet in ice-poor soil and 14 feet in ice-rich soil. Roadways and storage areas within the project site should be constructed of gravel fill at least five feet thick placed over the tundra.

II INTRODUCTION

LIST OF ILLUSTRATIONS

II INTRODUCTION

A. Project Setting

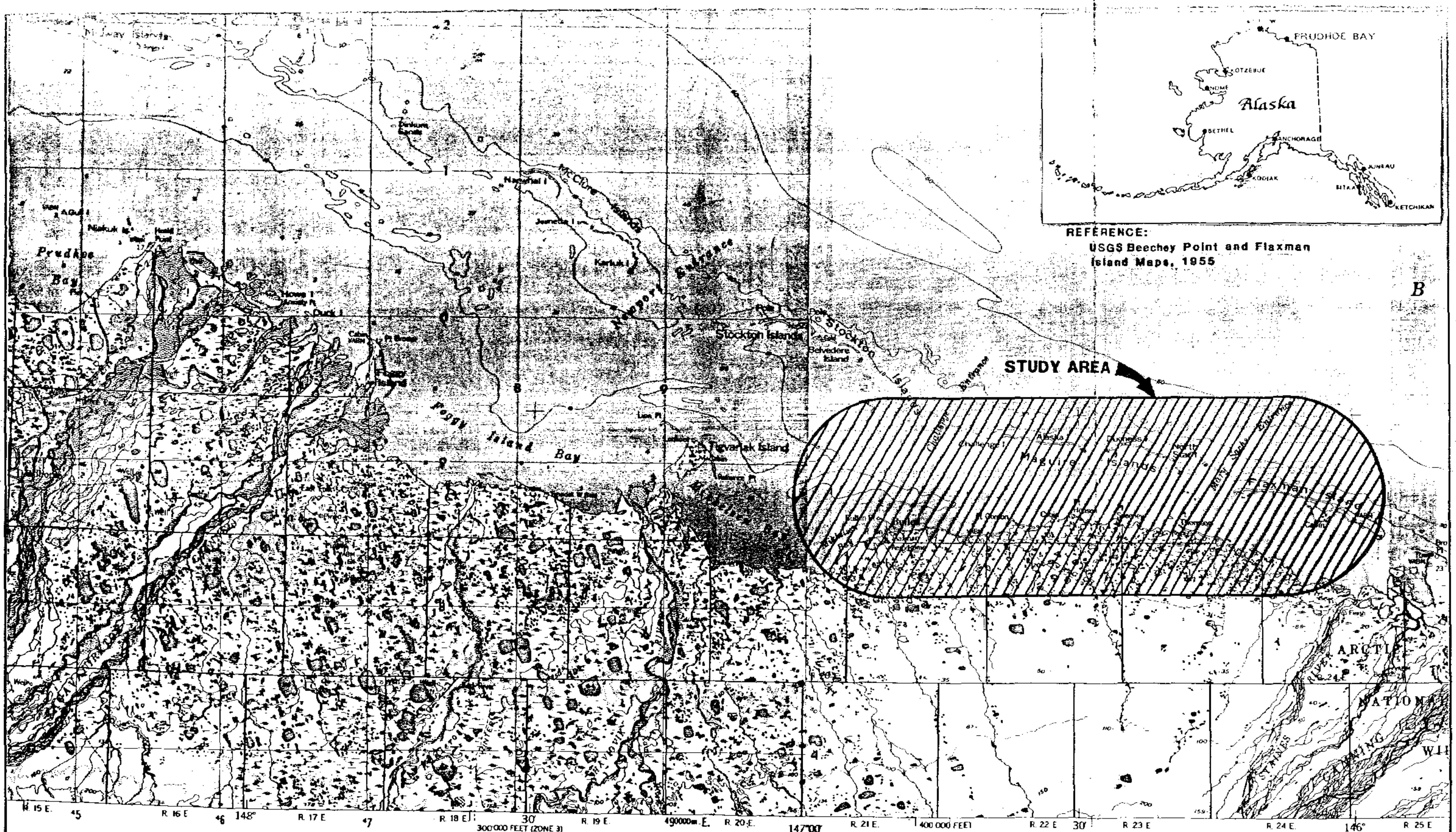
The Point Thomson Development area (PTD) is situated between Bullen Point and Brownlow Point northwest of the Staines and Canning River Delta, and about 50 miles east of Prudhoe Bay. This report presents general geotechnical engineering data needed for regional planning and preliminary design of oil production facilities.

The PTD area shown on Plate II-1 extends approximately 3 miles onshore and 5 miles offshore. Lagoon, shoal and deep water conditions exist within the offshore area, with water depths as great as 35 feet outside the barrier islands and 10 to 15 feet inside the barrier islands. The coastline within the area is a delta front with sandy beaches, mud flats and islands separated by small sandy peninsulas and channels of the Staines and Canning Rivers.

The climate is cold with monthly average air temperatures below freezing nine months of the year. The offshore area is covered with sea ice most of the year. Permafrost, the frozen soil condition associated with arctic cold, is present onshore and offshore and exerts a major influence on both the geologic processes and the engineering properties of soils in the area.

B. Proposed Development

At this time, the type, location and number of specific development facilities have not been defined. We anticipate that the development will include production well facilities requiring onshore gravel pads, offshore gravel islands and gravel pads on the barrier islands, both onshore and offshore pipelines, and support and transportation facilities such as roads, bridges, camps, pump stations, causeways and docks. A substantial requirement for



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Regional Study Area

Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **II-1**

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gravel fill material to construct both the onshore and offshore facilities is anticipated. Typical production islands may have clusters of 70 to 80 wells that will produce warm crude oil. Wells for gas reinjection and waterflooding will also be needed in the producing areas. A separate island in deeper water or a pad located on one of the barrier islands may be required to produce and treat water for waterflooding.

Fluids from the production wells will be piped to production facilities either onshore or on a barrier island where they will be treated to separate oil, gas, and water. The production facilities may consist of either modular structures, such as those used in Prudhoe Bay, or an integral barge structure using island fill to protect the sunken barge from sea ice. To accommodate both drilling and production facilities, island and pad surfaces could be as large as 30 acres.

The production islands will be interconnected by pipeline systems which will convey crude oil, natural gas, sales oil, and waterflood water. Communication and power supply systems and fuel lines will also interconnect the islands.

Transporting the sales product from the PTD to the Alyeska system and the proposed gas conditioning facility at Prudhoe Bay will probably be by subsea pipelines and causeways from offshore islands to an onshore collecting station. The causeway will protect the pipeline from sea ice and also allow year-round vehicle access from the onshore camps to offshore structures. The pipeline would extend from the PTD coastline to the vicinity of the Duck Island Development (DID) area where it could follow the proposed DID pipeline corridors. An onshore operations camp will be needed for the construction of the pipeline, road, causeway, and island system and for operation of the unit.

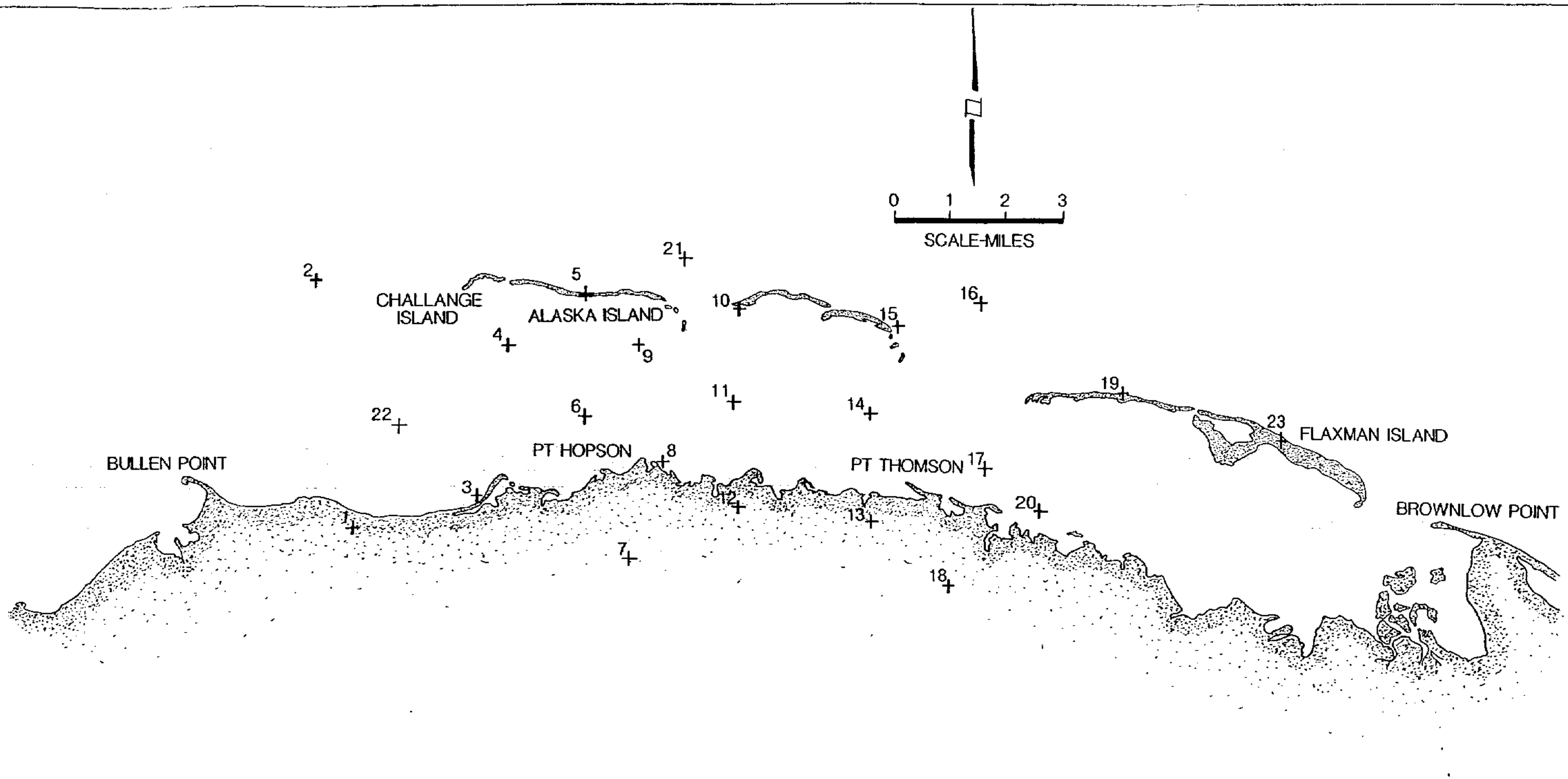
C. Purpose and Scope

This report presents the findings of a geotechnical investigation performed by Harding Lawson Associates for the Point Thomson Development area. The work was performed under Exxon Contract Agreement Number PTD-8201. The purpose of our investigation was to provide baseline geotechnical data, conclusions, and recommendations for regional planning and preliminary design of the project.

Our work included onshore and offshore test drilling and soil sampling, laboratory testing of the samples recovered during the investigation, geological interpretation of the field data, and engineering analyses to provide conclusions and recommendations for preliminary design. The results of this investigation have been correlated with previous studies.

Twenty-three borings were drilled to depths ranging from 25 to 80 feet below the ground surface/mudline in the PTD area at the locations shown on the Site Plan, Plate II-2. Of the twenty-three borings, fourteen were drilled overwater, four were drilled on the barrier islands and five were drilled onshore. Engineering properties of both frozen and unfrozen samples were determined by laboratory testing. Stabilized temperature profiles were obtained in seven of the borings by installing and reading thermistors.

This report is presented in two volumes. Volume I presents the findings of our field and laboratory investigations and our conclusions and recommendations for preliminary design. Volume II contains the data base for the conclusions presented in Volume I. A summary of previous studies performed in the area is presented in Appendix A. Detailed descriptions of the drilling program are presented in Appendix B along with individual boring logs. Ground temperature measurements and the laboratory testing program are described and the test data are presented in Appendixes C and D respectively.



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Site Plan
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE II-2

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Apendix E presents an explanation of analytical procedures used to define mechanical properties of fill materials, pile settlement and laterally loaded piles in permafrost.

D. Acknowledgements

Project Director was Jay England, Technical Project Manager and Field Project Manager was Donald Bruggers. Authors of the geological section of the report were Craig Rodeick, Lawrence Toimil and Steven Johnson. Field geologists for the drilling were Richard Prescott, Mark Musial and Peter Ondra. Director of the laboratory testing program was Dr. Kai Wong. The engineering analyses and chapters on engineering considerations are credited to Donald Bruggers, Dr. Kai Wong, and Dr. Richard Christensen. Author of Volume II and project engineer was Mark Musial. Technical editor was Martha Jokela.

Day-to-day coordination and support were provided by Leslie Paxton-Rousseau of Exxon Company, U.S.A. Additional support was provided by Dr. Anton Prodanovic of Exxon Production Research Company.

III REGIONAL SETTING

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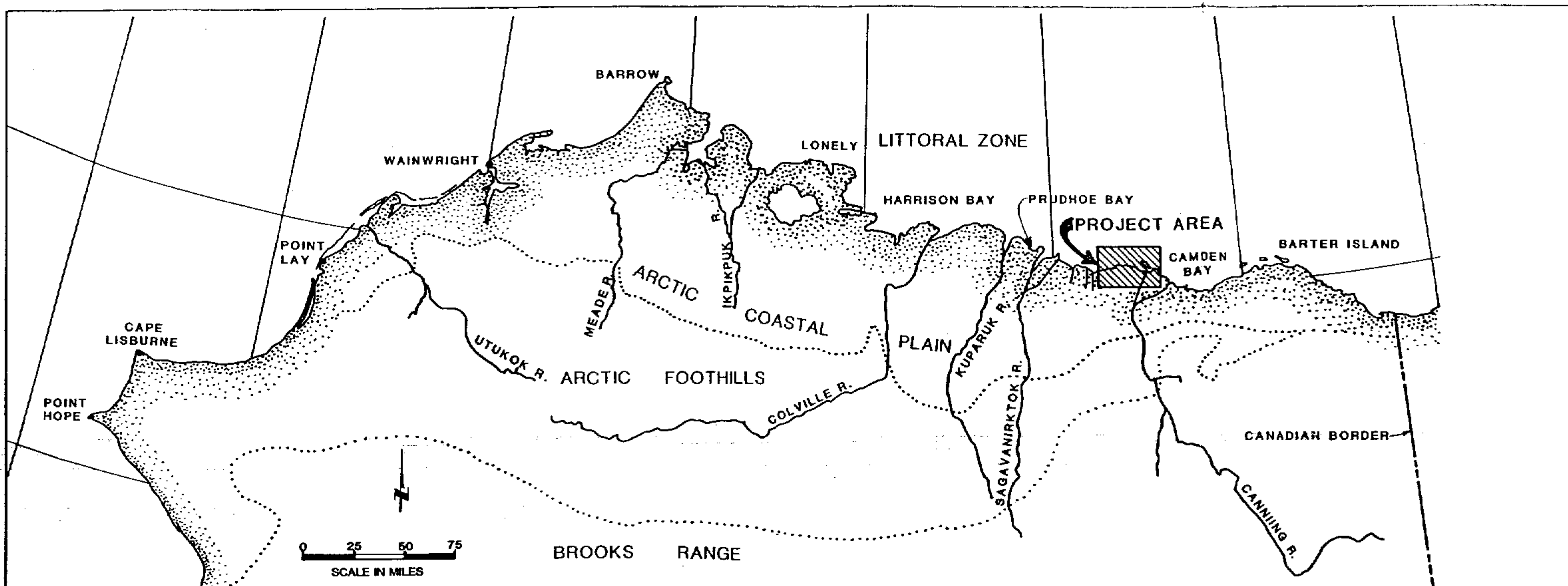
III REGIONAL SETTING

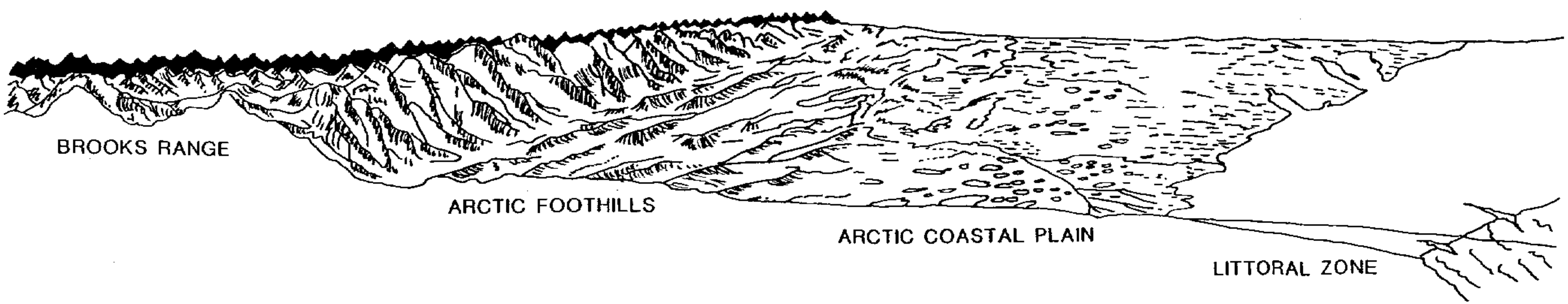
A. Physiography

1. The Arctic Region

The Arctic region of Alaska encompasses the drainage basins of all rivers flowing north from the divide of the Brooks Range into the Chukchi and Beaufort Seas. As shown on Plate III-1, this area stretches more than 600 miles from the Canadian border west to Cape Lisburne and nearly 250 miles north to south from Point Barrow to the crest of the Brooks Range near Howard Pass. The total area of the region is approximately 81,000 square miles, about the size of the state of Idaho. Although it contains approximately 14 percent of Alaska's land, the region has only about 2 percent of its population. Permanent and parttime residents number approximately 8000 (U.S. Army Corps of Engineers, 1980), and the population density is about 0.10 person per square mile.

Wahrhaftig (1965), Hartwell (1973), and others have divided the Arctic region of Alaska into four provinces: the Brooks Range, the Arctic Foothills, the Coastal Plain, and the Littoral Zone. The high rugged peaks of the Brooks Range end abruptly at the Arctic Foothills, which are long arcuate ridges descending to the rolling tundra of the Coastal Plain. The Coastal Plain province which extends from sea level to an elevation of 400 feet, is about 30 miles wide near the project area, but it widens significantly to the west and is over 90 miles wide south of Point Barrow. Offshore, the Continental Shelf continues under the Beaufort and Chukchi Seas to form the Littoral Zone. The shelf is only about 35 to 50 miles wide from Point Barrow eastward but extends several hundred miles offshore to the west. It is





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Physiographic Provinces of the Arctic Region of Alaska

Pt. Thomson Development Project, Winter 1982,
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PLATE **III-1**

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terminated by the relatively steep continental margin which drops off into the Arctic Ocean Basin. Only the Coastal Plain and Littoral Zone are of interest for Point Thomson development and these two are discussed in more detail below.

a. Coastal Plain Province

The Coastal Plain Province in the project area is characterized by subtle topography, thaw lakes, meandering north flowing stream channels, and ice-bonded permafrost soils. The soils are the product of fluvial, glaciofluvial, and marine processes active since Tertiary time (1 to 70 million years ago). Materials eroded from land areas have been deposited near the coast and offshore. These materials include mineral soils ranging in size from clay to boulders and, in some areas, large amounts of organic matter.

Stream, current, wave, and ice processes have winnowed, transported, and deposited a wide range of materials into the sea, forming a coast consisting of prograding delta fans, low truncated bluffs, shallow lagoons, and barrier islands. The shaping of the present coast has taken place over the last 3000 years during a period when the sea level has remained fairly constant.

In the vicinity of the project area, the Coastal Plain Province can be subdivided into the Upland Fan and Coastal Zone regions; these regions differ in their topography, hydrology, and morphology.

(1) Upland Fan

The Upland Fan region is adjacent to northward sloping abandoned deltas of the Staines and Canning Rivers. The topography of the Upland Fan region is very subdued. Elevations range from about 25 feet five miles inland of Point Thomson to zero feet on the beach at Point Thomson. The abandoned delta slopes northward at a gradient of about 1 foot vertical to

1300 feet horizontal. Upland relief is generally minor, with local changes in topography of approximately six feet. Abrupt changes in topography, on the order of 15 feet, are associated with banks along small, north flowing streams.

One of the dominant features of the Upland Fan region is the continuous layer of perenially frozen ground known as permafrost. Permafrost is generally defined as ground having a temperature below 0^0 Celsius (C) continuously for two or more years. Permafrost extends almost to the ground surface except for unfrozen pockets, which are typically located beneath deep lakes and abandoned or active major river channels. Permafrost is found in materials ranging from dense gravel to silts and clays.

Because of the impermeable nature of the shallow permafrost and its low relief, drainage of the Upland Fan is poorly developed. Small, northern flowing streams, most less than four miles long, form the drainage system of the Upland Fan. Thaw lakes along the streams result in a "beaded" drainage pattern. The beaded streams form as a result of the melting of massive ground ice in a process called thermokarst. Water derived from rain, snow melt, and summer thawing of the subsoil accumulates above the permafrost table and this moisture combined with the thaw lakes results in the swampy character of low lying areas during the summer.

The landscape between the thaw lakes is characterized by a variety of patterned ground forms. The most common of these are the high and low centered polygons and frost boils which constitute the principal microrelief.

The major rivers in the study area are the Staines and Canning Rivers. These rivers are of the braided type, i.e., they are characterized by continuously shifting active channels separated by gravel and sand bars. The headwaters of the Staines and the Canning are in the Brooks Range, and they terminate in a delta approximately four miles wide directly east of the project area. The rivers freeze over during the winter but deeper pools, particularly in the Canning, remain unfrozen. The river ice is normally five to six feet thick by the end of winter but overflows can cause the development of aufeis which can considerably thicken the ice cover. Beneath the ice, the river may maintain a discontinuous winter flow. Although freeze-over generally occurs during October, it sometimes takes place as late as mid-November. River breakup generally occurs in early June, and the associated floods are estimated to contain 60 to 80 percent of both rivers' annual flow.

(2) Coastal Zone

The Coastal Zone includes the ancient and modern delta front of the Staines and Canning Rivers and its beaches. The beachline is characterized by numerous small triangular points and high angle bars which have been named Points Gordon, Hopson, Sweeney and Thomson from west to east, respectively.

The shoreline from the Staines River delta west to Bullen Point is characterized by an abrupt break in the slope between the relatively flat Upland Fan and the gently sloping sea floor (Hartwell, 1973). The beach in this area is formed by slumping of the bluffs which are generally about 10 feet high. Slumping results from the thermal and mechanical erosion induced by wave activity along the base of the beach bluffs: blocks of frozen overhanging tundra mats collapse when ice wedges within the tundra melt or are undercut by waves, scattering large clumps of organic material along the beach front.

b. Littoral Zone

(1) Offshore Topography

The Littoral Zone is that portion of the study area offshore of the mainland beach and includes the barrier islands. Seabed slopes between the mainland beach and the barrier islands are relatively flat. An abrupt steepening of the seabed occurs seaward of the barrier islands where seabed gradients are 0.2 percent or a slope ratio of 1 vertical to 500 horizontal (1:500) (Harding Lawson, 1979).

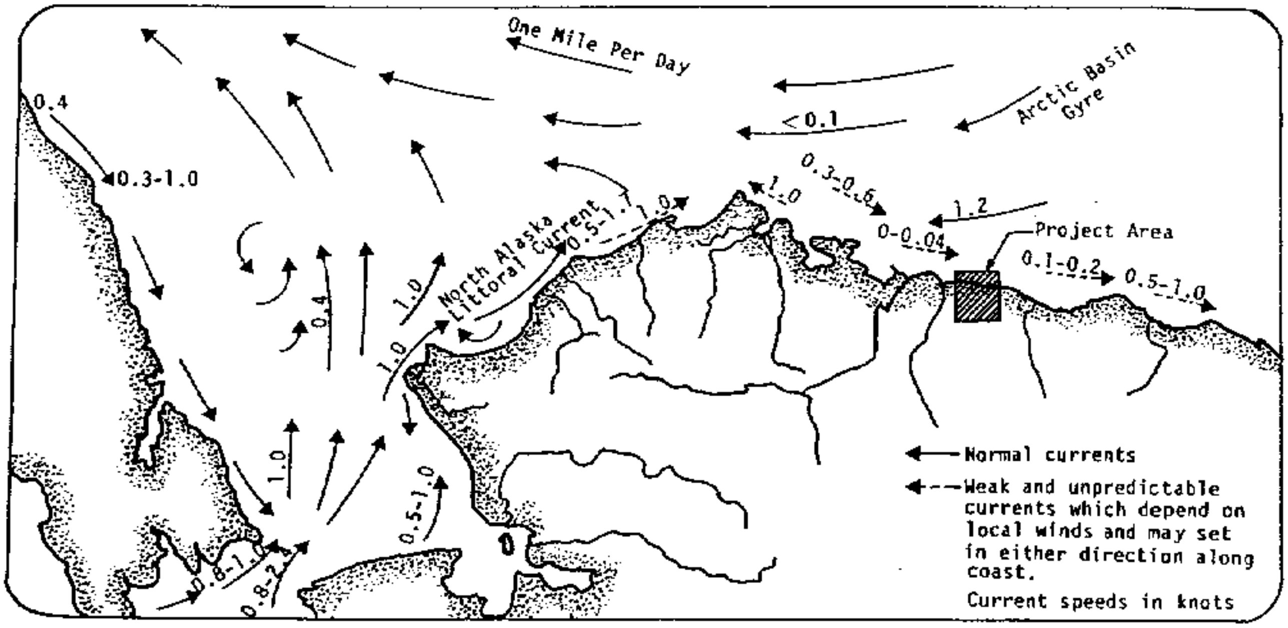
The barrier islands are the dominant topographic features of the Littoral Zone. Two kinds of barrier islands are found in the PTD study area: those that represent the erosional remains of the relict coastline and those recently constructed by physical processes. Flaxman Island is an example of an erosional remnant island. Flaxman Island stands as high as 25 feet above sea level and, along its western end, supports a continuous cover of tundra vegetation and small thaw lakes. The beaches along the western portion of the island are formed by the sloughing of bluffs up to 15 to 20 feet high. Sloughing results from the same mechanical and thermal processes that are eroding the mainland beaches.

The Maguire Islands are examples of more recent constructional features. Nowhere higher than ten feet, these islands are much longer than they are wide, with a maximum length of about three miles and widths ranging from 40 to 150 yards. They are covered with sand and gravel and are separated by frequent inlets. Plant growth is sparse to nonexistent and because vegetation is usually required for dune stabilization, there are no stable dunes. Sand and gravel beach ridges in excess of two feet high have been observed on the western end of Flaxman Island. These ephemeral ridges are the result of "plowing" of the islands by ice during the winter months (Leffingwell, 1919). The constructional islands are all migrating toward the southwest (Wiseman, et. al, 1973).

(2) Tides, Currents, and Waves

Summer water circulation patterns on the Beaufort Shelf offshore of the PTD study area are complex and not completely understood. The surface circulation on the outer Beaufort Shelf is dominated by a clockwise, westward flow associated with a gyre in the Arctic Basin midway between Alaska and the North Pole, as shown on Plate III-2 (Selkregg et al., 1975). Mountain (1974) reports an eastward flow of water along the inner shelf which appears to be associated with local westerly winds produced by storms; these storms occur mainly in late summer and fall and cause higher sea levels. Current reversals have been observed a few hours after a change in wind direction from easterly to westerly or vice versa (Namtvedt et al., 1974). The data indicate that the nearshore currents and sea levels along the Beaufort Sea coast from the Sagavanirktok Delta to the Canning Delta are strongly influenced by local winds.

The mean lunar tidal range for the Beaufort Sea coast in the vicinity of the PTD study area is 6 to 12 inches (Kinney et al., 1972; Reimnitz and Barnes, 1974), and the associated tidal currents are weak. Because lunar tides are relatively insignificant, nontidal factors have an important effect on sea levels, particularly wind. The available data indicate a strong correlation between high tides and strong westerly winds. Storm surges can cause major changes in sea level. In 1970, for example, a major storm surge caused by gale-force westerly winds inundated low-lying



Currents of the Beaufort and Western Chuckchi Seas (After Selkregg, et al., 1975)

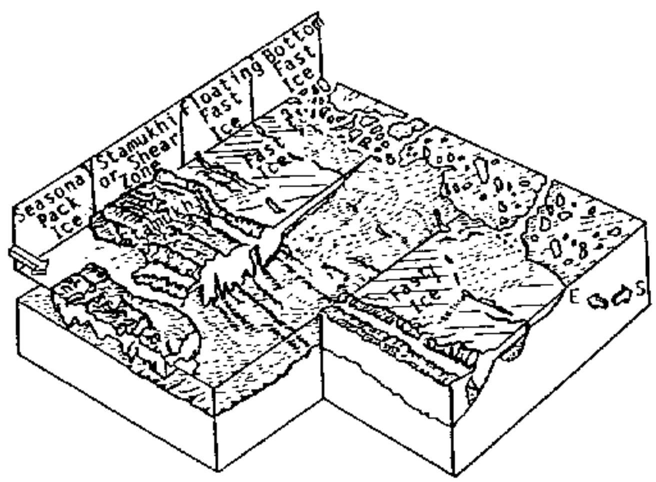


Diagram of an idealized cross section of ice zonation along the Alaskan Beaufort Sea in spring

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OCEAN CURRENTS AND ICE ZONATION
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PLATE **III-2**

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tundra plains and deltas as far as 15,000 feet inland and left a driftwood line as much as 7 feet above normal sea level in the Bullen Point - Point Gordon area (Reimnitz and Maurer, 1979).

Although major sea level surges are generally associated with the open water season, they can also occur in winter. Winter surges recorded in 1973 at Oliktok Point are reported to have had heights of 3.1, 4.6, and 2.2 feet (Reimnitz and Maurer, 1979). Henry (1975) recorded two surges about 3 feet high in the Canadian Sector of the Beaufort Sea during the winter of 1973-74.

Surface waves are generated only during open-water months in summer. The fetch, or distance across open water, available for wave generation is usually small because of the polar pack ice offshore. Under certain wind conditions the polar pack ice may retreat far to the north, increasing the fetch and permitting swells up to six feet high (Reimnitz and Maurer, 1979). During such periods the constructional barrier islands can be significantly modified. On the coast, where there are no beaches, or where they are poorly developed, storm waves can greatly accelerate erosion of the coastal bluffs.

(3) Sea Ice

For about nine months of the year, the Beaufort Sea is almost completely covered by ice. As shown on Plate III-2, Reimnitz, Toimil, and Barnes (1977) have divided the Beaufort Sea seasonal ice sheet into four distinct zones: bottom fast ice, floating fast ice, stamukhi, and seasonal pack ice.

In the bottom fast ice zone, ice freezes to the sea floor. By winter's end, it is normally about 5 to 6 feet thick and extends as far as 8 miles offshore.

The floating fast ice zone stretches seaward from the boundary of the bottom fast ice to the stamukhi zone. Water in the floating fast ice zone is typically between 3 to 60 feet deep. The ice in this zone is underlain by seawater and is stabilized by the barrier islands and the grounded ice within the stamukhi zone; its canopy, therefore, experiences little deformation throughout the winter.

The stamukhi, or shear zone, consists of a belt of grounded ice ridges and hummocks seaward of the fast ice zone. Commonly occurring near the 60-foot isobath, the stamukhi zone represents a zone of slippage between the floating fast ice and the seasonal and polar pack ice.

The seasonal pack ice zone continues outward 70 to 100 miles seaward of the stamukhi zone to the toe of the Continental Shelf. The ice in this zone is mobile, unstable, and highly deformed. During winter, the seasonal pack ice and polar ice sheet act as a cohesive mass which rotates westward at rates from 1/2 to 2 miles per day.

2. Climate

No long-term climate records are available for the Point Thomson region. However, the study area lies equidistant from Prudhoe Bay on the east and Barter Island on the west. Comparison of climatic records at these two locations should indicate the range in climatic conditions experienced in the Point Thomson area.

Temperature data collected at the ARCO airfield and Barter Island are presented in Table III-1. The data indicate Prudhoe Bay and Barter Island have average annual temperatures of about -13^{o} and $-12^{o}C$ respectively. Summer temperatures at Prudhoe Bay and Barter Island remain relatively cool because of the combined effects of the Arctic Ocean to the north and its

TABLE III-1. MEAN MONTHLY AIR TEMPERATURES FOR PRUDHOE BAY REGION (ARCO AIRFIELD) AND BARTER ISLAND (°C)

Station	Year	Temperature (°C) J	F	M	A	M	J	J	A	S	O	N	D	Yr	Thaw degree days	Distance to coast (km)
Prudhoe Bay at ARCO Airfield	1976	-30.8	-31.9	-29.0	-16.5	-5.9	3.2	6.8	6.6	1.6	-11.4	-16.6	-30.3	-12.7	571	6.0
	1977	-23.1	-28.0	-31.8	-19.2	-5.5	3.7	5.4	8.2	2.5	-4.7	-21.4	-23.4	-11.4	643	
Barter Island	1976	-29.2	-30.7	-27.9	-19.3	-6.7	1.1	3.5	4.3	0.9	-11.0	-15.0	-30.2	-31.1	326	0.1
	1977	-23.4	-27.3	-32.3	-20.1	-5.9	1.4	3.1	5.2	2.1	-4.9	-19.8	-21.7	-12.0	375	
	29-yr means (49-77)	-26.2	-28.6	-25.9	-17.7	-5.4	1.2	1.8	3.8	0.2	-8.7	-17.7	-24.7	-12.2	--	

associated cold air mass and the Brooks Range to the south which blocks the flow of warm air from the interior. Summer temperatures inland are higher because of the reduced cooling effect of the Beaufort Sea. Based on limited temperature data, thawing indexes of about 375^{o}C-days occurring over a time period of 4 months and 550^{o}C-days are estimated for coastal areas and locations 10 miles inland, respectively, in the Point Thomson region.

Extremely low winter temperatures at Prudhoe Bay and Barter Island result from the low level of incoming solar energy. Since sea ice covers the coastal waters after mid-October, winter temperatures along the Beaufort Sea coast are not modified by the ocean. A freezing index of 5200^{o}C-days occurring over a time period of 8 months is estimated for the Point Thomson area (Hartman and Johnson, 1978).

The wind in the Point Thomson area is generally from the east and northeast during the late spring, summer, and early fall. Strong westerly and southwesterly winds with occasional easterly and northeasterly winds are more common during the late fall, winter, and early spring. Based on data from Prudhoe Bay and Barter Island, the mean annual wind velocity is approximately 13 miles per hour, with the highest monthly mean velocities during spring, early summer, and fall. The lowest mean velocities occur in late winter and are associated with extremely low temperatures. The strong westerly winds in winter often occur with snow storms, resulting in ground blizzards. Easterly winds often redistribute the snow.

Precipitation records for the Point Thomson area are not available and precipitation records for Prudhoe Bay prior to 1977 are unreliable because of the difficulty associated with measuring small amounts of precipitation under very windy conditions. A gauge designed for high wind conditions was installed in October 1976; it has recorded precipitation totals considerably higher than those previously recorded at other stations on the Arctic Coastal Plain, as shown on Table III-2 (Walker et al., 1980). The available data indicate that about 65 percent of the total annual precipitation along the Beaufort Sea coast between Prudhoe Bay and Barter Island falls as snow. Most of the snowfall occurs during September and October when there is still open water in the Beaufort Sea to provide a source of moisture.

TABLE III-2. PRECIPITATION (mm) FOR 1977 AND 1978 AT PRUDHOE BAY AND BARTER ISLAND

Year	Rain	Prudhoe Bay(1) Snow(2)	Total	Barter Island(3) Total
1977	81	142	223	85
1978	58	125	183	91

(1) Modified Wyoming snow gauge
(2) Water equivalent
(3) Standard National Weather Service instruments

Drizzle, fog, and light rain are the most common forms of precipitation during the summer months. Fog is common in the mornings along the coast and it often lingers in coastal areas after it has dissipated at Deadhorse and other inland areas.

B. Geology

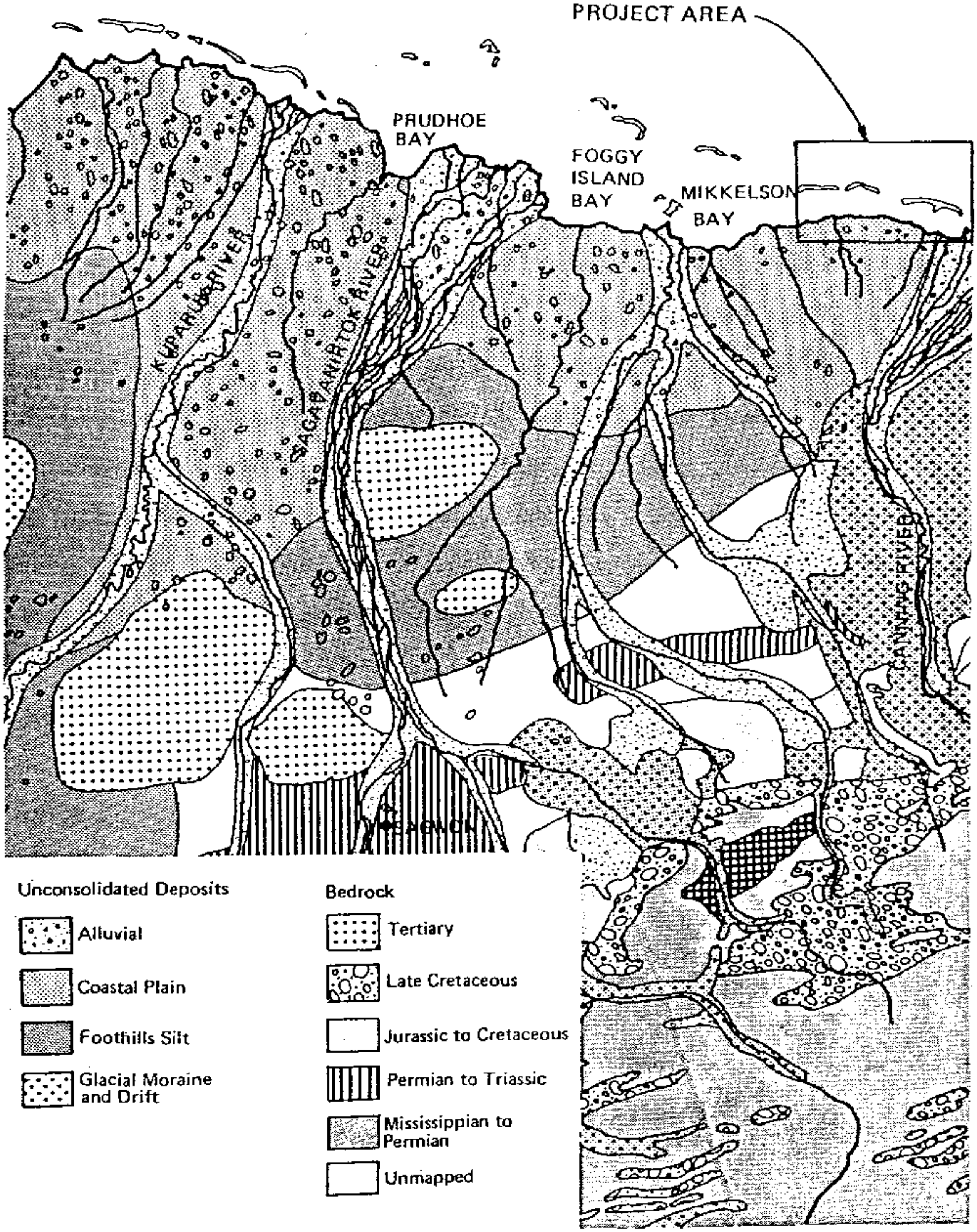
1. History

a. Pre-Quaternary Geology

The surficial geology of the Arctic region from the Brooks Range northward to the Beaufort Sea coast is shown on Plate III-3. The major structural feature underlying the Prudhoe Bay area is the Barrow Arch, which consists of uplifted lower Paleozoic (older than 400 million years) rocks. Above the Barrow Arch lie approximately 12,000 feet of post-Devonian [350 million years Before Present (BP)] sediments consisting of organic-rich shale, sandstone, conglomerate, and dolomitized limestone units of both marine and nonmarine origin. The thickness of the sediments increases southward toward the axis of the Colville Trough.

During late Jurassic (135 to 180 million years BP) and early Cretaceous times (70 to 135 million years BP), the northern part of the Arctic platform and the northern terrain source were pulled (rifted) away, leaving a continental margin facing northward. During the same time period, the Arctic platform was tilted downward to the south and overridden by the emerging Brooks Range. A foreland basin, the Colville Trough, formed north of the Brooks Range. The northern apex of the rifted and tilted Arctic platform formed the Barrow Arch that trends parallel to the Beaufort coast.

Nearly 8000 feet of marine and nonmarine sediments accumulated in the Colville Trough during Cretaceous and Tertiary times. The upper Tertiary bedrock consists of weakly cemented sand, gravel, clay, and silt known as the Sagavanirktok Formation, upon which Quaternary (present to 1 million years) sediments are unconformably deposited.



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Surficial Geology of a Portion of the Arctic Region

Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE **III-3**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08		4/82		

b. Quaternary Geology

(1) Sea Level Changes

Eustatic sea level changes caused by worldwide glacial epochs have exposed the present onshore and offshore regions of the study area to subaerial erosion and continental and marine deposition. At least six major changes of sea level influenced the stratigraphy of the area during the Pleistocene. A number of these changes appear to have been complex, including more than one episode of high sea level transgression separated by regressions of substantial duration (Hopkins, 1973). There is evidence to suggest that sea level reached its highest position, greater than 60 feet and perhaps as great as 300 feet higher than present, during the middle Pleistocene (0.7 to 1.9 million years B.P.). As recently as 25,000 years ago, the shoreline stood at least as low as the present 60-foot isobath and perhaps as low as the 280-foot isobath.

(2) Quaternary Geologic Units

The definitions and symbols for the geologic units used on the geologic maps and cross sections discussed in the following sections are presented in Table III-3.

TABLE III-3. QUATERNARY GEOLOGIC UNIT DESCRIPTIONS

Geologic Symbol		Name	Unit Description
Holocene	QHt	Thaw Lake Deposits	Organic ice-rich silts with lenses and wedges of massive ground ice. These materials are unstructured in some areas and contain coarse-grained soil because of reworking by frost processes.
	QHs	Shoal Deposits	Constructional units having positive relief and composed of clean, well-sorted sands and minor amounts of gravel.

TABLE III-3. QUATERNARY GEOLOGIC UNIT DESCRIPTIONS
(continued)

Geologic	Symbol	Name	Unit Description
Holocene	QH	Lagoon Deposits	Organic-rich silts, clays, and minor lenses of sand which have been deposited in protected lagoon and bay environments.
	QHd	Delta Deposits	Usually consist of interbedded sequences of clayey and sandy silts with occasional interbeds of silty sand. These materials are unstructured in some areas because of reworking by sea ice and strudel scour.
	QHnm	Nearshore Marine Deposits	Unstructured mixtures of silty sands and sandy silts which have been reworked by grounding sea ice and deposited in a nearshore environment. This unit may contain material from underlying units.
	Qf	Flaxman Lag	A lag deposit, which consists of gravel, cobbles, and boulders resulting from the erosion of the Pleistocene Flaxman formation.
Pleistocene	QPf	Flaxman Unit of Gubik Formation	A marine unit consisting of inter bedded sandy silt and clay with ice-rafted glacial boulders. The unit may contain boulders up to 9 feet in diameter but, most are less than 2-1/2 feet in diameter.
	QPnm	Nearshore Marine Deposits	Includes soft to medium-stiff silts and clayey silts. Similar to QHnm; locally contains ice-rafted glacial boulders up to 3 feet in diameter. Where boulders are present, referred to as Flaxman Formation.
	QPm	Marine Deposits	Interglacial marine clays and clayey silts with thin seams of fibrous organic material. Overconsolidated, becoming stiff to hard in some test borings.

TABLE III-3. QUATERNARY GEOLOGIC UNIT DESCRIPTIONS
(continued)

Geologic Symbol		Name	Unit Description
Pleistocene	QPb	Beach Deposits	The base of the Pleistocene marine section primarily composed of gravelly sand, but in some boreholes it consists of silty sand.
	QPo	Outwash Deposits	Fluvial and glaciofluvial interbedded gravelly and silty sands. Thins onshore, grading into QPb in some areas.
	QPa	Alluvium	Fluvial and glaciofluvial material consisting predominantly of interbedded sand and gravel, which may contain lenses of fine-grained material.

(3) <u>Onshore Geology</u>

A generalized map of the surficial geology of the PTD project area is presented on Plate III-4. During episodes of Pleistocene sea level regression, the Canning River transported massive quantities of fluvial and glaciofluvial materials to the coast to form the Upland Fan Shelf. These materials now form a thick mantle of Quaternary deposits that overlie the Tertiary Sagavanirktok formation. Most of these Quaternary deposits are unconsolidated sands and gravels (geologic symbol QPa) composed of reworked Tertiary materials and materials derived from the Brooks Range to the south. These deposits are similar to the terrace and floodplain deposits of the present-day Canning and Staines Rivers.

The Flaxman Unit (geologic symbol QPf) of the Gubik Formation outcrops as a thin band along the shoreline of the mainland and on Flaxman Island. The formation is named from Flaxman Island where it is well exposed. The unit contains sporadic gravel, cobbles and boulders, a diagnostic feature of the unit.

At Flaxman Island and along the coastal bluffs surrounding the study area, scattered gravel, cobbles, and boulders are incorporated in the QPf deposits. Sightings of boulders incorporated within QPf units of the Gubik Formation have been recorded by Leffingwell (1919), MacCarthy (1958), and Rodeick (1975). The boulders, commonly called Flaxman boulders, are characterized by rock types foreign to the geology of northern Alaska and include red granite, granulite-facies, metamorphic rocks, pyroxenite, diabase, pink quartzite, and large amounts of dolomite. It is generally believed that the Flaxman boulders represent glacial dropstones that were ice-rafted to their present positions during periods of elevated sea level, most likely during the Sangamon interglacial period (70,000 to 100,000 years BP) or perhaps when the coast of northern Alaska was isostatically depressed (Hopkins, 1979).

Overlying the QPa and QPf on the Upland Fan are from two to twenty feet of ice-rich silts (geologic symbol QHt) with variable amounts of organic matter. The silts are probably equivalent to the Barrow member of the Gubik Formation (Black, 1964). Because they would obscure the surficial geology, the upper few feet of Holocene sediments are not shown on the geologic map.

Thaw lakes have modified the landscape of the PTD study area during Holocene time (last 10,000 years) through the combined processes of thermokarst and thermo-erosion (Everett and Parkinson, 1977). The thaw lake cycle and intense frost activity have reworked the upper two to eight feet of ice-rich silts. Melting of the ground ice has lowered the land surface in some areas by as much as several tens of feet. Eolian and frost activity have been locally important in reworking the Holocene lake deposits.

These processes are discussed in more detail in the following sections.

(4) Offshore Geology

(a) Geologic History

The geologic history of the offshore area from Bullen Point to Brownlow Point and the Canning River Delta area is similar to the offshore area around the Duck Island Development area. Sedimentation has been primarily controlled by eustatic changes in sea level during the Pleistocene.

Numerous glacial periods in the foothills and Brooks Range have supplied vast quantities of material to the arctic coastal plain and offshore continental shelf. The oldest deposits encountered (described in detail below) are sand and gravel of fluvial and glaciofluvial origin (map symbol QPa). The ubiquitous nature of these deposits, often just below the surface near the coast, attests to the volume of water and material supplied by melting glaciers. The top of this unit dips to the north away from the coast. The deposit is more than 400 feet thick beneath Reindeer Island.

These coarse-grained deposits were subsequently inundated by rising sea level and overlain by a marine clay and silt unit possibly 175,000 years BP during the pre-Illinoisan interglacial period. The fine-grained deposits are normally less than 28 feet thick in the lagoon, except near the east end where deposits may be in excess of 60 feet thick. Seaward of the barrier islands, USGS/HLA Test Boring 18 was drilled 300 feet below mudline and did not encounter the bottom of this fine-grained unit. Most of these deposits are overconsolidated, a condition caused by subsequent subaerial erosion during the next glacial period. That period was one of major modification for the marine clays and in some places the clays were stripped away completely.

Approximately 70,000 to 100,000 years ago, the sea level rose about 12 meters higher than present. During that time, a thick deposit of silt accumulated over much of the PTD study area. Ice fragments, calved from glaciers in the Canadian Archipelago and carrying gravel to boulder sized material, grounded in Alaskan waters and deposited rocks dissimilar to those found in the Alaskan Brooks Range. Years of accumulation formed a sandy silt deposit containing coarse material foreign to Alaskan geology. Known as the Flaxman Unit of the Gubik Formation, it is up to 37 feet thick in the study area.

Subsequent to the deposition of the Flaxman Unit, sea level fell during the Wisconsin glacial epoch to the present 60-foot isobath and perhaps as low as the 280-foot isobath. During this period major rivers and small melt water streams carried material onto the continental shelf, cutting through the Flaxman Unit and the older marine clay. Further modification of existing deposits occurred during another transgression and regression in the middle and late Wisconsin and the final Holocene rise to modern day sea level, which was established approximately 5000 years BP.

(b) Pleistocene Units

Plate III-4 shows a surficial geologic map of the offshore portion of the study area. The top meter of sediment has been stripped off to show near-surface deposits that differ from the lagoon sediments. The relationship of all identified geologic units is shown on the Fence Diagram, Plate III-5.

The Flaxman Unit (QPf) of the Gubik formation consists predominantly of sandy silt with subordinate layers of sand and silty clay.

The silt varies from soft to hard, but is generally stiff to very stiff and overconsolidated. Occasional lenses of fibrous organic materials have been noted on the boring logs. The thickest section of QPf encountered in the borings was a 36-foot-thick layer in Boring 19 (Appendix B) which did not penetrate the entire section. QPf appears to have at one time completely mantled older Pleistocene clay (QPm) and alluvial deposits (QPa) throughout the study area as shown by the ubiquitous distribution of Flaxman lag deposits (Qf).

The lowermost marine unit (QPm) consists of clay, silty clay, and minor amounts of silt and sand. The deposits are normally stiff to hard and overconsolidated although normally consolidated clay and silt deposits are present locally. The QPm unit appears fairly continuous except near the coast. In some cases QPm consists of sand, especially near the coast.

The QPm unit rests disconformably on Pleistocene fluvial deposits and its upper surface is probably erosional. Seismic reflection profiles from the Duck Island Development study (1981) show the upper surface of this unit as an erosional unconformity implying that it has been subjected to at least one episode of subaerial exposure before deposition of the overlying QPf silts.

The Fence Diagram, Plate III-5, shows that QPm pinches out against Pleistocene alluvium (QPa) before reaching the southern coastline. There are outcrops of the overlying QPf along the coast, but no evidence of QPm.

The combined thickness of fine-grained sediments probably does not exceed a thickness of 55 to 60 feet inside the lagoon, with the possible exception of the very east end near the eastern tip of Flaxman Island. In the offshore area, fine-grained soils in excess of 300 feet thick were encountered in USGS/HLA Boring 18, in excess of 150 feet in USGS/HLA Boring 17, and in excess of 126 feet in USGS/HLA Boring 16 (Appendix A).

Pleistocene alluvium (QPa) underlies most of the PTD study area. It is at the surface onshore and becomes progressively deeper to the north. The Fence Diagram shows that QPa dips more steeply to the north near the east end of the lagoon. The material was placed as ancient rivers meandered back and forth across the coastal plain. The material is probably a combination of fluvial and ancient glacial outwash material. QPa consists of sandy gravel and gravelly sand with a chert and limestone lithology common to the Alaskan Brooks Range.

(c) Holocene Units

Holocene deposits within the PTD study area consist of materials deposited in lagoon, beach, and nearshore marine environments. Areally the most extensive Holocene sediments are lagoon deposits (map symbol QH). The QH deposits consist predominantly of soft to medium stiff silts, often organic-rich silts, minor clay and sand deposited in a protected lagoon. The thickest QH deposits were drilled at Test Borings 2 and 4 (Appendix B) where the QH section extends to a depth of 25 feet below the mudline.

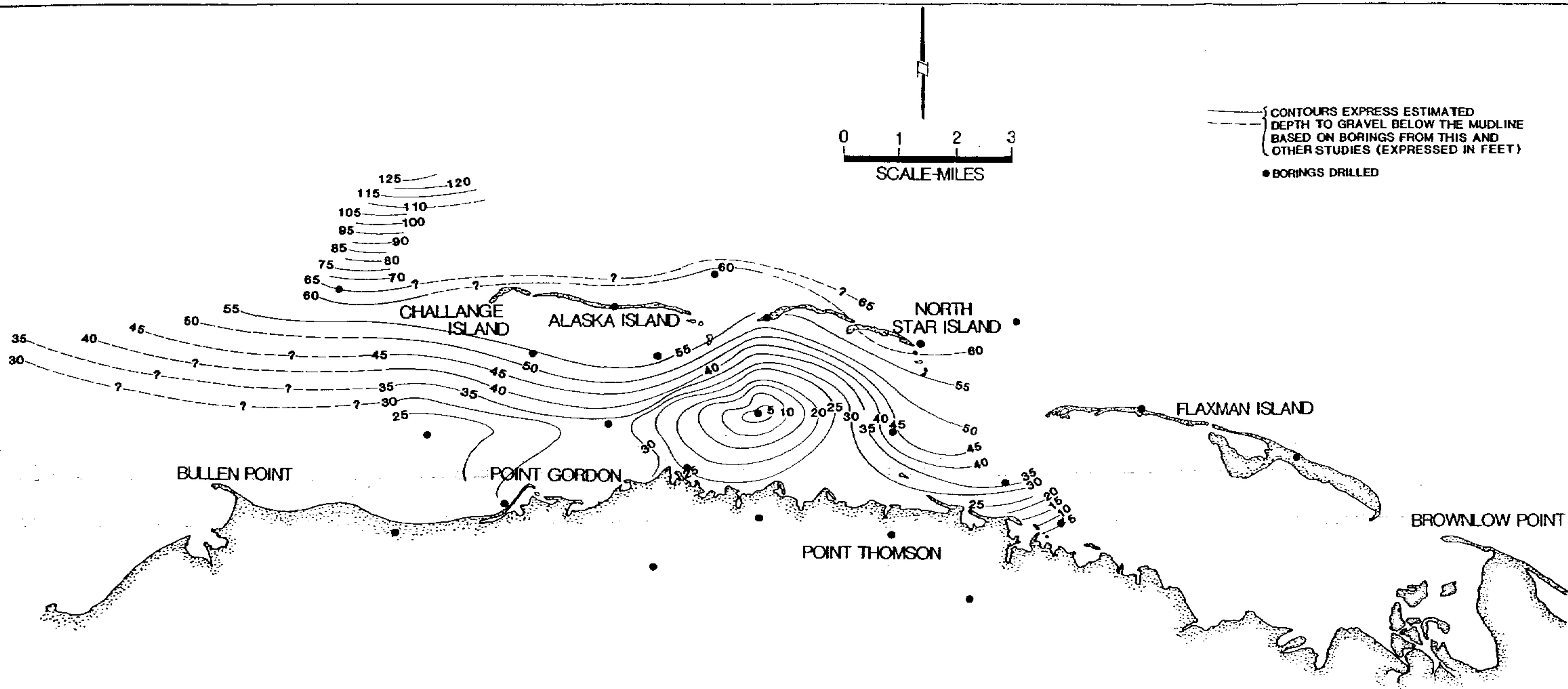
Holocene beach deposits (map symbol QHb) consist of medium- to coarse-grained sand and gravelly sand. In HLAs' "Interpretation of Geophysical, Geologic and Engineering Data" (1979) the beach deposits in the lagoon area were considered to be a lag deposit (map symbol Qf) created by the winnowing of an older unit. The beach deposit thickness of up to 14 feet suggests more recent sand has been transported to the study area by the Staines and Canning Rivers. Therefore, QHb may contain Qf material. QHb has been used to designate two different Holocene beach deposits. At USGS/HLA Test Boring 15 (Appendix A), QHb is the basal transgressive deposit, a beach that was created during the last rise in sea level and subsequently inundated, whereas the beaches of the Maguire Islands and the sand spit of Flaxman Island are more recent features.

Holocene nearshore marine deposits (map symbol QHnm) consist of loose sand, silty sand and soft clay. They are present as a thin veneer in the offshore study area and have been subjected to reworking by ice gouging in an open marine environment.

Surficial concentrations of gravel, cobbles, and boulders are present on many of the beaches within the study area and are also exposed on the seabed. These Holocene lag deposits (Qf) are the remains, after erosion, of the boulder-rich Flaxman Unit (QPf) of the Gubik Formation. The boulders rest upon eroded outcrops of the Gubik and may be up to 14 feet thick locally; however, this unit is generally less than 3 to 4 feet thick.

(5) Offshore Gravel Resources

Plate III-6 shows our interpretation of the depth to gravel in the lagoon based on the test boring data. The two most promising sites for ground extraction are in the vicinity of Borings 11 and 20 (Appendix B). The water depths at these borings are 9.5 and 8.8 feet, respectively.



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Depth to Gravel Map

Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **III-6**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08		4/82		

Gravel in Boring 11 is part of the QPa unit discussed above, as shown by the typical Alaska chert lithology of the gravel. The gravel at Boring 20 is probably Qf and separated from QPa by a 7-foot-thick layer of QPf. Thus, the better gravel site is the vicinity of Boring 11 where an uninterrupted sequence of sand and gravel is present to at least 50 feet below mudline.

Conservative estimates, based on our Depth to Gravel Map, Plate III-6, suggest that within a 1/2 mile radius from Boring 11 there may be in excess of 10 million cubic yards of gravel available within 50 feet of the mudline.

2. Seismicity

The Point Thomson area is within seismic risk Zone 1 of the Uniform Building Code. The risk of the project area being affected by a significant (greater than 5.0 Richter magnitude) seismic event is considered low. The only recorded seismic event greater than magnitude 5.0 occurred in 1937 northeast of Herschel Island in MacKenzie Bay, approximately 200 miles east of the PTD project site (Woodward-Clyde, 1978). The closest known active fault to the project site is the Kobuk Fault which borders the southern foothills of the Brooks Range about 230 miles south of Point Thomson. The mapped length of the Kobuk Fault extends more than 300 miles; however, only a portion of the fault displays evidence of Quaternary activity (Patton, 1973).

Alyeska Pipeline Service Company used the following ground motion parameters for the design of facilities at Prudhoe Bay: maximum contingency level acceleration of 0.12g, maximum contingency level velocity of six inches per second, maximum operating level acceleration of 0.06g, maximum operating velocity of three inches per second and a duration of 5 seconds (Corps of Engineers, 1980).

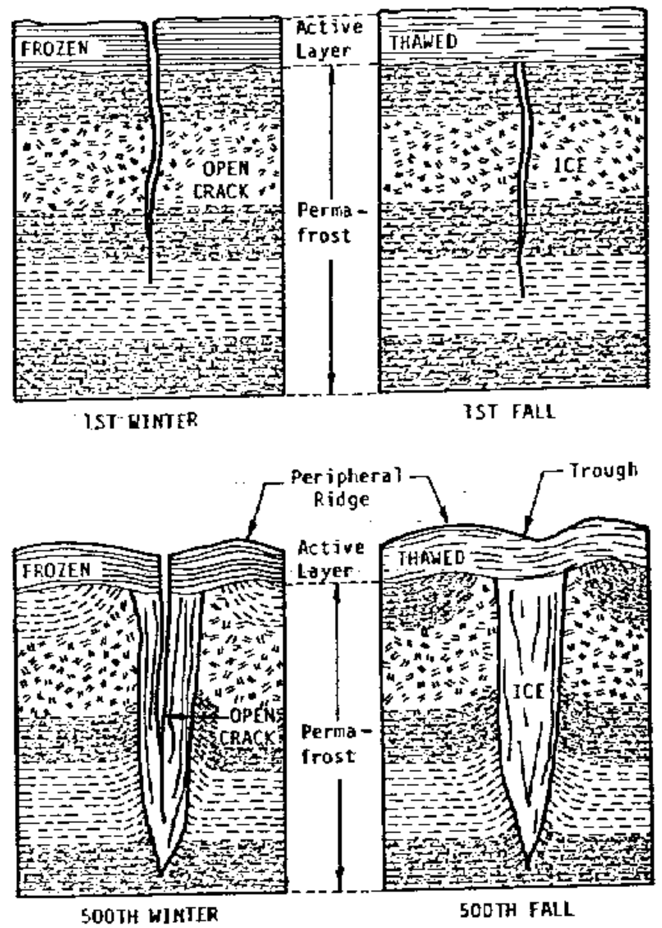
3. Permafrost

a. Onshore Permafrost

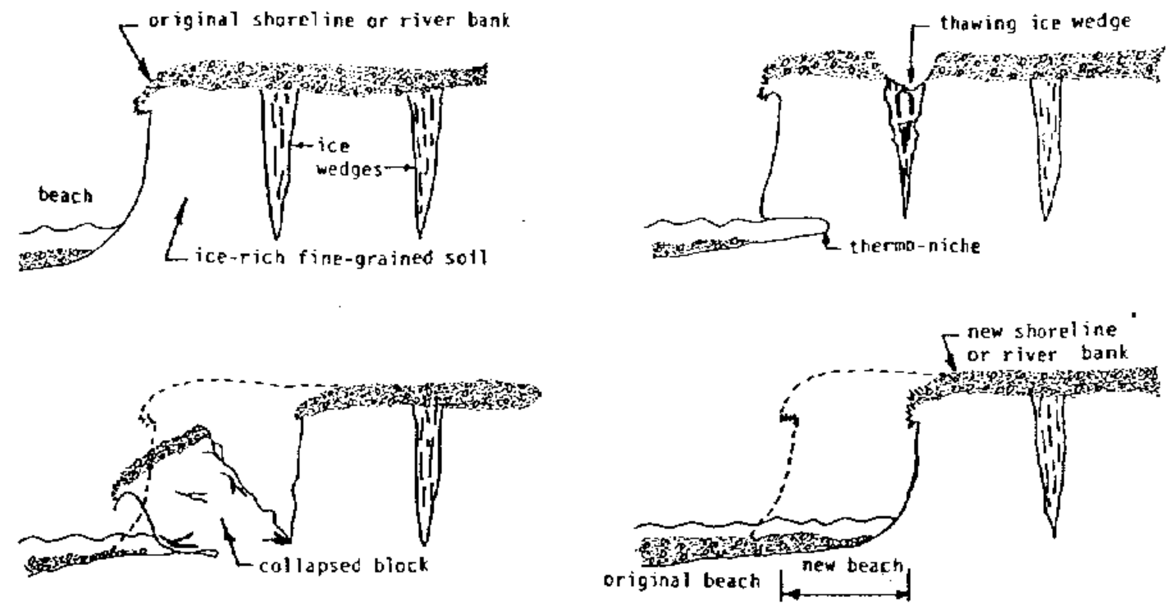
The dominant feature of the Upland Fan region is the continuous layer of perenially frozen ground known as permafrost. The maximum depth of permafrost at Prudhoe Bay is approximately 1800 feet (Howitt, 1971) and similar depths are expected in the Point Thomson Development area. Permafrost extends almost to the ground surface, except for thaw pockets typically located beneath deep lakes and abandoned or active major river channels.

Permafrost may be ice free at $0^{o}C$ or below, depending on the salinity of the interstitial water. When permafrost occurs in clay beds, substantial amounts of unfrozen moisture can persist at temperatures several degrees below $0^{o}C$ due to the freezing point depression produced by capillary forces (Ferrians, Kachadoorian, and Green, 1969).

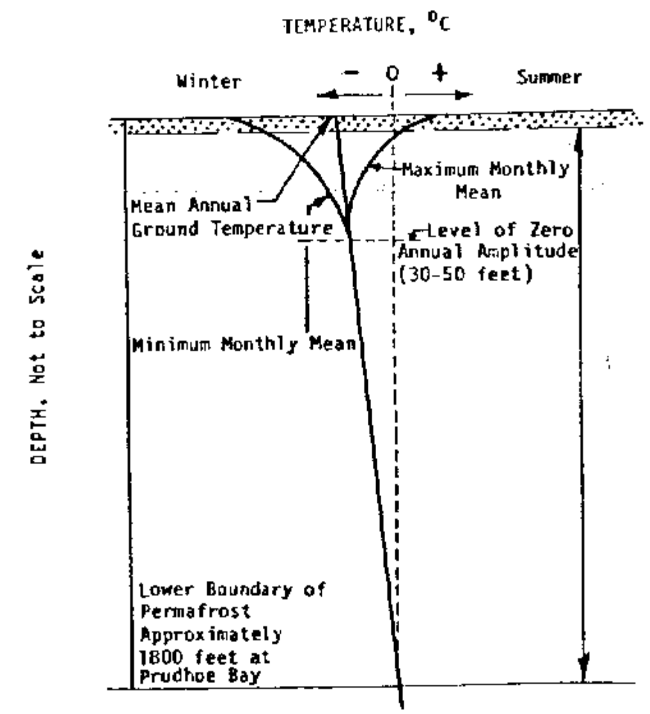
As shown on Plate III-7 the temperature of the permafrost zone is lowest near the ground surface during the winter months and gradually increases with depth until it reaches $0^{o}C$. The depth of the permafrost is controlled by heat flow from the inner core of the earth. Beneath a depth of 30 to 100 feet below the ground surface the temperature remains almost constant. We would expect ground temperatures at a depth of about 25 feet to vary annually from $-8^{o}C$ to $-12^{o}C$. The temperature lag relative to air temperatures at this depth is five to six months (Lachenbruch, 1959). Warmer temperatures are found near bodies of water such as lakes, rivers, and the ocean. Ground temperatures observed during the Point Thomson study are presented in Chapter IV.



Schematic representation of the evolution of an ice wedge according to the contraction-crack theory (After Lachenbruch, 1963)



Schematic of thermo-erosional niche



Schematic of typical ground temperature regime in permafrost areas

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ICE WEDGE EVOLUTION, THERMO-EROSIONAL NICHING, AND GROUND TEMPERATURE REGIME

Pt. Thomson Development Project, Winter 1982, Geotechnical Study, EXXON Company, U.S.A.

PLATE **III-7**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
SP	9612,031.08	DEB	4/82		

Permafrost is overlain by a shallow "active layer" which thaws in the summer and freezes in the winter. Away from the influence of streams and lakes, the active layer is generally between several inches and several feet thick, depending upon the amount of incoming solar radiation, the texture and water content of the soil, the depth of snow, and the thickness of the vegetative cover.

Tundra vegetation provides a good insulating mat and in the PTD study area the active layer beneath the mat is relatively thin (1.5 to 3.5 feet). The high moisture content of the tundra soils also contributes to the slowing of heat exchange between the atmosphere and the soils, making the soils relatively cool in summer. Sands and gravels, such as are found in the active and abandoned floodplains of the Canning River, generally thaw to depths of five feet if not covered with a vegetative mat. Thaw depths as much as eight feet have been encountered in similar gravels along the Sagavanirktok River.

b. Offshore Permafrost

The mean annual bottom water temperature for the inner shelf of the Beaufort Sea is between -1.09^{o}C and -1.50^{o}C. Bottom water temperatures below 0^{o}C are expected in water less than 500 to 1000 feet deep (Selkregg et al., 1975). Therefore, all sediments within the offshore project area meet the broad definition of permafrost.

In the following discussion, the term relict permafrost is used to describe old bonded permafrost that probably has remained from the last subaerial exposure of the seabed, as opposed to shallow, probably recently formed, permafrost. The following explanation for the existence of relict ice-bonded permafrost beneath the Beaufort Sea has been proposed by Hopkins and Hartz (1978):

> The position of the shoreline in the Beaufort Sea 18,000 years ago, lay somewhere seaward of the 20-meter isobath and borehole data thus suggest, in fact, that relative sea level fell at least 90 meters below present in the Beaufort Sea. The mantle of marine silt and clay, deposited during Sangamon time (approximately 120,000 years ago), became frozen as did the underlying gravels. The total thickness of bonded permafrost formed at any particular place, depended partly upon the duration of exposure to sub-aerial temperatures, the thicknesses of several hundred meters were formed in most areas of the shelf landward of the present 20-meter isobath. (sic)

The distribution of relict permafrost in the offshore project area was investigated during the HLA/USGS test boring program of 1979 (HLA 1979) and the test boring program performed during the winter of 1982 for this study. The data from these studies suggest that the ice-bonded permafrost in the offshore project area is relict, having formed during the last low sea level standstill, and that the depths to the ice-bonded layer are highly variable. Seasonal ice bonding and development of new permafrost in surficial sediments take place only within bottom fast ice zones in very shallow waters (about 2.5 feet deep).

The data obtained from the two test boring programs indicate that beyond a depth of about six feet below sea level and away from the immediate vicinity of the barrier islands, relict ice-bonded permafrost is restricted to the Pleistocene units of the Gubik Formation. The depths to ice-bonded permafrost in the project area are shown on the cross sections, Plates IV-4 through IV-11 and the Regional Soil Map, Plate IV-1 presented in Chapter IV.

Temperature measurements obtained in all of the borings indicate that both ice-bonded and unbonded sediments have temperatures well below $0^{o}C$. It was also found that all of the borings have negative thermal gradients, indicating that ice-bonded permafrost may be present at depths not reached by the borings.

4. Onshore Geologic Processes

a. Ice Wedging

Polygonal or patterned ground is the most conspicuous surface feature of the Upland Fan area. The polygonal patterns are formed by temperature-induced contraction cracks similar to those encountered on dry mud flats. These cracks fill with water and freeze. Continued cracking, filling, and freezing along the same lines eventually produces a network of ice wedges several yards deep and tens of feet apart.

A schematic representation of the evolution of an ice wedge is presented on Plate III-7 (Lachenbruch, 1963). The size of polygons and the spacing of ice wedges varies from less than 20 to more than 300 feet across. The depth of the wedges varies from 3 feet to more than 25 feet. Ice wedges may be from a few inches to up to 10 feet wide at the top, and they commonly taper with depth.

The compression of the ground adjacent to ice wedges causes peripheral ridges. The ridges can prevent drainage from the center of the polygon, resulting in small shallow ponds in the polygon centers.

b. Thaw Lake Cycle

There are thousands of thaw lakes, ponds, and pools dotting the Upland Fan area. The lake basins originate in areas of restricted drainage where warm surface temperatures cause the underlying ground ice to thaw, resulting in subsidence. Most of these small ponds are less than three feet deep. The lakes may be regular or irregular in shape, but there is a strong tendency for the larger lakes (approaching one mile long) to be elongated or elliptical and oriented parallel to one another.

The thaw lakes go through a cycle of development, expansion, drainage, and revegetation. Development of an oriented thaw lake begins with climatic change or disruption of the vegetation and organic cover of the polygonal tundra. Thawing of the ice-rich near-surface materials and melting of ice wedges result in standing water which eventually becomes a thaw lake. If the pond is large enough, the permafrost beneath it and along its sides thaws, resulting in expansion of the water body.

Thawing occurs most rapidly below the water level, eroding the surrounding tundra and rapidly melting and truncating ice wedges to produce an irregular shoreline. Eventually, thawed and frozen blocks of tundra collapse into the growing lake. The lake grows most rapidly along the axis perpendicular to the wind, partially because of increased current velocities and seasonally higher temperatures of the water at the ends of the lake (Carson and Hussey, 1962). Eventually, the lake intersects a natural outlet and drains. In time, vegetation recolonizes the lake bottom, and the thawed depression fills with peat and other organic materials. Ice wedges form new polygons, and the cycle begins again.

c. Beaded Stream Erosion Cycle

Beaded streams form the small, northward flowing drainages common in the PTD study area. A beaded stream consists of a series of elliptical or irregularly-shaped pools varying in depth from three to nine feet. They can vary from a few tens of feet to hundreds of yards in width and length.

The beaded streams are formed from the melting of the ice wedges which underlie the polygon rims. Warm surface waters melt the ice and form a depression on the tundra surface. A series of these thermokarst depressions along a drainage route forms a beaded stream (Lewellen, 1972).

Rex (1953) proposes four phases of development. Phases 1 through 3 are characteristic of most of the beaded streams in the area:

Phase 1 - A marshy feeder area of polygon trough intersected with pools usually located at the polygon intersections.

Phase 2 - A stream channel marked by a string of pools. The pools are circular if located at the ice wedge intersections or elongated if located between two polygons.

Phase 3 - A clearly defined and often entrenched stream between large circular pods; sometimes the stream surrounds a polygon.

Phase 4 - Several large bonded pools joining to form a thaw lake.

Chapman (1964) reports the headward erosion along the gully of a melting ice wedge that intersected a lake which subsequently drained. A good example of the headward erosion is shown on Plate III-8. This figure was obtained from the aerial photos of the project area immediately south of Point Gordon where a beaded stream has drained a large lake located in Sections 17 and 18, T9N, R22E.

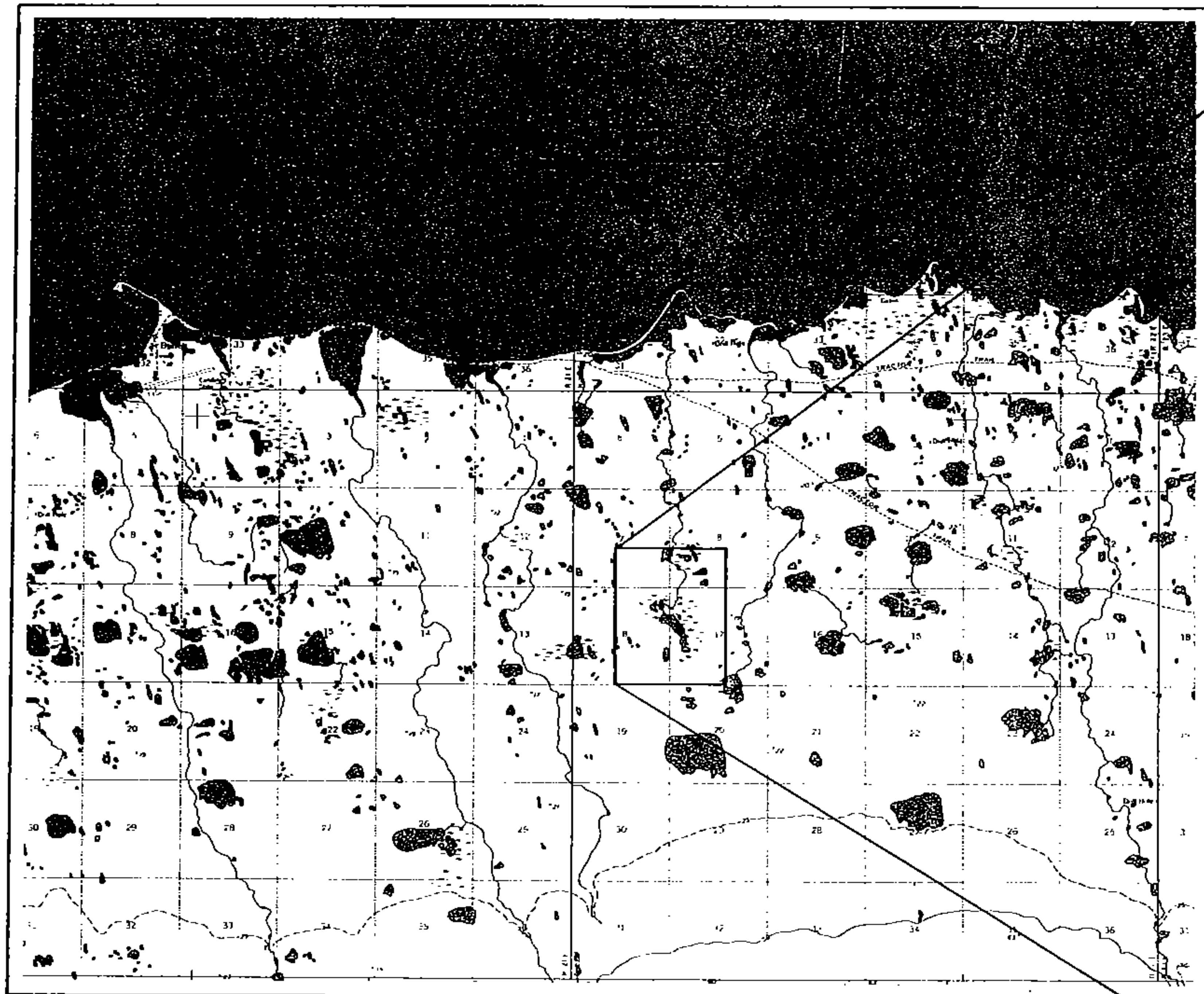


Figure 1 USGS area map

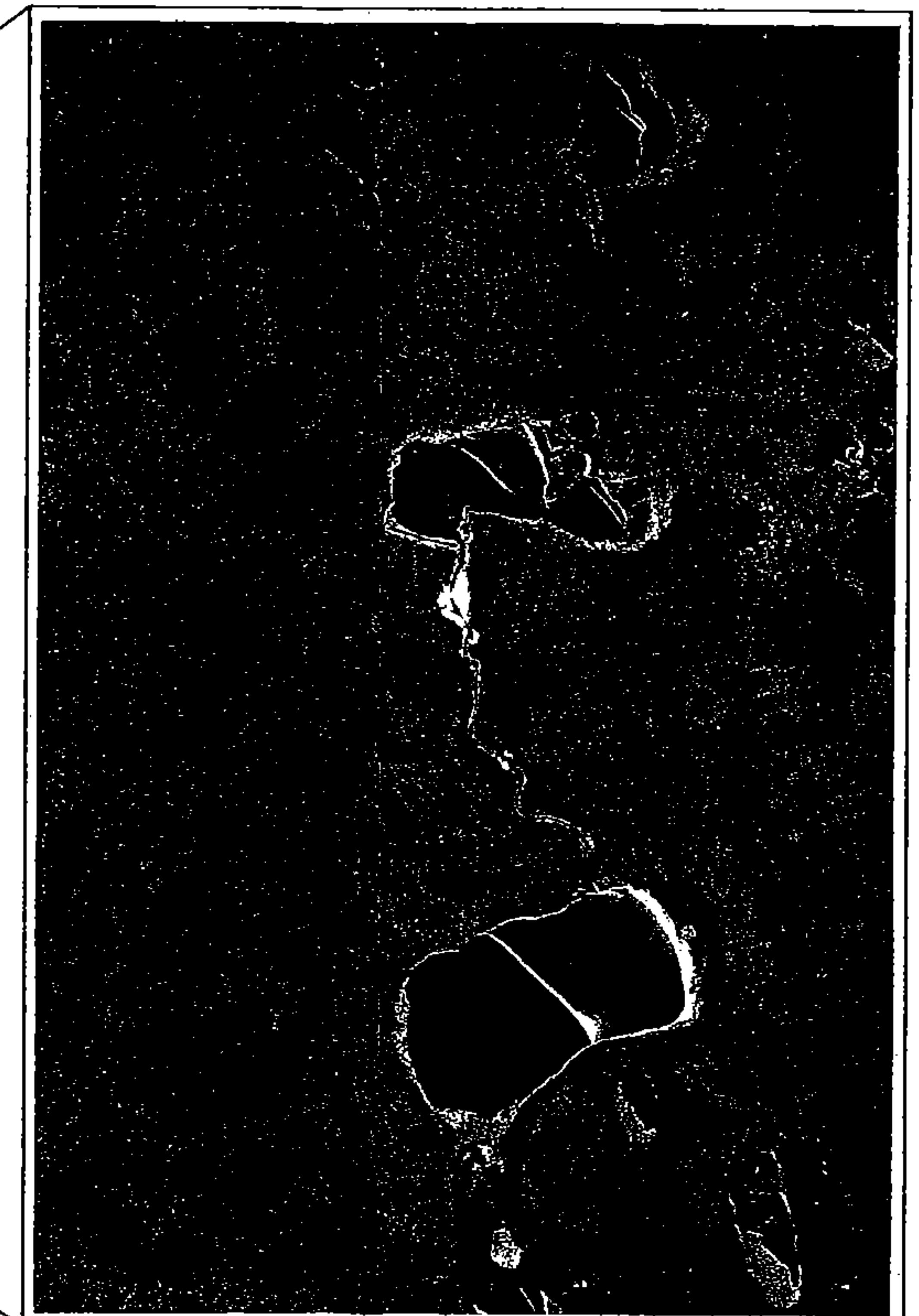


Figure 2 Beaded stream and erosion of lakes (from aerial photo 6-6, Exxon, 1980, Point Thomson.)

Harding Lawson Associates Engineers Geologists & Geophysicists	**Beaded Stream Erosion Cycle** Pt. Thomson Development Project Winter 1982, Geotechnical Study EXXON Company, U.S.A.	PLATE **III-8**
DRAWN JP	JOB NUMBER 9612,031.08	DATE 6/82

d. Frost Action

In the project area, frost action within the fine-grained active zone soils is intense. Pressures generated during the freezing of the active zone tend to jack the gravel-sized soil fractions to the surface, forming frost boils and stone rings. This process also causes frost churning or cryoturbation of the active zone soils. The depth of soils experiencing cryoturbation varies with soil gradation and moisture content.

e. Thermo-Erosional Niching and Shoreline Retreat

(1) Thermo-Erosional Niching

Thermo-erosional niching, the undercutting of river banks or coastal bluffs by the thermal action of flowing water, is the most active erosive agent along the beaches in the project area. Thermo-erosional niching is defined as lateral erosion resulting from melting of ground and interstitial ice accompanied by lateral current transport of the liberated fine materials (Hopkins and Hartz, 1978). Plate III-7 shows the stages of thermo-erosional niching of a wedge of fine-grained permafrost and its ultimate collapse and lateral transport.

(2) Coastal Erosion

Based on aerial photographs from the late 1940's to present, the coast within the project area is retreating at an average rate of six to nine feet per year (Hopkins et al., 1977). The coastline from Point Thomson to Brownlow Point is reported by Lewellen (1977) and Hartz (1978) to be retreating at rates of 22 and 12 feet per year, respectively. Leffingwell (1919) estimated that Brownlow Point was being eroded at the much higher rate of almost 30 feet per year.

The rapid retreat of the coast is a result of the process of thermo-erosional niching acting on beach bluffs composed primarily of fine-grained, ice-rich sediments. The erosion rates depend on the local morphology, the orientation and exposure of the coast, and, according to Lewellen (1977), the ice content and grain size of the coastal sediments being eroded.

Drawings of Flaxman Island prepared by Lewellen (1970) from aerial photographs taken during the period 1949 through 1968 (U.S. Army, 1949, 1955, 1968) are presented on Plate III-9. Comparison of Lewellen's drawings to outlines of the island taken from airphotos dated 1980 indicate the north face of the island has retreated approximately 70 feet in 12 years for an average annual rate of about 6 feet per year. Based on these data, the north face of the island has retreated approximately 360 feet for an average annual rate of about 12 feet per year. This rate of retreat compares well with Lewellen's estimates (Plate III-9) and suggests that the coastal promontories and barrier islands are undergoing higher annual rates of erosion than the rest of the coast in the project area.

5. <u>Offshore Geologic Processes</u>

The major processes affecting the seabed in the lagoon are strudel scour, ice gouging, and the longshore transport of sediments by ocean currents. The dearth of available data on the lagoon and shallow offshore environment makes it difficult to accurately assess the impact of these processes in the study area. However, some conclusions can be drawn by comparison to other similar settings.

a. Strudel Scour

Scouring of the sea floor occurs each spring when north flowing rivers break up sending their fresh waters flowing across the sea ice. The water drains through holes and cracks in the sea-ice canopy and acts as a jet of water to scour the seabed.

Major seabed modification by this phenomenon is restricted to areas around river mouths. As such, strudel scour should be restricted to the eastern part of the PTD project area within four to six miles of the mouth of the Staines River. Severe scouring will probably be restricted to that portion of the lagoon west of the center of Flaxman Island. Water depths east of the center of Flaxman Island are generally less than six feet and the ice canopy is normally frozen to the lagoon bottom by late spring. As strudel scouring is most effective where there is water under the ice canopy, the area east of the center of Flaxman Island will probably be protected from severe strudel scour.

No data are available on the depth of scour in the lagoon. However, Toimil (1979) reports strudel scours of undetermined depth in water eight to ten feet deep three miles east of Point Thomson.

b. Ice Gouging

Ice gouging is not expected to be a major hazard in the lagoon. The lagoon is shallow, well protected from large multi-year ice keels, and major shear zones are restricted to the seaward side of the barrier islands. The shelter offered by the barrier islands suggests gouging will be less intense in the lagoon than that found in Stefansson Sound. Gouge orientation is random rather than concentrated and the maximum gouge depths do not exceed two to three feet.

Seaward of the barrier islands ice modification of the sea floor is probably intense. The major winter shear zone forms close to the barrier islands and a gouge density of 50 gouges per kilometer with sub-seabed penetration up to 1 meter are expected. The extremely irregular bathymetry seaward of the barrier islands probably owes its origin to ice gouging.

c. Sediment Transport

The dominant direction of sediment drift along the Beaufort Sea coast is westward; however, there are many local variations. West of Brownlow Point longshore transport is to the west and east of Brownlow Point it is to the east. Thus, the majority of sediment supplied to the marine environment by the Canning River is funneled into Camden Bay and the smaller Staines River is the major supplier to the lagoon.

Our borings logs indicate sand and silt are being supplied to the lagoon. The most probable sources of this sediment are the Staines River and the eroding beach bluffs surrounding it. They also indicate little recent material is being supplied to the offshore shallow marine environment.

Relatively large amounts of sediment flow into the lagoon during spring break up. The flow tapers off throughout the summer season and is negligible during the winter months. Since the boring logs show sand is the most prevalent surface material, much of the silt and clay size material either remains in suspension or is put into suspension during summer storms and transported out of the lagoon.

6. Terrain Units

Terrain unit mapping was performed using color aerial photographs taken by North Pacific Aerial Surveys dated September 1980. Aerial photos covering Flaxman Island are at a scale of 1 inch equals 700 feet. The remainder of the project area was mapped using color aerial photos at a scale of 1 inch equals 500 feet. The terrain unit map is presented on Plate III-10.

a. Low Centered Polygons

Low centered polygons (map symbol LCP) are one of the most easily recognized landforms in the Point Thomson region. These polygons are caused by repeated intense freezing and thawing in fine-grained soils, resulting in the growth of intersecting ice-wedges, which form the polygon rims. Ice-wedge growth is discussed in Section 4 and is shown graphically on Plate III-7.

Low centered polygons generally occur in areas of poor surface and subsurface drainage. During the thaw season, the depressed central portion of the polygon can contain up to 1-1/2 feet of standing water. Low centered polygons with wet centers are common along the coast and in the Upland Fan adjacent to thaw lakes.

Soil conditions typical of low centered polygon areas were encountered in Boring 18 (Appendix B). In areas with low centered polygons, ice-rich silts (moisture contents greater than 50 percent) and massive ice predominate in the upper 10 to 15 feet and are commonly underlain by granular soils with a lower ice content than the silt. The ice in the granular soils generally occurs as grain coatings and as random crystals.

b. High Centered Polygons

High centered polygons (map symbol HCP) can be recognized by the deep troughs which form their perimeters. Within the study area typical exposures of high centered polygons occur on the coast at the break in slope between the Upland Fan and the beach and along the shores of the drained lakes immediately east of the Staines River.

High centered polygons are caused by thermo-erosion of the ice and ice-rich soils that form the perimeter of low centered polygons. The process is initiated by an improvement in the surface and subsurface drainage in an area containing low centered polygons. Melt water escaping from the ice wedges within the polygon perimeters causes the perimeter troughs to deepen, exposing the soil on the polygon rim to thermo-erosion. As this process is repeated over a number of years, the rim soils become so eroded that they form troughs around the polygon centers.

Soil conditions in high centered polygon areas are the same as those in low centered polygon areas. However, there are smaller amounts of massive ice in the rim soils of the high centered polygons because of thermo-erosion.

c. Reticulate Ground

Reticulate ground (map symbol RG) appears as a network of slightly convex polygons up to 5 feet in diameter. The center of each polygon is generally formed by frost-boil tundra having a hummocky micro-relief. The frost boils consist of irregularly shaped masses of bare soil up to three feet across. Because of frost activity, rocks are commonly segregated from the soil, forming stone rings around the perimeter of the frost boils.

Reticulate ground normally occurs in the upland areas adjacent to drainage ways, such as active and abandoned stream channels and thaw lakes. Soils in these areas consist of a thin mat of peat or organic silts underlain by gravelly sand.

The mineral soil exposed on the surface of the frost boil is exposed to the drying and erosive action of the wind. Removal of the fine grained soils by the wind often results in dish shaped scars in the tundra surface up to a few feet deep. In some areas the scars may be as much as 50 feet in diameter.

Frost boil tundra, in conjunction with high and low centered polygons, is the predominant terrain unit in the Upland Fan.

f. Thermokarst Terrain

Thermokarst terrain (map symbol TK) is a composite terrain consisting of high centered polygons and low center polygons undergoing topographic reversal and conversion to high centered polygons. Small, irregularly shaped thaw lakes are common, occurring at the intersections of the polygon rims. The micro-relief of this landform is quite high with topographic differences of four to six feet occurring between polygon rims and perimeters.

Thermokarst terrain is most common inland from Point Thomson westward to the Staines River and probably represents incomplete topographic adjustment to improved drainage northward to the coast and westward to the Staines River.

g. Low Energy Beaches

Low energy beaches (map symbol LEB) are rarely wider than 60 feet and are usually backed by coastal bluffs less than 15 feet high. They are composed primarily of sand and silt derived from erosion of the coastal bluffs. In areas where the Flaxman Formation outcrops in the bluffs, some coarse material in the form of gravel and occasional boulders may be present.

Wave activity along the low energy beaches is generally not sufficient to winnow out the silt and sand site material. As a result, the coarse gravels and boulders occur within a matrix of fine sand and silt size material.

Low energy beaches predominate along the mainland of the PTD study area and on the southern shore of Flaxman Island.

h. High Energy Beaches

Coarse grained materials predominate in high energy beaches where the winnowing action of the waves has removed the fine grained material leaving behind the coarse sand and gravel fractions. The high energy beaches are very dynamic and can undergo considerable variation in width and length on a seasonal basis. Due to their exposure to wind and waves they support very little vegetation.

High energy beaches form the bars at Bullen Point and Points Gordon, Hopson, Sweeney and Thomson. They are also well developed along the Maguire Islands and the western end of Flaxman Island. The high energy beaches along the seaward side of the eastern end of Flaxman Island are backed by coastal bluffs up to 20 feet high.

IV SOIL AND PERMAFROST CONDITIONS

LIST OF ILLUSTRATIONS

IV SOIL AND PERMAFROST CONDITIONS

A. Stratigraphy

1. Introduction

Soil and permafrost conditions were investigated by drilling and sampling 23 test borings in the PTD area. Representative samples from the borings were tested in our laboratory to measure their engineering properties. The soil conditions encountered in the 23 borings are summarized in Table IV-1 and are shown on the Regional Soil Map, Plate IV-1. Logs of the borings along with laboratory test results are presented in Volume II of this report. Plate IV-2 presents locations of cross sections. Geologic cross sections depicting soil conditions along east-west and north-south alignments are presented on Plates IV-3 through IV-10.

2. Onshore Soil Conditions

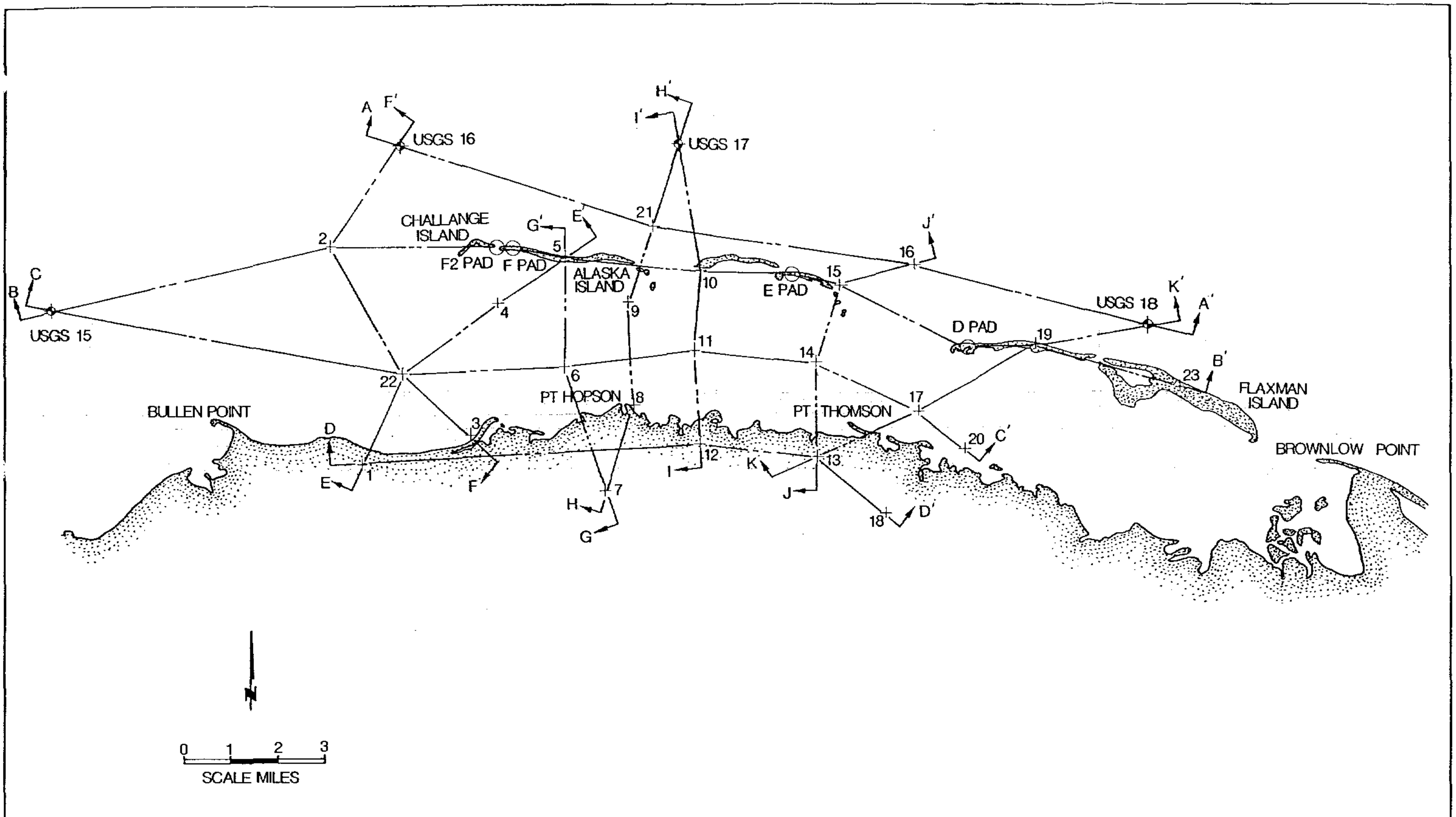
a. Soil Types

The onshore soil conditions in the PTD area generally consist of a thin layer of fine-grained Holocene silts and organic silts overlaying Pleistocene alluvial silts, sands and gravels. With the exception of the active layer, the soils are frozen throughout the year. As shown on Plate IV-11, the alluvial material comprises the bulk of the soils in the onshore area.

TABLE IV-1. SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring Number	Water Depth	Boring Depth	Depth to Bonded Permafrost	Thickness of Holocene Silt/Clay	Thickness of Holocene Sand/Gravel	Depth to Top of Pleistocene Clay/Silt	Depth to Top of Pleistocene Sand/Gravel
Offshore							
2	12.3	74.5	NE	26	4	30	55
3	5.8	51.0	38	4	--	--	4
4	11.8	51.5	NE	24	5	29	44
6	16.2	51.5	NE	4	8	12	33
8	8.5	51.5	37	2	8	10	25
9	9.5	61.5	NE	21	5	26	50
11	9.5	50.0	NE	--	2	--	2
14	9.6	51.5	NE	11	5	16	44
15	16.5	56.5	39	11	10	20	--
16	31.0	25.5	2.5	--	2.5	2.5	NE
17	9.0	51.0	NE	8	9	17	38
20	8.8	52.0	NE	2.5	13.5	16	24
21	25.0	65.0		2.5	--	2.5	NE
22	15.0	51.0	NE	10	--	--	10
Barrier Islands							
5	3.0	51.5	0	--	12.5	12.5	--
10	--	51.5	0	--	17.5	17.5	--
19	--	50.5	0	--	15	15	--
23	--	50.5	0	2	--	2	--
Onshore							
1	--	48.5	0	1.5	4 (Ice & ML)	--	5.5
7	--	49	0	2	4 (Ice & ML)	--	6
12	--	50.5	0	2	5 (Ice)	30	7
13	--	50	0	3	--	15	3/25
18	--	48.5	0	1.5	1.5 (Ice)	--	17

Note: NE = not encountered. All depths in feet.
*Unexpected shallow permafrost in deep water



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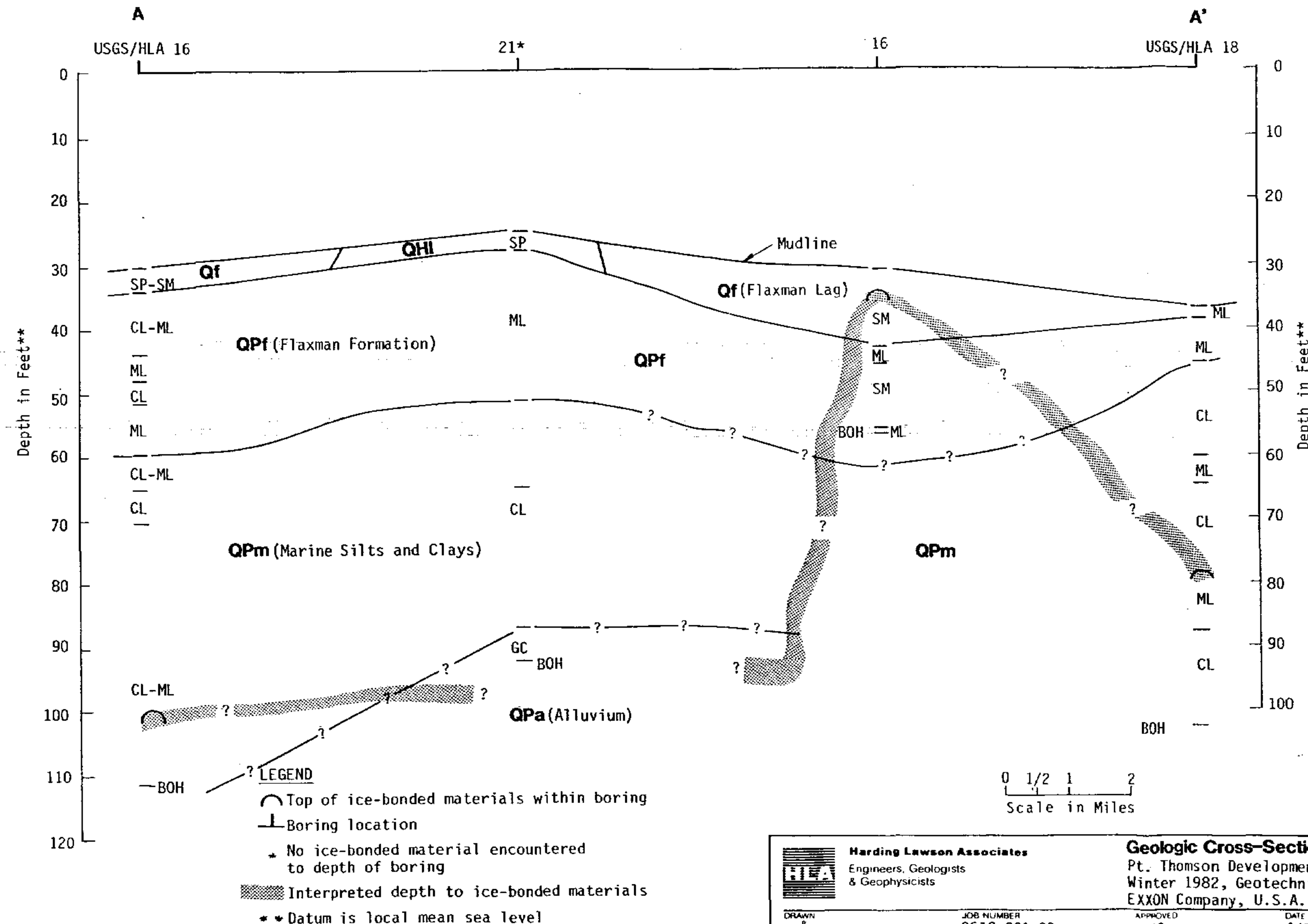
Locations of Geologic Cross Sections
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-2**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08		4/82		



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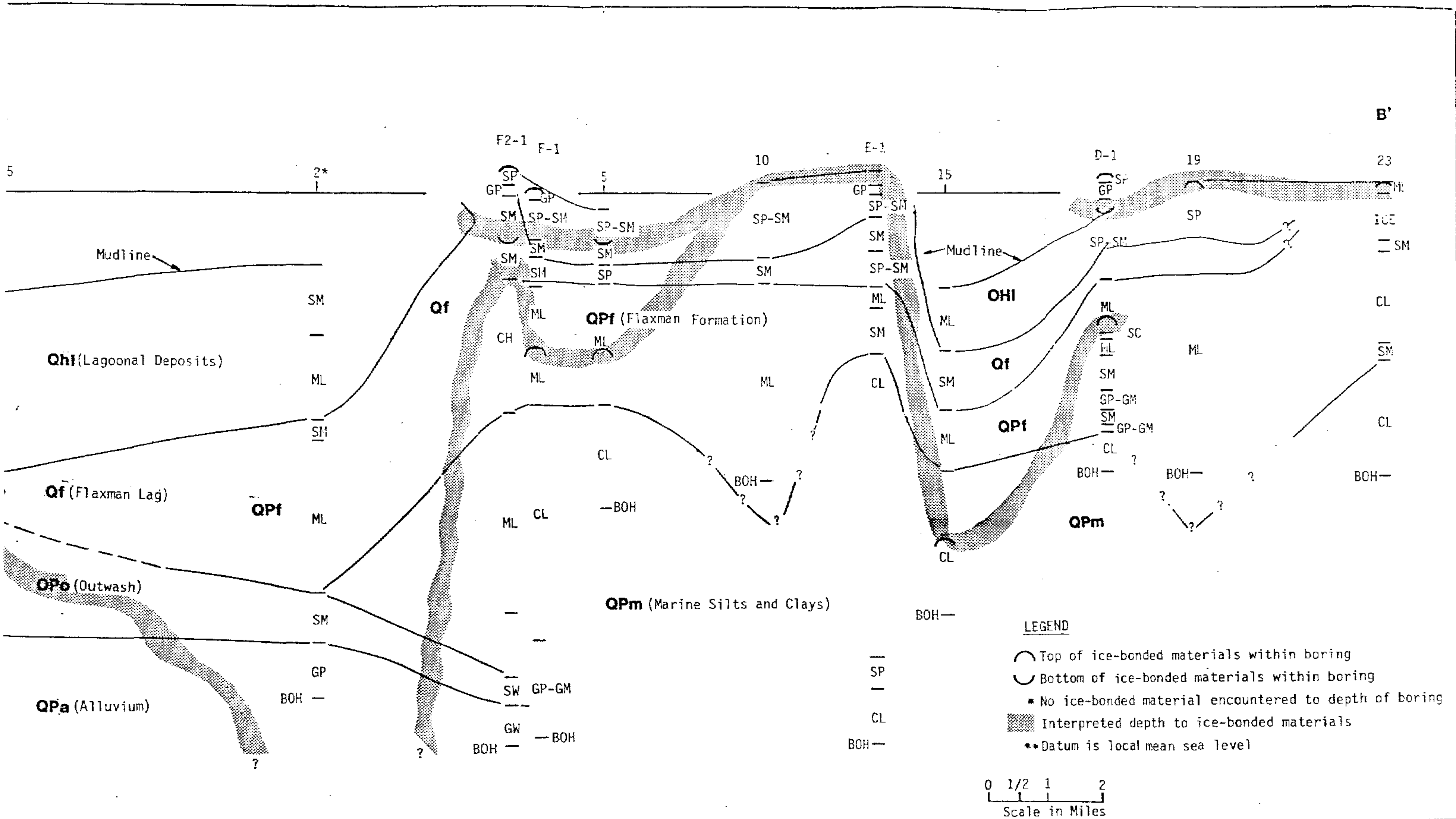
Geologic Cross-Section A-A'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-3**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08		4/82		



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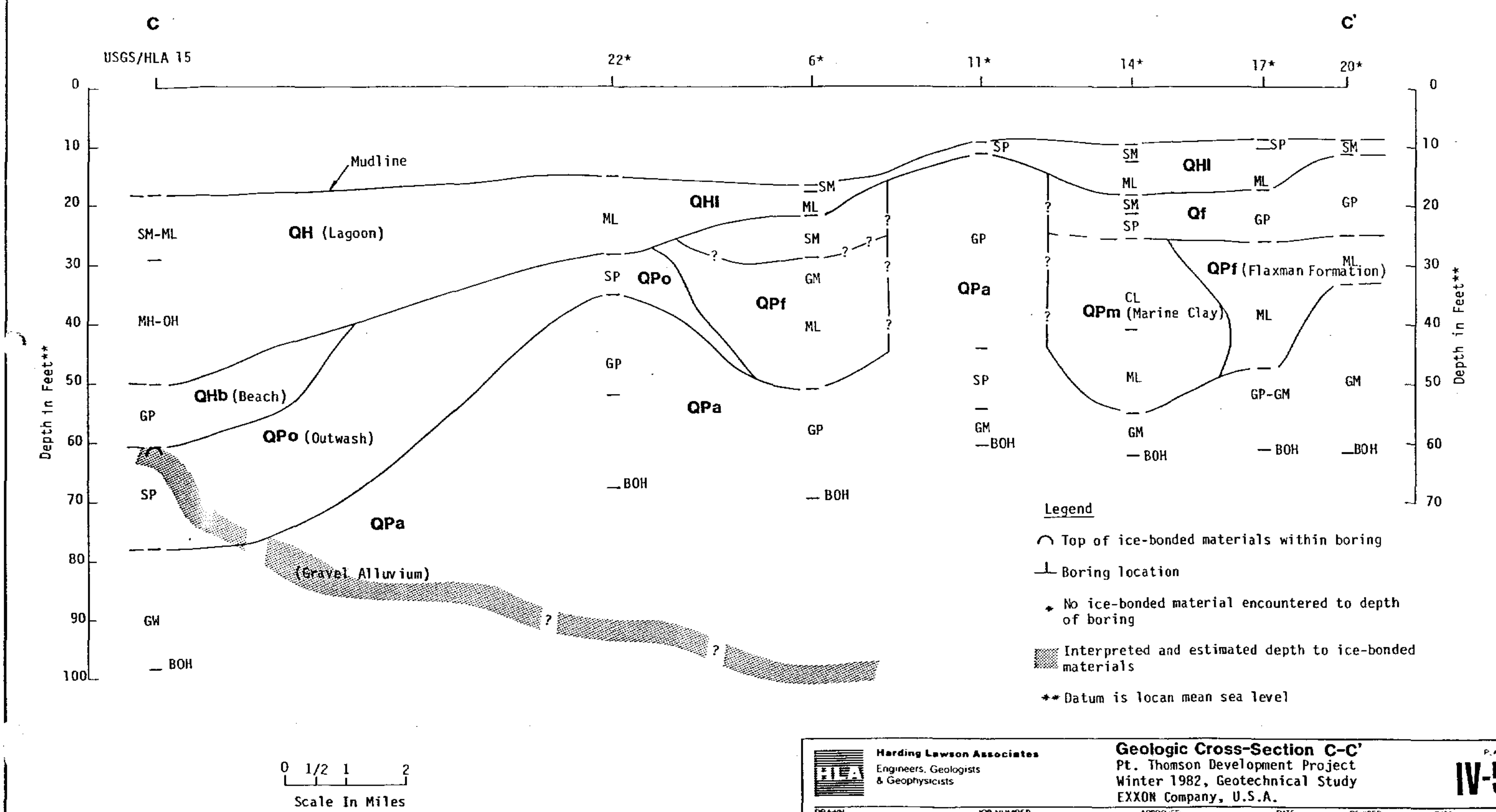
Geologic Cross-Section B-B'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-4**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
SP	9612,031.08	DEB	4/82		



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Geologic Cross-Section C-C'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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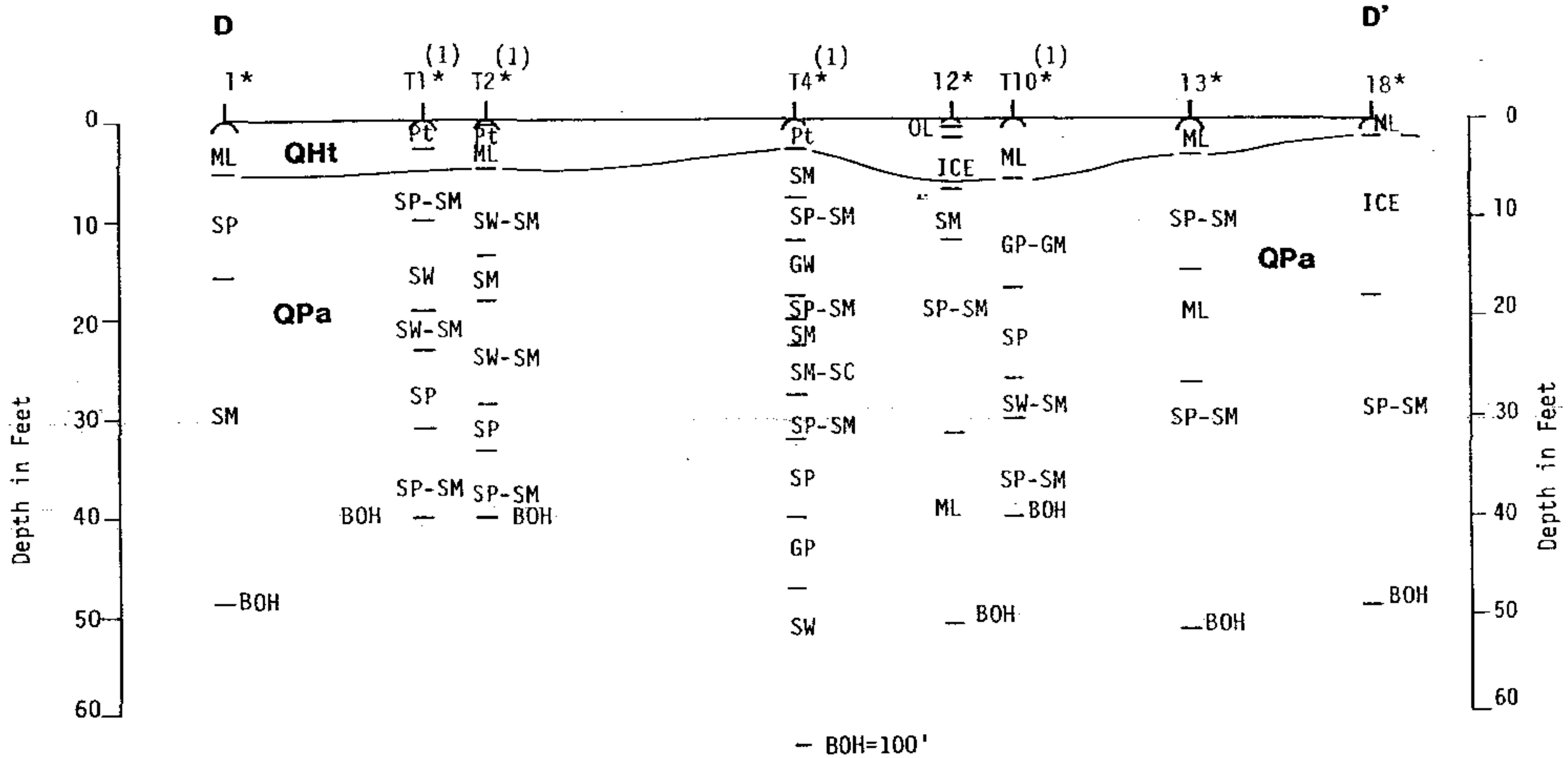
PLATE **IV-5**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JP	9612,031.08	DGB	4/82		



Legend

Top of ice-bonded materials within boring

Boring location

* Ice-bonded material encountered to full depth of boring

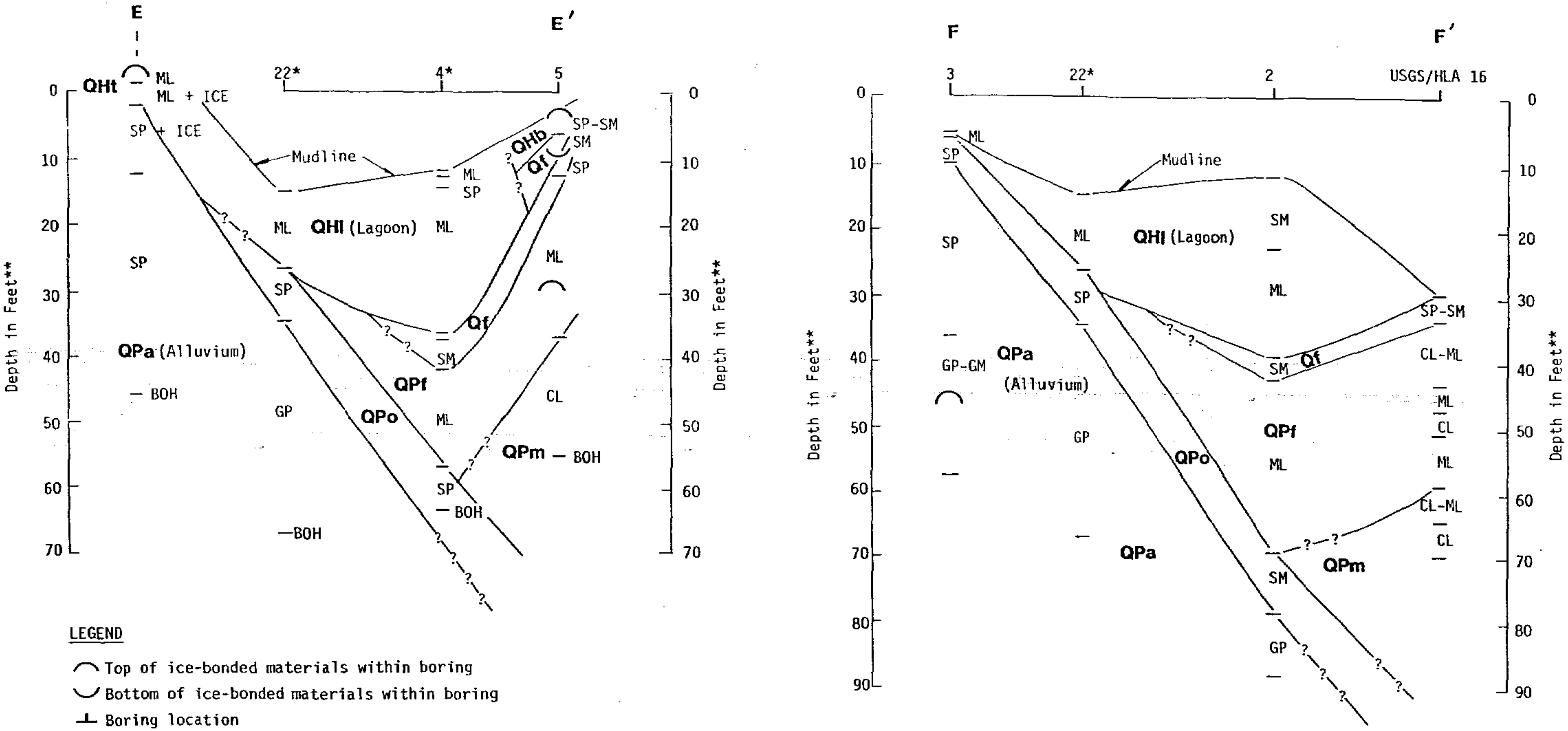
(1) Borings from Exxon Gravel Study, 1980.

0 1/2 1 2

Scale In Miles

Harding Lawson Associates Engineers, Geologists & Geophysicists	**Geologic Cross-Section D-D'** Pt. Thomson Development Project Winter 1982, Geotechnical Study EXXON Company, U.S.A.	PLATE **IV-6**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08	DEB	4/82		



LEGEND

- Top of ice-bonded materials within boring
- Bottom of ice-bonded materials within boring
- Boring location
- * No ice-bonded material encountered to depth of boring
- Interpreted depth of ice-bonded materials
- ** Datum is local mean sea level

0 1/2 1 2

Scale in Miles

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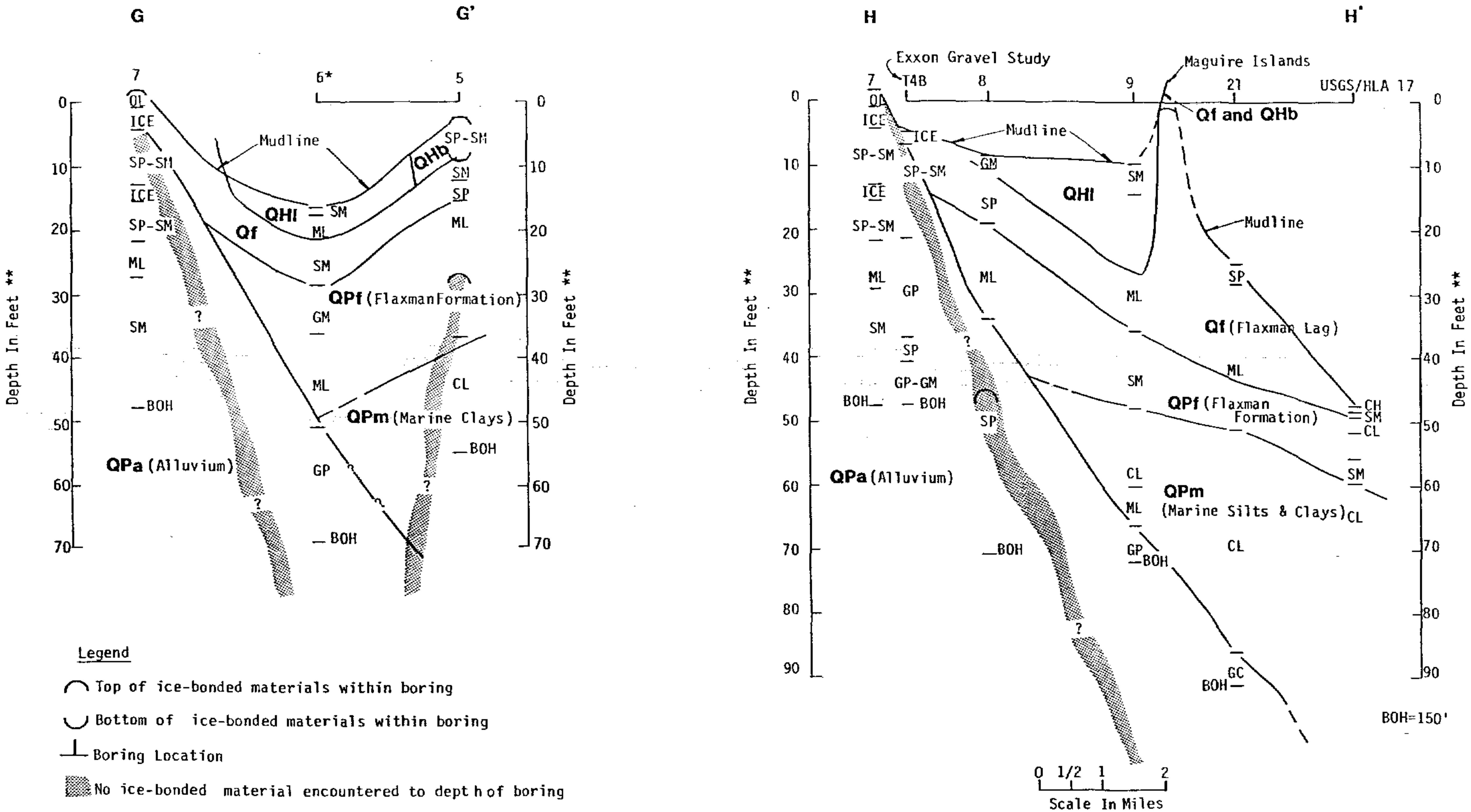
Geologic Cross-Sections E-E' and F-F'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-7**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JY	9612,031.08	DRB	4/82		



Legend

- Top of ice-bonded materials within boring
- Bottom of ice-bonded materials within boring
- Boring Location
- No ice-bonded material encountered to depth of boring
- **Interpreted depth to ice-bonded materials
- * Datum is local mean sea level

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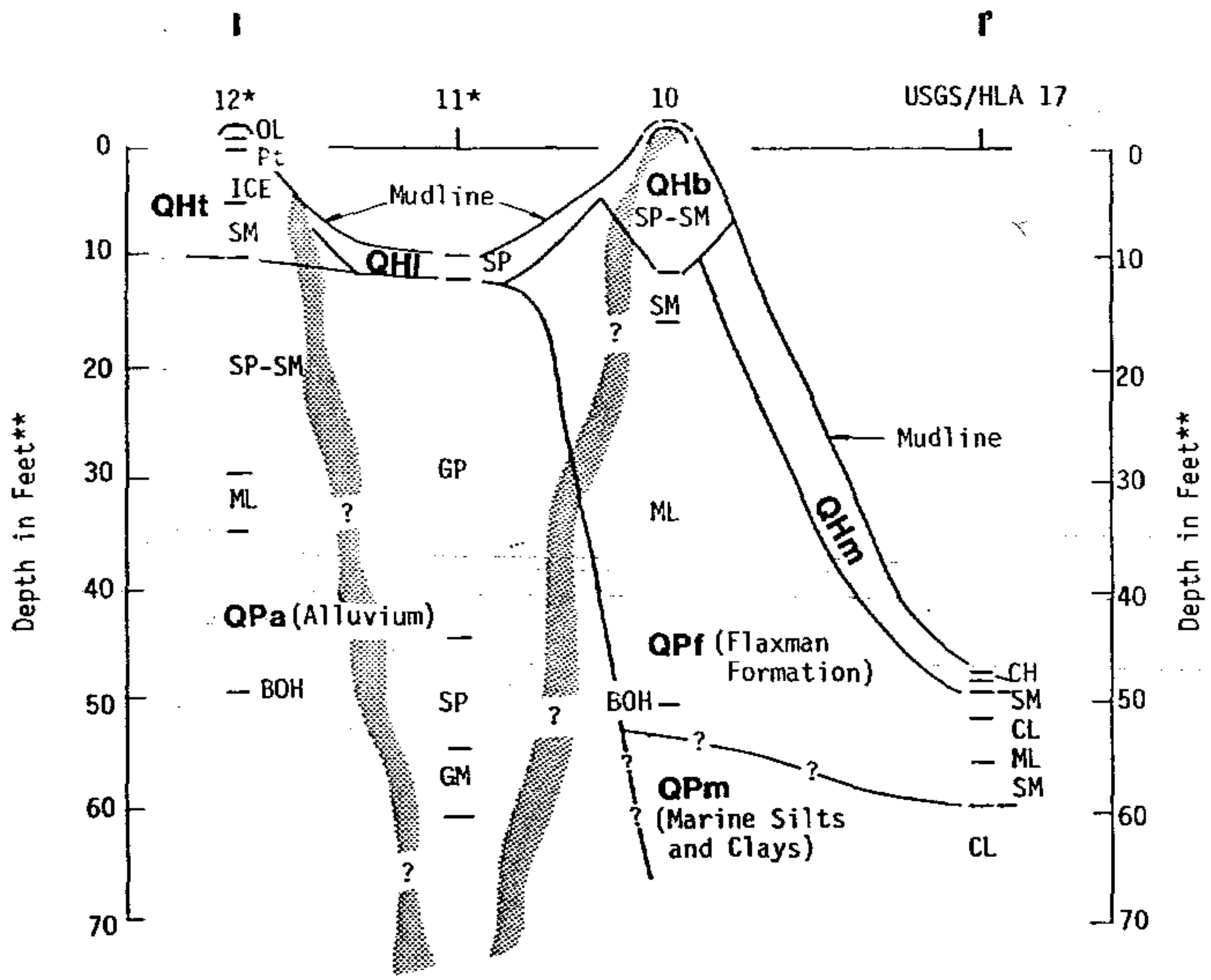
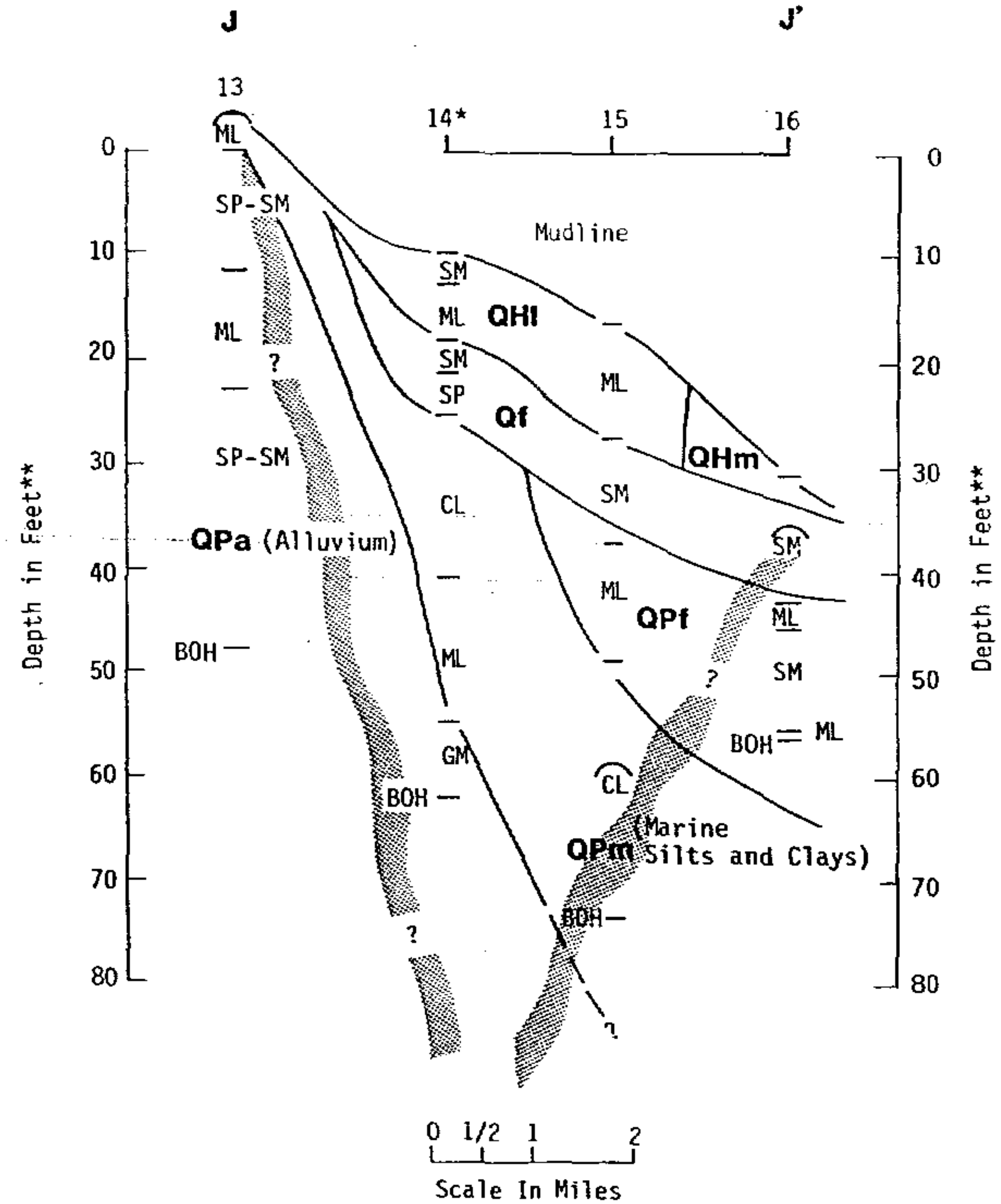
Geologic Cross-Section G–G' and H–H'

Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE IV-8

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
Y	9612,031.08	JEB	4/82		





LEGEND

Top of ice-bonded materials within boring

Bottom of ice-bonded materials within boring

Boring location

* No ice-bonded material encountered to depth of boring

Interpreted depth to ice-bonded materials

** Datum is local mean sea level

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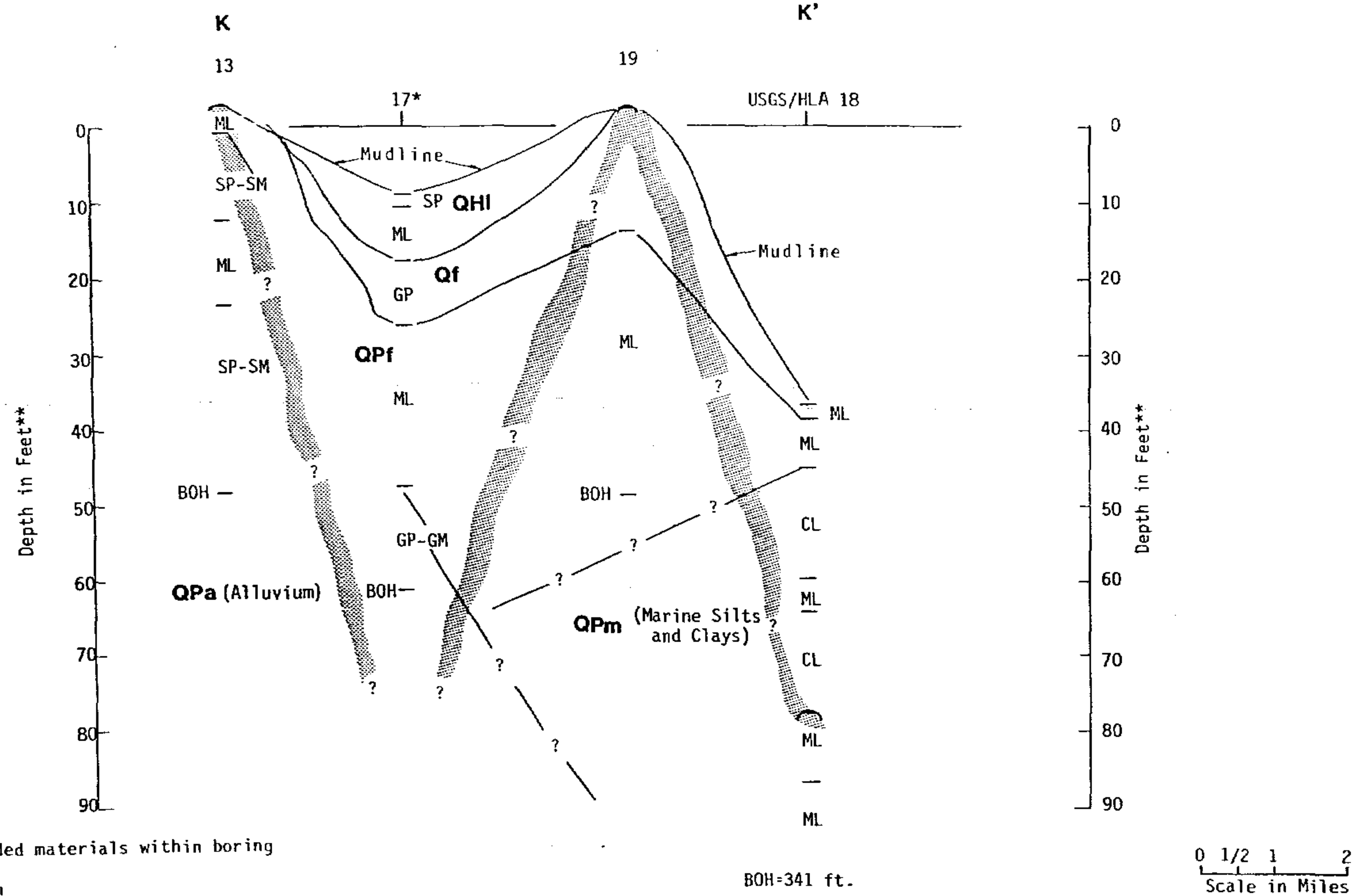
Geologic Cross-Sections I-I' and J-J'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-9**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
Y	9612,031.08	TFB	4/82		



Legend

Top of ice-bonded materials within boring

Boring location

* No ice-bonded material encountered to depth of boring

Interpreted depth to ice-bonded materials

** Datum is local mean sea level

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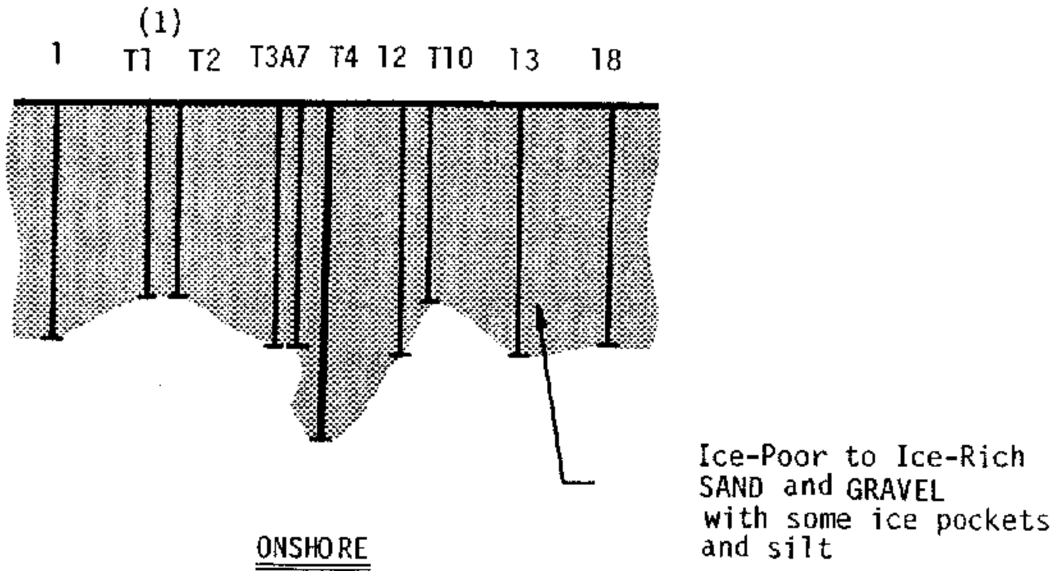
Geologic Cross-Section K-K'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

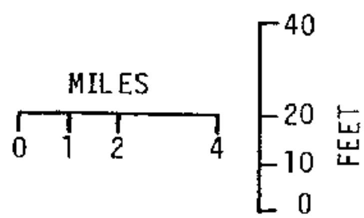
PLATE **IV-10**

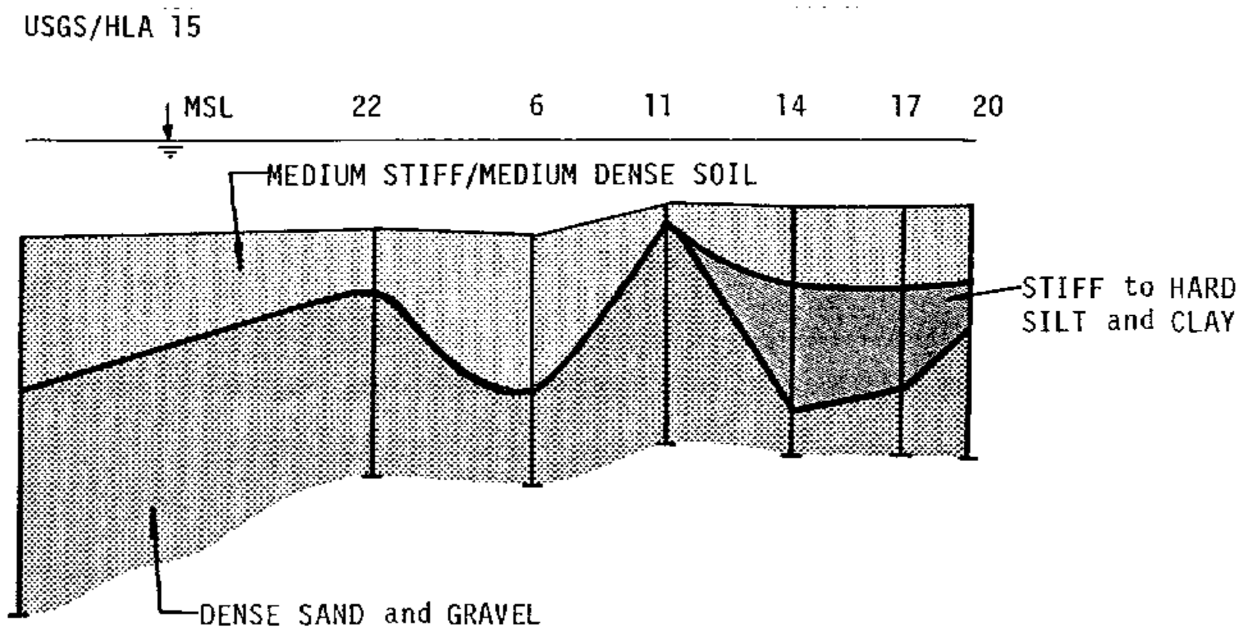
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
[illegible]	9612,031.08	[illegible]	4/82		



ONSHORE

(1) Borings denoted "T" are from Exxon Gravel Study, 1980.





LAGOON

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Generalized Soil Strata

Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-11**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JP	9612,031.08	DEB	4/82		

The load-bearing capacity and long-term settlement of foundations installed in ice-bonded soils are dependent on the moisture (ice) content of the soils. For this study we have treated soils with moisture contents greater than 50 percent as ice-rich in evaluating soil properties. In order to simplfy soil properties for the design of onshore foundations, we have grouped the soils in the onshore portion of the study area into three types based on their moisture (ice) content and particle size, as shown in Table IV-2.

TABLE IV-2. ONSHORE SOIL TYPES

Soil Type	Moisture (Ice) Content (Percent by Weight)	Classification
I	less than 50%	gravel and clean sand (GP), (GP-GM), (SP)
II	more than 50%	sand, silty sand, silt, organics, and massive ice (SP), (SM), (ML), (OL), (Pt), (ICE)
III	less than 50%	sand, silty sand, and and silt (SP), (SM), (ML)

The relationships between terrain units and the depth of soil types within the Point Thomson region are summarized in Table IV-3.

TABLE IV-3. TERRAIN UNITS AND DEPTH OF CHARACTERISTIC SOIL TYPES

		Soil Type[1] and Depth (ft)		
Terrain Unit	Map Symbol Abbreviation	Type I Coarse, Ice-poor	Type II Fine, Ice-rich	Type III Fine, Ice-poor
Low Centered Polygons	LCP	NE[2]	0-20	10-20
High Centered Polygons	HCP	5-40[4]	0-6	4-50
Reticulate Ground	RG[3]	--	--	--
Non-Patterned Ground	NPG	NE	0-15	3-40
Frost Boil Tundra	FBT	16-40	0-20	4-16
Thermokarst Terrain	TK[3]	--	--	--
Low Energy Beaches	LEB[3]	--	--	--
High Energy Beaches	HEB	0-50	NE	NE

Notes:

1. Refer to Table IV-2 for descriptions of soil type.
2. NE = Not encountered.
3. Terrain unit not drilled during boring program.
4. Test Boring T3A, Point Thomson Gravel Study, HLA #9612,008.08

Low centered polygonal ground (LCP) and non-patterned ground (NPG) landforms were generally underlain by ice-rich Type II soils to depths of at least 15 to 20 feet. Massive ice was encountered in the upper 20 feet in 6 of the 10 borings drilled in these areas (Exxon, 1980). Where encountered, ice-poor Type I and III soils were generally found approximately 10 feet below the ground surface.

Ice-rich Type II soils were found to a depth of about 6 feet in the high centered polygon (HCP) units. Massive ice was found in two of the five borings drilled in this terrain unit. Ice-poor Type I or III soils were encountered below the upper ice-rich material in all five borings. The gravel content of the Type I and III soils generally increased with depth. Ice contents of the Type I and III soils decreased with depth.

The frost boil tundra (FBT) units generally occurred in areas of mixed LCP and HCP landforms. This type of unit exhibited the greatest variability in soil type of all the terrain units mapped in the study area. Ice-rich Type II soils were found to depths ranging from 2 to 20 feet. Massive ice was encountered in the upper 20 feet in 4 of the 10 borings. Ice-poor Type I or III soils were encountered below the upper ice-rich material in all 11 borings. Type I materials were found in the upper 20 feet in 8 of the 10 borings (Exxon, 1980).

No test borings were drilled in thermokarst (TK) areas. Soil conditions in TK areas should be similar to those found in the FBT areas with the exception that higher massive ice contents are expected in TK areas.

Test Boring 3 was drilled in a high energy beach (HEB) unit on Point Gordon. Unbonded Type I materials were encountered just below the surface and were present throughout the test boring to a total depth of 38 feet. Ice-bonded Type I materials were present from 38 to 50 feet.

No test borings were drilled in low energy beaches (LEB). Unbonded Type II and III soils with high organic contents and occasional gravel seams should be expected.

b. Ground Temperatures

Ground temperatures were measured in two of the onshore borings on March 13 and April 17, 1982. Boring 7 was drilled in a high centered polygon terrain approximately two miles south of Point Hopson. Boring 13 was drilled in a terrain unit containing high and low centered polygons approximately one mile south of Point Thomson. The equipment and procedures to measure the ground temperatures are described in Appendix C, Volume II.

Temperature versus depth profiles from Borings 7 and 13 are presented on Plate IV-12. Temperature data from Test Borings 7 and 13 are virtually identical. Temperature variations between borings in the upper 30 feet are a maximum of 1.0°C. Below a depth of 30 feet the maximum variation is $\pm$0.1°C. The temperature versus depth curve has a negative slope indicating that colder temperatures exist at depths below that reached by the test borings.

The low ground temperatures measured in the upper 30 feet of Test Borings 7 and 13 are the result of cooling of the surface during the winter months. Ground temperatures below a depth of 30 feet in Test Borings 7 and 13 compare very well to data obtained in Test Boring 1 drilled in August, 1981 along the coast (on the beach) in the DID area (Exxon, 1981c). Comparison of the data indicates that the most significant seasonal variations in temperature occur within the upper 30 feet of the ground surface in both areas.

Ground temperatures in the DID area were found to be warmer inland from the coast. Based on the comparison of the Point Thomson and Duck Island data, we conclude that ground temperatures in the Point Thomson region will also be warmer inland. Warmer inland ground temperatures result from warmer air temperatures inland during the summer as indicated by the estimated thaw indices of 375°C days for coastal areas versus 550°C days for inland areas.

3. Offshore Soils

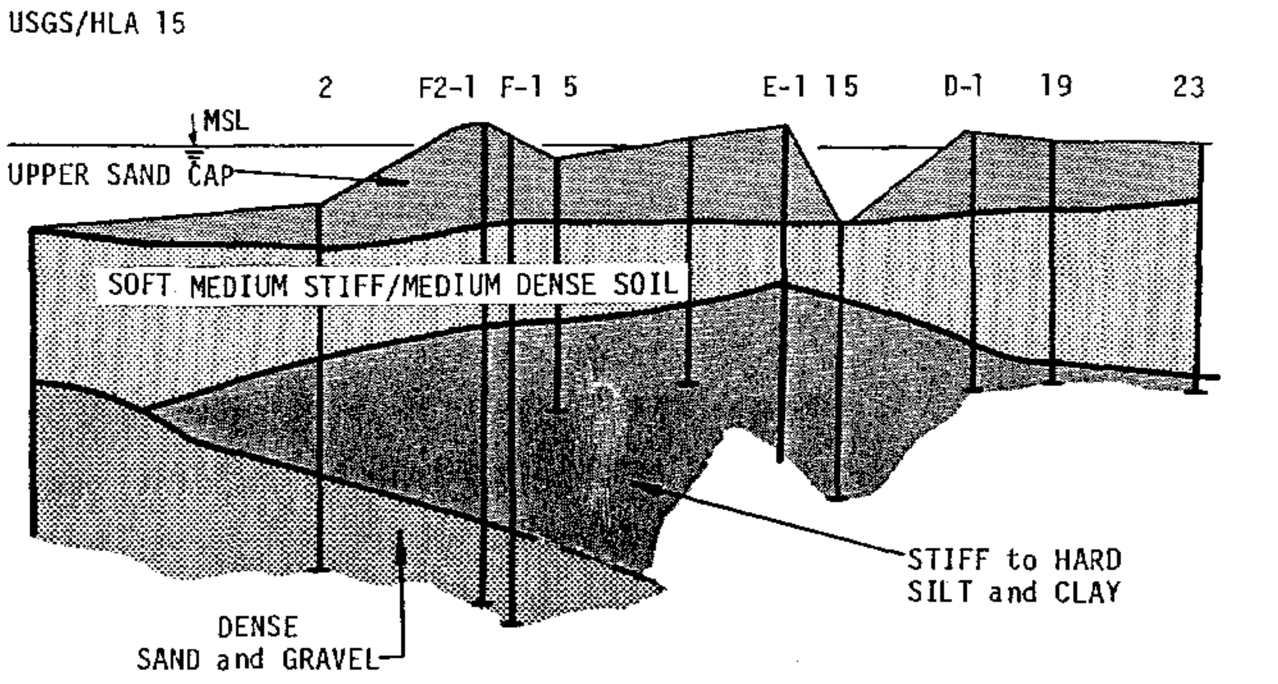
a. Soil Groups

The offshore sediments in the project area consist of a complex sequence of Holocene and Pleistocene marine units that overlay Pleistocene fluvial and glaciofluvial sands and gravels. For the purpose of engineering analysis, it is possible to group soils with similar properties into three idealized cross-sections. These three cross-sections are shown on Plates IV-11 and IV-12 for typical lagoon, barrier island, and deep water areas.

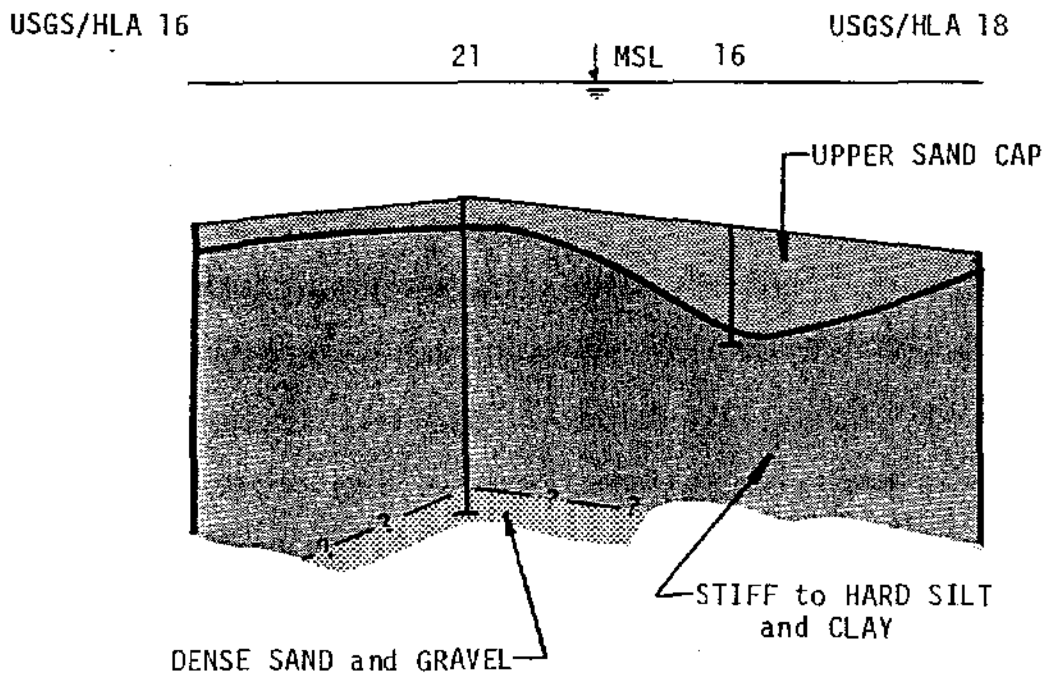
The offshore soils profiles presented on Plates IV-11 and IV-12 have been divided into four general groups based on their moisture content and dry density as shown in Table IV-4. Related geologic classifications are also shown in Table IV-4. Detailed descriptions of the geologic units were presented in Table III-3.

TABLE IV-4. OFFSHORE SOIL GROUPS

Group	Unified Soil Classification	Geologic Map Symbol	Description
1	SP - SM	QHs	medium to very dense poorly sorted fine sands
2	SM - ML	QHl, QNd, QHnm, Qf	soft to medium stiff silts and loose to medium dense sands
3	CL - ML	QPf, QPm	stiff to hard clays and silts
4	GP - SP	QPb, QPo, QPa	medium dense to dense sands and gravels



BARRIER ISLANDS



DEEP WATER

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Generalized Soil Strata
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **IV-12**

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Group 1 is most clearly depicted on Plate IV-12. This layer is the sand cap that forms the barrier islands. It is also present as a thin veneer of sand and silty sand, up to five feet thick, north of the barrier islands (HLA/USGS, 1979). Geologically, it is composed of Holocene beach and Flaxman lag deposits.

Group 2 is composed of a mixture of soft to medium stiff Holocene lagoon and lag deposits. This soil type is found in the lagoon area, up to about 30 feet below mudline and is present beneath the barrier islands. The soil types found in this layer range from silt to sand, along with sporadically occurring gravel, cobbles, and boulders.

Group 3 is a sequence of stiff to hard silts and clays which commonly overlies the dense sand and gravel deposits in the lagoon, barrier islands, and deep water areas. This soil is composed primarily of Pleistocene marine sediments that were deposited when sea levels were considerably higher than they are today. Although the stratum pinches out before it reaches the coastline, it thickens offshore and is in excess of 300 feet thick at USGS/HLA Boring 18.

Group 4 is a layer of dense sand and gravel that is composed primarily of Pleistocene alluvium and underlies the entire project area. As shown on Plates IV-11 and 12, this material comprises the majority of the onshore sediments. Although it is found close to the surface onshore, it generally dips steeply to the north offshore, and was not encountered in any of the deep water test borings. However, in the lagoon area at Test Boring 11, Pleistocene sands and gravels were encountered less than five feet below the mudline and extended to a depth of at least 50 feet.

b. Ground Temperatures

Ice-bonded permafrost was encountered in the barrier islands test borings and in Test Borings 3, 8, 15, and 16. Although no ice-bonded soil was observed in the other offshore test borings, it is probably present at greater depths than were explored.

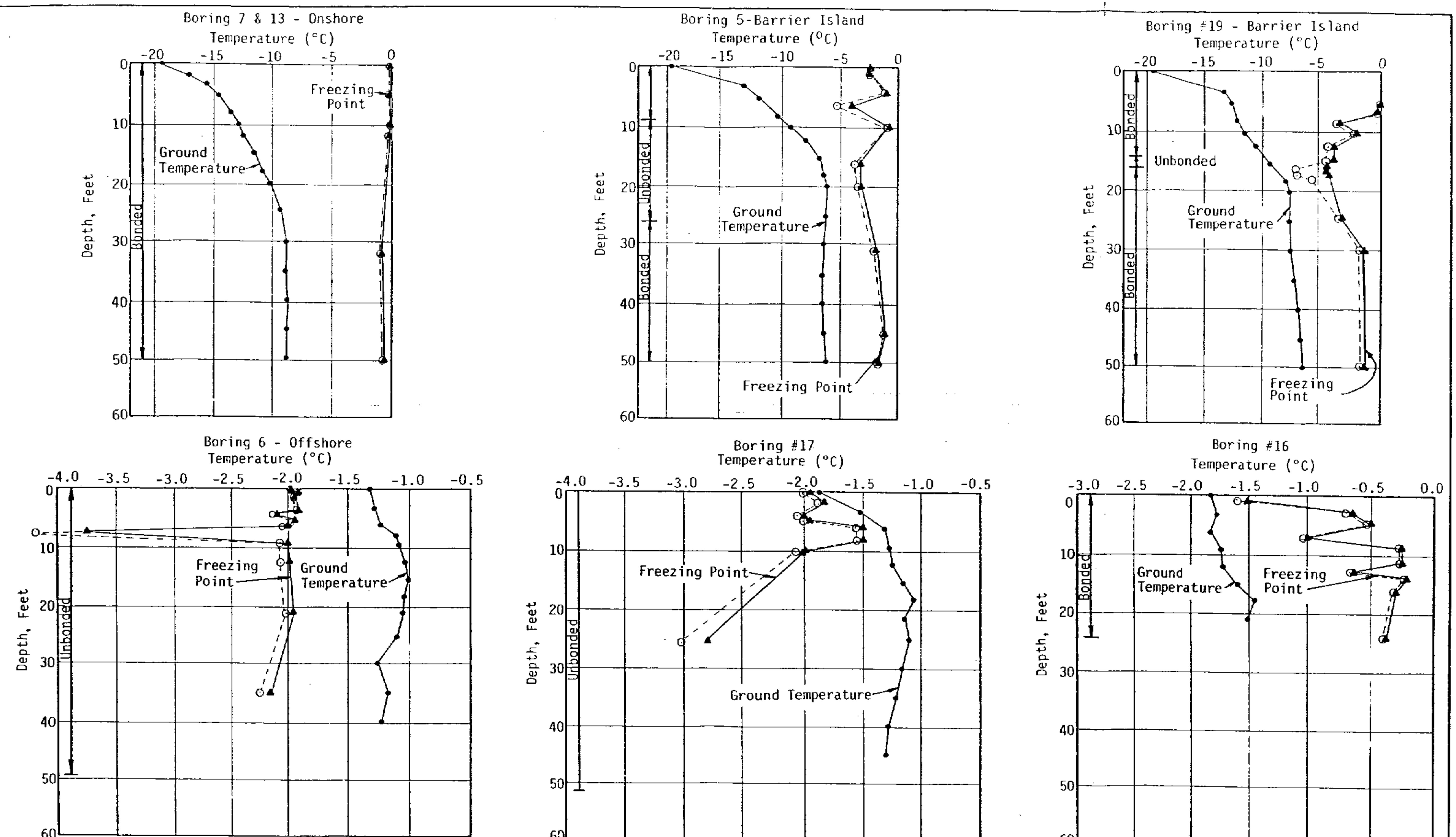
Ice bonding was confirmed by the presence of visible ice in the borings or estimated to be present on the basis of secondary information such as the temperature and salinity relationships and sampling resistance of the materials. The visible ice generally consisted of thin layers and laminations, individual crystals and ice coatings. The ice layers and laminations were predominantly 1/8 to 1/2 inch in thickness and were present in fine-grained deposits only. The laminations and layers of ice, although occasionally horizontal, frequently had an orientation that was steeply inclined or even vertical. In the sand and gravel deposits, the ice was predominantly in the form of small ice crystals and/or grain coatings and was frequently only visible with a hand glass.

The presence of ice-bonded subsea permafrost was expected since conservative estimates of coastline retreat indicate that much of the Point Thomson offshore area could have been above sea level and exposed to winter air temperatures as recently as 3000 years ago. Furthermore, some of the subsea soils have also been exposed to a colder thermal regime as the barrier islands migrate to the southwest. Therefore, since the present offshore permafrost is actually a relict onshore permafrost, variations in ice content should be similar to those observed in deeper onshore permafrost.

Ground temperatures were measured in the offshore borings, and have been correlated with the estimated freezing point based on soil salinity as shown on Plate IV-13. These data show that the offshore ground temperatures are considerably warmer than those found either onshore or on the barrier islands. By extrapolating the data, as discussed in Appendix C, the average sea floor temperature is about -0.8^{o}C, while the average ground surface temperature, both onshore and on the barrier islands, is about -9^{o}C. The average surface temperature of Test Boring 5 is slightly warmer at about -6^{o}C because it is located on the shoreline of the island. There will be large seasonal variations from these average surface temperatures.

Measured ground temperatures and calculated freezing points (FP) typically correlate well with the zone of ice-bonded permafrost. Exceptions to this relationship are observed in the unbonded, coarse-grained soil on the barrier islands. There is good correlation between the unbonded soil and high salinity in the barrier islands test borings. The FP values shown were calculated using freezing point-salinity relationships for seawater (57th Edition, Chemistry Handbook). The compound sodium cloride (NaCl) comprises 68 percent of the salts in seawater. If NaCl relationships are used, the freezing point approaches the actual measured ground temperatures in zones of high salinity. Since the pore fluids tested are composed of 90 percent Na and Cl salts, the NaCl assumed relationships might be more appropriate for determining the freezing point. The FP based on the NaCl relationship is presented on Plate IV-13 for comparison.

The surface of offshore permafrost is quite variable and complex. The area around Test Boring 16 has the greatest variation in the permafrost surface. In Test Boring 16, permafrost was encountered three feet



Note:

▲—▲ Denotes freezing point determined using sea water/salinity relationship

o---o Denotes freezing point determined using NaCl/salinity relationship

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Summary of Ground Temperature
Point Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **IV-13**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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below mudline. It does not occur until 39 and 41 feet below mudline in Test Boring 15 and HLA/USGS Boring 18, respectively. Differences in the permafrost table of as much as 40 vertical feet in a 350-foot horizontal distance have been reported in the DID area.

These large variations in the surface of the permafrost are similar in appearance to those found in marginal, discontinuous permafrost zones, such as in the interior of Alaska. Variations in vegetation, soil type, moisture, and solar exposure are the principal causes of discontinuities on land. Offshore, the fairly uniform seawater temperatures and soil conditions appear to preclude thermal factors as causes of the variations. For example, subsurface temperature measurements from other Beaufort Sea studies indicate a range of average sea floor temperatures of about $\pm 0.5^{o}C$. Additionally, thermal conductivity tests of the offshore unfrozen silts and sands also indicate only slight differences between materials.

We believe that the large vertical variations in the surface of the subsea permafrost could be due to downward permafrost decay caused by salt advection, i.e., the diffusion of salt into the sediments. Salt advection is strongly indicated as the cause of permafrost thawing along the shoreline and outside the barrier islands where the salinity ranges from 30 to 50 parts-per-thousand (ppt).

The lagoon is characterized by fine grained Holocene deposits which have never frozen overlying Pleistocene deposits which were probably frozen prior to deposition of the Holocene soils. As with the permafrost outside the barrier islands, we believe that salt advection is the cause of the Pleistocene sediments thawing with this thawing process ongoing duing deposition of the Holocene sediments.

As mentioned previously, large vertical variations in the permafrost surface could also be due to the migration of the barrier islands. As the islands move across an area, the permafrost surface is raised in that region.

B. Soil Properties

1. Unfrozen/Unbonded

The soil properties that are discussed in the following section are representative of the conditions that are encountered in the unfrozen/unbonded sediments in the lagoon, deep water, and barrier islands areas. In several cases where there is little overall difference in behavior between strata, only a single generalized parameter with a discussion of local exceptions is presented.

a. Specific Gravity

The average specific gravity that was measured on typical soil types is presented in Table IV-5. These results are within the range of values that are considered normal for each type of soil.

TABLE IV-5. SPECIFIC GRAVITY

Upper Sand Cap SP, SM	Medium Stiff Silt Medium Dense Sand SM, ML	Stiff to Hard Silt and Clay CL, ML	Dense Sand and Gravel SP, GP
2.69	2.70	2.75	2.69

b. Moisture Content and Dry Density

There is a wide variation of natural moisture contents over the depth of the test borings in the lagoon and deep water areas. Furthermore, there is often significant local variation of these values due to numerous, thin interbeds of increased sand, silt, clay or organic content. Sandy or gravelly soils typically have the lowest moisture contents and organic soils have the highest.

TABLE IV-6. AVERAGE WATER CONTENT AND DRY DENSITY IN OFFSHORE SEDIMENTS

Upper Sand Cap		Medium Stiff Sand Medium Dense Sand(1)		Stiff to Hard Silt and Clay		Dense Sand and Gravel	
w (%)	γ (pcf)	w (%)	γ (pcf)	w (%)	γ (pcf)	w (%)	γ (pcf)
15-25	90-110	25-35	70-90	15-25	90-110	5-15	115-130

(1) For unfrozen barrier islands sediments, w avg = 10-20% and γavg = 110-115pcf.

The high and low values of moisture content and dry density fall within a relatively narrow band in all of the soil types except for the medium stiff/medium dense layer. In this material, there are significant local variations in the maximum and minimum values. Additionally, the stiff Pleistocene silts and clays in the eastern portion of the lagoon have natural moisture contents approximately 5 percent lower than the average for that strata. A similar decrease in moisture content is also evident in the dense sands and gravels in this area.

c. Salinity

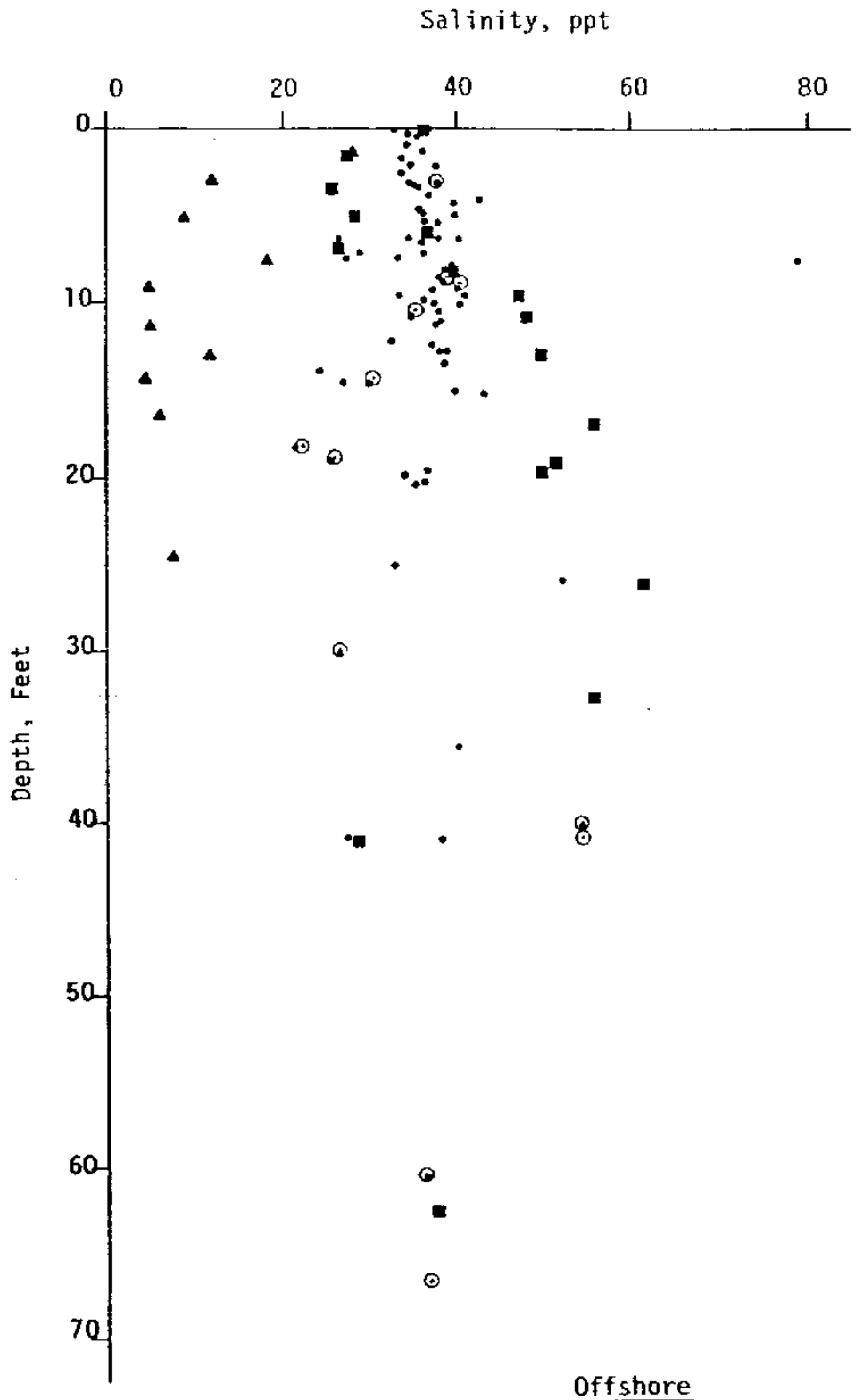
A number of salinity measurements were made throughout the area. The results of these tests are summarized on Plates IV-14 and IV-15; salinity profiles for individual test borings are presented in Appendix D on Plates D-55 through D-59. Additionally, the relationship between salinity, ground temperature and freezing point depression is discussed in Section A of this chapter.

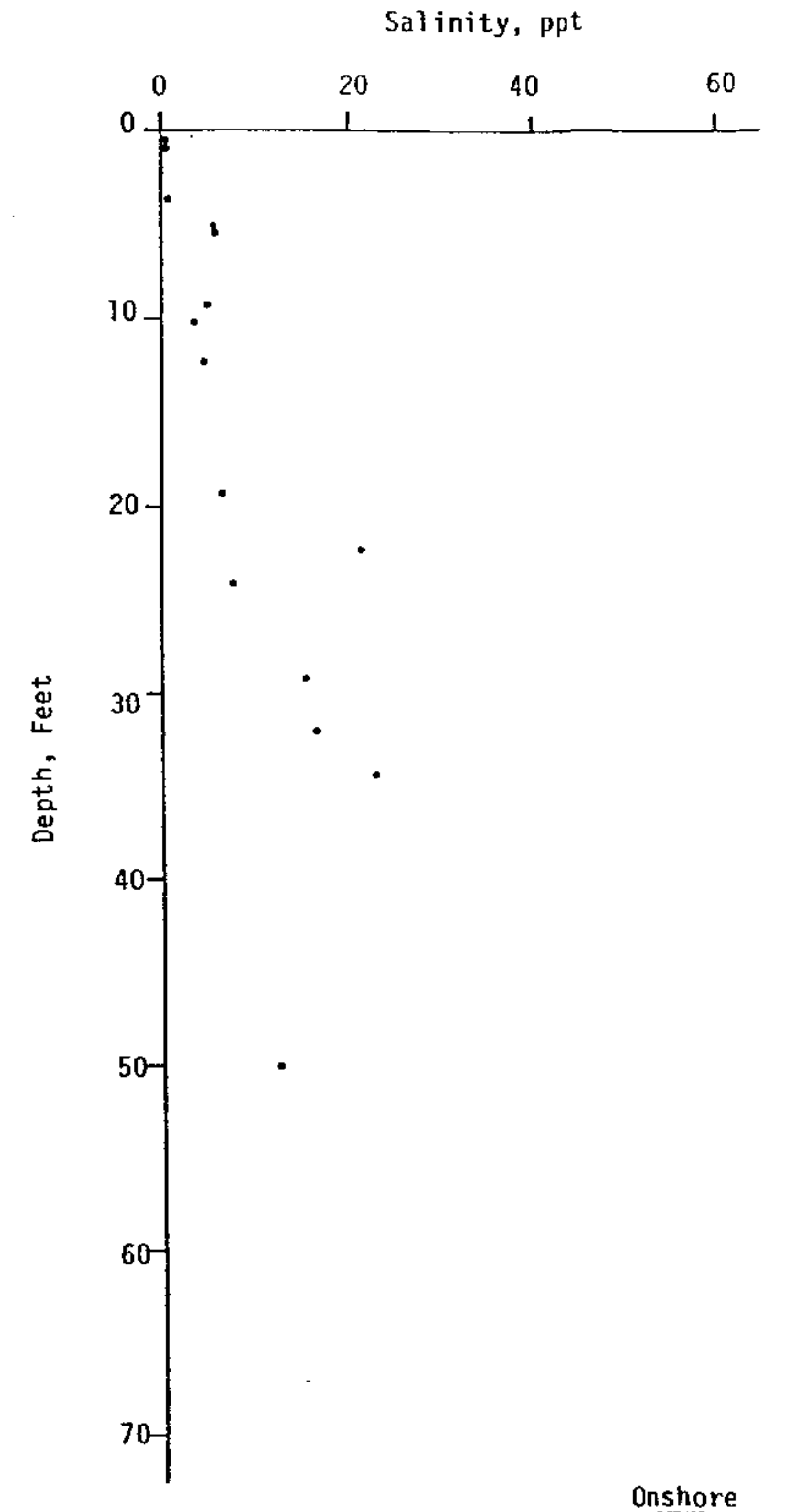
In all of the test borings, highest salinities correlate well with zones of unbonded soil. This relationship is especially evident in data obtained from the barrier islands, where salinity in the unbonded zones was greater than 100 ppt. The high salinity concentration is suspected to be a result of brine rejection as the upper soil freezes. In the lagoon area where no permafrost was encountered and in the beach test borings the salnities are generally greater than 30 ppt and tend to range between 35 ppt and 42 ppt. Onshore salinities were significantly lower than those measured in any other area and tended to increase slightly with increasing depth.

d. Organic Content

The offshore samples that were tested contained less than 16 percent organic matter by weight. The majority of the soils within this area contain only trace amounts of organic matter. Those samples with high organic contents were typically found as thin seams of material.

In general, the presence of organics tends to decrease the dry density and increase the moisture content of a soil. Also, with the exception of peat or highly fibrous sediment, Beaufort Sea organic silts typically do not exhibit either a noticeable reduction in the shear strength nor an increase in compressibility as compared to non-organic soils. This finding is





Notes:

- • Denotes data from Lagoon borings
- ⊙ Denotes data from Boring 2
- ▲ Denotes data from Boring 16
- ■ Denotes data from Boring 21

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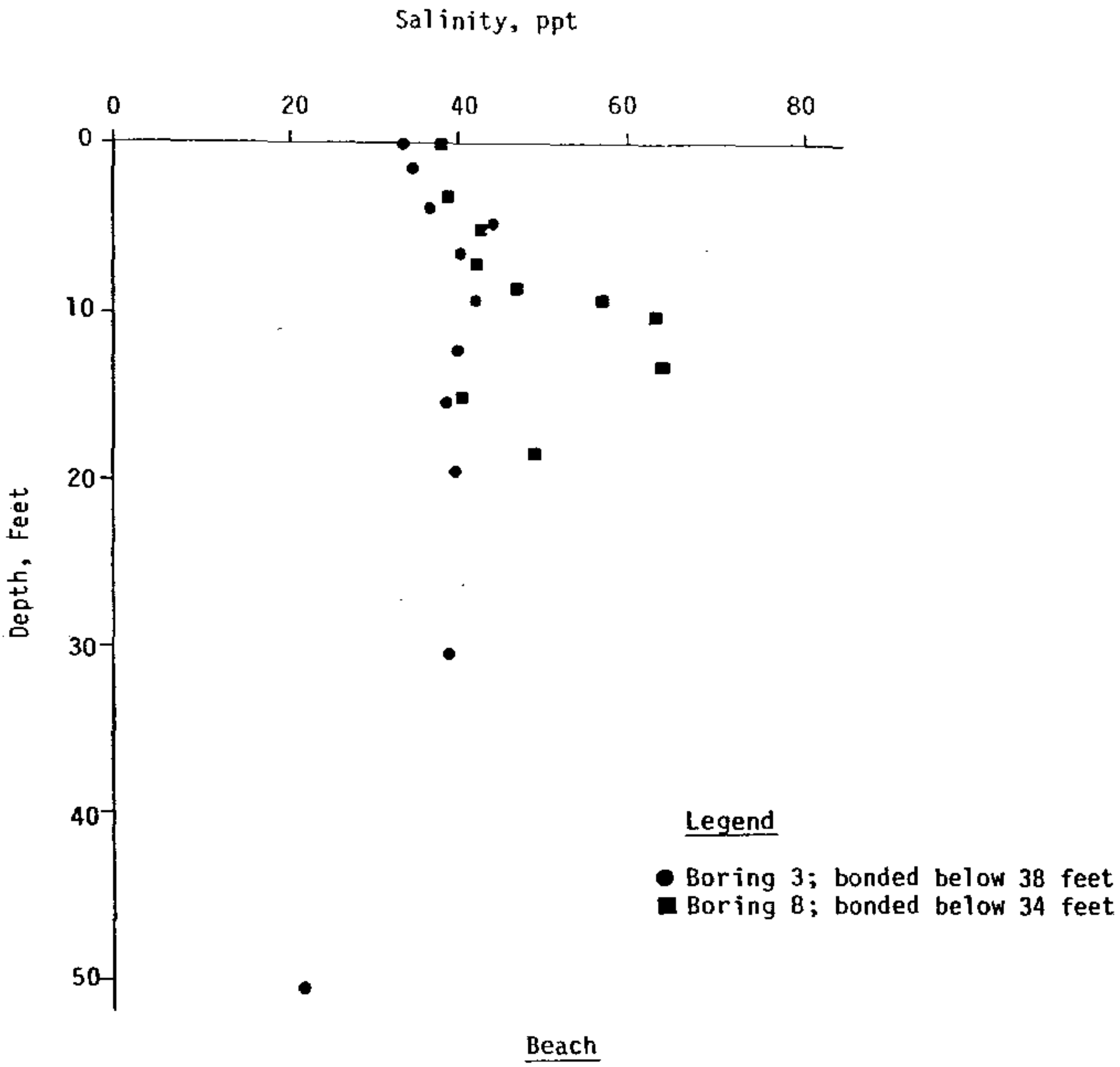

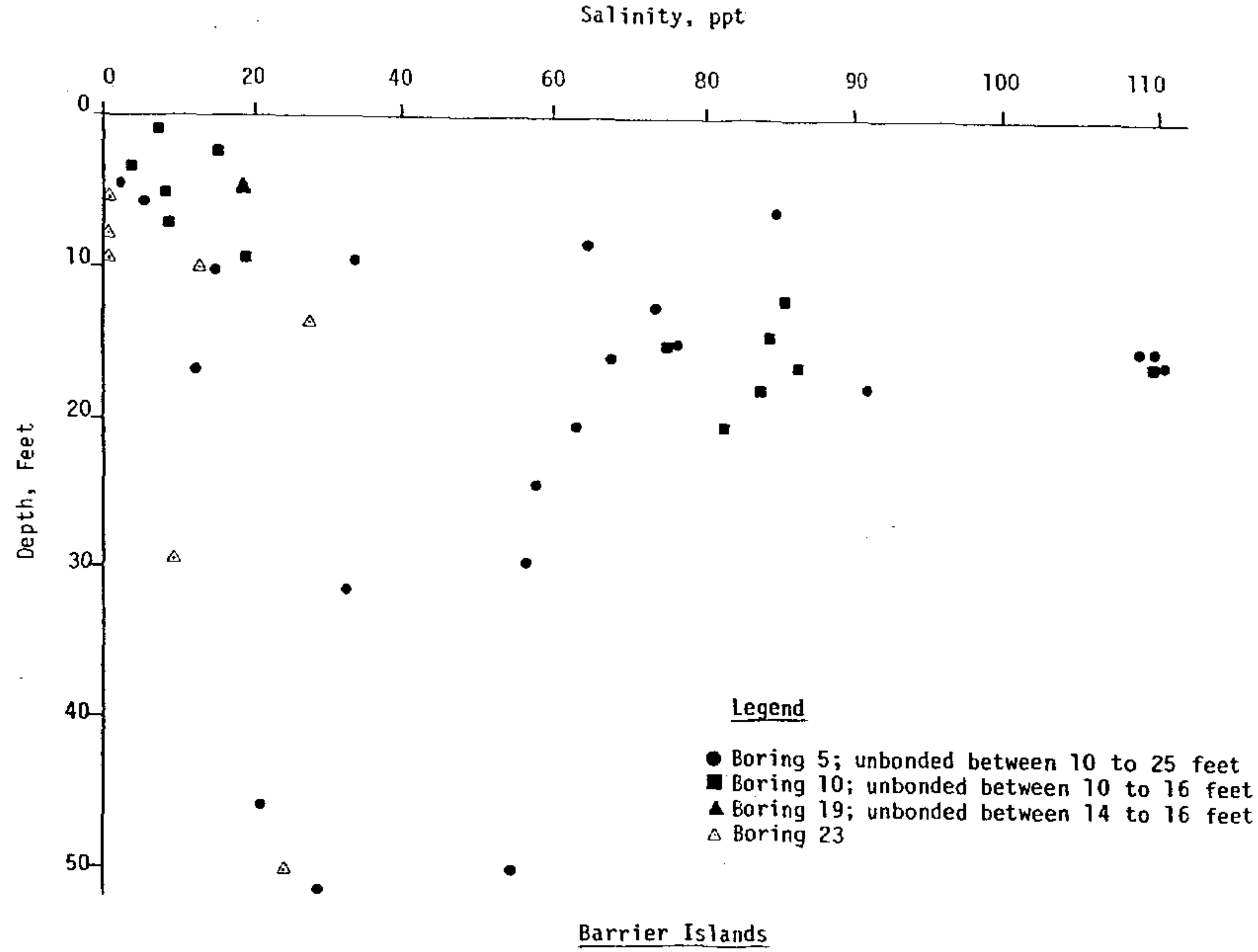

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Summary of Salinities,
Beach and Barrier Islands Borings
Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE **IV-15**

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in agreement with Franklin et al. (1973), where it was reported that when the organic content is less than about 20 percent, its effect is less important than that of minor mineralogical or structural differences. Therefore, it may be concluded that differences in the mineral constituents of the soils have a greater influence on their mechanical properties than does a small percentage of organics.

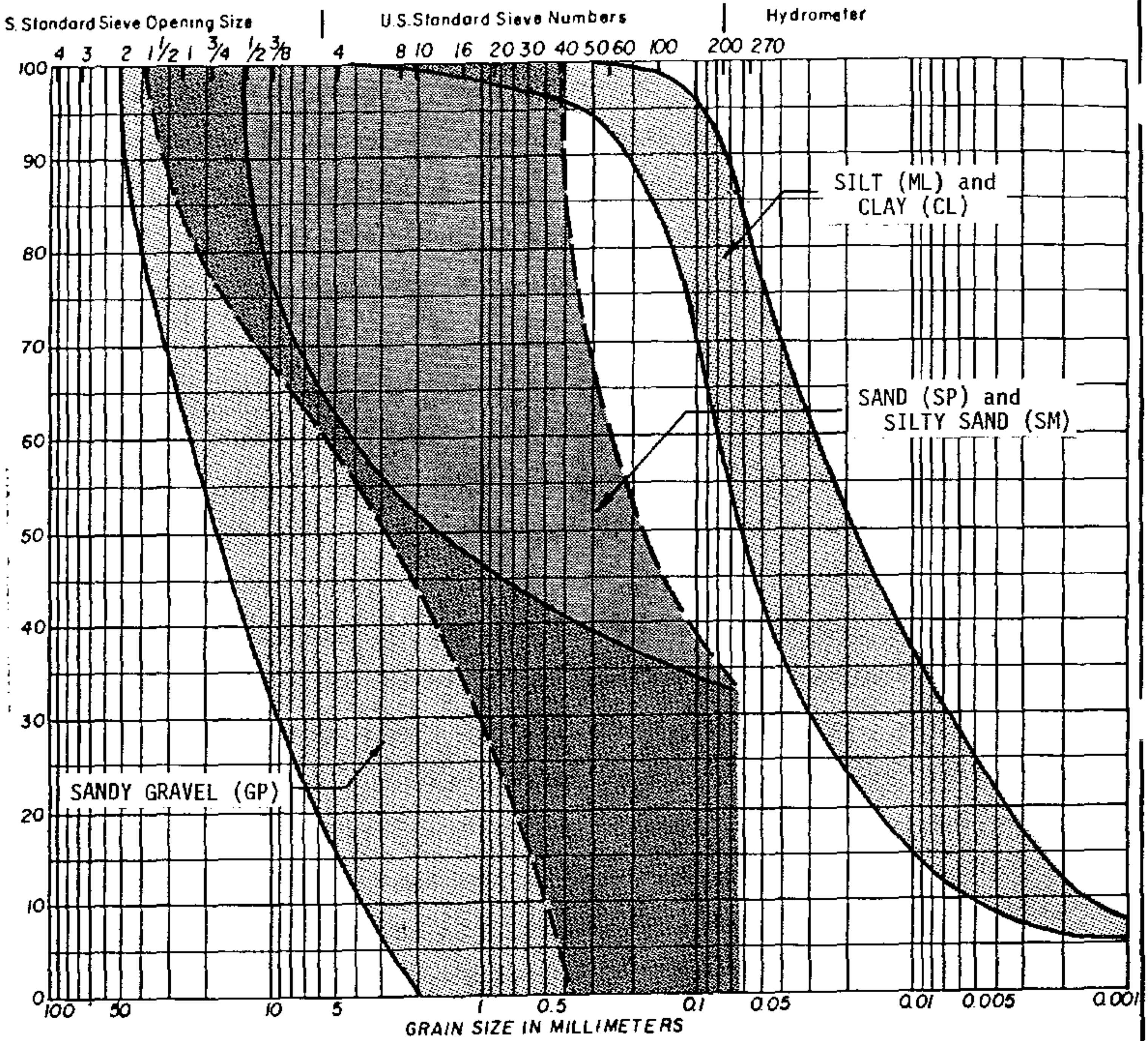
e. Particle Size Analysis

The quantitative distribution of particle sizes was determined by performing sieve and hydrometer tests on representative samples. Individual tests results are summarized on Plates D-44 through D-47 and graphically on Plate IV-16.

The gravels in the project area are generally less than 2 inches in diameter, and contain more than 15 percent sand and less than 37 percent silt. The sands that were tested have mostly coarse to medium size particles. Furthermore, these sands typically contain up to 40 percent gravel finer than 1-1/2 inches in diameter, and less than 35 percent silt. The silts and clays in the area exhibit a much narrower band of particle sizes than either the sands or gravels. All of the silts and clays that were tested contain between 6 to 40 percent fine sand particles.

f. Atterberg Limits

The Atterberg limits of the samples that were tested are summarized on Plate IV-17. The fine grained soils range from non-plastic to moderately plastic. The non-plastic specimens are sandy silts that were sampled at various depths throughout the lagoon area. The remaining samples are either silts or clays that are clustered around the A-Line. This clustering of silts and clays around the A-Line is also a common feature of the soils in this region.



BBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

ymbol	Sample Source	Classification

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Summary of Particle Size Analyses

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PLATE
IV-16

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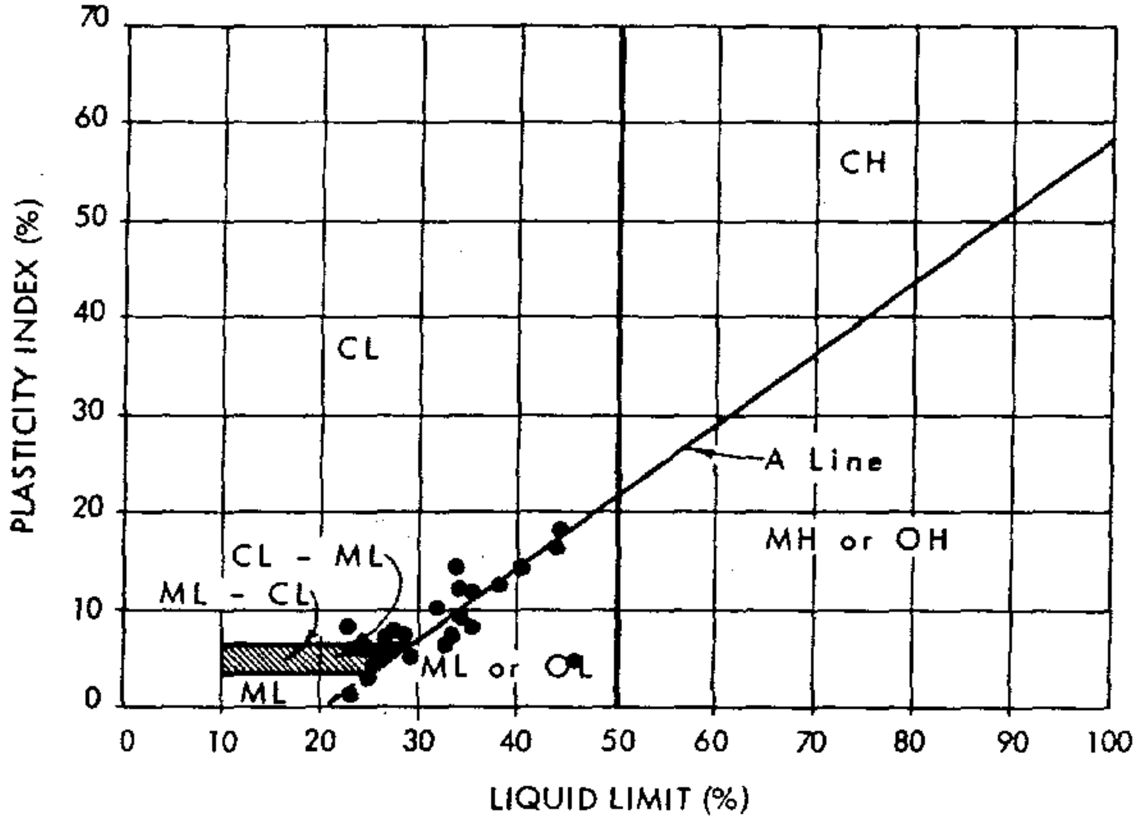

Classification	Number of Samples	Liquid Limit (%)		Plastic Limit (%)		Plasticity Index (%)	
		Average	Deviation	Average	Deviation	Average	Deviation
ML-OL	11	32	8	24	5	7	4
CL	10	33	6	22	3	11	4
CL-ML	5	25	2	20	2	6	1/2

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Summary of Plasticity Indices

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PLATE **IV-17**

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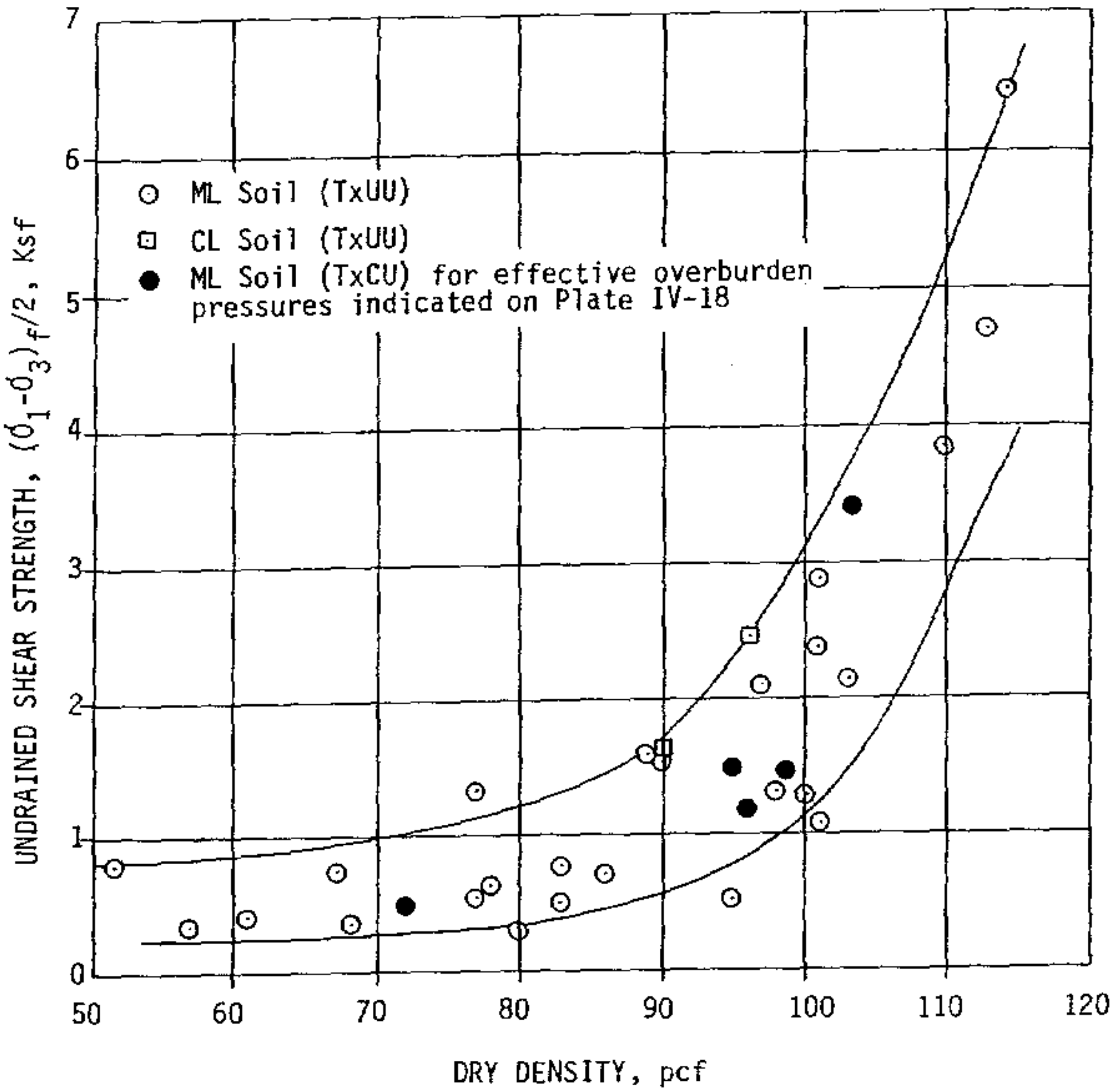
g. Shear Strength

The shear strength of unfrozen fine-grained soils is greatly dependent upon the pore water drainage conditions during loading. If drainage cannot occur, excess pore pressures develop and the undrained condition exists. This condition occurs where there has been a sudden change in the stress state of the soil, such as is common in short-term or end-of-construction situations. Conversely, when a load is applied sufficiently slowly so that the pore pressures can dissipate, drained tests can be used to evaluate the effective stresses. Pore pressures dissipate most rapidly in sand, followed by silt and clay, respectively.

Typically, the undrained shear strength of fine-grained, homogeneous soil correlates well with depth. However, during this investigation a meaningful correlation between the undrained shear strength versus depth could not be established due to the heterogeneous nature of the soil. Rather, the shear strength correlates very well with dry density, as is shown on Plate IV-18.(1) The undrained shear strength is also related to the effective consolidation pressure, σ'_{3c}. This relationship is shown on Plate IV-19. In general, the ratio of increase in shear strength to increase in σ'_{3c} is approximately 0.5 for the silts and clays. In addition, the soil is generally overconsolidated having an overconsolidation ratio (OCR) between 1 and 3.

(1) The dry density can be related to other soil properties such as the void ratio by applying standard soil mechanic fundamental definitions.



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Undrained Shear Strength vs Dry Density

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PLATE **IV-18**

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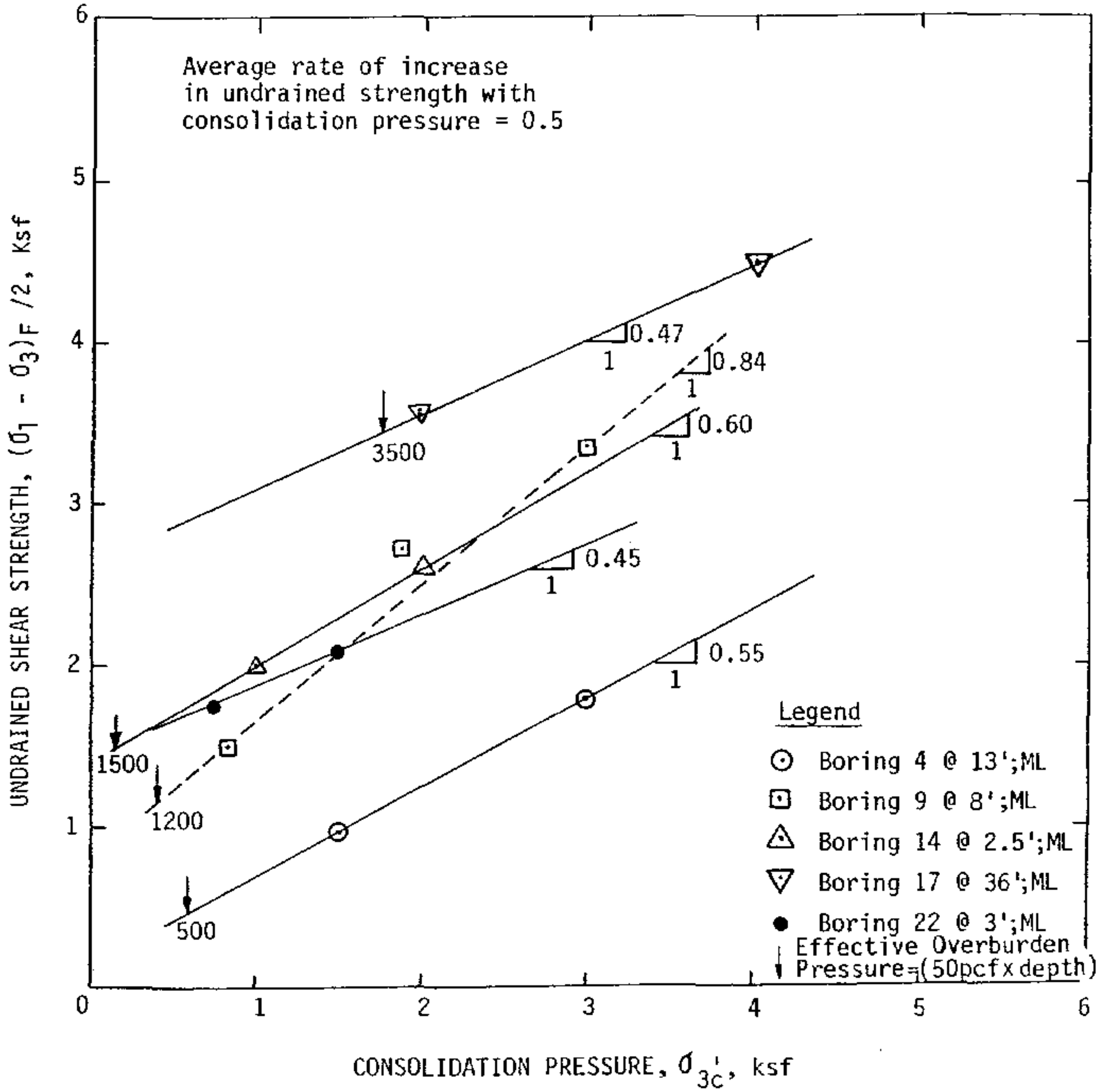

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Undrained Shear Strength vs Effective Consolidation Pressure
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PLATE **IV-19**

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Plate IV-20 shows the peak effective shear strengths that were measured for samples of sands, silts, and clays throughout the project area. Under effective stress conditions, all of these soil types behave as cohesionless soils with an effective angle of internal friction of 40°, which is consistent with the values that have been reported for similar Beaufort Sea soils. (Exxon, 1981; Exxon, 1980).

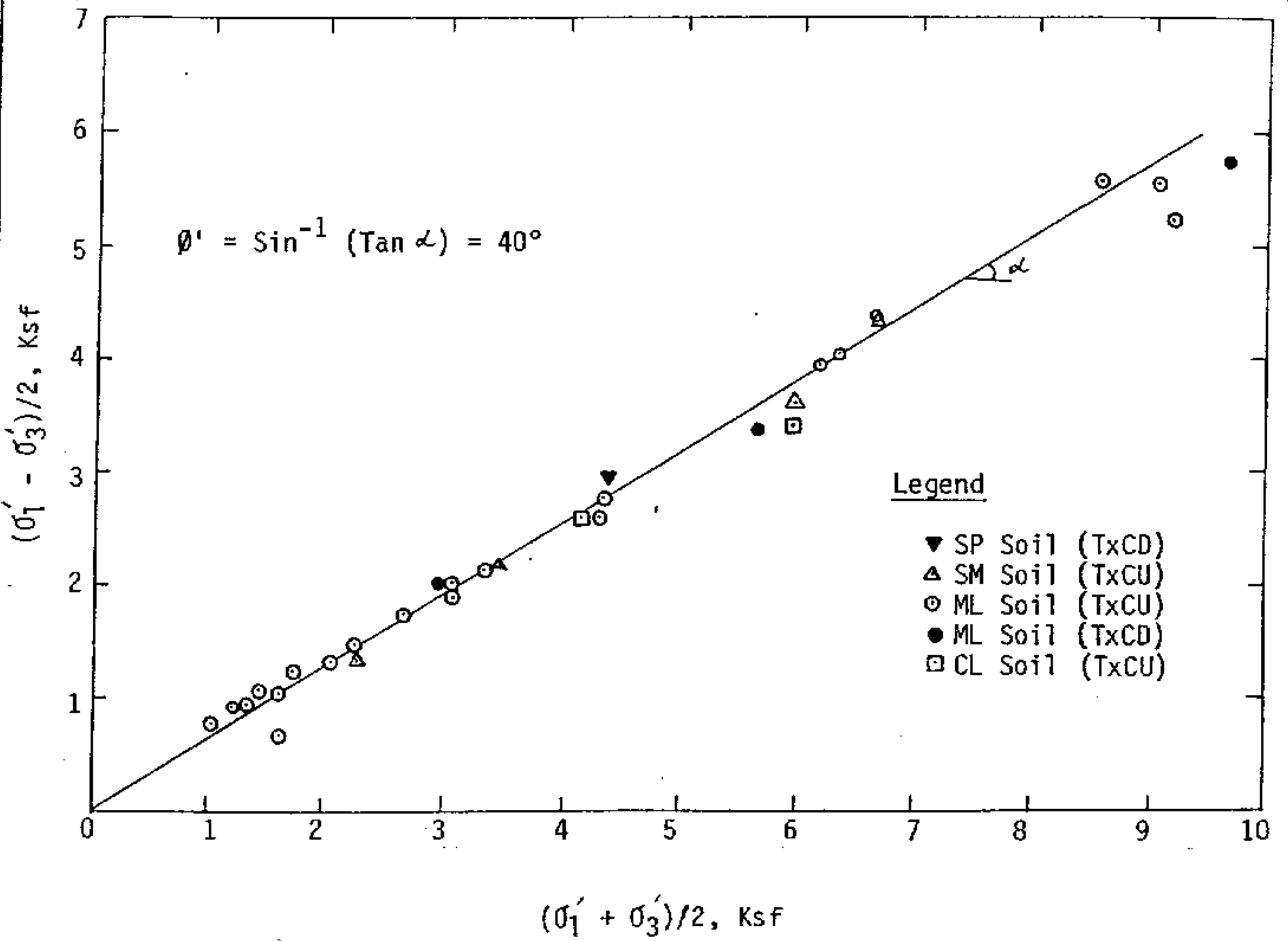
h. Compressibility

Consolidation tests were conducted on selected representative specimens. For each test, an average compression ratio was calculated, as described in Appendix D, for the range of expected design loads. Furthermore, recompression ratios were also calculated. These ratios are plotted versus dry density on Plate IV-21. From this plot, it is seen that the average compression ratio approaches the recompression ratio as dry density increases.

Plate IV-22 shows the relationship between the coefficient of consolidation and dry density for silts and clays. The two points that are typically shown for each dry density correspond to coefficients that were calculated for different consolidation pressures. In general, the soils in the Point Thomson area consolidate relatively quickly compared to similar Beaufort Sea sediments (Exxon, 1981; Exxon 1980).

2. Frozen

Ice-bonded permafrost was observed in all soil strata. The fine-grained soil contains visible ice that ranges in size from small crystals to layers of massive ice. The presence of ice has the effect of lowering the dry density of the frozen soils and increasing the distribution of moisture contents.



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Effective Shear Strength
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PLATE **IV-20**

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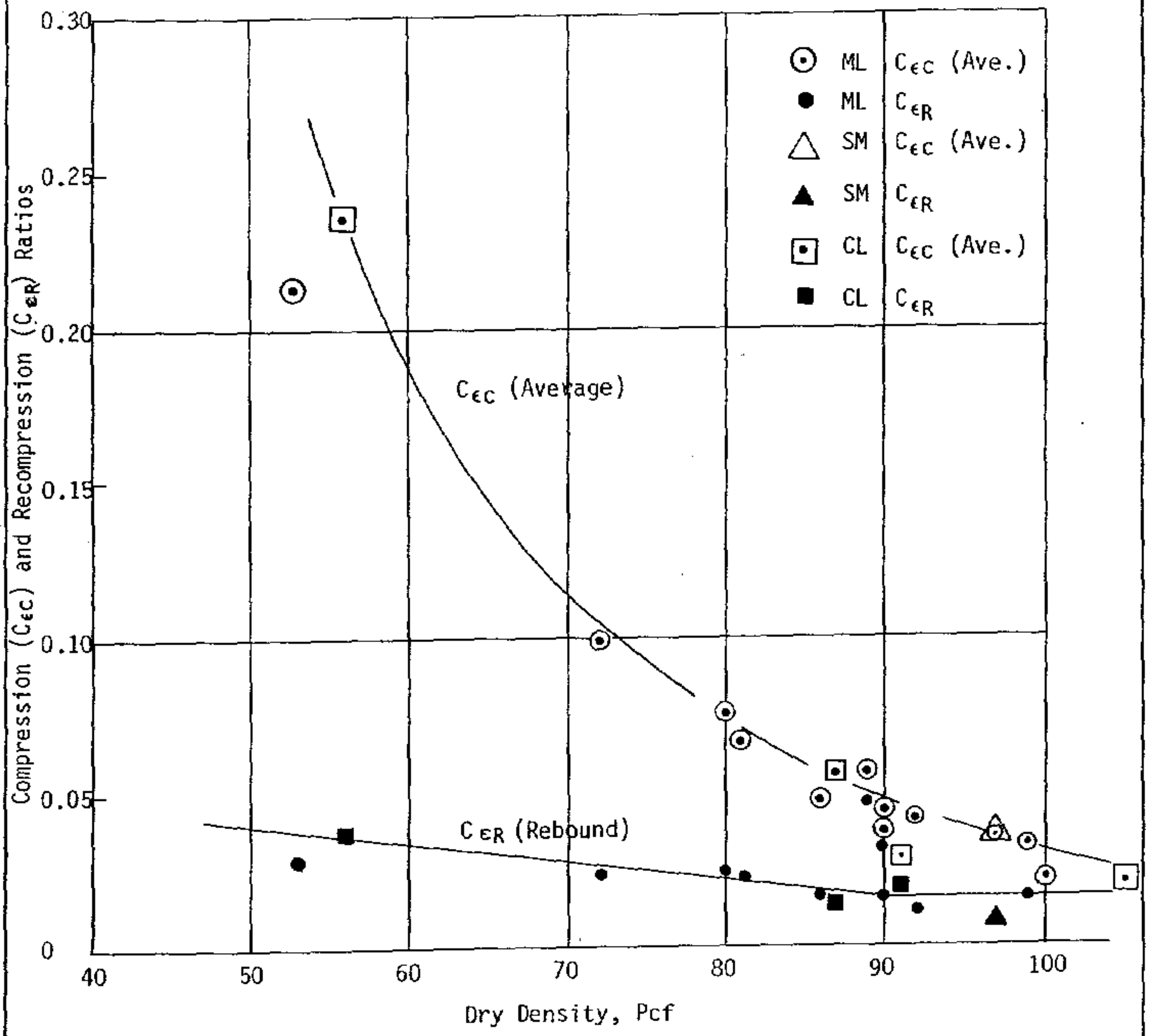

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Compression and Recompression Ratios vs Dry Density
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PLATE **IV-21**

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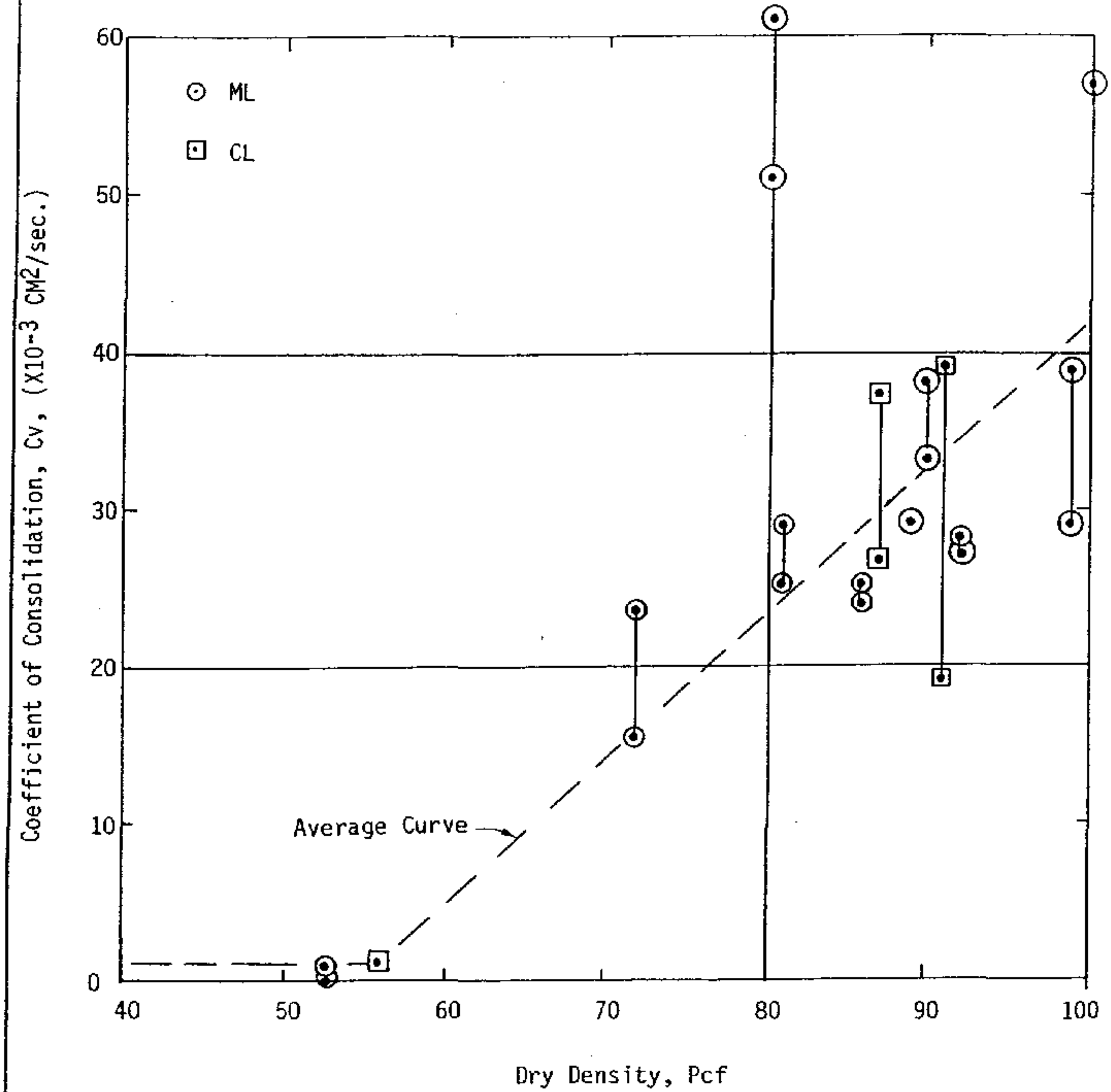

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Coefficient of Consolidation vs Dry Density

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PLATE **IV-22**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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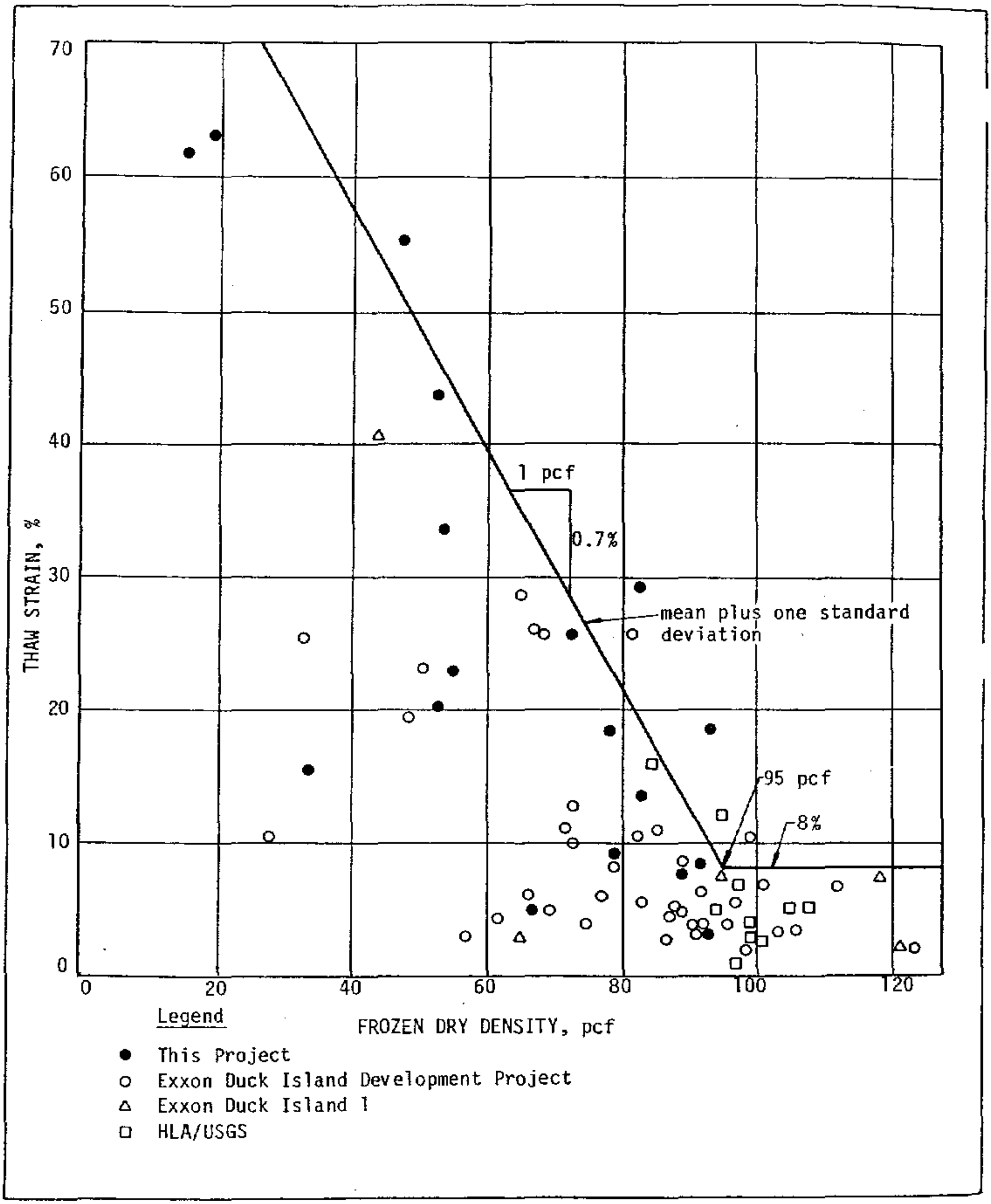
a. Thaw-Strain

Ice-bonded, fine-grained samples were tested for thaw consolidation in our laboratory. Test procedures are described and data presented in Appendix D. A summary plot of the results is presented on Plate IV-23 along with data from previous studies (Exxon, 1981a, c; HLA, 1979). The data from other studies, because of the similarity of soil types, are included to allow the development of a relationship between dry density and thaw-strain over the full range of dry densities anticipated. Despite the broad scatter on the plot, an envelope of points computed as the mean of strain values plus one standard deviation is considered a reasonable curve for design. This curve is consistent with other published data (Luscher and Afifi, 1973). Considering the average density of frozen, fine-grained soils as 85 pcf, an average thaw-strain of 15 percent should be expected. Variations in thaw-strain from 3 to 30 percent are indicated by the data. Based on experience with onshore permafrost, the variation in ice content and resulting thaw-strain can occur in the permafrost within distances as short as 10 to 50 feet.

When the frozen dry density is greater than approximately 95 pcf, the thaw-strain is generally less than 8 percent, with an average value of approximately 4.5 percent. As the frozen dry density decreases, the scatter becomes greater. The increase in thaw-strain is approximately 0.7 percent per pcf of decrease in dry density.

Suitable gravel samples were not obtained for thaw-strain tests for this study. The results of thaw-strain tests on gravel from previous studies are presented in Chapter V.



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Thaw Strain vs Frozen Dry Density

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PLATE **IV-23**

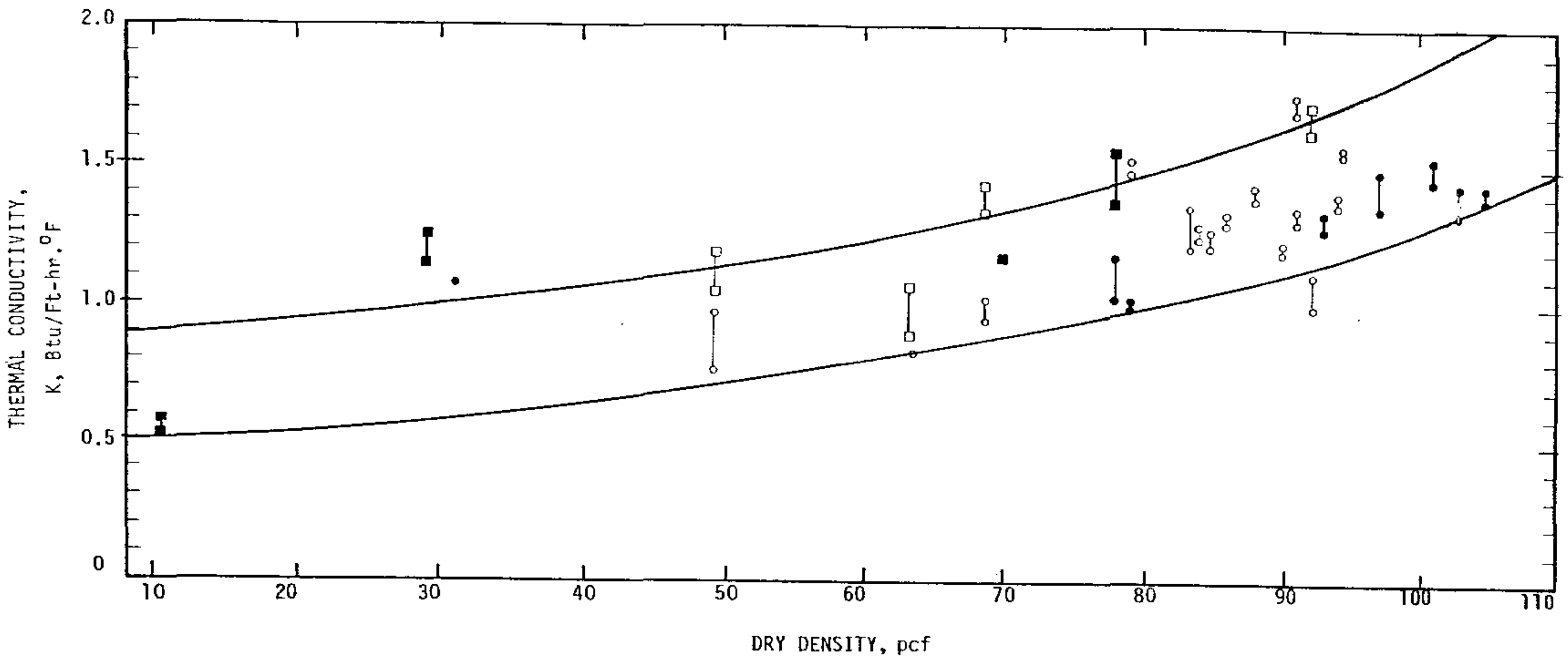
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JP	9612,031.08	DEB	4/82		

Since the offshore gravels are the same geologic unit as the onshore gravels (see Chapter III), they are expected to have similar properties. Luscher and Afifi show that gravels with dry densities in excess of 115 pcf will experience thaw-strains of two percent or less. As noted in Chapter V, information from onshore gravel studies shows that in-place ice contents are generally in the 5 to 15 percent range, with the exception of the C-1 material site, which had an average ice content of 25 percent. Moisture contents of 5 and 15 percent correspond to an in place dry density of 130 and 115 pcf, respectively. A thaw-strain value of two percent for in place sandy gravel is thus considered a conservative basis for design. For gravel material such as that encountered in the C-1 material site, a thaw-strain value of 15 percent for in place gravel is recommended for design.

b. Thermal Conductivity

The variation of thermal conductivity with dry density, as determined in our laboratory investigation is summarized on Plate D-48 and plotted on Plate IV-24 along with data from the DID study (Exxon, 1981). The data from the DID Study were included because of the similarity of soil types and to assess the development of a relationship between dry density and thermal conductivity. Furthermore, it is seen that thermal conductivity increases with increasing dry density for both frozen and thawed soil.

In general, the thermal conductivity of a soil is a function of its soil texture. At a given dry density and moisture content, the thermal conductivity of coarse-grained soils, such as sands and gravels, tends to be higher than that for fine-grained soils, such as silts and clays. This is in agreement with our test results. Samples of sandy soils tended to have higher



Legend

This Project:

●—● Thawed Test
■—■ Frozen Test

Duck Island Development Project:

○—○ Thawed Test
□—□ Frozen Test

Note: The two dots represent the two runs that were performed for each test.

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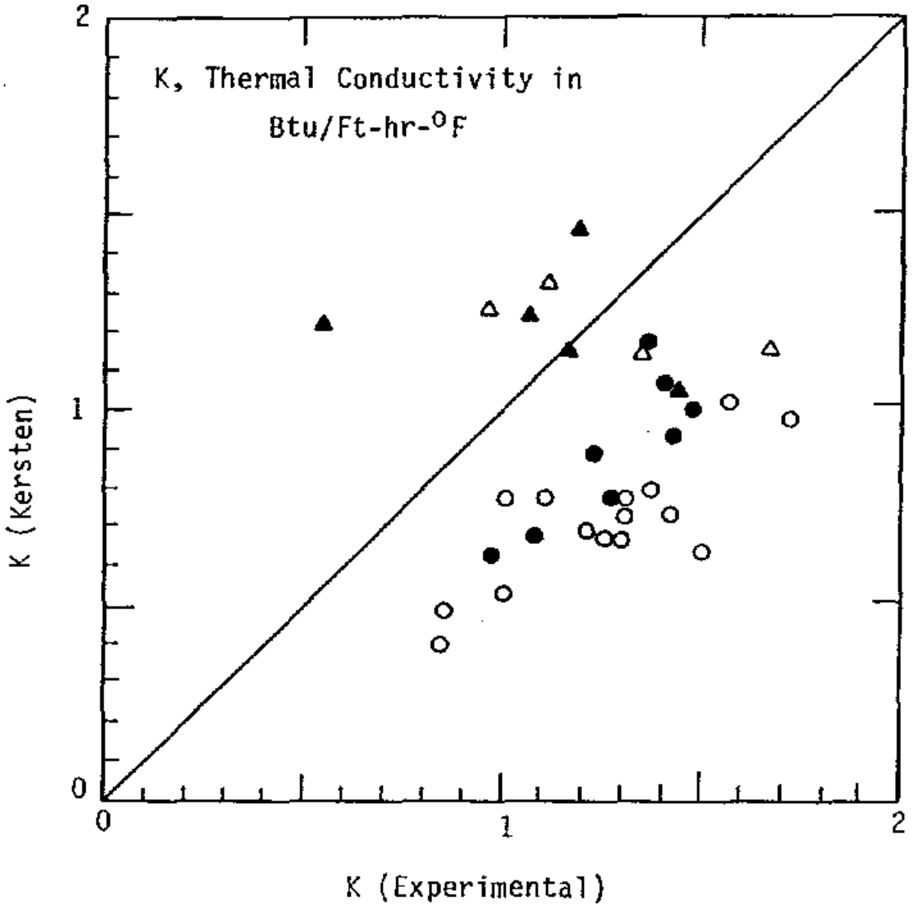
Variation of Thermal Conductivity and Dry Density
Pt. Thomson Development Project, Winter 1982, Geotechnical Study, EXXON Company, U.S.A.

PLATE IV-24

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08		4/82		



Legend

This Project:

● Thawed Samples

▲ Frozen Samples

Duck Island Development:

○ Thawed Samples

△ Frozen Samples

Harding Lawson Associates
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Comparison of Thermal Conductivity Values

Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

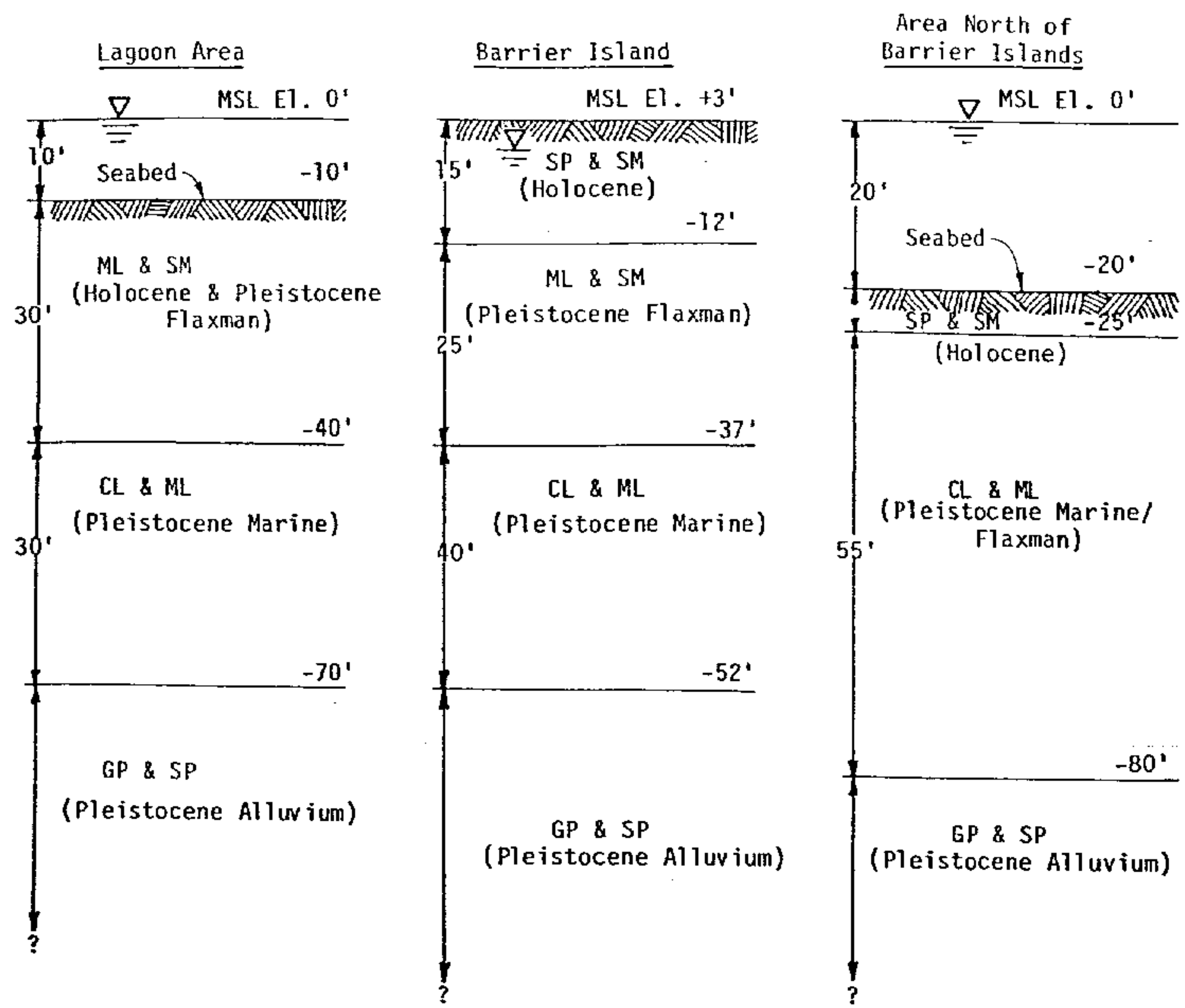
PLATE IV-25

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
DCR	9612,031.08	DEB	4/82		

thermal conductivities than those of fine-grained soils, while highly organic material had the lowest values. Without specific thermal conductivity test data, the thermal conductivity of soil is generally determined using dry density or moisture content-thermal conductivity relationships developed from Kersten's work (Kersten, 1949). On Plate IV-25, thermal conductivity data from this and the DID study are plotted against the data reported by Kersten (1949). It is seen that the thermal conductivities of thawed soils in the Point Thomson area are greater than those predicted by Kersten. These results also agree well with those presented for the DID project.

3. Recommended Design Parameters

The recommended typical geotechnical parameters for offshore soils are summarized on Plate IV-26 along with the three generalized soil profiles that were used in our design analysis. Onshore soil conditions have been defined in Section A of this chapter. Frozen soil design parameters are presented in those sections which discuss the design of structures using onshore materials. Where indicated, data from this study were supplemented with test data from other studies to develop the recommended design parameters for soil properties where tests were not conducted or where it was determined that data from other studies were suitable for this project and would allow development of more representative design parameters.



TYPICAL SOIL PROFILES AND LAYER THICKNESSES

Typical Soil Properties / Soil Type	SP & SM Soil	ML & SM Soil	CL & ML Soil	GP & SP Soil
Index Properties				
Specific Gravity	2.69(0.01)[1]	2.70(0.02)	2.75(0.03)	2.69(0.03)
Moisture Content, %	24(9.1)	32(12.3)	25(6.2)	10(8.8)
Void Ratio	0.70(0.11)	1.07[2]	0.65(0.09)	0.30***
Dry Density, pcf	100(10)	80(13)	100(6)	130***
Buoyant Unit Weight, pcf	60	50	60	80
Atterberg Limits				
Liquid Limit, %	--	31(10)	32(7)	--
Plastic Limit, %	--	23(3)	22(8)	--
Shear Strength Parameters				
S_u, Ksf	NA	$0.5+0.5\Delta\bar{P}$[3]	$2.0+0.5\Delta\bar{P}$	NA
Ø', degrees	40	40	35	40
C', Ksf	0	0	0	0
Consolidation Parameters[4]				
Average Compression Ratio	0.03	0.075	0.03	0.015
Coefficient of Consolidation (cm^2/sec)	NA	0.023	0.042	NA
Coefficient of Secondary Consolidation	NA	0.009	0.009	NA
Thermal Conductivity				
Frozen (BTU/ft-hr-°F)	1.8*	1.5**	1.4**	2.4*
Thawed (BTU/ft-hr-°F)	1.1*	1.3**	1.4**	1.8*
Thawed Settlement Parameters				
Frozen Dry Density, pcf	81(14)	93(13)	97(8)	120***
Thaw Strain, %	15(10)	13(10)	7(4)	2***

1. Parenthesis denotes standard devia
2. Maximum = 1.933; Minimum = 0.382
3. $\Delta\bar{P}$ = External load at surface expressed in pounds per square foot (psf)
4. Parameters determined using Plates IV-20 and IV-21 for thawed dry density values listed above.

* Kersten Values
** Average of data from this study and Duck Island Development Project, 1981
*** As reported in Duck Island Development Project, 1981

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Recommended Geotechnical Design Parameters
Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE IV-26

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JP.	9612,031.08		4/82		

V FILL MATERIALS

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V FILL MATERIALS

A. Introduction

The Point Thomson Development Project will be built largely of earth fill material. The number and final configuration of drill pads, islands, pipelines, causeways and roadways will determine the quantity of gravel fill required. We estimate the project will require up to 20 million cubic yards of fill material if the following structures are constructed:

1. Ten on-shore drilling pads (500 ft x 1000 ft x 5 ft)
2. One production facility pad (2000 ft x 2000 ft x 5 ft)
3. Fifty miles of road (50 ft wide, 5 ft height)
4. Two offshore causeways (50 ft wide, 10 ft above water, 10 ft water depth, length of 3 miles)
5. Five offshore islands (1500 ft diameter, 10 ft water depth, 15 ft above water)
6. Five barrier island pads (500 ft x 1000 ft x 10 ft)

Fill materials will be required for many purposes, but probably the largest requirement will be for island and causeway construction. Fill used to build islands and causeways should have strength and deformation characteristics that provide positive foundation support and safe lateral resistance to ice loads. Other uses for fill materials include roadway construction, erosion control, structural backfill, slurry backfill for adfreeze-piles, sand bag fill, and pipe backfill and protection.

The source(s) of fill material should be located so the fill can be economically excavated, transported and placed within the constraints of the arctic seasons. The winter season is almost three times longer than the summer season, but because of ice in the material, fill placed during the winter is seldom as dense, thaw stable or as strong as well-compacted fill placed during the summer season.

This chapter discusses the types of fill materials, known locations, and properties of fill after placement. Site specific investigations of onshore or offshore sources of fill materials were not within the scope of our services.

The two principle types of available fill material are (1) marginally well-graded, sandy gravel/gravelly sand with a mean grain size of about 1/4 inch (5mm) and (2) fine-grained silty sand. The sandy gravel is found both onshore and offshore; the silty sand is found offshore near the shoreline.

Until this year, Exxon's source of gravel in the Point Thomson area was the onshore Exxon C-1 Pit, located three miles from shore, southeast of Point Thomson. This material site has been converted to a water reservoir. Exxon believes that the Point Thomson area contains sufficient quantities of sandy gravel for the entire project (Exxon communication). The fill material can be transported by trucks over ice roads constructed in winter or via causeways in the summer. Alternative transportation systems for summer construction include barges or conveyors.

Gravel is present offshore within the lagoon. Outside the barrier islands, no gravel was found. The gravel is covered by 0 to 35 feet of thawed fine-grained soil. Unfrozen, thawed gravel was present offshore in Boring 11 from the sea floor to the total depth explored (50 feet). With the exception of the near shore and barrier islands borings, the gravel is not frozen.

All the offshore gravel could be mined and transported using suction dredges and barges. Thawed fine-grained silty sand is present at shallow depths beneath the sea floor near the shoreline and could be dredged and hydraulically placed.

B. Sources and Material Types

1. Onshore

a. Locations and Quantities

The onshore North Slope area contains both potential and developed borrow sources for gravel fill materials. The largest exposed sources are deposits of sand and gravel in active and abandoned channels of the Kuparuk, Putuligayuk, and Sagavanirktok Rivers. These sources are 60 to 80 miles from the Point Thomson area and are not considered to be feasible sources due to the long haul distance. Because site-specific analyses of new onshore sources of fill were not within the scope of our investigation, the remainder of this discussion is confined to potential sources of gravel in the Point Thomson area.

Exxon has been granted permits by the State of Alaska to extract gravel from an extensive area within the Point Thomson Development area. This area includes the 5-acre C-1 Pit, Exxon's primary source of onshore gravel material in the Point Thomson region. The C-1 Pit is located about three miles southeast of Point Thomson.

Gravel extraction from the C-1 Pit began in 1980. A total of about 300,000 cubic yards has been mined, producing a pit approximately 35 feet deep and covering about 5 acres (Exxon communication). The thickness of overburden averaged approximately 5 to 8 feet. The pit was converted to a potable water reservoir in 1982.

During HLAs' 1980 gravel investigation for Exxon (HLA, 1980), information pertaining to subsurface conditions was obtained for the Point Thomson onshore development area, including the C-1 Pit. Boring locations for the gravel investigation are shown in Appendix A on Plate A-1.

b. Material Descriptions

Onshore gravel deposits consist of sandy gravel with occasional gravelly sand, silty sand, and gravel interbeds. Particle size analyses of representative samples from the Putuligayuk River, Exxon Duck Island, and Point Thomson gravel pits are shown on Figure 1, Plate V-1.

In general, the materials from the Point Thomson pit are more sandy than the materials from the other two pits. The Point Thomson materials, by weight, contain 40 to 70 percent gravel, 30 to 60 percent sand, and 5 to 30 (but generally less than 5) percent silt. They are classified as poorly-graded gravel (GP) or sand (SP) according to the Unified Soil Classification System. The gravel pit samples have less than 3 percent by weight finer than 0.02 millimeters (mm), and are therefore classified as non-frost susceptible (NFS) (State of Alaska Department of Transporation and Public Facilities, 1978). Specific gravity tests performed on the gravel samples resulted in values that ranged from 2.65 to 2.70 and averaged 2.67.

Onshore gravel is generally ice saturated. Ice contents for gravel in the Prudhoe Bay, Duck Island and Point Thomson areas typically range from 5 to 15 percent. However, the ice content for gravel fill sampled from the Point Thomson Pit during the construction of drill pads in 1981 varied from 18 to 37 percent with an average of 25 percent. (HLA, 1981b).

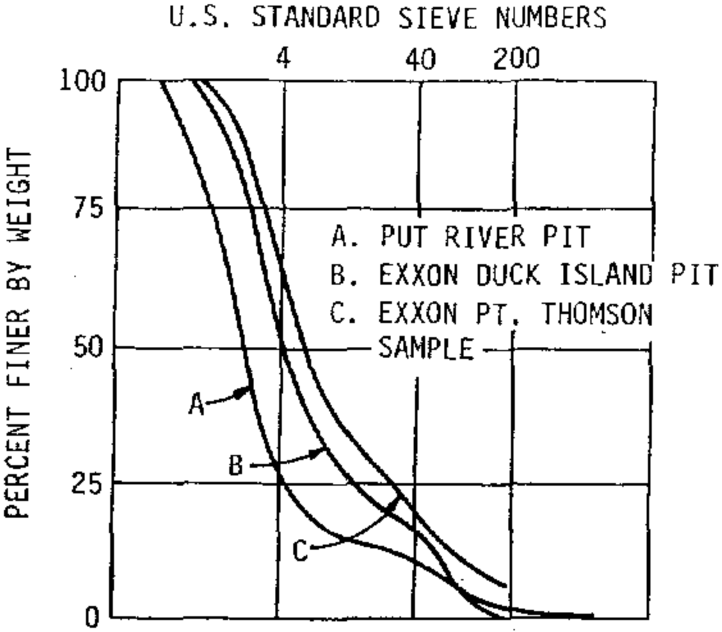

Figure 1 PARTICLE SIZE ANALYSES OF REPRESENTATIVE GRAVEL SAMPLES NORTH SLOPE

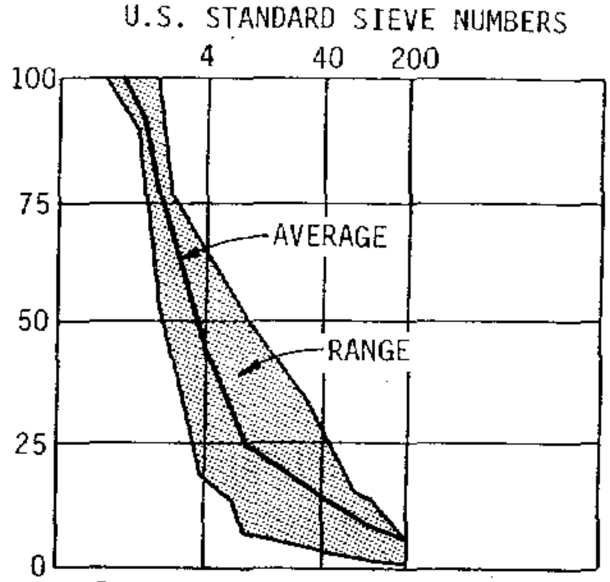

Figure 2 PT. THOMSON OFFSHORE GRAVEL GRADATION

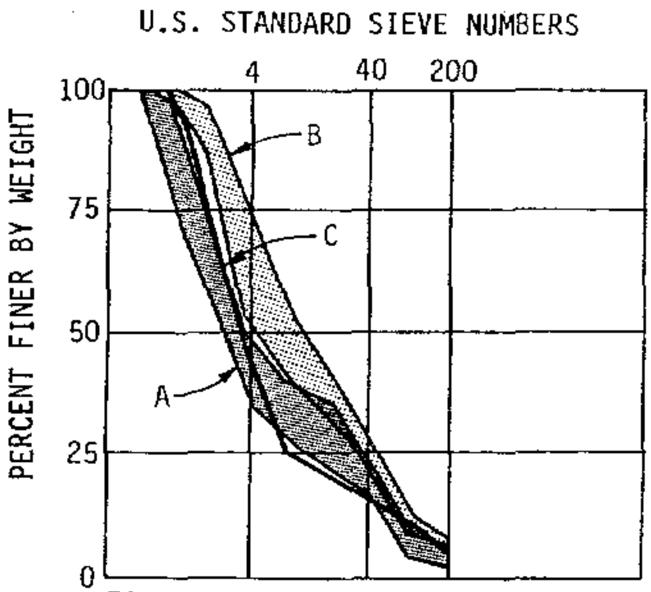

Figure 3 PARTICLE SIZE ANALYSIS OF REPRESENTATIVE GRAVEL SAMPLES POINT THOMSON AREA

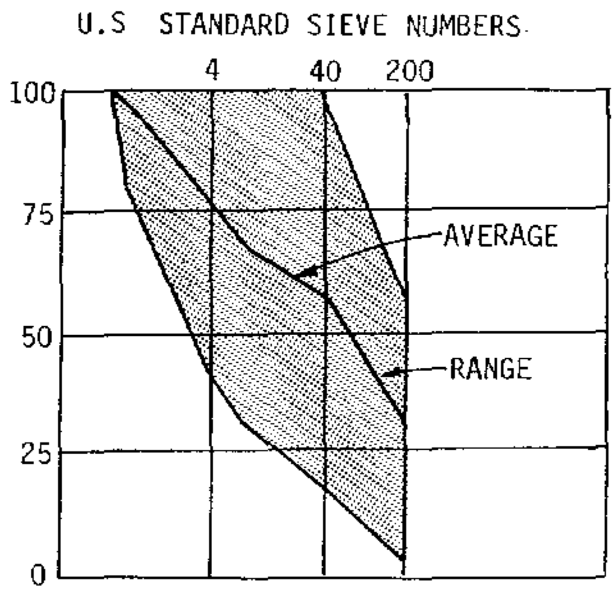

Figure 4 PT. THOMSON OFFSHORE, NEAR SURFACE SILTY SAND/GRAVEL GRADATION

Legend

A. Pt. Thomson C-1 Material Site, 1980
B. Pt. Thomson Gravel Study, 1980
C. Gravel - Offshore Borings 6, 11, 14 & 2, 1982

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Particle Size Analysis of Fill Materials

Pt. Thomson Development Project, Winter 1982,
Geotechnical Study, EXXON Company, U.S.A.

PLATE
V-1

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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2. Offshore

a. Locations and Quantities

The offshore portion of the study area contains potential borrow sources for both silty sand and sandy gravel fill materials. The sources of finer materials are the Holocene nearshore marine and deltaic deposits that blanket the offshore area. The sources of coarser materials are the Pleistocene outwash and alluvial deposits that underlie the finer-grained offshore Holocene and Pleistocene soils. Plate III-6 shows our interpretation of the depth to gravel in the lagoon contoured from borehole data. Clay soils are also found offshore, but are not considered suitable for use as fill material.

The Holocene materials consist of sands and silts with occasional thin layers of clay and organics in the lower portions of the unit. In terms of gradation, the near shore upper silty sands are a feasible source for offshore borrow materials. Further offshore, such as the center of the lagoon, the sandy fine-grained soils contain increasing quantities of clay and organics making them less suitable as fill sources.

Pleistocene sands and gravels were encountered in all the offshore borings drilled between the shoreline and barrier islands at varying depths. In Boring 11, gravel was present from the surface to the total depth explored, 50 feet. In general, the depth to gravel is highly variable but deepest outside the barrier islands and shallowest inside the lagoon, in particular around Boring 11 and in the east end of the project area. The gravel encountered in the lagoon was not frozen.

b. Material Descriptions

(1) Gravels

The onshore and offshore gravels are from the Pleistocene formation. These deposits consist of sandy gravel, gravelly sand with occasional silty sand, and gravel interbeds. Figure 2, Plate V-1 shows the particle size range of the gravel materials obtained from our offshore borings. Comparison with onshore gravel gradations, Figure 3, Plate V-1, shows that the grain size distributions of offshore gravels are similar to those of gravels from onshore material sources. For comparison, the ranges of both onshore and offshore material types are summarized in Table V-1. Gravel from the C-1 material site and offshore have similar gradation characteristics and are coarser grained than the overall onshore gravels. Gravel materials were classified as GP and sands as SP and are considered non-frost susceptible.

TABLE V-1. FILL MATERIAL GRADATIONS

Material Source	Material Types (percent) Gravel	Sand	Fines
C-1 Material Site[1]	50 to 63 average = 59	35 to 48 average = 38	2 to 5 average = 3
General Onshore Area[1]	24 to 48 average = 40	44 to 69 average = 55	5 to 8 average = 5
Offshore[2]	34 to 82 average = 55	17 to 65 average = 40	1 to 30 average = 5

1 HLA, 1980 Exxon Gravel Study
2 This Study

Both frozen and unfrozen gravel materials were encountered offshore. Unfrozen gravel soil conditions were encountered in all the borings drilled in the lagoon area. Frozen gravel was encountered only in those borings drilled near the shoreline or on barrier islands. Unfrozen gravel deposits were relatively dense. Although in situ dry densities for offshore gravels have not been determined, moisture contents indicate the densities are similar to the onshore values of 115 to 130 pcf.

(2) Silty Sands

The fine-grained Holocene deposits in the offshore areas consist of silts and fine sands in varying percentages. These materials are classified as SM and SP-SM in accordance with the Unified Soil Classification System; their frost classifications are F2, F3 and F4, i.e., moderately to highly frost susceptible. As shown on Figure 4, Plate V-1, the silt contents range up to 50 percent but average about 25 percent. Specific gravity values for the silts and sands range from 2.63 to 2.73, with an average of 2.67. The deposits contain small seams of organic materials.

In the two borings drilled nearshore (Boring 3 and 8), the silts and sands were unfrozen. The deposits were loose to medium-dense, and their in situ dry densities ranged from 78 to 121 pcf, with an average of 105 pcf. Moisture contents of the sands and silts averaged 10 and 25 percent, respectively.

C. Excavation and Transport

1. Onshore Gravels

Onshore gravels, due to their relatively high moisture content and ice-bonded nature, are commonly extracted by blasting and ripping with heavy equipment. Past experience has indicated that powder ratios in the range of

0.5 to 1.5 (pounds of explosive per cubic yards excavated) are generally required. The powder ratio employed depends on the moisture (ice) and silt contents of the material and the degree of breakage desired. In ice-cemented gravels, lower powder ratios generally produce larger quantities of cobble- and boulder-sized lumps. These lumps are undesirable and must be removed or crushed. Crushing is commonly performed before loading by making repeated passes over the material with tractors.

Gravel can be excavated in the summer without blasting, if the gravel is exposed to the air and allowed to thaw. The thawed material has to be frequently scraped into stockpiles to expose new frozen surfaces to the air. For normal summer air temperatures of about $5^{O}C$, a surface layer 4 to 6 inches thick will thaw each day. Therefore for each acre of working pit, 500 to 800 cubic yards of fill material could be excavated in a day.

The stockpiled gravels are generally wet. Some water drains from the stockpiles but a sufficient amount remains so that when the material refreezes in the winter the stockpile is usually a hard, frozen mass. It is possible to dry the thawed gravels naturally during the summer months and reduce the ice content by frequent blading and turning of the stockpile. Past experience has indicated that the low production rates achieved are not compatible with the short summertime construction season.

Gravel from onshore pits is commonly transported by large dump trucks. During winter months, construction of ice roads permits truck transport to offshore sites. Boulder-sized lumps of ice-cemented gravel can cause unloading problems, particularly when belly-dump trucks are used. In summer months, trucks are used on gravel roads and, if docks or conveyor systems are available for loading, barges are used offshore.

The amount of fill settlement resulting from thaw consolidation and compaction is directly proportional to the placement ice content of the borrow material. Although little can be done during winter season construction to alter the natural ice content of the borrow material, care should be taken during mining and transport to minimize the amount of ice and snow in the borrow material.

2. Offshore Gravels

The depth of fine-grained soil cover varies randomly within the lagoon, as shown on Profile CC' (Plate IV-4). The fine-grained soil is 12 feet thick at Boring 22, 35 feet thick at Boring 6, non-existent at Boring 11 and, in the eastern half of the site, 45, 37, and 24 feet thick at Borings 14, 17, and 20, respectively. It should be pointed out that a near-surface layer of gravel was observed in Borings 14, 17, and 20. This gravel deposit overlays stiff Pleistocene clays which, in turn, overlay the deep gravel layer.

In the western half of the site, gravel is buried beneath 12 to 55 feet of fine-grained material and was thawed to the depths explored (75 feet). Use of these gravels could involve disposal of large quantities of fine-grained materials if the fine-grained materials are not used for fill. To our knowledge, dredging of Alaskan Beaufort Sea gravels has never been accomplished.

Depending on the distance from the dredge source to construction sites, it is expected that gravels dredged will be transported through submerged or floating hydraulic lines or by barges to the construction sites. A substantial spread of dredging equipment, tugs, and barges will be needed to excavate and transport the required quantities of gravel during the short open-water season.

3. Offshore Silty Sands

Sandy nearshore sea floor deposits could be dredged from areas near the construction sites and pumped directly to the sites during the open-water season. Shoreline stability and erosion control are two environmental factors which should be evaluated prior to design of operations. Permit requirements for the shoreline region should be examined in order to evaluate the practicality of this scheme.

D. Placement and Compaction

1. Gravel

There are a number of placement and compaction combinations to be considered for gravel fill materials:

a. Below-Water Placement

(1) Winter, gravel-ice mixture

(2) Winter, drained gravel

(3) Summer, thawed gravel

b. Above-Water Placement

(1) Winter, gravel-ice mixture

(2) Winter, drained gravel

(3) Summer, thawed gravel

Category a: Below-Water Placement. This category covers all the fill placed from the sea floor up to sea level.

Case a(1): Winter, Gravel-Ice Mixture. The practice in winter has been to build ice roads from shore over the ice sheet and to dump the fill material through a hole cut in the ice. The cold gravel-ice mixtures from onshore borrow pits have been observed to bond in chunks in the near-freezing seawater and form a honeycomb-like structure; however, this may be a temporary condition. There is no practical method of compacting fill placed in this manner. Without introducing heat into the fill, we would not expect the fill to thaw except along the edges exposed to open water during the summer. Experience at the West Dock causeway has shown that a freeze front forms in the gravel, which in all probability will progress into the underlying sea floor soils.

Winter diving observations have shown there is no perceptible current in the seawater beneath the winter ice cover. We would expect little or no deposition of unbonded fine sands and silts from the gravel mixture outside the fill during winter gravel fill placement.

Case a(2): Winter, Drained Gravel. There has been no large-scale experience with excavating and stockpiling onshore gravel in such a manner as to form a truly drained, ice-free material that could be hauled over winter ice roads and placed in the sea. Large, heaped stockpiles drain slowly except on the surface. It may be possible to produce large quantities of drained gravel if a stockpile were built up in small lifts over a large area with continual in-place turning and processing by graders and scarifiers. One difficulty with this method is the overcast weather common in the summer; fog and drizzle slow the evaporative drying process. Determining the practicality of such a scheme would require (1) experimenting with the process over a large area and (2) assuring that tundra permits could be obtained to cover the required area. We estimate the stockpile should not be more than four to six feet deep for such a process to work. If this method were used, a stockpile of two million cubic yards of fill would require an area of 250 acres. Drained gravel should produce a higher density fill than gravel-ice mixtures, but a test program would be necessary to determine the actual densities attainable.

Case a(3): Summer, Thawed Gravel. Two sources of thawed gravel for summer placement are offshore dredge sites and onshore pits. Offshore dredged material probably would be pumped or carried to the structure sites by barge and then dumped directly into the water. According to Whitman (1970), the resulting fill would have a relative density of about 50 percent. While dynamic compaction (repeated dropping of a massive weight lifted by a crane) might improve the below-water densities, it is questionable whether there would be enough improvement to justify the time and expense required.

Alternatively, the below-water densities could be improved using techniques such as displacement piles or vibroflotation. These techniques are probably only practical in deep water, i.e., greater than 25 feet. Improvement methods should be used before the fill freezes; consequently, placement and compaction should be performed during the same summer season.

Down-current sedimentation should be minimal during summer placement of gravel fill. Down-current sedimentation distances for medium and fine sand and medium silt particle sizes in summer offshore currents average 0.5 feet per second (Corps of Engineers, 1980) and are presented in Table V-2. Water turbulence caused by waves will tend to increase the sedimentation distances under actual construction conditions. The calculated distances indicate that the gravel and coarse to medium sand fraction, about 95 percent by weight of the fill material, will settle in the immediate dump area; only the silt fraction will settle outside the dump area.

TABLE V-2. DOWN-CURRENT SEDIMENTATION

Particle Size	Settling Velocity (in./sec.)	Distance of Down-current Sedimentation (ft.) 5' water	10' water	20' water
Medium Sand (0.50mm)	2.8	10	20	40
Fine Sand (0.07mm)	0.28	100	200	400
Medium Silt (0.01mm)	0.008	3700	7500	15,000

Category b: Above-Water Placement. After the fill placement reaches sea level, spreading and compaction will be performed by conventional means in either winter or summer. Similar methods will be used for fills placed onshore.

Compaction methods suitable for gravel fill include roller compaction and in-place treatment techniques such as vibroflotation, compaction piles, and dynamic compaction. Drained or thawed gravel fill will be compacted most readily by rollers. Gravel-ice mixtures compacted by rollers will still have low densities. After thawing, these materials could be further compacted by in-place treatment techniques.

Roller compaction can be performed by spreading the fill material in thin lifts (about 12 inches thick) and compacting each lift with smooth-wheeled, rubber-tired, grid, or sheepsfoot equipment. Vibratory grid rollers are quite effective for compacting granular fill. Based on past experience, it should be possible to compact unfrozen, ice-free gravel to at least 70 percent relative density (about 92 percent relative compaction*).

(*) Relative compaction is the ratio of the in-place dry density to the compacted maximum dry density.

The roller compaction program, including lift thickness and number of roller passes, can be modified in the field if necessary to meet compaction specifications.

2. Silty Sands

Silty sand materials from offshore borrow sources can be placed by pumping directly from the dredge site or dumping from barges. Material pumped in open water or on dry land will probably require fill enclosures formed by perimeter dikes or sheetpile walls to contain the material. Barged material can only be dumped in open water deeper than the draft of the barge.

Particle segregation during sand fill placement offshore is unavoidable. The fine sand fraction has a relatively fast settling velocity and will be deposited near the discharge point. The silt fraction has a much slower settling velocity, tends to stay in suspension, and will be deposited in ponded areas farther from the discharge point. Most of the silt will be carried downstream as indicated by the down current sedimentation distances presented in Table V-2. Open water placement will result in a predominantly fine sand fill. Underwater fill slopes for open water placement will be about 10:1 based on experience with similar fill conditions in the Canadian Arctic (Boone, 1980).

Roller compaction will be difficult because of the wet and loose nature of the recently-placed sand. Other compaction techniques include vibroflotation and compaction piles which densify material by soil displacement and vibration effects, blasting (results are usually erratic) and dynamic compaction. Vibroflotation and blasting are generally limited to cohesionless soils with silt contents less than 20 percent (Mitchell, 1970), while compaction piles and dynamic consolidation can be used to compact soils with somewhat higher amounts of silt. The sand fill can probably be compacted to about 70 percent relative density by all techniques (Mitchell, 1970), provided the improvement techniques are used before the fill freezes. The effectiveness of the method selected should be determined by field testing prior to production operations.

E. Fill Properties

In this section, the geotechnical properties of potential fill materials are considered. Fill materials, as delivered to onshore or offshore construction sites, could include:

- Ice-free gravel placed in the summer by barge, truck or dredge
- Gravel-ice mixtures placed in the winter by truck
- Ice-free silty sand placed in the summer by dredge

In addition to the basic fill material type and the method and season of placement, the geotechnical properties of the in-place fill will be influenced by the amount of compaction and the thermal regime within the fill after placement. In general, the fill material properties described in this chapter are based on our experience with similar material and on published data and empirical relationships. No laboratory tests to measure in-place fill properties were performed for this study.

1. Index Properties

The index properties that will most significantly influence the in-place behavior of the potential fill materials include:

- Void ratio (e)
- Moisture content (W)
- Relative density (R_d)

- Ice saturation (S_i)
- Brine content (S_b)

The weight-volume relationships for these and related index properties are shown on Plate V-2.

Relative density, moisture content, ice saturation, ice content, and brine content are often expressed as percentages; however, for simplicity in writing certain equations, we have adopted the convention of expressing all index property values as ratios. For unfrozen soils, R_d ranges from zero to 1.0; for ice-rich frozen soils R_d can have negative values because of the space occupied by the ice.

The ice saturation is defined as the ratio of ice volume to total void (ice plus air) volume. This index property is not used for frozen soils below sea level; for these soils brine content is a more appropriate parameter.

To distinguish between ice contents before and after placement, the brine content is defined as the ratio of brine weight to total weight of brine, sea ice, and fresh-water ice. The brine volume, ν , is the temperature-dependent equilibrium condition between brine and sea ice as shown on Plate V-2.

In the following sub-sections, relevant index properties are discussed for each potential fill material type, with reference to the method of placement, degree of compaction, and thermal regime.

a. <u>Ice-Free Gravel - Summer Placement</u>

Table V-3 summarizes HLA file data on relative density and compaction properties for gravel fill materials similar to those available for this project. The materials are from the Exxon gravel pits in the Duck Island and Point Thomson areas. For the tabulated data, the average maximum dry

WEIGHT VOLUME

$$e = \frac{\text{void volume}}{\text{solid volume}}$$

$$w = \frac{\text{weight of water}}{\text{weight of solids}}$$

$$R_d = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{1/\gamma_d min - 1/\gamma_d}{1/\gamma_d min - 1/\gamma_d max}$$

$$S_i = \frac{i}{e}\frac{G_s}{G_i}$$

$$S_b = \nu\left(1 - \frac{i_i \quad G_s}{e}\right)$$

WHERE:

e_{max}, e_{min} = void ratio for loosest, densest states respectively

γd = dry density

$\gamma_d(min)$, $\gamma_d(max)$ = dry density for loosest, densest states respectively

$$i = \frac{\text{weight of ice}}{\text{weight of solids}} = \text{ice content}$$

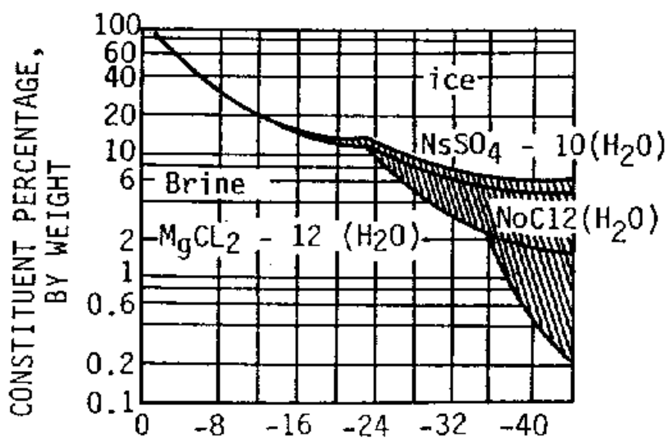
i_i = initial ice content

G_s, G_i = specific gravity of soil and ice, respectively

$$\nu^{1} = \frac{\text{weight of brine}}{\text{weight of ice and brine}} = \text{brine volume}$$

(1) This term is defined by weight; however, it is referred to as "brine volume" to be consistent with the original source (Michel, 1978).

Phase diagram for frozen sea ice
(After Michel, 1978)



ν = Brine volume (percentage by weight)

$\nu = s(0.532 - 49.12/\theta)$

s = salinity (parts per 1000)

θ = temperature (°C) (for: $-0.5°C < \theta < -22.9°C$)

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Weight–Volume Relationships

Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **V-2**

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density determined by the Relative Density Test using both wet and dry vibratory methods is 138 pcf, whereas the average compacted maximum dry density is 142 pcf. The compacted dry density maximum is greater than the relative density maximum because of particle fracturing during compaction (HLA, 1981a,b). Therefore, 138 pcf and 118 pcf are used as the maximum and minimum densities in calculating relative densities. These maximum and minimum dry densities correspond to minimum and maximum void ratios of 0.21 and 0.41, respectively.

TABLE V-3. GRAVEL FILL RELATIVE DENSITY AND COMPACTION TEST SUMMARY

HLA Project	Relative Density		Compaction Test	
	Minimum Dry Density (pcf)	Maximum Dry Density (pcf)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Duck Island (HLA 1981c), Testing Program	118	138	136	7
Duck Island Material Study (HLA, 1981a)	118	139	137	7
Point Thomson Gravel Study (HLA, 1980)	118	138	146	5
C-1 Material Site (HLA, 1981b)	118	139	145	6
Average	118	138	142	6

Ice-free gravels placed below sea level are expected to have a relative density of about 0.5 and a void ratio of 0.31 (Whitman, 1970); however, this needs to be confirmed by field data. Compaction of ice-free gravel above sea level or on land is expected to produce relative densities of at least 0.7, which is equivalent to 92 percent relative compaction and a void ratio of 0.27.

Ice-free gravel fills on land or above sea level are expected to have negligible ice or brine contents. Therefore, design parameters for these materials can be based on their behavior in the unbonded condition. The same materials placed below sea level will initially be saturated with sea water and behave as if they were unbonded. After freezing, the voids will contain brine and ice in the proportions indicated on Plate V-2 as a function of temperature.

A summary of expected index properties for ice-free gravel fill materials is presented in Table V-4, Plate V-3.

b. Gravel-Ice Mixtures - Winter Placement

If the void ratio for a gravel-ice mixture is based on non-ice void volume and ice plus soil solid volume, the average specific gravity of solids will be reduced in proportion to the volume occupied by ice. The resulting expression for dry density is as follows:

$$\gamma_d = \frac{\gamma_w}{(1+e')} \; \frac{G_s}{[1+i_i(G_s/G_i)]} \qquad \text{(V-1)}$$

where: γ_w = unit weight of water
e' = void ratio including ice in solids volume

Bulk samples of gravel fill material taken from the Duck Island Pit during the winter of 1980-81 had ice contents that ranged from 5 to 15 percent. The Duck Island Pit was the source of fill material used to construct Island BF37 and Duck Island 2 during the same winter. Applying this

TABLE V-4. SUMMARY OF INDEX PROPERTIES: ICE-FREE GRAVEL [1]

	Void Ratio[2] e	Dry Density γd (pcf)	Rel. Density Rd	Unfrozen Water Content	Ice Content i	Ice Sat. Si	Brine S_b
Placed dry, compacted							
Unfrozen	0.27	131	0.7	0.03	---	---	---
Frozen[3]	0.27	131	0.7	---	0.03	0.03	---
Placed below sea level, uncompacted							
Unfrozen	0.31	128	0.5	0.12	0	---	1.0
Frozen[3]	0.31	128	0.5	0.05	0.07	---	0.4

NOTES:

1. Ice-free refers to the condition of the fill prior to summer placement.
2. $e_{max} = 0.41$; $e_{min} = 0.21$.
3. Properties for frozen condition based on temperature of -5°C.

TABLE V-5. SUMMARY OF INDEX PROPERTIES: GRAVEL-ICE MIXTURES[2]

Winter Placement		Void Ratio[1] e	Dry Density γd (pcf)	Rel. Density R_d	Unfrozen Water w	Ice i	Ice Sat. Si	Brine S_b
Placed in the dry, compacted frozen								
[3]Case 1		0.63	102	-1.1	0	0.10	0.46	---
Case 2		1.19	76	-3.9	0	0.25	0.61	---
Placed below sea level, uncompacted								
Unfrozen	Case 1	0.83	91	-2.1	0.21	0.10	---	0.68
	Case 2	1.45	68	-5.2	0.29	0.25	---	0.75
Frozen	Case 1	0.83	91	-2.1	0.08	0.23	---	0.27
	Case 2	1.45	68	-5.2	0.12	0.42	---	0.30

NOTES:

1. $e_{max} = 0.41$; $e_{min} = 0.21$.
2. Properties for frozen condition based on temperature of -5°C.
3. Case 1 initial ice content assumed = 10 percent;
   Case 2 initial ice content assumed = 25 percent.

range of ice content to Equation (V-1) with $e' = 0.41$ (for ice-free gravel), $G_s = 2.67$ and $G_i = 0.92$ gives dry densities ranging between 82 and 103 pcf, which are consistent with drive sample data obtained during the construction of Island BF37 and Duck Island 2.

Bulk samples of gravel fill material taken from the Point Thomson C-1 Pit during the winter of 1980-81 had ice contents that ranged from 15 to 35 percent. The C-1 Pit was the source of fill material used to construct the Alaska State D-1 (Flaxman Island), Alaska State E-1 (North Star Island), and Point Thomson 3 Drill Sites during the same winter. Applying this range of ice content to Equation (V-1) with $e' = 0.41$ (for ice-free gravel), $G_s = 2.67$ and $G_i = 0.92$ predicts dry densities ranging between 63 and 82 pcf. This suggests that placement of gravel-ice mixtures below sea level results in an ice and soil fill approximating its loosest state, resulting in low strengths and high settlement characteristics.

Gravel-ice mixtures placed on land or above sea level can be compacted with conventional compaction equipment. The dry density of compacted gravel-ice mixtures should be dependent upon the ice content and the compaction characteristics of the material. Assuming these materials will compact to a void ratio (solids = ice and soil) equivalent to 70 percent relative density ($e' = 0.27$), Equation (V-1) reduces to Equation (V-2).

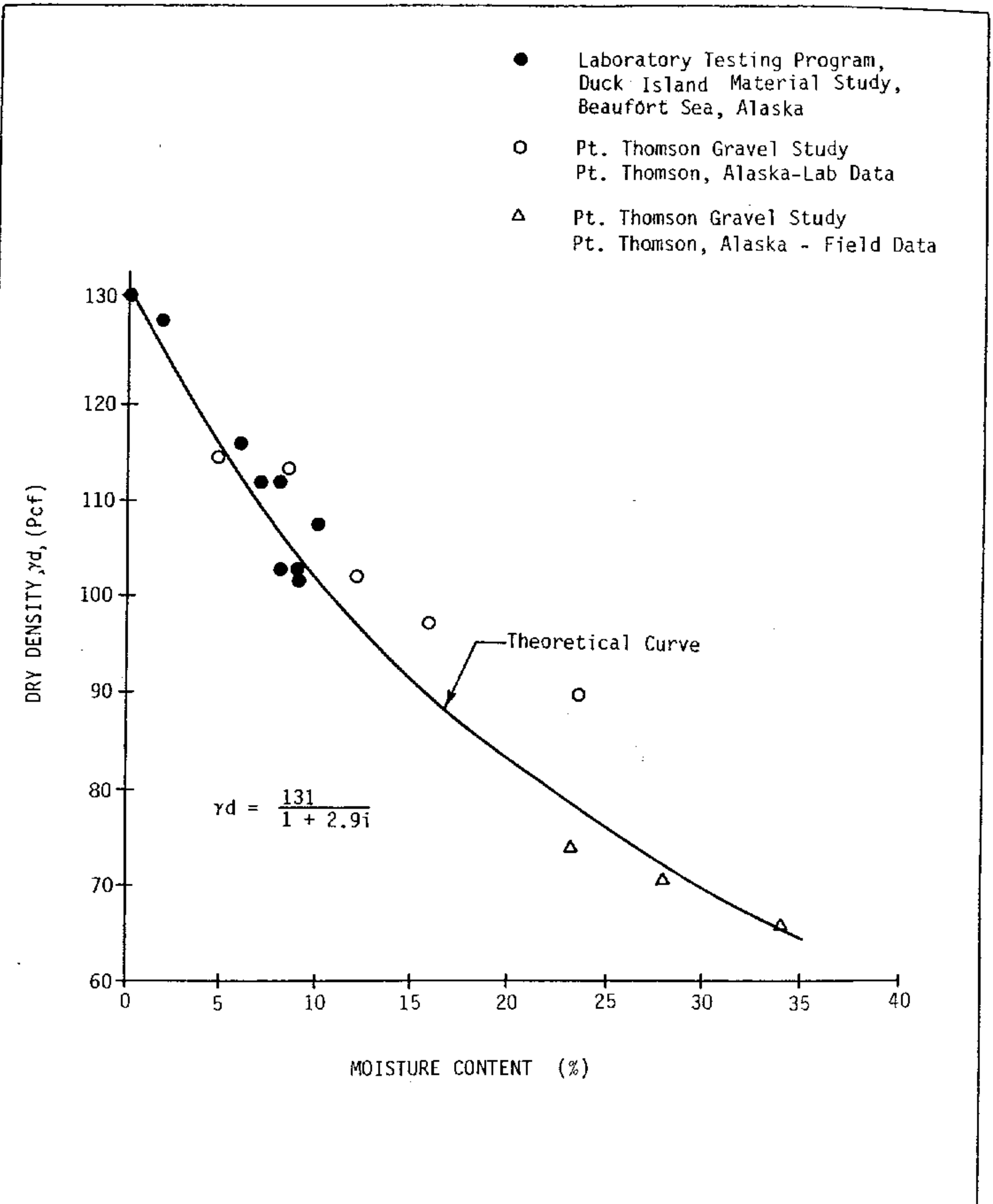
$$\gamma_d = \frac{131}{1 + 2.9i} \qquad \text{(V-2)}$$

For

$G_s = 2.67$

$e' = 0.27$

Equation (V-2) predicts that compacted dry density will be given as shown on Plate V-4. (HLA 1981, a, b). Plate V-4 also summarizes laboratory compaction



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Frozen Compaction Test Summary
Pt. Thomson Development Project
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PLATE **V-4**

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test data and field data for Point Thomson C-1 gravel-ice mixtures. Since these data show generally good agreement with Equation (V-2), this equation can be used to approximate compacted dry densities in the field. Table V-5, Plate V-3 summarizes the expected index properties of gravel-ice mixtures.

c. Ice-Free Silty Sands - Summer Placement

Ice-free silty sands, hydraulically placed and not compacted other than by the weight of overlying fill, will be loose. According to Whitman (1970), the relative density for hydraulically placed clean granular fill usually ranges from about 0.45 to about 0.55, i.e., it is medium dense. However, if the borrow material contains more than about 15 percent fines, the fill is likely to be very loose and heterogeneous. The discussion and case histories cited by Whitman (1970) suggest that hydraulically placed ice-free silty sands may achieve relative densities of only about 0.30.

From the Duck Island study, samples of potential fill materials that were predominantly sand had maximum and minimum dry densities of 102 and 99 pcf. Samples in which silt predominated had a maximum dry density of 107 pcf based on test method ASTM D1557 (compaction test) (HLA, 1981a). The minimum dry density was 59 pcf measured after allowing the soil particles to settle by gravity from a slurry. With surcharge loadings of 100 to 500 psf, the loose slurry consolidated to relative densities in the range of 0.2 to 0.3 (γ_d = 69 pcf to 73 pcf), tending to confirm Whitman's conclusion.

Assuming that the silty sand fill will be about 30 percent silt, the as-placed characteristics for hydraulically placed silty sands are expected to be as follows: $\gamma_{d(max)}$ = 105 pcf, $\gamma_{d(min)}$ = 75 pcf, γ_d = 82 pcf; R_d = 0.3; e = 1.03. In-place compaction of these materials by methods such as vibroflotation or dynamic compaction is assumed to be capable of improving the relative density to about 0.7 (Mitchell, 1970) or e = 0.78 (γ_d = 94 pcf).

Initially the silty sand fill will be saturated with sea water and is expected to remain so until it becomes frozen. Significant drainage before freezing is not anticipated above sea level because of capillary effects and low permeability. Since the initial pore fluid is all sea water, the brine content will be equal to the brine volume when frozen (Plate V-2).

The expected index properties for ice-free silty sand fill materials are summarized in Table V-6, Plate V-5.

2. Mechanical Properties of Unbonded Fill Materials

Both materials placed in the summer and gravel-ice mixtures placed in the winter will behave as unbonded soils until freezeback of free pore water. The parameters governing the behavior of fill materials in the unbonded condition include elastic modulus and Poisson's ratio, shear strength and compressibility. Estimated values for these parameters, based on index properties, laboratory test data and correlations available in the literature for similar soil types, are presented on Plate V-6 along with the relationships used to compute the values. (HLA, 1981a).

3. Mechanical Properties of Ice-Bonded Fill Materials

The behavior of fill materials in the ice-bonded condition are governed by creep, strength, elastic and compressibility parameters. Lacking reliable experimental data, we have performed a qualitative assessment of the mechanical properties of ice-bonded fill materials. Direct measurements are needed to evaluate the validity of the methods employed and the results and conclusions derived from the methods. A detailed discussion of the methods

TABLE V-6. SUMMARY OF INDEX PROPERTIES: ICE-FREE SILTY SANDS[1]

Summer Hydraulic Fill Placement	Void Ratio e^2	Dry Density γd (pcf)	Rel. Density Rd	Unfrozen Water w	Ice i	Ice Sat. Si	Brine Content S_b
Uncompacted							
Unfrozen	1.03	82	0.3	0.39	0	---	1.0
Frozen	1.03	82	0.3	0.16	0.23	---	0.40
Compacted							
Unfrozen	0.78	94	0.7	0.29	0	---	1.0
Frozen[3]	0.78	94	0.7	0.12	0.17	---	0.40

NOTES:

1. Ice-free refers to the condition of the fill prior to placement.
2. e_{max} = 1.22; e_{min} = 0.59.
3. Properties for frozen condition based on temperature of -5°C, with no frost heave.

Table V-7. Empirical Relationships - Unbonded Fill Materials

Shear strength - represented by the Mohr-Coulomb failure criterion:

$$s = c + \sigma_n \tan \phi$$

where:
s = shear strength
c = cohesion intercept
σ_n = normal stress on the failure plane
ϕ = angle of internal friction

Elastic moduli - presented as initial tangent moduli (E_i) in the form: (Duncan et al., 1980):

$$E_i = \left(\frac{\sigma_1 - \sigma_3}{\epsilon_1}\right)_i = k(P_a)\left(\frac{\sigma_3}{P_a}\right)^n$$

where:
σ_1, σ_3 = major and minor principal stresses, respectively
ϵ_1 = major principal strain
k = modulus number
n = modulus exponent
P_a = atmospheric pressure

Tangent modulus, E_t, for any stress condition, $(\sigma_1 - \sigma_3)$, σ_3, can be computed from:

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2c(\cos\phi) + 2\sigma_3(\sin\phi)}\right] E_i$$

where: R_f = "failure ratio" = 0.7 for moist soils. Assumes a hyperbolic stress-strain relationship.

Compressibility - presented in terms of the one-dimensional modulus (M) (Janbu, 1967):

$$M = \frac{d\sigma_v}{d\epsilon_v} = m(P_a)\left(\frac{\sigma_v}{P_a}\right)^{1-a}$$

where:
σ_v = vertical stress in one-dimensional compression
ϵ_v = vertical strain in one-dimensional compression
m = modulus number
a = stress exponent
P_a = atmospheric pressure

Table V-8 Mechanical Properties of Unbonded Fill Materials

	Initial Tangent Modulus		Poisson's Ratio	Shear Strength		[2] One-Dimensional Compression Modulus	
	k	n	μ	Friction Angle ϕ	[3] Cohesion c, psi	m	a
Ice-Free Gravel							
Placed in the dry, compacted to R_d = 0.7	250	0.4	0.30	40°	0	1000	0.9
Placed below sea level, R_d = 0.5	200	0.4	0.30	36°	0	800	0.9
Gravel-Ice Mixtures							
Placed in the dry, compacted	100	0.4	0.30	20°	0	400	0.75
Placed below sea level	60	0.4	0.35	15°		230	0.50
Silty Sands-Hydraulic Fill							
uncompacted, R_d = 0.3	75	0.25	0.45	28°	0	40	0.10
compacted, R_d = 0.7 1	150	0.25	0.45	32°	0	80	0.25

1. $E_i = \left(\frac{\sigma_1 - \sigma_3}{\epsilon_1}\right)_i = k\, P_a\left(\frac{\sigma_3}{P_a}\right)^n$ (After Duncan et.al., 1980)
2. $M = \frac{\sigma_1}{\epsilon_1} = m\, P_a\left(\frac{\sigma_1}{P_a}\right)^{1-a}$ (After Janbu, 1967)

   where: P_a = atmospheric pressure in same unit as σ_1 and σ_3
3. All unbonded fill materials considered cohesionless, except gravel-ice mixtures for which a nominal cohesion is assigned to account for some ice bonding when very cold gravel-ice mixtures are deposited underwater.

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Mechanical Properties-Unbonded Fill

Pt. Thomson Development Project, Winter 1982, Geotechnical Study, EXXON Company, U.S.A.

PLATE **V-6**

DRAWN: JP | JOB NUMBER: 9612,031.08 | APPROVED: DKB | DATE: 4/82 | REVISED | DATE used to develop the qualitative assessment was presented in the Duck Island report and can be found in Appendix E (Exxon, 1981c). In summary, the creep and strength properties of bonded fill were evaluated as a function of material type, strain rate, temperature, void ratio, brine content, and confining pressure. The elastic moduli for frozen soil were assessed as a function of temperature and strain rate. The compressibility was evaluated as a function of soil type, density and unfrozen water content. Estimated values for these parameters, based on index properties, laboratory test data and correlations available in the literature for similar soil types are presented along with the empirical relationships used to compute the values in Tables 9 and 10 (Plate 7). Plate V-8 presents the relationships used to determine the compression parameters.

4. Thermal Properties

Table V-11 summarizes the expected thermal properties of the various fill materials for representative densities and moisture contents. The thermal properties include thaw-strain, and frozen and unfrozen thermal conductivity. The formulation of these design values is discussed below.

TABLE V-9. Summary of creep properties for bonded fill materials

$\dot{\epsilon} = \dot{\epsilon}_c \left(\frac{\sigma}{\sigma_c}\right)^n$ ③	Ice-Free Gravel	Gravel-Ice Mixtures		Ice-Free Silty Sands	
	placed below sea level (R_d = 0.5)	placed in the dry, compacted (Recrystalized)	placed below sea level, uncompacted	uncompacted (R_d = 0.3)	compacted (R_d = 0.7)
Creep Exponent, n	7.5	7.5	7.5	7.5	7.5
Creep Modulus, σ_c(ksf) (@ $\dot{\epsilon}_c$ = 0.15 min^{-1})	115	②57	115	90	90
Adjustments to Creep Modulus ①					
(1) Temp. Exponent, m	0.5	0.5	0.5	0.5	0.5
Temp. factor	2.4	2.4	2.4	2.4	2.4
(2) Void Ratio Factor	0.38	0.97	0.87	0.69	0.87
(3) Ice Sat. Factor	---	0.34	---	---	---
(4) Brine Content	0.2	---	0.2	0.2	0.2
(5) Conf. Pressure Factor ϕ (degrees)	26	10	5	18	22
(N_ϕ - 1)	1.56	0.42	0.19	0.89	1.20
Adjusted Creep Modulus, σ_c	$21.0+1.56\,\sigma_3$	②$(45.0+0.42\,\sigma_3)$	$48.0+0.19\,\sigma_3$	$2.98+0.89\sigma_3$	$3.76+1.20\sigma_3$

NOTES:

1. For average fill temperature = -5°C.
2. Depends upon degree of recrystallization.
3. Ladanyi (1972) proposed creep law where
   - σ = applied stress
   - σ_c = creep modulus (a reference stress)
   - $\dot{\epsilon}$ = strain rate resulting from applied stress, σ
   - $\dot{\epsilon}_c$ = strain rate corresponding to stress, σ_c
   - n = creep exponent

TABLE V-10. Mechanical properties of bonded fill materials

	Initial Tangent Modulus (ksf)		Poisson's Ratio	Shear Strength ksf		One-Dimensional Compression	
	Short Term	Long Term	μ	Short Term (24hrs.)	Long Term (25yrs)	m	a
Ice-Free Gravel Placed below sea level, uncompacted σ_3 = 1.34 ksf @ 20 ft.	1400	200	0.25	4.6	1.4	1500	0.90
Gravel-Ice Mixtures Placed in the dry, compacted (recrystallized) σ_3 = 0.61 ksf @ 10 ft.	3000	550	0.25	10.0	3.7	400	0.75
Placed below sea level, uncompacted σ_3 = 1.19 ksf @ 20 ft.	2850	400	0.15	9.5	2.8	625	0.90
Silty Sands--Hydraulic Fill Uncompacted σ_3 = 0.90 ksf @ 15 ft.	1850	250	0.25	6.1	1.8	125	0.75
Compacted σ_3 = 0.94 ksf @ 15 ft.	2300	350	0.30	7.6	2.3	220	0.90

NOTES:

1. For average fill temperature = 5°C.
2. σ_3 assumed = 0.5 x (overburden pressure) for average depth within the fill.
3. Short-term assumed to be 24 hours; $\frac{Ei}{Su}$ = 300.
4. Long-term assumed to be 25 years; $\frac{Ei}{Su}$ = 150.
5. Properties will vary from unbonded to fully bonded depending upon degree of recrystallization.

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Mechanical Properties - Bonded Fill
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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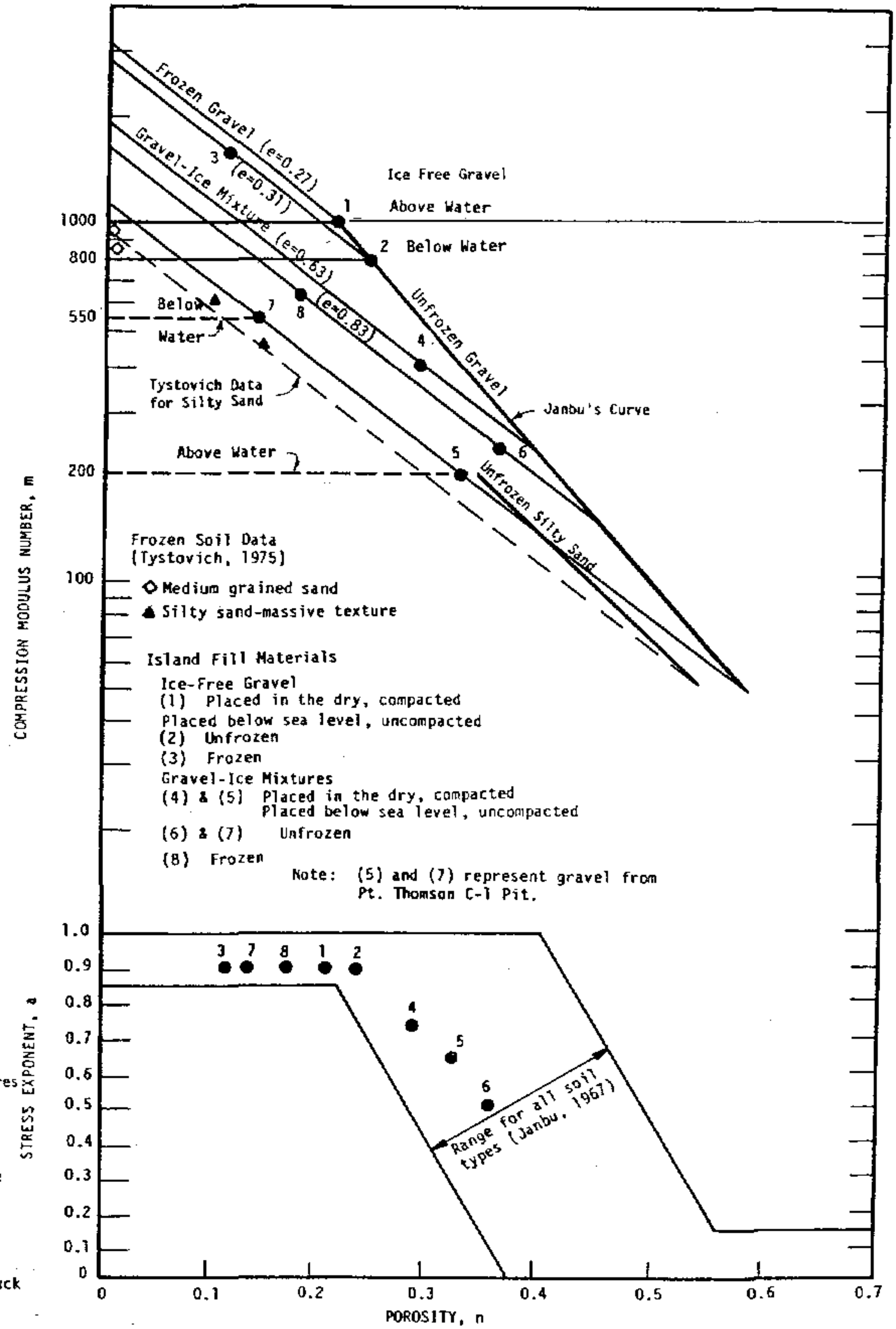
PLATE **V-7**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JS	9612,031.08	DCB	4/82		



NOTES:
(1) See Appendix E for explanation of procedures used to develope this Plate
(2) $M = mPa\left[\frac{\sigma_0}{P_a}\right]^{1-a}$

where:
P_a = atmospheric pressure

$M = \frac{\Delta\sigma}{\Delta\varepsilon}$

Ref: HLA, Exxon Duck Island, 1981

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Compression Parameters for Island Fill Materials

Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE **V-8**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
DCR	9612,031.08	DEB	4/82		

TABLE V-11. THERMAL PROPERTIES OF FILL MATERIALS

	Thaw Strain (%)	Thermal Conductivity(1) Unfrozen	Frozen
Ice-Free Gravel			
Placed in the dry	---	0.6	0.6
Placed below sea level	---	1.5	1.8
Gravel-Ice Mixtures			
Placed in the dry:			
Case 1	14	0.9	0.9
Case 2	35	0.6	0.9
Placed below sea level:			
Case 1	23	1.1	1.3
Case 2	42	0.7	1.1
Silty Sands-Hydraulic Fill			
Uncompacted	5-15	0.7	1.3
Compacted	5-15	0.8	1.3

(1) (BTU-ft./ft.^{2}-hr.^{o}F)

a. Thaw-Strain

The thaw-strain of ice-free gravel placed in the summer will be negligible because of particle-to-particle contact during placement. Movement from frost-heaving will also be negligible since the material is non-frost susceptible.

Based on our experience (HLA 1981a,b), the thaw-strain of gravel-ice mixtures will depend primarily on the initial dry density of the mixture. Gravel-ice mixtures with very low dry densities are expected to thaw-strain to approximately the minimum dry density of the gravel. Thaw-strains predicted on this basis are described by Equation V-3.

$$E_t = \frac{118-\gamma_d}{118} \qquad \text{(V-3)}$$

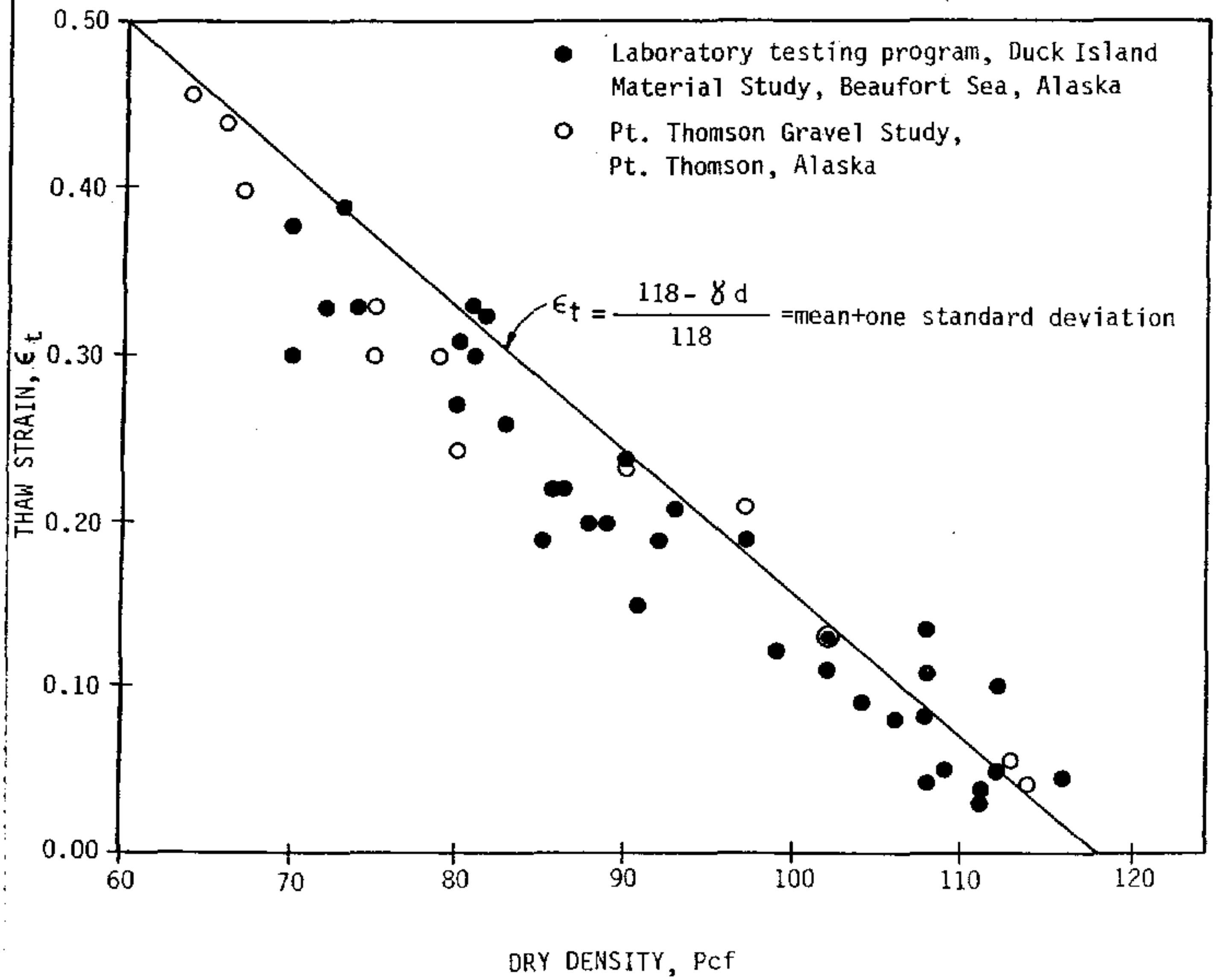
Thaw-strain data for gravel-ice mixtures compacted to various dry densities are presented in Plate V-9. Equation V-3 represents the mean plus one standard deviation of the test data. The thaw-strain tests were performed with surcharge loads of 500 and 1000 psf, and no significant difference in thaw-strain was observed for the difference in surcharge load.

These data are somewhat scattered but compare reasonably well with Equation (V-3) for dry densities less than about 115 pcf. Equation (V-3) has been used to predict thaw-strains for the gravel-ice mixtures anticipated for this project. Gravel-ice mixtures with initial dry densities above 115 pcf are expected to experience thaw strain on the order of 2 percent.

Thaw-strain of the unbonded fine sand and silt fill fractions will be zero. These materials are highly frost susceptible and, upon freezing, substantial ice lensing can be expected. If these materials later thaw, the thaw-strain is likely to be large and variable. Thaw-strains of 5 to 15 percent with large variations over short distances should be anticipated.

b. Thermal Conductivity

Unfrozen and frozen thermal conductivities were estimated using the average values for sandy soils shown in the U.S. Army Technical Manual TM 5-852-6 (1966). These values are within ten percent of typical values measured in our laboratory for unfrozen and frozen gravel samples compacted with fresh water to various densities (HLA, 1981a). To account for the mixture of ice and unfrozen water, average values for unfrozen and frozen thermal conductivities were used for gravel-ice mixtures placed below sea level.



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Thaw-Strain Relationship

Pt. Thomson Development Project, Winter 1982, Geotechnical Study, EXXON Company, U.S.A.

PLATE **V-9**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
SP	9612,031.08	DEP	4/82		

F. Conclusions

Sandy gravel is present in sufficient quantities for construction of islands, causeways, roads, and drill pads within the development area with an overburden thickness varying from 0 to 35 feet thick at offshore and onshore locations. Offshore, shallow unfrozen gravel is present between Point Hopson and Point Thomson in the lagoon (Borings 11 and 22) and at the east end of the project, Borings 17 and 20. Onshore, shallow frozen gravel is available throughout the project area and in particular in the vicinity of the old C-1 Pit, 3 miles from the coast.

The best fill material for the construction of gravel fill structures is the Pleistocene sandy gravel that underlies the majority of the offshore lagoon. Ideally, the gravel will be dredged from sites close to proposed structures and transported to the structures as a slurry through pipelines. This method may be feasible in the vicinity of Boring 11 where the gravel surface is at the mudline and the gravel is not frozen. In other areas within the lagoon the amount of overburden may make the development of a local source of dredged gravel infeasible.

The fill placement can be performed during either the winter or summer seasons, but the densest fill embankment is achieved during the summer construction season by placing and compacting unfrozen, ice-free gravel. Either the onshore or offshore sources of sandy gravel could be used. The onshore material is frozen, but during the summer with the time and disturbance required to excavate, stockpile, load, and haul the gravel, the material will probably have a significantly reduced ice content when it is placed.

The unfrozen gravel can be compacted above the water to a dense condition using conventional equipment such as heavy vibratory rollers. The sandy gravel is sufficiently free-draining that some below water compaction can be achieved with vibratory rollers used at sea level. Depending on the equipment used and the number of roller passes, the gravel fill can be compacted to a medium density to depths of 5 to 8 feet below sea level (D'Appolonia, Whitman and D'Appolonia, 1969). In water too deep for surface rollers to be effective, the gravel can be densified using deep compaction techniques such as vibroflotation or drop weights. The summer season for placement and compaction of unfrozen gravel is only about 100 days long, and to be effective, these two construction operations should not be separated by a winter season.

The strength of the unfrozen gravel increases by about 25 percent from the loosest to densest condition. The strength of the unfrozen gravel depends on its angle of internal friction and the effective pressure confining the material. The major component of the confining pressure in an island is the weight of the fill material. Unfrozen gravel fill weighs about 125 pcf, exclusive of water content.

The gravel fill will subsequently freeze. The gravel is non-frost susceptible; frost heave forces should have little if any impact on the design of piles used as foundations through the embankment. The gravel will be stronger after freezing, and because of the initial grain-to-grain contact, will be relatively resistant to creep deformations when loaded for long periods of time. If the gravel subsequently thaws, the resulting settlement will be minor.

Fill placement can be performed in the winter using sandy gravel excavated from onshore sources similar to the old Exxon C-1 Pit. Material from this pit was used during the winter of 1980-81 to build the drill pads on Flaxman Island, North Star Island and at Point Thomson.

Gravel fill placed during the winter season has a lower density than fill placed in the summer. The 25 percent average ice content by weight (based on the C-1 site) in the frozen gravel occupies 67 percent of the fill volume because of the difference in specific gravities of the ice and soil. Because of the volume occupied by the ice, the dry density of frozen gravel fill, even with proper compaction, is less than the loosest density of unfrozen gravel. With 25 percent ice content, the dry weight of the frozen fill after compaction will be about 70 pcf.

When very cold gravel-ice fill is placed in freezing seawater it is believed that the seawater freezes as it floods the fill material. The amount of frozen seawater probably depends on the temperature, gradation, and ice content of the fill, the depth of water, and the rate and method of fill placement. Field or laboratory data on fill properties are not available for this method of placement. Strength values extrapolated from data for tests on frozen sands and silts and on sea ice indicate that the gravel-ice mixture in an island will have 15 to 75 percent less strength than the loose but unfrozen gravel condition. The effect of the difference in weights of the two fills is included in the strength difference.

VI OFFSHORE GEOTECHNICAL CONSIDERATIONS

VI OFFSHORE GEOTECHNICAL CONSIDERATIONS
(continued)

LIST OF TABLES

LIST OF ILLUSTRATIONS

VI OFFSHORE GEOTECHNICAL CONSIDERATIONS

A. Introduction

Planned construction in the study area includes artificial islands, causeways, offshore pipelines, bridges and dock structures. The islands will probably support heavy modular units and drilling facilities. The modular units may be several stories high and the drilling facilities may include a multistory structure containing a drilling deck, a production deck, and a cellar block. A waterflood intake structure may be located in the deep water north of the barrier islands.

Geotechnical considerations for the artificial islands include analyses of liquefaction, slope stability, compression and thaw-strain settlements, base sliding, shallow and deep foundations, and dock structures. Along the causeway, design consideration for culverts, pile foundation for bridges and sheet pile walls for bridge abutments were studied. Potential problems associated with offshore pipelines were evaluated.

For preliminary design purposes, the offshore geotechnical conditions can be divided into three areas based on water depths, soil conditions and thermal regime: (a) the lagoon area (between the shoreline and barrier islands), (b) the barrier islands and (c) the area north of the barrier islands. Representative soil profiles along with general soil properties for the three areas have been defined on Plate IV-26.

B. Geotechnical Considerations for Island Design

Island design requires knowledge of fill and foundation soil properties, thermal regime throughout the design life of the island and external loads, such as wave, ice, wind and earthquake. In this preliminary study, we have limited our analyses to gravel slope stability, compression settlement, thaw-strain settlement, base sliding and liquefaction potential.

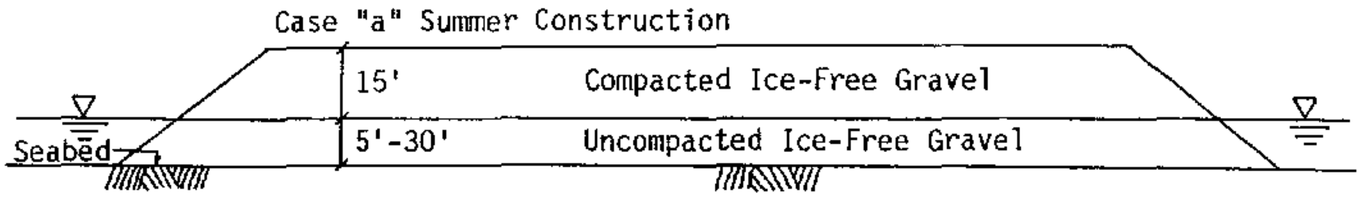
1. Fill Sections

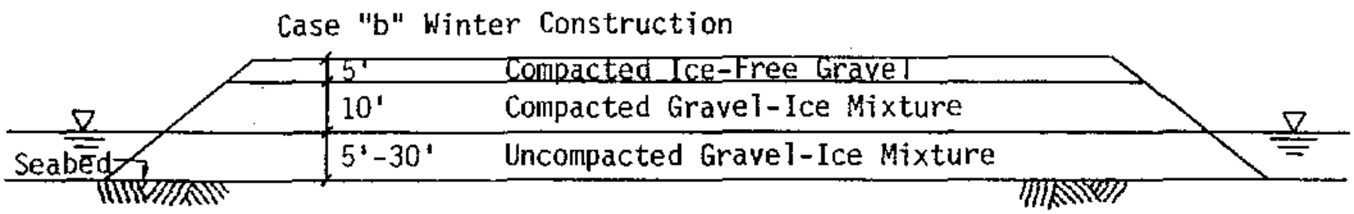
Because of the availability of gravel in the area, the islands will probably be constructed of ice-free gravel or gravel-ice mixtures. The islands will become frozen with time. The freeze front will advance from the surface and progressively penetrate deeper in subsequent winters, eventually extending below the fill into the natural sediments. To prevent annual cycles of frost heave and thaw settlement, the surface active layer of the fill should be of non-frost susceptible material. We anticipate this active layer to be five to eight feet thick. We recommend that the upper eight feet of fill should be constructed of ice-free gravel with less than three percent fines.

Two potential fill sections for island design are shown on Plate VI-1. Index properties and mechanical and thermal properties of gravel fill materials are discussed in Chapter V.

2. Slope Stability

There are basically two potential modes of slope failure: (a) shallow slope failure within the island fill and (b) deep seated failure through the foundation soils. Shallow slope failure within the fill slope (surface sloughing) can be induced by seepage forces from wave runup, ice or wave scouring undercutting the slope, loss of strength when the fill thaws, or a seismic event. A weak foundation soil with a steep fill slope is usually





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Probable Island Fill Sections
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE
VI-1

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the cause of a deep seated slide and often occurs during or shortly after construction. Surface sloughing is mainly a maintenance problem, but the slopes can be designed to reduce this nuisance. Deep-seated slope failure must be avoided.

a. Deep-Seated Stability Failure

The most critical period for deep-seated stability failure is during or shortly after construction when the strengths of the fill and the underlying natural sediments are at a minimum. The fill becomes stronger as the freeze front advances with time and the natural sediments consolidate. Therefore, only the end-of-construction stability is analyzed. The analyses were made using Bishop's modified method with the aid of a computer program. Earthquake loading was not included for the end-of-construction case because both conditions are transient and very unlikely to occur simultaneously.

The lagoon area is blanketed by soft to medium stiff sandy silt sediments overlying very stiff clay and silt. The upper silty sediment is interbedded with many sand and gravel sublayers; therefore, this stratum is expected to be relatively fast draining. To be conservative, the undrained shear strength was used without giving consideration to pore pressure dissipation during construction. Our stability analysis for a representative range of fill strength and unit weight yielded a factor of safety of at least 1.5 for side slopes of 2 to 1 (horizontal to vertical) or flatter.

Deep-seated stability failure is not a problem on the barrier islands. The height of fill is less than 15 feet and the underlying soil is a medium dense sand with a strength approaching that of the gravel fill. In many areas the permafrost is very shallow; therefore, analyses were not performed for this case.

The area north of the barrier islands is underlain by a deep stratum of very stiff clay with an undrained shear strength of over 2000 psf. The allowable bearing pressure exceeds that imposed by the island; hence, no further stability analysis was made.

b. Shallow Slope Failure

The possibility of shallow slope failure within the fill was analyzed using the infinite slope method. Four cases were analyzed: (a) end-of-construction, (b) seepage parallel to the slope, (c) pseudo-static earthquake loading (horizontal ground acceleration, a = 0.05g) and (d) pseudo-static earthquake loading with seepage parallel to the slope.

In all cases, the analyses are only applicable to the thawed active layer of the fill slope during the summer season. Assuming the fill near the surface is relatively well-compacted, the effective friction angle should be in the range of 35 to 40 degrees. The factors of safety for three different side slopes are shown in Table VI-1.

TABLE VI-1. FACTORS OF SAFETY FOR SHALLOW SLOPE FAILURE

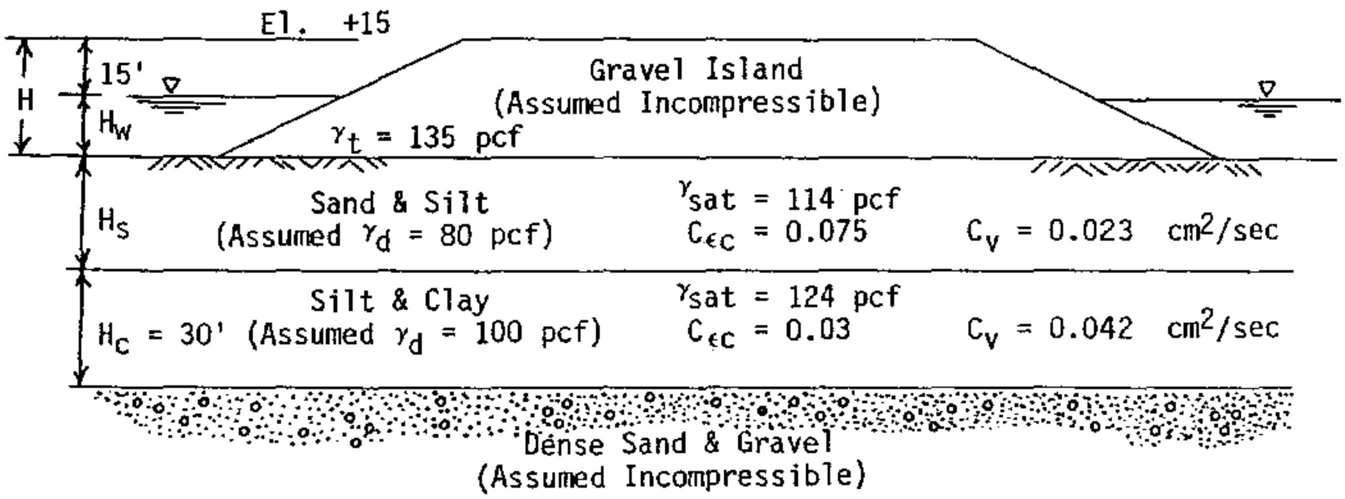
	Factor of Safety					
	2 to 1 Slope		3 to 1 Slope		4 to 1 Slope	
Case	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$
End-of-Construction	1.4	1.7	2.1	2.5	2.8	3.4
Seepage Parallel to Slope	0.7	0.9	1.1	1.3	1.5	1.7
Pseudo-static (a = 0.05g)	1.2	1.5	1.8	2.2	2.3	2.8
Pseudo-static with Seepage Parallel to Slope (a = 0.05g)	0.6	0.8	0.9	1.1	1.2	1.4

It may be uneconomical to select a fill slope that is safe against shallow slope failure for all four cases discussed. For example, the case that considers pseudo-static loading with seepage parallel to slope is only applicable to a seismic event that occurs during the short summer season. The chance of this happening is fairly remote. It may be more economical to maintain the slope periodically since only the relatively shallow active zone would be affected. If slope protection is used, the safety factor for all cases will be increased.

3. Compression Settlement

Compression settlement will result from consolidation of the natural sediments under the weight of the island fill. The total and time rate of settlement have been computed using Terzagi's one-dimensional consolidation theory. The parameters used in the analyses are based on the greatest fill weight (ice-free gravel), average dry densities of the compressible strata, and the correlation between dry density and the compression parameters established in Chapter IV (see Plates IV-20 and IV-21).

In the lagoon area, the water depth and thicknesses of the compressible strata are location dependent. In general, they increase from east to west and larger settlement and a longer period of consolidation are expected at the west end of the lagoon. Results of our analyses for various water depths and soil thicknesses are presented on Plate VI-2. The largest total settlement (at the west end) is on the order of two feet, with 90 percent of the settlement expected to occur within three months after construction.



Water Depth (ft)	Thickness of Sand & Silt, Hs (ft)	Total Settlement (ft)	Time at 90% Consolidation (months)
10	10	1.2	0.5
	20	1.6	1.5
	30	1.8	3.0

NOTE: Increase settlement by 10% if water depth is 15 feet instead of 10; time at 90% consolidation remains unchanged.

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Settlement of Gravel Island within Lagoon Area
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Winter 1982, Geotechnical Study
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At the barrier islands, the depth to permafrost varies from 0 to 30 feet. In areas of shallow permafrost, the compression settlement is negligible as long as the soil remains frozen. If the depth to permafrost is 30 feet, the compression settlement is estimated to be approximately one foot, with 90 percent of the settlement to be completed within three months after construction (see Plate VI-3).

The area north of the barrier islands is covered by a deep stratum of heavily overconsolidated clay and underlain by dense sand and gravel. The pressure imposed by the weight of an island does not exceed the preconsolidation pressure of the clay. Therefore, the compression settlement will be small and the time rate of settlement relatively fast. The computed settlement is about one foot and will be approximately 90 percent complete in seven months. Results of the analyses are shown on Plate VI-4.

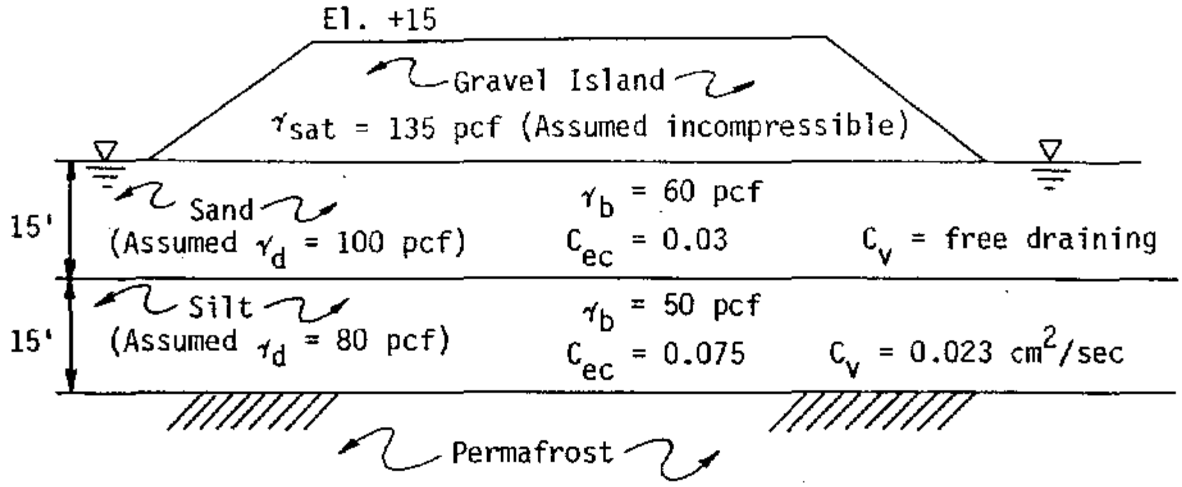
4. Thaw-strain Settlement of Natural Sediments at the Barrier Islands

The barrier islands and new fill will settle if the permafrost thaws. Thawing could be caused by the heat from structures placed at-grade, warm buried utilidors, or the well area. Based on laboratory test results from this and other studies (Exxon, 1981), thaw-strain settlements for the frozen sediments can be estimated using the following equation.

$$TS = 0.05D_1 + 0.10D_2 + 0.02\,D_3$$

Where
- TS = Total thaw-strain settlement in feet
- D_1 = Thickness of the thawed zone within the sand and silt layer in feet
- D_2 = Thickness of the thawed zone within the stiff clay layer in feet
- D_3 = Thickness of the thawed zone within the dense gravel layer in feet

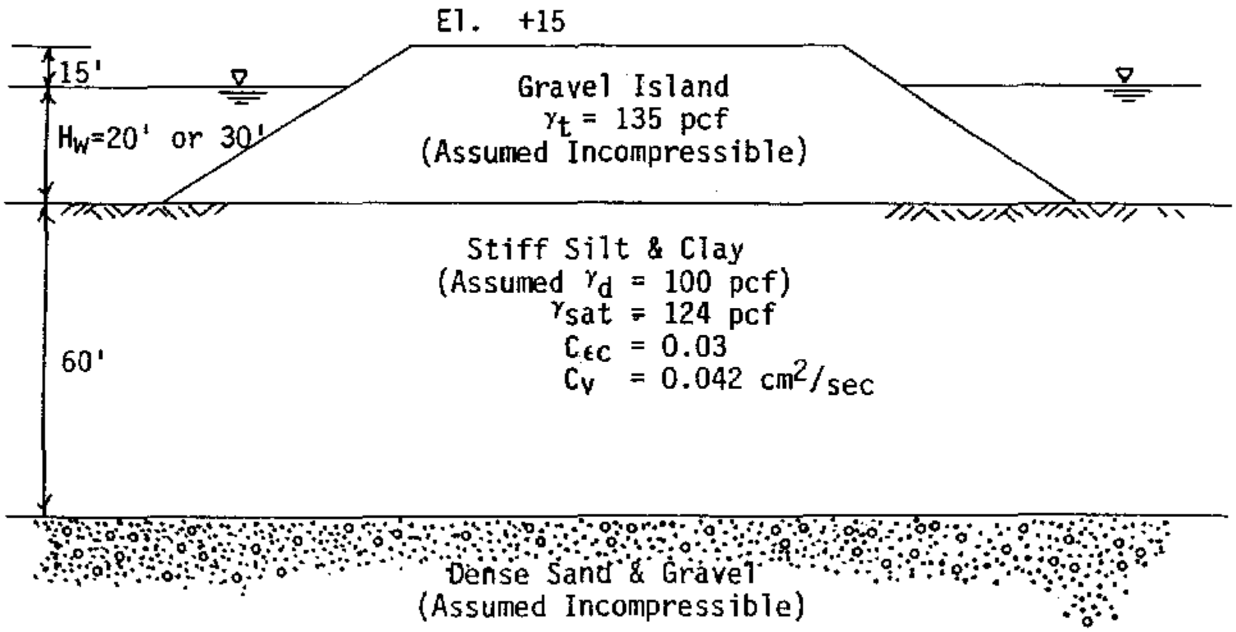
Settlement Of Gravel Pads On Barrier Islands



Total Settlement = 1 foot

Time at 90% Consolidation = 3 month

NOTE: If the permafrost is shallow and remains frozen after construction, the compression settlement is negligible.



Water Depth (ft)	Total Settlement (ft)	Time at 90% Consolidation (months)
20	0.9	7
30	1.0	7

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Settlement of Gravel Island North of the Barrier Islands
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Thaw-strain settlement may be non-uniform because of the presence of ice lenses and wedges.

5. Thaw-strain Settlement of Island Fill

If the island is constructed of ice-free gravel, the thaw-strain settlement will be negligible. If a gravel-ice mixture is used and subsequently thaws, the thaw-strain settlement can be estimated using the following equation:

$$TS = 0.36D_1 + 0.42D_2$$

Where TS = Total thaw-strain settlement in feet
D_1 = Thickness of the thawed zone within the gravel-ice mixture above water in feet
D_2 = Thickness of the thawed zone within the gravel-ice mixture below water in feet

This equation is based on our previous studies of the gravel fill in the area and ice contents of the fill above and below water assumed to be 25 and 42 percent, respectively.

6. Resistance Against Base Sliding

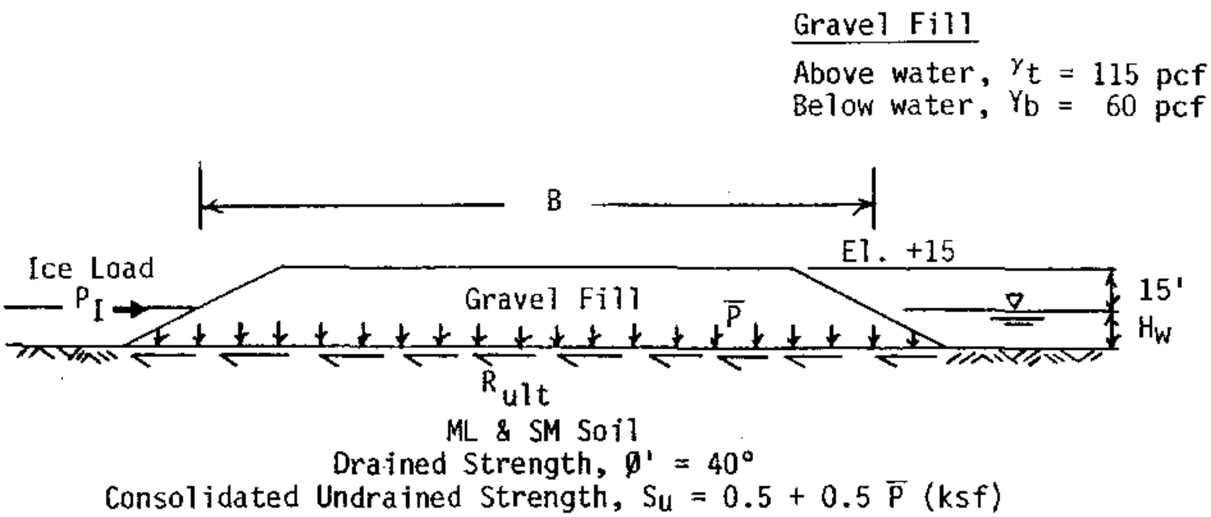
The resistance against base sliding is dependent on the thermal regime of the island. The freeze front will advance from the surface and eventually penetrate into the natural sediments. At the end of construction when the gravel fill is at its weakest, a large lateral ice load impinging on the island could cause a local passive failure of the fill. We have assumed that when the island becomes frozen, it will tend to move as a frozen block under a lateral ice load with the plane of failure located at the frozen/-unfrozen boundary in the natural sediments.

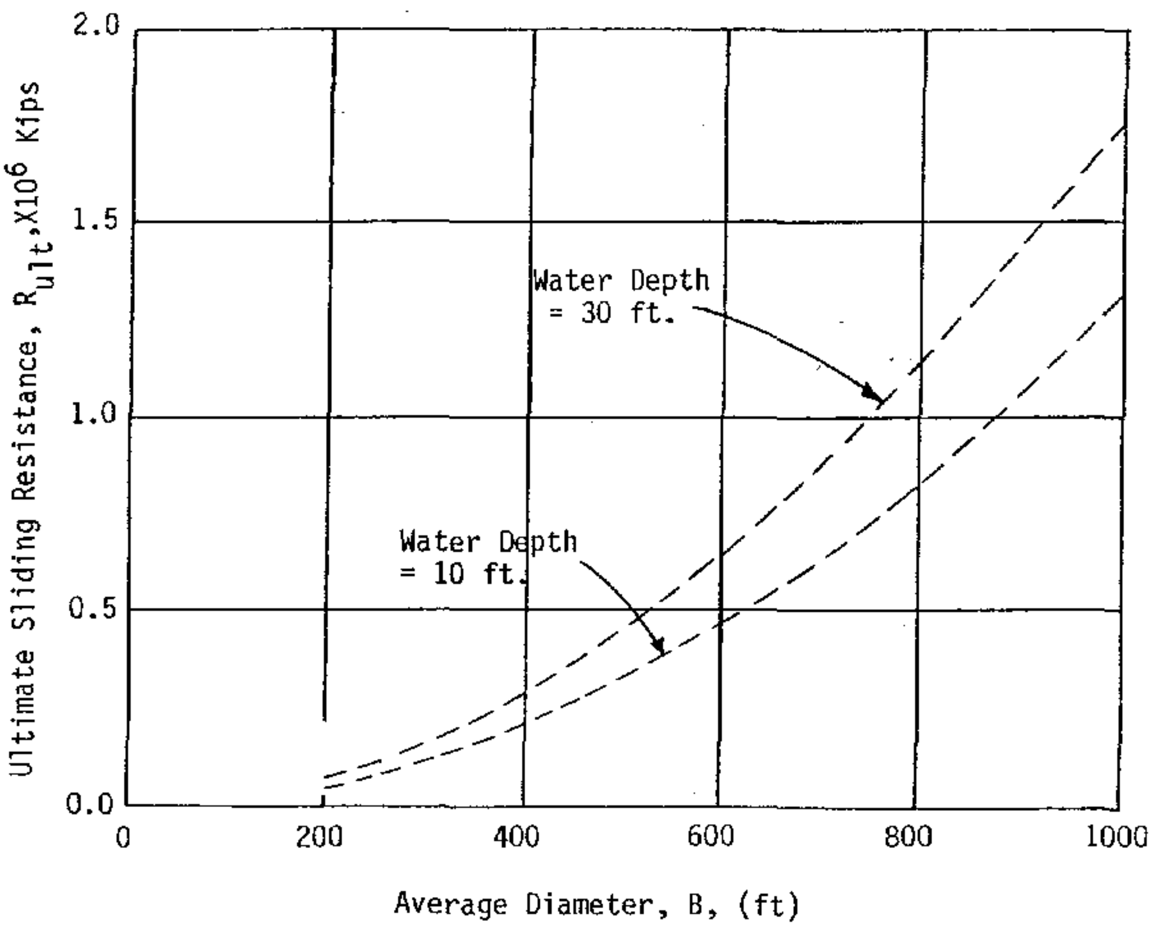
When a lateral ice load impinges on a frozen island, two things occur simultaneously: (1) the island compresses elastically in the direction of the load, and (2) shear stresses are induced into the underlying sediments which cause shear deformation. Therefore, the initial island movement is the result of elastic compression of the island plus the shear deformation of the foundation soil. Most of the elastic compression occurs near the point of application of the ice load, which mobilizes the shear strength of the soil in that area at the base of the frozen island. As the ice load increases, more soil strength becomes mobilized. Eventually, if the ice load becomes large enough, slippage occurs along the base of the frozen island.

The ultimate resistance is computed using the undrained strength of the natural sediments. These strength criteria are applicable to all areas offshore, except in the barrier islands where the permafrost is at the surface. Since we have assumed the gravel island will be frozen, sliding will occur at the base. Assuming an initial moisture (ice) content of 24 percent, the total and bouyant unit weights of gravel-ice fill placed in the winter are 115 and 60 pounds per cubic foot, respectively. Sliding resistances have been computed for island diameters varying from 200 to 1000 feet in water depths of 10 and 30 feet. The surface elevation of the island is assumed to be at 15 feet above mean sea level. Results of the analyses are presented on Plate VI-5.

7. Liquefaction Potential

In general, the offshore area is blanketed by interbedded layers of silt, sand and gravel. These soils are underlain by permafrost at the barrier islands and by a layer of stiff clay in other areas, neither of which are considered susceptible to liquefaction. The liquefaction potential of the upper sediments was evaluated using the simplified method developed by Seed (1978). Two seismic design criteria were considered in the analysis: (1) a





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Sliding Resistance Against Base Sliding for Gravel Islands
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PLATE **VI-5**

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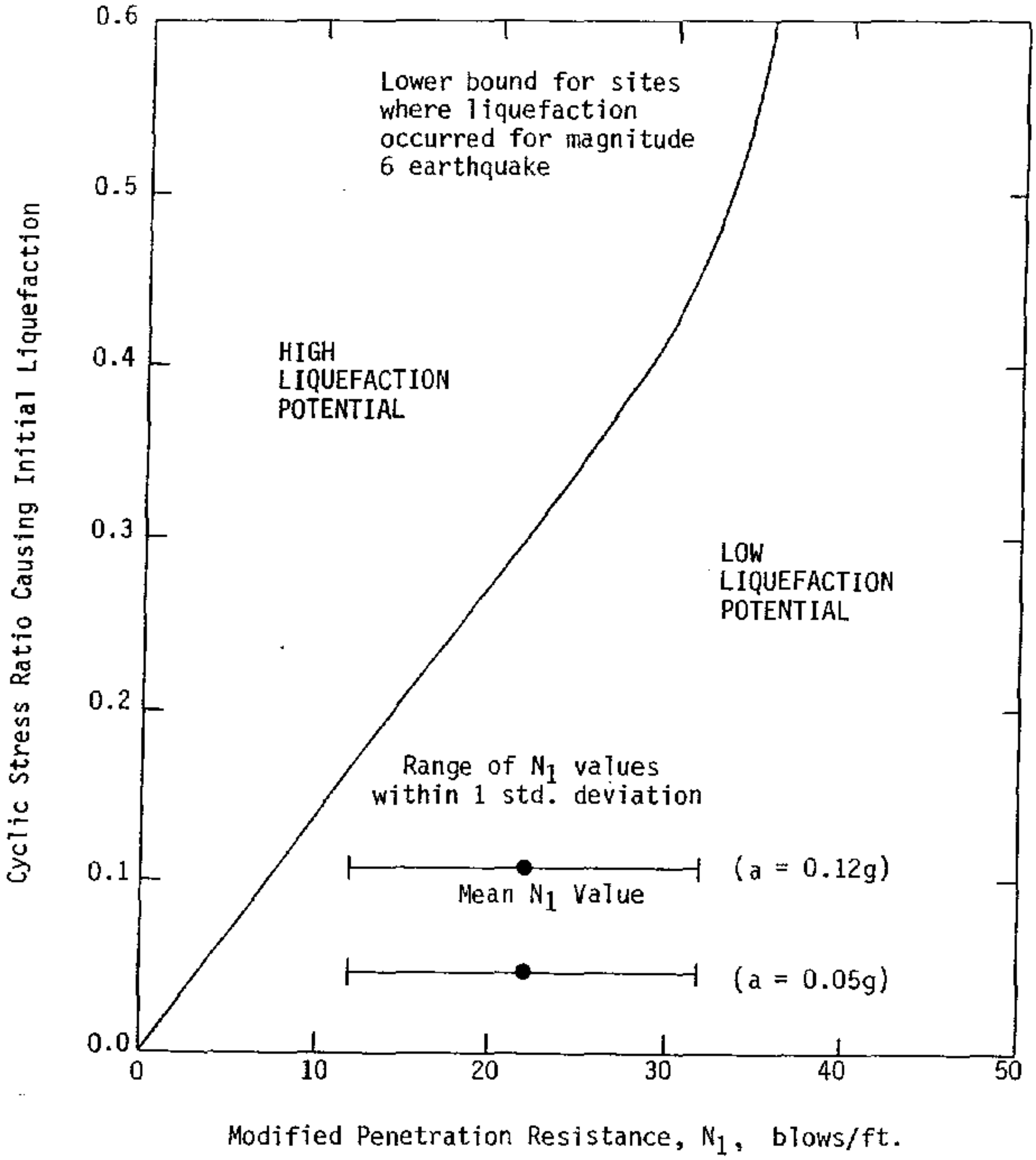
design earthquake of Richter magnitude 5.5 with an effective duration of 5 seconds and a horizontal ground acceleration of 0.12 g, as developed for the Alyeska Pipeline Service Company (Donovon and Singh, 1978); and (2) a ground acceleration of 0.05 g for Zone 1 seismicity in accordance with the 1976 Uniform Building Code.

Seed's simplified method of liquefaction analysis relates values of cyclic stress ratio (τ_{av}/σ_o') known to be associated with liquefaction, or the absence of liquefaction to the modified penetration resistance (*), N_1, of the soil deposit involved. A plot presented by Seed (1976), relating stress ratio and penetration resistance for a magnitude 6 earthquake, is reproduced in Plate VI-6. The solid line delineates cases where liquefaction did or did not occur for various stress ratios and N_1 values.

The blow count data from our field exploration have been converted to standard pentration test (SPT) values to account for differences in hammer energy and sampler size. The converted blow count values (with a range of one standard deviation), are from 9 to 23, and the corresponding N_1 values at 30 feet below mudline are 12 and 32, respectively, with a mean value of 22. The computed cyclic stress ratio at the 20-foot depth (with 25 feet of fill above mudline) ranges from 0.05 to 0.10 kips per square foot (ksf) for the UBC and Alyeska criteria, respectively. The data presented on Plate VI-6 indicate that the seabottom sediments at the site are not likely to liquefy.

(*) N represents Standard Penetration Test (SPT) blow counts. (N_1) values are blow counts normalized to an effective overburden pressure of 1.0 ton per square foot.



(After Seed, 1976)

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Liquefaction Potential

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PLATE **VI-6**

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C. Foundations

The structures on the islands can be supported on either pile foundations gaining support from the materials below the island fill or on shallow footings bearing on the fill material. The selection of the appropriate foundation system depends on the structural loads and tolerances to total and differential settlement, the subsea soil conditions, the type of fill and the season and method of fill placement, and the elapsed time between the construction of the island and the installation and loading of the foundation system.

Most foundation design experience is from buildings requiring a substantial period of time for construction, i.e., the work starts with the foundation and the load increases gradually as the building goes up. For the modular system of construction, the total building load is applied almost instantaneously. When the module is lowered from the carrier support system onto the foundation system, the foundation will be immediately loaded to almost 100 percent of the structural deadload.

As discussed in the previous section on compression settlement, the weight of the new fill will compress the unfrozen fine-grained sediments below the seafloor and result in island settlement. The compression settlement will affect any foundation systems installed before the compression settlement is complete. The remaining compression settlement could produce settlements of shallow footings or downdrag forces on piles. Results of the time rate calculation indicate that 90 percent of the settlement will occur within 3 months after construction.

1. Pile Foundations

a. Axial Capacity

Structures on the islands can be founded on piles to reduce settlement. Driven piles are an economical method of installation since the underlying saturated sands and silts would have to be fully cased to prevent caving if drilling techniques were used. Predrilling or jetting may be necessary for the piles to penetrate through the gravel fill.

In our analyses, we assumed non-displacement piles, such as "H" piles or open-ended pipe piles, would be used. We assume the pipe piles will be backfilled with dense gravel and the pile capacity derived from side friction only. The method of analysis is similar to that recommended by Poulous and Davis (1980) with modifications appropriate for the soil conditions encountered in this study. For example, a conservative side friction resistance was assigned to the upper silty sediments in the lagoon area using the blow count method proposed by Meyerhof (1976) and the side friction was limited to 2000 psf for all soils. We feel the preliminary nature of this study warrants the use of a conservative approach in predicting pile capacities. Therefore, because of the lack of practical experience with these soils, and field test data such as pile load tests, the blow count method was used in lieu of other relationships that predict frictional resistance based on effective angle of internal friction $\emptyset'$, or undrained shear strength. These other methods are felt to be applicable during site-specific design.

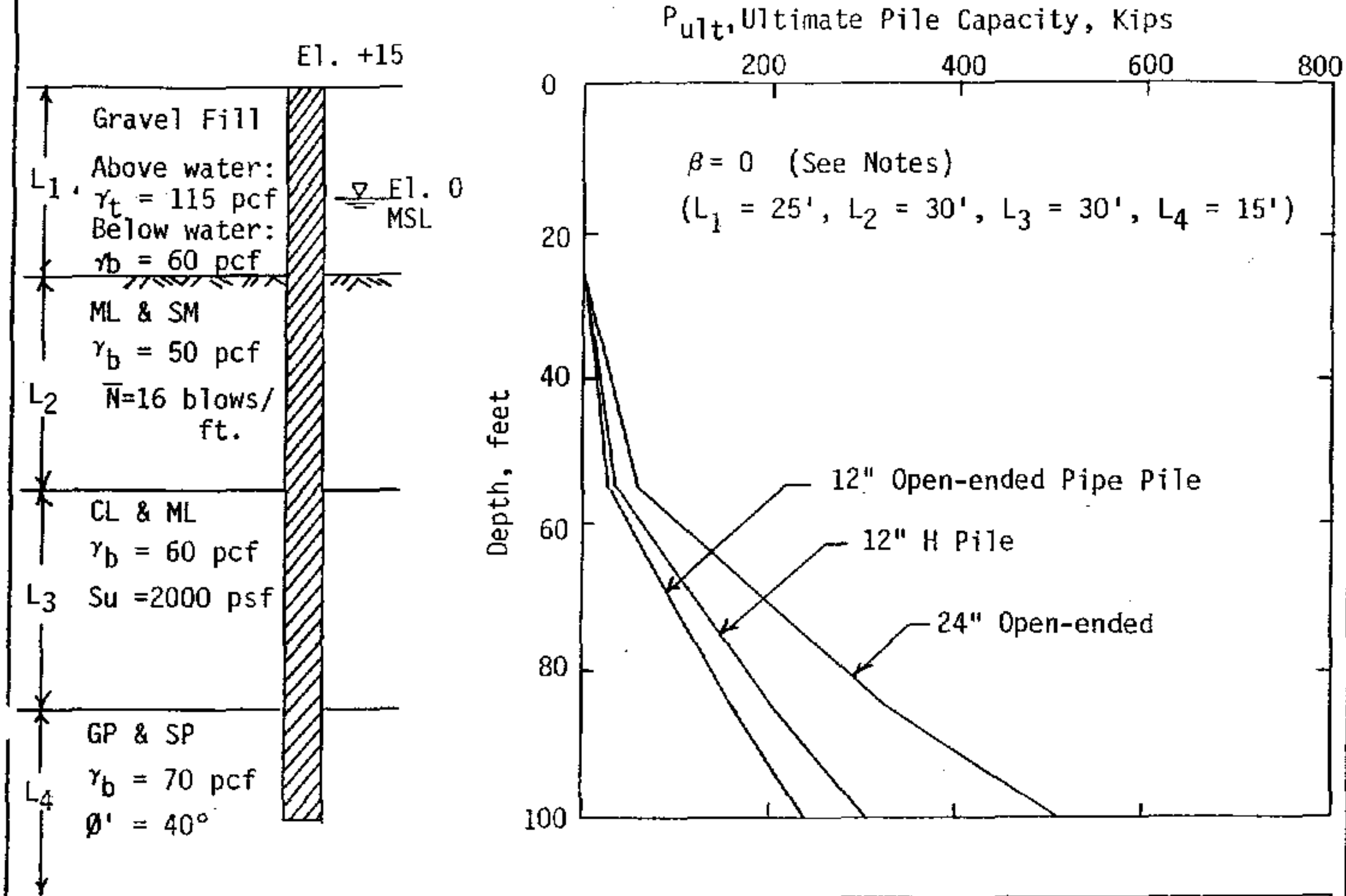
Side friction in the gravel fill depends on the quality of the fill, the thermal regime and the method of pile installation. If a gravel-ice mixture is used, excessive creep settlement will occur under a sustained load, eliminating any side friction support within the fill. If the gravel-ice fill thaws, the fill will compress and create downdrag on the piles. The downdrag force is computed assuming the thawed gravel will have a friction angle of 28 degrees.

If the island is built of gravels with low ice content, it is too conservative to discount the frictional resistance from the fill in computing pile capacity. Therefore, the frictional resistance may be computed using a friction angle of 35 degrees. If predrilling, jetting or spudding is used, the friction resistance should be reduced 50 percent.

The ultimate pile capacities are expressed in equation form, reflecting the variation of water depth and thicknesses of the soil strata. The axial capacity can be computed by substituting the appropriate water depth and layer thicknesses. Results of the analyses for the lagoon area and the area north of the barrier islands are presented on Plates VI-7 and VI-8, respectively. Examples showing pile capacities for the typical soil conditions in the two areas are also presented. The pile capacity equations yield ultimate axial capacities. For allowable design load, a safety factor of 2.0 should be used for dead loads and 1.5 for dead plus live loads.

In the lagoon area, no permafrost was encountered to the depth drilled (50 to 80 feet below sea floor). Hard driving is anticipated in the dense sand and gravel underlying the stiff Pleistocene clay. If refusal is encountered when the hammer is operating property, the full structural capacity of the pile can be utilized.

In areas north of the barrier islands, permafrost was encountered as shallow as 3 feet below sea floor (Boring 16). If a pile penetrates into the permafrost, consideration should be given to possible



H Piles & Pipe Piles = $P_{ult} = R\left[(1.13L_1 - 4)\beta + 0.32L_2 + 1.4L_3 + 1.6L_4\right]$ Kips

where R = pile perimeter in feet

24" Pipe Piles = $P_{ult} = 2\pi\left[(2.0L_1 - 12.2)\beta + 0.32L_2 + 1.4L_3 + 2.0L_4\right]$ Kips

NOTES: (1) For allowable design load, use F.S. = 2.0 for dead load, and F.S. = 1.5 for dead + live load

(2) β = side friction factor to account for axial capacity or downdrag force on pile from the gravel fill
For ice-rich fill = $\beta = 0$ if fill remains frozen
$\beta = -0.2$ if fill becomes thawed
For fill with low ice content = $\beta = 0.5$ if pile was predrilled thru fill
$\beta = 1.0$ if no predrilling nor jetting

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Pile Capacity in Lagoon Areas
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PLATE **VI-7**

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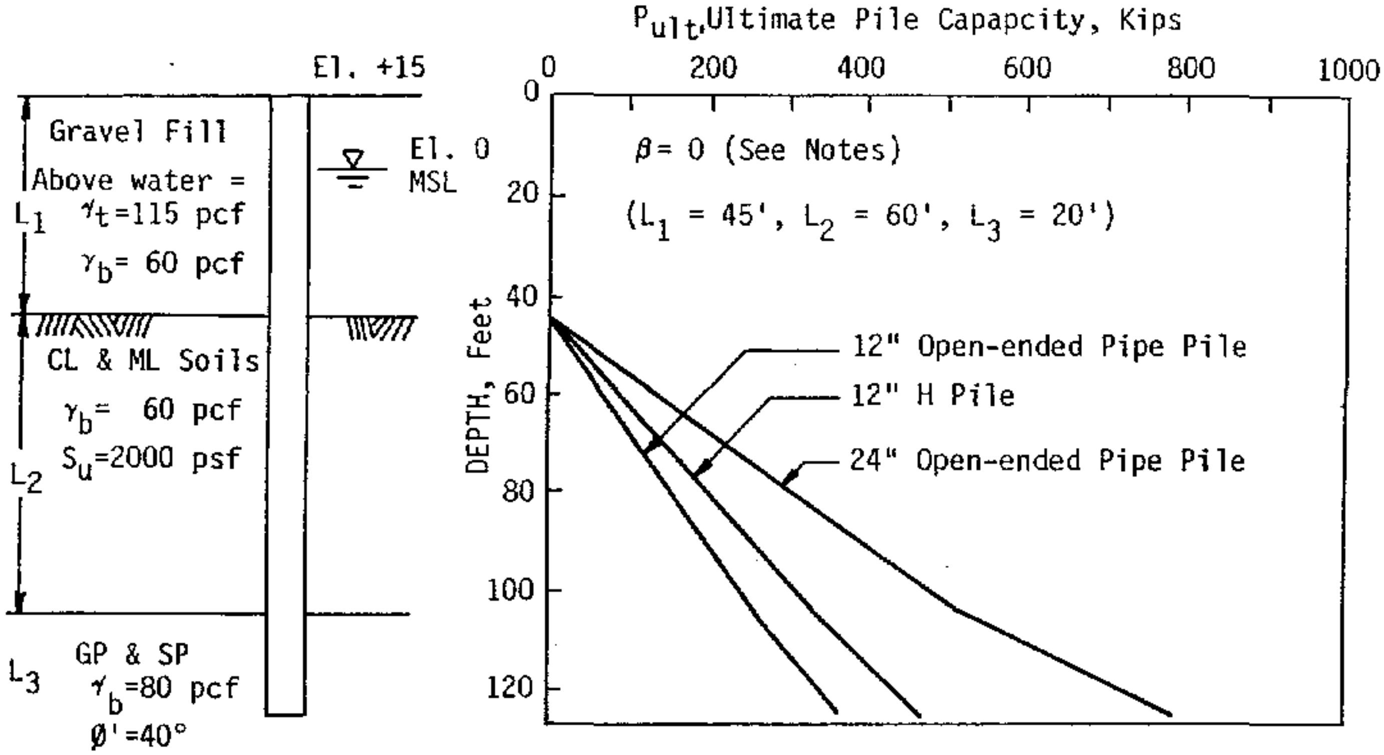
Purpose = Pile Capacity for Areas North of Barrier Islands



H & Pipe Piles: Pult = $R\left[(1.13\,L_1 - 4)\beta + 1.4\,L_2 + 1.6\,L_3\right]$ Kips
(10" to 14") where R = pile perimeter in feet

24" Pipe Piles: Pult = $2\pi\left[(2.0\,L_1 - 12.2)\beta + 1.4\,L_2 + 2.0\,L_3\right]$ Kips

NOTES:

1. For allowable design load, use safety factor of 2.0 for dead load and 1.5 for dead + live load.

2. β = side friction factor to account for axial capacity or downdrag force on the pile from the gravel fill.

   For ice rich gravel fill = β = 0 if fill remains frozen
   β = -0.2 if fill becomes thawed (24" piles)
   β = -0.3 if fill becomes thawed (12" piles)

   For fill with low ice content = β = 0.5 if pile predrilled thru fill
   β = 1.0 if no predrilling nor jetting.

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Pile Capacity–for Areas North of Barrier Islands
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creep movement at the pile tip. At this time, there is insufficient information available to analyze creep settlement from a concentrated load at the pile tip. A pile load test is warranted before final pile design. To avoid this problem, a larger pile can be used to derive adequate frictional resistance in the unfrozen soil.

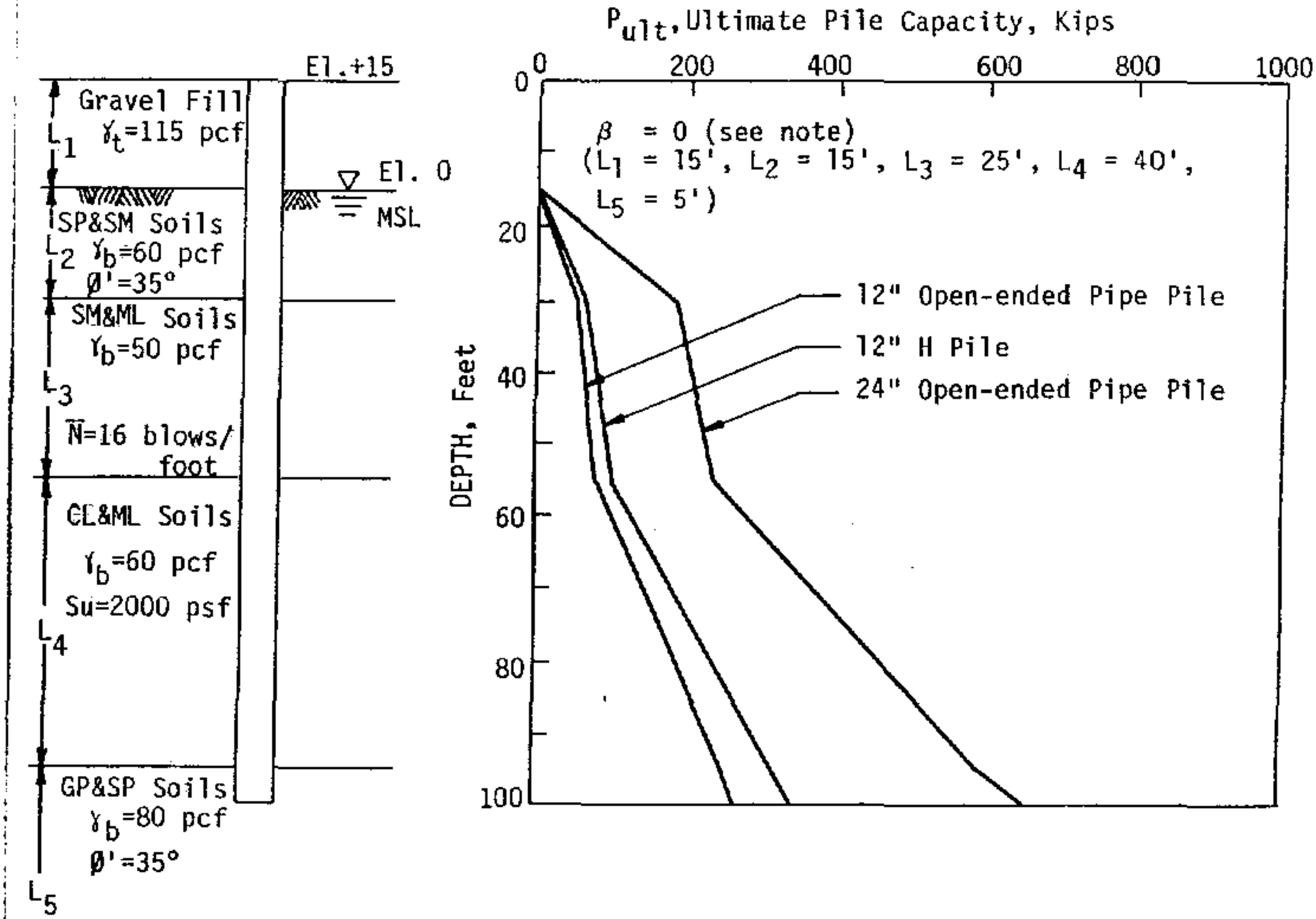
For pile groups, we recommend a center-to-center spacing of at least three pile diameters. A group efficiency of 1.0 can be used for preliminary design (Morgenstern, 1980).

The settlements for a single pile and a group of four piles embedded within the stiff clay stratum are estimated to be on the order of 1 and 2 inches, respectively. For piles penetrated into the underlying dense sands and gravels, the corresponding settlements may be reduced by one half. These settlements are the result of soil compression beneath the pile tip and do not include the elastic compression of the pile.

The design soil profile for the barrier islands is shown on Plate VI-9. Permafrost is present at shallow depths and includes a zone of unbonded soil of varying thickness. The unbonded zone may extend from 10 to 30 feet below the islands surfaces and is comprised of SP-SM and SM-ML soil layers. The underlying stiff clay layer (CL-ML soil) is bonded, at a temperature roughly $3^{o}C$ colder than the calculated freezing point as shown on Plate IV-12.

The pile design criteria shown on Plate VI-9 were developed using unfrozen soil strength parameters. Unfrozen strength criteria were used in lieu of frozen strength parameters because, for a design temperature of $-3^{o}C$, the frozen long-term strength approaches the unfrozen strength at high confining pressure (Alkire and Andersland, 1973). A friction angle of 35^{o}

Axial Pile Capacity at Barrier Islands



H Piles & Pipe Piles = Pult = $R\,[13\beta + 1.13\,L_2 + 0.32\,L_3 + 1.4\,L_4 + 1.13\,L_5]$ Kips
where R = pile perimeter in feet

24" Pipe Piles = Pult = $R\,[18\beta + 2.0\,L_2 + 0.32\,L_3 + 1.4\,L_4 + 2.0\,L_5]$ Kips

NOTES:

1. For allowable design load, use F.S. = 2.0 for dead load, and F.S. = 1.5 for dead + live load.

2. β = side friction factor to account for contribution from gravel fill
   For ice-rich fill = β = 0 if fill remains frozen
   For fill with low ice content = β = 0.5 if pile was predrilled thru fill
   β = 1.0 if no predrilling nor jetting
3. If the gravel fill and permafrost become thaw, the pile capacity and thaw-strain settlement should be evaluated on individual basis.

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Axial Pile Capacity at Barrier Islands
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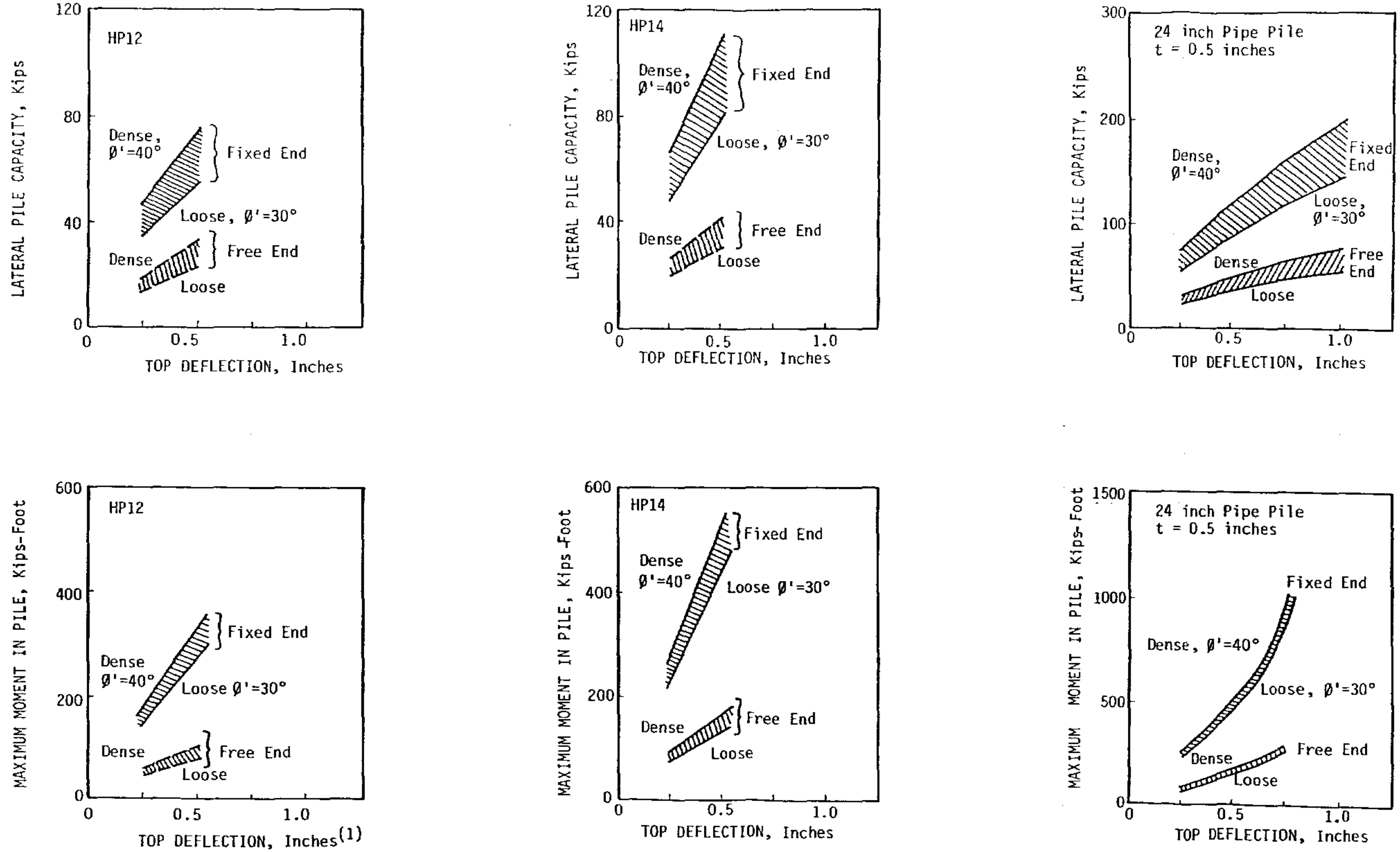
was used in the analysis based on data from Alkire and Andersland which indicates that the long-term friction angle for frozen soil is roughly 6° less than the thawed value. We believe that soil strength defined by this friction angle is less than the ultimate soil creep strength. However, large deformation may occur and creep settlement may govern pile design. Pile load tests should be performed to determine pile load capacity based on limiting creep deformation.

b. Lateral Capacity

The lateral pile capacity is largely governed by the soil properties near the ground surface. In late winter when the ground is frozen, the lateral capacity is very high and large transient loads can be resisted. The actual capacity is a function of the magnitude and duration of the load, the ground temperature and the quality of the gravel fill. Due to the uncertainties concerning the effects of these variables, analyses were not performed for the frozen gravel.

In the late summer when the thaw depth is greatest, thawed soil properties should be used in the design to resist lateral loads. Lateral capacities and pile moments for this case were calculated using the method described by Matlock and Reese (1961). Two uniform unfrozen soil profiles, a dense sand ($\phi' = 40^{\circ}$) and a loose sand ($\phi' = 30^{\circ}$), were analyzed to establish the upper and lower bound values for lateral pile loads applied at the ground surface with either fixed or free end conditions. Results of the analyses are shown on Plate VI-10. These curves are approximate but appropriate for preliminary design purposes for both "fixed" and "free end" piles. We recommend that a safety factor of 1.2 and 1.5 (for transient and sustained loads, respectively) be applied to the ultimate values obtained from the charts.



(1) Top deflection defined at soil surface

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Lateral Pile Capacity

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PLATE **VI-10**

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2. Shallow Foundations

The suitability of shallow foundations for support of structures on the islands depends upon a number of factors, including:

- type of fill material;
- season of construction;
- placement of fill and compaction methods;
- thermal regime;
- magnitude and duration of foundation loads;
- tolerance of structures to total and differential movements.

a. Allowable Bearing Pressure

The allowable bearing pressure is governed either by the maximum allowable pressure for bearing stability or the pressure that will cause a settlement of one inch. Analyses were made for the bonded and unbonded conditions which establish upper and lower limits on the allowable bearing pressures. The fill properties used in the analyses are described in Chapter V.

The bearing stability analysis is computed using the classical bearing capacity theory for continuous footings (Terzaghi and Peck, 1967). Consideration was given to different footing widths, embedment depths, and the different fill properties beneath the footing. Analyses were made for unfrozen fills only, which is the governing case for bearing capacity.

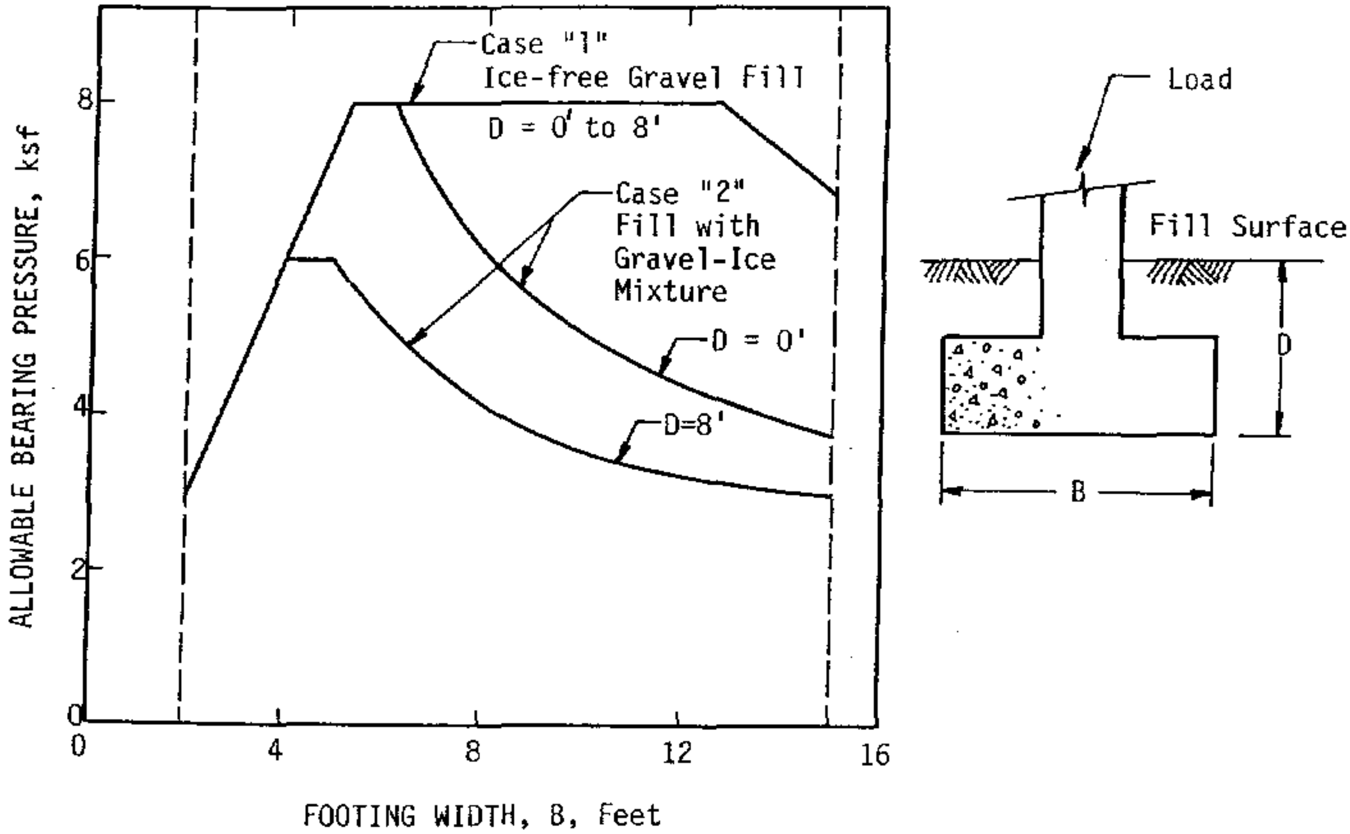
Settlement analyses were made for both bonded and unbonded conditions. For the bonded condition, the settlements are based on creep deformation theory using the cavity expansion method proposed by Ladanyi (1975). An average bonded fill temperature of $-5^{0}C$ and a creep period of 25 years were assumed for all cases. The effects of footing embedment depth were assumed to be self compensating. Increasing the embedment depth reduces the settlement in the upper unbonded layer but increases the stress (and therefore the creep settlement) on the bonded portion of the fill. Within the limits of accuracy of the creep parameters and the computational methods, the computed creep settlements are assumed to be equally valid for surface or or embedded footings. For the unbonded condition, settlements were computed using Janbu's method (1967) with the compression modulus data from Chapter V. Settlements were computed for footings placed on the surface of the fill and at the bottom of the active layer, which was assumed to be eight feet deep.

Results of the analyses are presented on Plate VI-11, which shows the variation of allowable bearing pressure with footing width, embedment depth and the type of fill. For narrow footings, bearing stability governs; for larger footings, creep settlement controls.

If the fill is frozen when the footings were placed and then subsequently thaws, large thaw-strain settlements are anticipated. The settlement can be computed as previously discussed.

b. Lateral Resistance

The footings can resist lateral loads by frictional resistance on their base. We assume the foundation type will be either timber or steel grillages. Frictional resistance can be calculated by multiplying the structural dead load by an appropriate friction factor. Footings bottomed on dense sandy gravel can be designed using a friction factor of 0.25 and 0.30 for steel and wood, respectively. These friction factors include a factor of



NOTES:

1. Allowable bearing capacity values include a factor of safety of three for dead plus sustained live loads.

2. For total design loads including wind and seismic, the allowable bearing capacity can be increased by 50%.

3. Allowable bearing capacity is based on a total settlement due to footing pressure of one-inch.

4. Footings should be at least two feet wide, and loaded areas larger than 15 feet wide should be evaluated on an individual basis.

5. See Plate V-3, for descriptions of island fill cases "1" and "2".

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Bearing Capacity Chart for Shallow Spread Footings

Pt. Thomson Development Project, Winter 1982 Geotechnical Study, EXXON Company, U.S.A.

PLATE **VI-11**

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safety of two and are based on published data for unfrozen soil (Tomlinson, 1969). When the soil freezes, the footing will be restrained by an adfreeze bond substantially larger than the unfrozen frictional value.

c. Construction Considerations

The two probable fill sections illustrated on Plate VI-1 are assumed to have a compacted, ice-free gravel pad at the surface. This gravel pad is considered to be non-frost-susceptible, relatively moisture-free and thick enough to insulate the underlying fill material from seasonal freeze-thaw effects. There is no restriction on embedment depth for footings founded in this material. However, if the fill contains excessive fines, the footing should be placed below the depth of annual freeze-thaw to minimize frost heave movements. Alternatively, insulation can be installed in the fill near ground surface to limit the depth of summer thaw.

D. Retained Fill on the Barrier Islands

On the barrier islands the fill may be retained with sheet pile walls. The depth to permafrost beneath the islands varies from 0 to 30 feet. In many areas, a thaw zone is sandwiched between two frozen layers. Cantilever sheet pile walls with a wall height of 15 feet above ground and a uniform surcharge load of 400 psf on the surface, were analyzed assuming that the soil was not frozen. Since the data are insufficient to define the thawed properties of the natural sediment and the soil is frost susceptible and may have been weakened by freeze-thaw cycles, a friction angle of 35 degrees was assumed in the analysis.

Results of the analysis are presented on Plate VI-12. The required pile section is PZ-27 with high strength steel (ASTM A690 steel). The required depth of penetration is 17 feet below ground surface. We recommend that a 3 foot gravel blanket be placed in front of the wall to reduce wave and ice scouring. Periodic maintenance will be required to restore the gravel blanket. To reduce excessive wall movement and stresses, backfill behind the wall should consist of non-frost susceptible gravel fill with low ice content, compacted to at least 50 percent relative density within a horizontal distance of one-half the wall height.

Hard driving is anticipated in the permafrost and it may be necessary to steam thaw the soil before driving.

E. Dock Structure

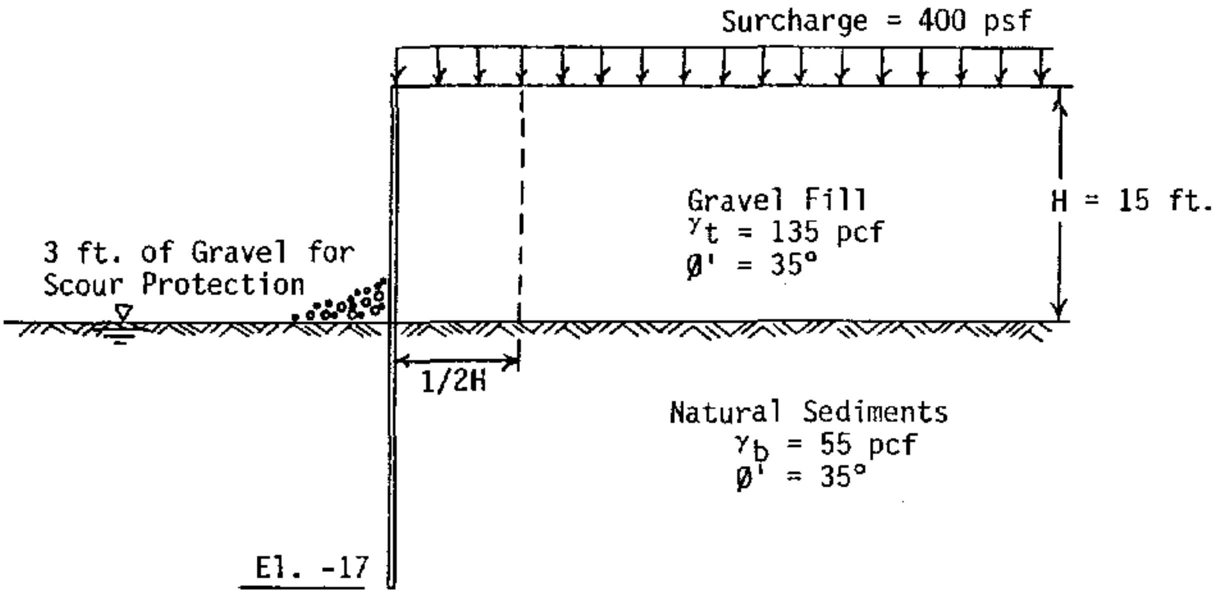
A dock structure may be constructed at an artificial island, at the end of a causeway extending from the shore or at the barrier islands.

Cellular cofferdams and pile-supported relief platforms are possible alternatives. This study only analyzes the relief platform but for final design the relative economy of both alternatives should be evaluated.

1. Relief Platform

Anchored bulkheads were analyzed to retain fill and to support a surcharge load of 400 psf. A pile-supported relief platform can be used to support the loading and unloading of the heavy modules. The relief platform can be supported on "H" piles or pipe piles. Axial load capacities can be determined from Plates VI-7 through VI-9, as previously discussed. The lateral load capacities presented on Plate VI-10 are only applicable for lateral loads directed away from the wall.

The retaining walls were analyzed using the free earth support method with Rowe's moment reduction. Results of the analyses for bulkheads constructed at the barrier islands, in the lagoon, and in areas north of the barrier islands are presented on Plates VI-12, 13 and 14.



Method of Analysis	=	Conventional Method for Cantilever Walls
Reference	=	USS Steel Sheet Piling Design Manual, 1975
Piling Selection	=	PZ-27 piling (ASTM A690 Steel)
Embedment Depth	=	17 feet below natural ground surface
Maximum Wall Moment	=	1230 inch-Kips per foot
Backfills	=	Non-frost susceptible ice-free gravel should be used within a distance of one-half the wall height.

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Sheet Pile Wall at Barrier Islands
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PLATE **VI-12**

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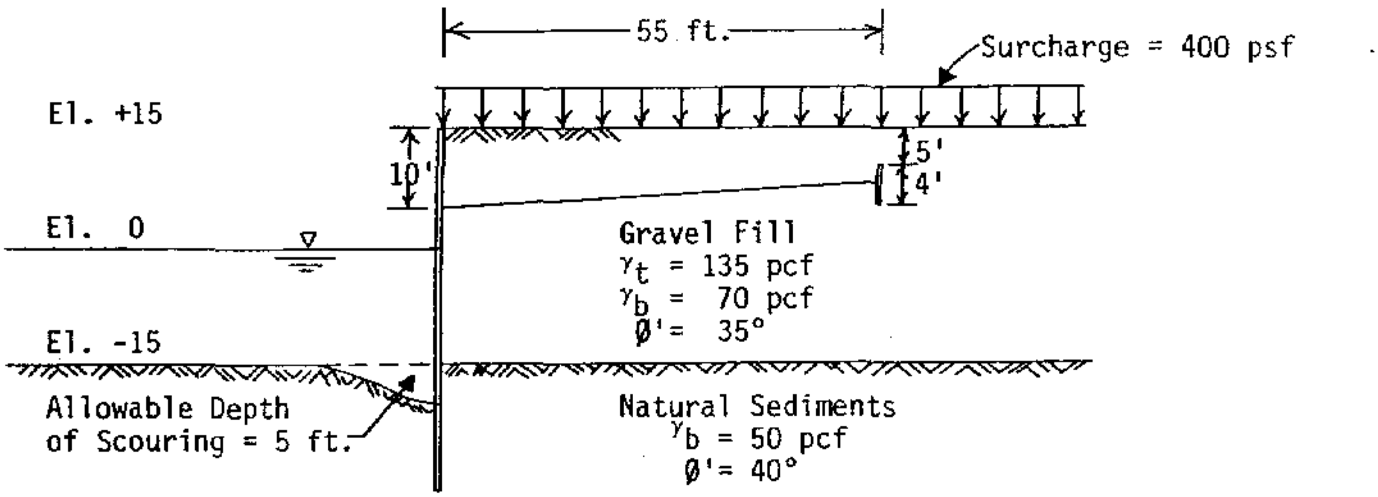

Depth of Penetration = 16 ft.

Method of Analysis = Free Earth Support Method with Rowe's Moment Reduction

Sheet Pile Wall = PZ-27 (ASTM A690 Steel)
Embedment Depth = 16 ft. below seabed
Maximum Wall Moment = 1000 in-Kips/ft.
(before moment reduction)
= 670 in-Kips/ft.
(after moment reduction)

Tie Rod = Use 2-1/2 inch diamter rod upset to 3-1/4 inch (A36 Steel)
= Rod Spacing = 6 ft.
= Rod Tension = 108 Kips per rod (F.S. = 1.3)

Wales = Use 2 C12x20 Channels bolted together back-to-back

Anchor Wall = Continuous 4-foot wall with top of wall at 5 feet below surface.
Use PZ-27 piling with A328 Steel

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Anchored Bulkhead in Lagoon Area
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **VI-13**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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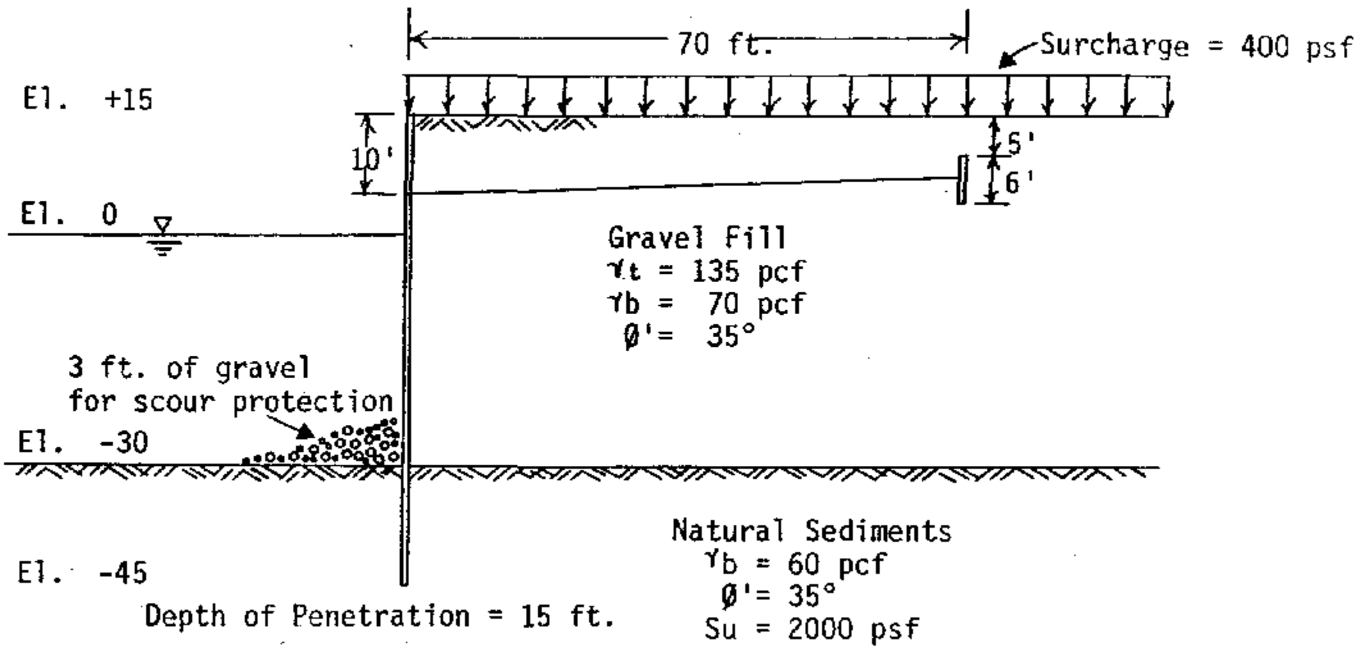

Method of Analysis = Free Earth Support Method with Rowe's Moment Reduction

Sheet Pile Wall	= PZ-38 piling with 1-in. cover plate (ASTM A690 steel) Embedment Depth = 15 ft. below sea level Maximum Wall Moment = 2800 in-Kips/ft. (before reduction) = 2382 in-Kips/ft. (after reduction)
Tie Rod	= Use 3-inch diameter upset to 3-3/4 inch. (A36 Steel) Rod Spacing = 6 ft. Rod Tension = 160 Kips per rod (F.S. = 1.3)
Wales	= Use 2 C12x25 Channels bolted back-to-back
Anchor	= Continuous 6-foot wall with top of wall at 5 feet below surface. Use PZ-27 piling (ASTM A328 Steel)

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Anchored Bulkhead -
North of Barrier Islands

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PLATE **VI-14**

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In general, the gravel fill will provide good support for the tension rods. In areas where thaw-strain settlement is anticipated, a protective pipe can be installed for each rod. The inside diameter of the pipe should be larger than the anticipated settlement so that the tie rod will be free inside the pipe as the ground and pipe settle.

2. Backfill Material and Compaction Requirements

We recommend that a non-frost susceptible and low ice content gravel soil (less than 3 percent finer than 0.02 mm by weight) be used as backfill behind the wall for a distance of one-half the wall height or greater. The same fill material should be used in front of the anchor wall for a distance equal to two times the wall height and a depth extending from the surface to the bottom of the anchor wall. Fill placed below water is assumed to be uncompacted; fill placed above water should be compacted to a relative density of at least 0.5.

3. Lateral Ice Load

The effects of lateral ice loads on the wall were not analyzed. According to the Prudhoe Bay Unit Waterflood Study (HLA, 1981), a similar structure with unfrozen backfill would displace about 4 feet under an ice load of 300 kips per foot. In the Prudhoe Bay waterflood analyses, it was assumed that the vertical piles supporting the relief platform behind the wall would not provide any lateral resistance. The wall moment induced by the ice loads is less than the maximum allowable moment. When the gravel fill becomes frozen, the stiffness and strength of the frozen materrial will probably be higher than that of the ice acting on the wall. Therefore, we do not anticipate any significant yielding of the frozen fill; the amount of wall movement and the induced moment on the wall would be insignificant.

4. Corrosion of Steel Sheet Piling

The steel pilings exposed to open water are susceptible to corrosion. Corrosion experience in the Arctic is limited, but according to a study by Mullen (1968) the average rate of corrosion in temperate climates for piling of marine grade steel (ASTM A690) within the splash zone is on the order of 0.01 inches per year. If the design life for the piling is 30 years, the required wall thickness would be on the order of 0.3 inch in addition to the thickness required to carry the design moment. However, because of the short open-water season, the rate of corrosion could be much slower than that recorded in the temperate zone. The additional thickness requirement may be drastically reduced. In lieu of the uncertainties involved, we recommend that heavier piling and high carbon, marine grade ASTM A690 steel be used. The application of protective coatings and cathodic protection along with periodic inspections are also recommended.

5. Depth of Scour

The sheet pile walls in the lagoon area are designed for a scour depth of 5 feet. For bulkheads located in areas north of the barrier islands, a protective gravel blanket or sand bags should be placed in front of the wall for scour protection. Periodic measurements should be made to monitor the depth of scour; we recommend that this be done at least annually before the barges arrive each summer. The scoured area in front of the wall should be backfilled when the scour depth approaches the allotted design depth.

F. Geotechnical Considerations for Causeway Design

1. General

Causeways may extend from the shore into the lagoon area or extend offshore from the barrier islands. Causeways may be used as roads to barrier islands and offshore islands or protection for subsea pipelines. Assuming the causeways are built of the same gravel fill used in the island construction, all analyses related to gravel island construction are applicable to the causeway; therefore, the same design considerations and parameters can be used.

2. Pipeline Burial

Pipelines can be buried beneath or within causeways. There are two disadvantages to burying the pipelines beneath causeways. First, the pipeline may experience uneven settlement due to compression settlement of the sediments under the weight of the causeway fill, which may induce excessive stresses in the pipe. Secondly, thaw-strain settlements may occur in areas where the permafrost is shallow. It is preferable to bury the pipelines within the embankment.

If the pipeline is buried in the embankment, the fill should be placed in the summer so that it can be properly compacted. Fill placed during winter will probably have unacceptable thaw settlement properties unless extreme care is taken to use gravel having a low ice content and to avoid incorporating ice and snow in the embankment.

A pipeline placed above sea level in a causeway will allow excavation of the trench, placement of the necessary insulation, and installation and burial of the pipe in dry conditions. The higher elevation would also improve thermal insulation of the pipeline.

3. Causeway Breaches

Two typical methods to breach through causeways are culverts in shallow water and bridge crossings in deeper water.

a. Culverts

Circular or elliptical culverts can be used to allow fish passage and current flow through the causeway.

The culverts should be constructed on level, firm bedding. The bedding material should be easily compacted under water and have low erosion potential. We recommend that clean, ice-free gravel be used for bedding material and the maximum size of individual particles not exceed 1-1/2 inches. The bedding material should be uniformly compacted to a dry density equivalent to a relative density of 50 percent or greater.

Large multi-plate culverts at Prudhoe Bay have failed due to inadequate backfill support. Frozen fill material should be avoided since the frozen material will thaw behind the culvert and result in a loss of lateral support. Summer construction is recommended because it is very difficult to prevent ice and snow from entering the fill during winter construction.

The area adjacent to and below the spring line of the culvert should be backfilled with properly compacted gravel. The fill should be compacted to 70 percent relative density.

b. Bridge Structure

A typical bridge structure consists of a clear span deck supported on piles. The abutments would be cantilever sheet pile walls backfilled with gravel and the surface of the bridge deck would be level with the causeway probably at about Elevation +15.

(1) Driven Piles

The bridge can be supported on either "H" piles or open-ended pipe piles. The piles will probably be located behind the sheet pile wall to protect them from sea ice loads. The ultimate axial load capacities for different piles and sizes presented on Plates VI-7 and VI-8 are suitable for preliminary design.

(2) Cantilever Sheet Pile Wall for Bridge Abutments

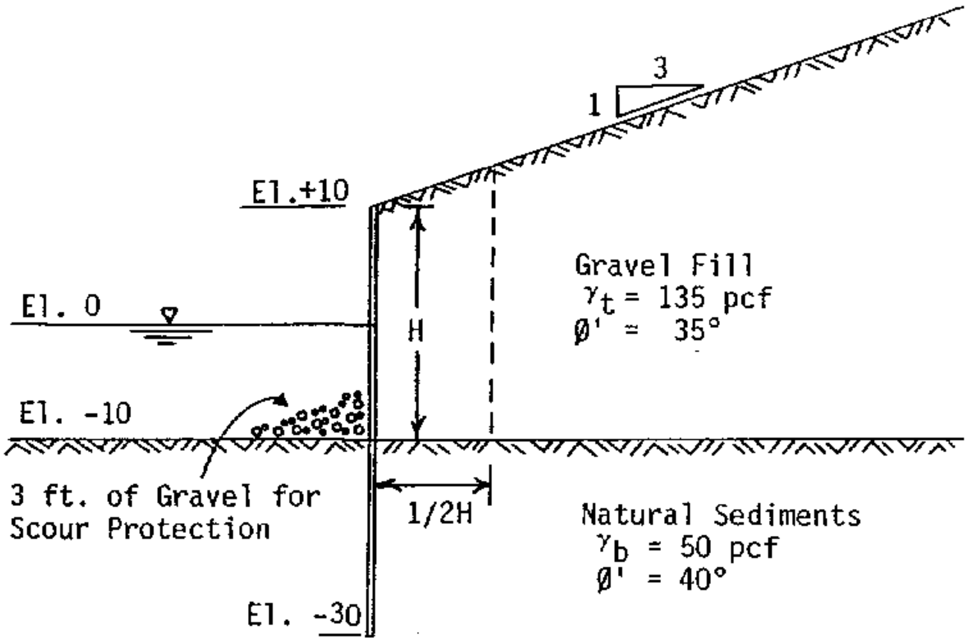
The top of the cantilever sheet pile wall for the abutments is assumed to be at Elevation +10 with backfil slopes of 3:1. The required pile length is 40 feet, and the depth of embedment below mudline is 20 feet. The required pile section is PZ-38 (ASTM A690 steel). A gravel blanket (3 feet thick) should be placed in front of the wall for scour protection and should be checked and maintained periodically. Non-frost susceptible gravel fill with low ice content should be used as backfill for a horizontal distance behind the wall equal to one-half the wall height or greater. Fills above water should be compacted to a relative density of 0.5 or greater. The design requirements are presented on Plate VI-15.

G. Offshore Pipelines

1. Introduction

A system of offshore pipelines will connect offshore drill pads and islands with the onshore production facilities. These pipelines will include produced fluid lines, waterflood lines, artificial gas lines, operating fuel lines, and power and communication lines. As in the Duck Island Development area, the size of the pipes may vary from 2 inches in diameter for the power and communication lines up to 24 inches in diameter for the produced fluid lines.

Because island and drill pad locations have not been identified, the location of pipeline routes are unknown at this time. However, it can be assumed that pipelines will extend north offshore from the barrier islands, between individual barrier islands and from barrier islands and drill pads



Method of Analysis	=	Convention Method for Cantilever Walls
Reference	=	USS Steel Sheet Piling Design Manual, 1975
Piling Section	=	PZ-38 Piling (ASTM A690 Steel)
Embedment Depth	=	20 feet below seabed
Maximum Wall Moment	=	1170 inch-Kips per foot of wall
Backfills	=	Non-frost susceptible ice-free gravel should be used within a distance of one-half the wall height.

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Sheet Pile Wall for Bridge Abutments
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PLATE **VI-15**

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located in the lagoon to an onshore location. We have assumed that, for protection from ice movement, all lines will be buried below the sea floor or in gravel causeways. Water depths in the lagoon area vary from 5 to 15 feet. Outside the barrier islands, water depths extend up to 35 feet. The final configuration of piping could be either individual pipelines laid parallel in the same trench or a bundled set of pipelines laid as a single unit.

Operating temperatures of the pipelines will vary from approximately ambient ground temperature for utility lines to about 200°F (93°C) for the produced fluid lines. The waterflood intake line will be cool, approximately 50°F (10°C) (Exxon Duck Island Report, 1981). The sales gas and oil will be transported to shore via an offshore pipeline corridor or a causeway pipeline route.

The following sections present general design and construction considerations for pipeline burial and restraint based on the soil conditions encountered in our test borings. A study of sea floor hazards unique to ice-covered coastal waters such as ice gouging, strudel scour, and ice pounding was not within the scope of this work. Rather, we have assumed that these processes may occur to some degree; consequently, sea floor protection such as pipeline burial is required. A detailed discussion of these processes has been presented in the DID report and will not be repeated here.

2. Soil Conditions

Within the expected maximum depth of pipeline burial, 15 feet below the sea floor, soil conditions vary considerably. Geologically, the surface soils consist of Holocene and Pleistocene nearshore marine and deltaic deposits. The soils are predominantly mixtures of unfrozen silt and fine sand, small amounts of clay and pockets and layers of organics. Boulders and cobbles associated with the Flaxman lag formation are anticipated to be present on the sea floor within the PTD area, but none were encountered in the sediments beneath the sea floor.

Below the pipeline burial depth, soil conditions are also heterogenous; for example, untypical shallow gravel alluvium was encountered in Borings 11, 17 and 20. Permafrost conditions are also quite variable. No permafrost was encountered within the lagoon area up to the maximum depth drilled (80 feet). The depth to permafrost varies from 0 to 30 feet at the barrier islands. Along the barrier islands shoal area and along the shoreline, the annual bottom-fast ice and shallow water appear to be causing an aggrading permafrost condition with bonded soils becoming thicker with time. The surface of bonded permafrost within areas of shallow waters is expected to be a few feet below the sea floor.

3. Pipeline Burial and Restraint

Excavating the silt and sand to the depth expected for pipeline burial, up to 15 feet, should pose no unusual problems for equipment such as draglines, hydraulic dredges, or plows. The stiff clay associated with the sea floor outside the barrier islands will be more resistant to excavation than the silt and sand.

Boulders and cobbles might be present on the sea floor as discussed in Chapter III. After initial pipeline corridors have been selected, the routes should be mapped and the location of boulders and cobbles identified. Prior to construction it may be necessary to rake those segments of the alignments where boulders are expected, so that they do not interfere with excavation or cause pipe bedding problems.

The side slopes of the trench are expected to stand at between 2:1 and 3:1. Based on observed slopes of 20^0 in strudel scour holes in the DID area, 3:1 side slopes should result in a conservative estimate of excavation and backfill quantities. Trenches will tend to infill and slopes will flatten during heavy wave action.

If gravel is used for backfill, we assume that the sand and silt excavated from the trench will be cast to the down-current side of the trench so that currents and waves do not rework the material back into the excavation. In most cases the down-current side will be either shoreward or westward of the trench. The down-current sediment plume will depend on the settling velocity of the excavated material and the turbulence of the water. Settling velocities for sand, silt, and clay are discussed in Chapter V.

The pipeline will be installed in the trench and backfilled at seawater temperature. As indicated by the ground temperatures, seawater varies from about 28^0F (-2^0C) in winter to about 36^0F ($+2^0$C) in summer. When the pipeline begins operation, the flow of oil through the newly constructed pipe will result in two additional design considerations regarding soil-pipeline interaction:

1. During operation, the flow of relatively hot oil will increase the temperature of the pipeline to at least 200^0F (93^0C). Longitudinal thermal expansion of the pipeline steel will impose substantial forces on the surrounding soil.

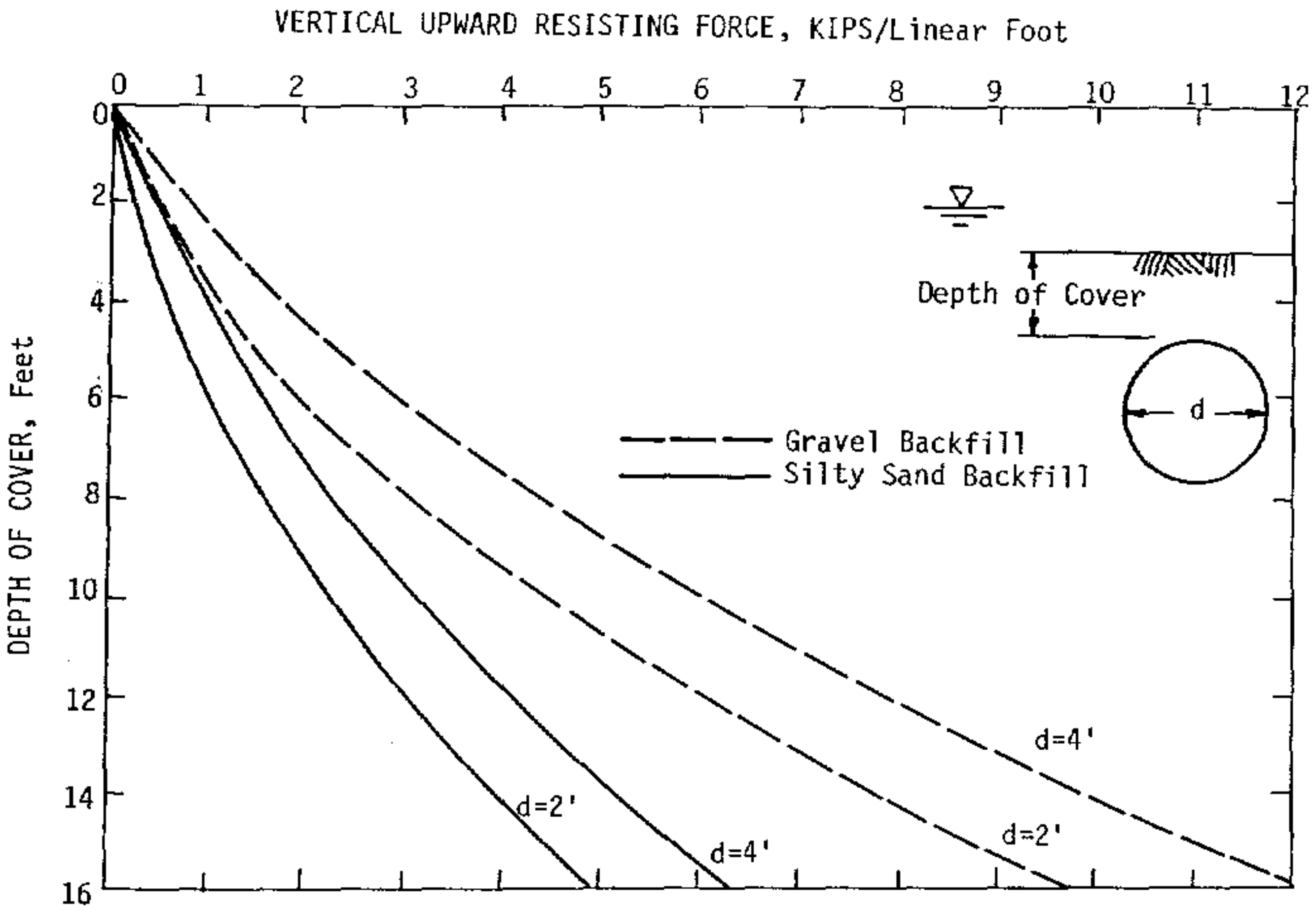
2. The pipeline will impose lateral forces on the adjacent soil resulting from surges and momentum changes at pipeline bends.

Without proper restraint, pipeline buckling could result from these phenomena. Pipeline restraint offered by the surrounding soil can be separated into two cases: (1) vertical restraint, upward or downward and (2) horizontal or lateral restraint.

Upward vertical restraint to pipeline movement will be provided by the weight of the soil wedge resting on top of the pipeline and the weight of any material incorporated in the pipeline design. In order for the pipeline to move vertically, it must lift the soil wedge. The geometry and weight of the soil wedge is dependent on the strength and material properties of the overlying soil. In our analysis, we have assumed that the pipeline is buried under either a sandy gravel fill or fine sand placed at a relative density of 50 percent with no compaction. The results of our analysis of vertical pipeline restraint versus depth of pipeline cover are shown graphically on Plate VI-16.

Downward vertical resistance is calculated using bearing capacity values for a strip footing. For silty sand, we have assumed a conservative angle of internal friction of 35 degrees to compute the resisting force shown on Plate VI-17. For a clay subgrade at the bottom of the trench the bearing resistance per lineal foot of pipe should be reduced to 2000 pounds times the pipe diameter in feet. These bearing capacity values include a factor of safety of three as compared to the ultimate values.

Horizontal or lateral pipeline restraint varies with pipeline strain, or movement, through the surrounding soil mass. Assuming that the pipeline will have long radii of curvature at bends, it can be treated as a long continuous anchor slab in estimating the ultimate resisting load. The lateral



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Vertical Upward Resistance of Buried Pipeline

Pt. Thomson Development Project, Winter 1982
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PLATE **VI-16**

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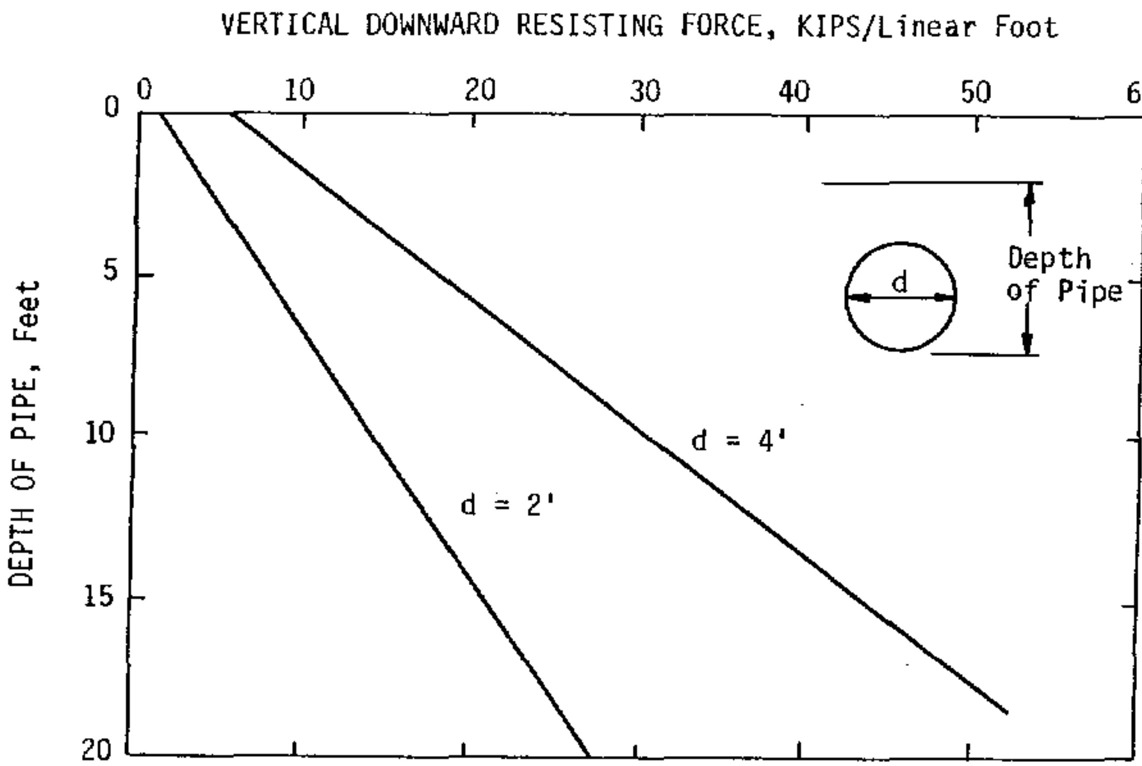

NOTES:

1. Values are for sandy trench bottom.

2. For clay trench bottom the resisting force is 2d Kips/Linear Ft.

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Vertical Downward Resistance of Buried Pipeline

Pt. Thomson Development Project, Winter 1982
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PLATE **VI-1**

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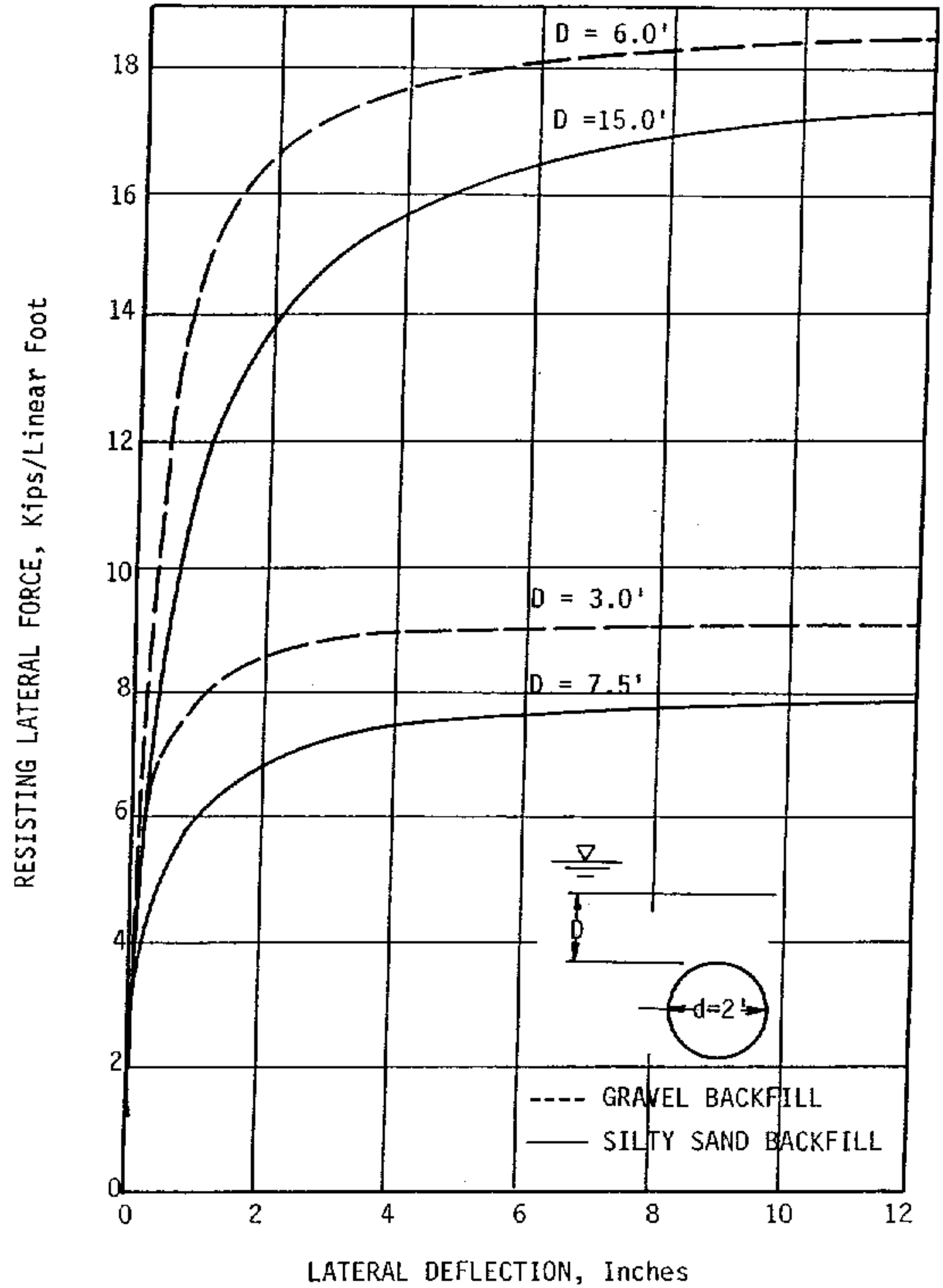
soil resistance was analyzed using methods developed by Audibert and Nyman (1977) to determine the variation in resisting lateral force with pipeline strain at various burial depths. The analysis was performed for gravel backfill and sand backfill cover depths of 3 to 15 feet using a two-foot-diameter pipeline. The results of the analysis are shown on Plate VI-18.

Resistance to pipeline expansion will also be supplied by longitudinal frictional forces between the pipe coating, or protective wrap, and the adjacent soil. This resistance will be a function of soil pressure against the pipe and the friction angle between the outer pipeline material, which is unknown at this time, and the surrounding soil. The soil pressure will vary from full effective overburden pressure on the top and bottom of the pipe to an at-rest pressure on the sides of the pipe.

If multiple pipelines are laid parallel on the trench floor, additional analysis should be performed to determine interactive forces that may affect pipe restraint in all directions. These forces are dependent upon pipeline spacing and relative movement and cannot be ascertained until a more specific pipeline design is established.

4. Thaw Strain

Thawing of the permafrost and the resulting thaw-strain below the heated pipelines will depend on heat flux (a function of the heat source and the thermal properties of the backfill around the pipe, the natural soils underneath, and the permafrost) and on the thaw-strain characteristics of the permafrost. Heat flux computations, including rates and depths of thaw, were



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Lateral Resistance of Buried Pipeline

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PLATE **VI-18**

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not within the scope of this study. The thermal and thaw-strain properties of the subsea materials have been summarized in Chapter IV. These values can be used in thermal computations to calculate total and differential settlement of a pipeline due to thawing of frozen soil.

VII ONSHORE GEOTECHNICAL CONSIDERATIONS

LIST OF TABLES

LIST OF ILLUSTRATIONS

VII ONSHORE GEOTECHNICAL CONSIDERATIONS

A. Introduction

Development of the PTD area will require construction of numerous onshore structures such as drill pads, pump station facilities, pipelines, roadways, and a base camp with associated service facilities, i.e. storage pads, a potable water source, and waste disposal site. These structures must be designed and constructed so that they will not degrade the thermally sensitive permafrost. For soil identified as thaw stable, including low ice content granular soils that are frozen below the foundation level, foundation design is basically identical to conventional practice in warm regions.

Foundation systems in thaw-unstable soil are designed to preserve the existing permafrost. In general, degradation of permafrost beneath the foundation is acceptable only when foundation materials are thaw stable, only short-term design life is involved, or the resulting settlements can be tolerated.

This chapter discusses typical foundation systems used to support structures onshore to avoid unacceptable degradation of the permafrost. General design recommendations for pile and shallow foundation systems, drill pads and roadways are presented. Geotechnical design considerations for support systems such as a potable water supply and sewage and solid waste disposal are also addressed.

B. Foundation Support

Onshore structures should be supported above the ground surface so that heat from the structures does not affect the ground temperature. The traditional means of implementing substructure ventilation is by supporting the structure on piles embedded in permafrost. Light structures can be supported on posts extending from footings founded in a gravel pad. Heavy structures supported by this method may experience large creep settlement. A clear space of at least 36 inches should be provided between the base of the structure and the ground surface. A larger separation should be provided where snow accumulates or where deep fronting members, pipe trays, or very wide structures are planned.

The onshore area of the PTD project area is underlain by ice-rich, thaw unstable, permafrost. Without a well designed insulation and cooling system warm structures at grade would cause thermal degradation of the ice-rich permafrost, leading to a myriad of engineering and environmental problems.

Experience on the North Slope of Alaska has shown that elevating warm structures above grade is usually the most economical method of insulating and cooling the permafrost. However, we have not reviewed specific cost comparison studies. If the Arctic winds have clear access beneath the structure so that snow drifts do not seal the passage in winter, the permafrost will be thermally maintained at close to its natural condition. The active layer may deepen slightly if the surface albedo changes, but, if normal air temperatures are maintained, a slightly warmer, stable thermal level will be reached within a year or two.

Most structures on the North Slope, including pipelines, are elevated above grade on piles. Occasionally it is necessary to place heated structures or pipelines at grade or below grade; in such cases the cost of artificial insulation and cooling systems is justified by special conditions.

1. Shallow Foundations

Relatively light or temporary structures can be supported on post and pad type foundations bottomed on a gravel pad. Temporary structures are structures which impose a load on the soil for less than one year. The foundations can consist of a timber or steel grillage with wood or steel columns extending above ground to support the floor and provide a ventilated airspace. Some vertical movement of this type of foundation should be expected as the gravel fill annually freezes and thaws. If footings are underlain by at least four feet of thaw stable compacted gravel fill, bearing pressures of 3000 psf can be used for total design loads. The allowable bearing capacity includes a safety factor of 3 or greater and is based on a total settlement, due to footing pressure of 1-inch. Footings should be at least 2 feet wide and loaded areas larger than 10 feet wide should be evaluated on an individual basis. Thaw stable compacted gravel fill is defined as fill material which has a dry density greater than 115 pcf.

Design of an above-grade structure may not be economical for large heated buildings with heavy floor loads and vehicle access requirements, such as garages or warehouses. For these structures, the permafrost can be maintained using an insulated floor with a cooling system below the insulation. The cooling system can consist of ventilation ducts or passive liquid-vapor heat transfer systems buried in the fill. Design of the cooling and insulating system depends on building size and amount of heat generated by the building.

2. Pile Foundations in Permafrost

One possible method and perhaps the most common type of design for permanent construction on thaw-unstable permafrost incorporates a pile foundation system with an air space between the structure and the ground surface. Possible types of piles include timber, steel pipe or "H" sections.

Piles can be installed as either augered-slurried piles or as driven piles. The piles can be designed for vertical and lateral loads using the criteria presented in the following sections.

Steel pipe, "H", and timber piles have all been used for foundation support in permafrost (Davison, 1978). Most heavy structures in Prudhoe Bay are supported on pipe piles because of their high vertical load capacity and the increased torsional and lateral resistance that their shape provides in comparison with that of "H" sections. Timber piles are frequently used for lightweight or temporary structures.

Piles are installed in permafrost by augering an oversized hole, placing the pile in the hole, and then backfilling with a sand/water slurry which freezes. Pile capacity depends upon the bond between the pile surface and the frozen slurry or, in some cases, the bond between the frozen slurry and the wall of the drilled hole; consequently, the pile system is referred to as a slurry adfreeze pile.

Pile driving methods for installing steel piles have been tried at Prudhoe Bay. Hydraulic resonant pile drivers, electric vibratory drivers, and diesel impact hammers have all been used both experimentally and for production driving. Permanent driven piles have only been used for supporting light structures and pipelines, and the history of their use has been too short to adequately evaluate their performance.

3. Pile Design for Vertical Loads

Pile foundations in permafrost must be designed to support sustained downward vertical loads and to resist seasonal uplift forces -- frost jacking -- caused by frost action in the active layer. Typical forces acting on a pile in permafrost are shown in Figure 1 on Plate VII-1. The resistance to frost jacking and vertical loading is provided by an adfreeze bond in the permafrost. This adfreeze resistance is dependent on pile diameter and type, embedment depth, slurry type if preaugered, water content of the soil, ground temperatures, construction techniques, and the duration of the applied loads. Pile settlement and pile adfreeze support capacity are two considerations for the design of piles in permafrost and are discussed below.

a. Pile Settlement

(1) Ice-Rich Soil

There is very little published load test data on the long-term use of piles in permafrost. Morgenstern, Roggensack and Weaver (1980) have performed a comprehensive review of field and laboratory long-term creep tests on ice and ice-rich soil and have proposed a flow law for piles in ice and ice-rich soils:

$$\frac{U}{a} = \frac{3^{(n+1)/2} \, BT^{n}}{n-1} \qquad \text{(VII-1)}$$

Where:
- U = pile velocity
- a = pile radius
- T = constant tangential shear stress on ice-rich soil
- n = stress exponent
- B = creep parameter at a uniform constant ground temperature

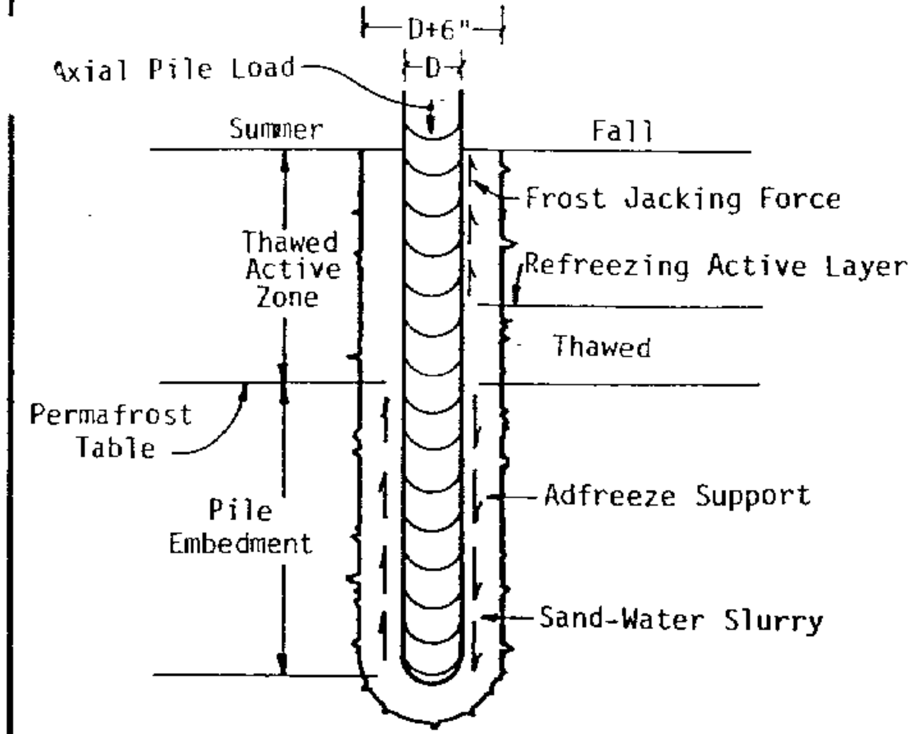
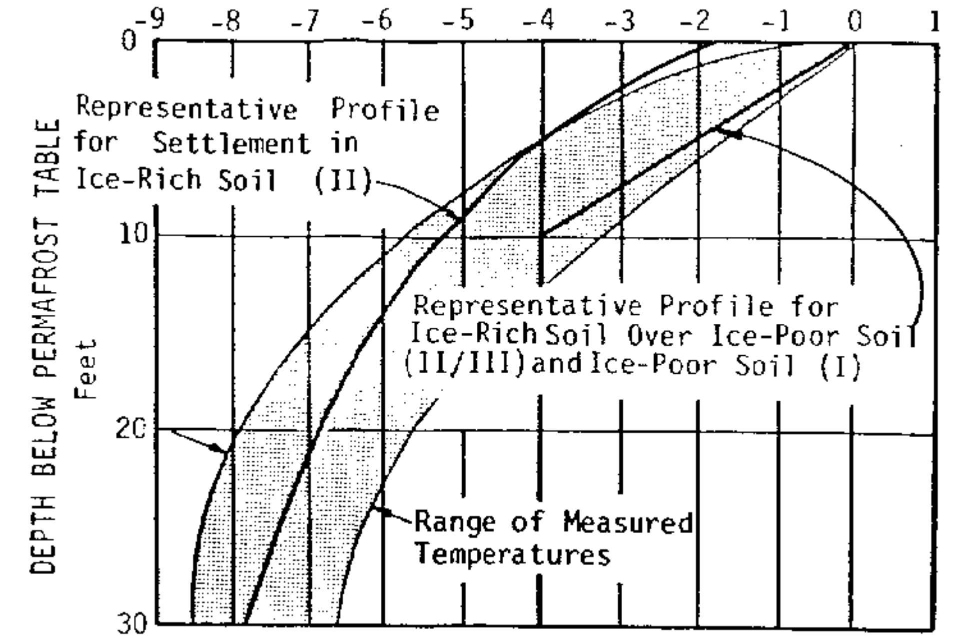


Figure 1. Schematic of onshore pile

Design Case (Soil Type)	Depth of Active Zone (Feet)	Required Pile Embedment Below Permafrost Table to Prevent Frost Jacking (Feet)
Ice-Poor Soil (Type I)	8	11
Ice-Rich Soil (Type II)	3	14
Onshore Drill Pads or Camp Sites (Type II/III)	Fill Pad Plus 1-foot; 6 feet total	14
1-Foot Ice-Rich soil over Ice-Poor Soil (Type II/III)	3	14

Table 1. Onshore pile design criteria



Fibure 2. Temperature profiles used in analysis

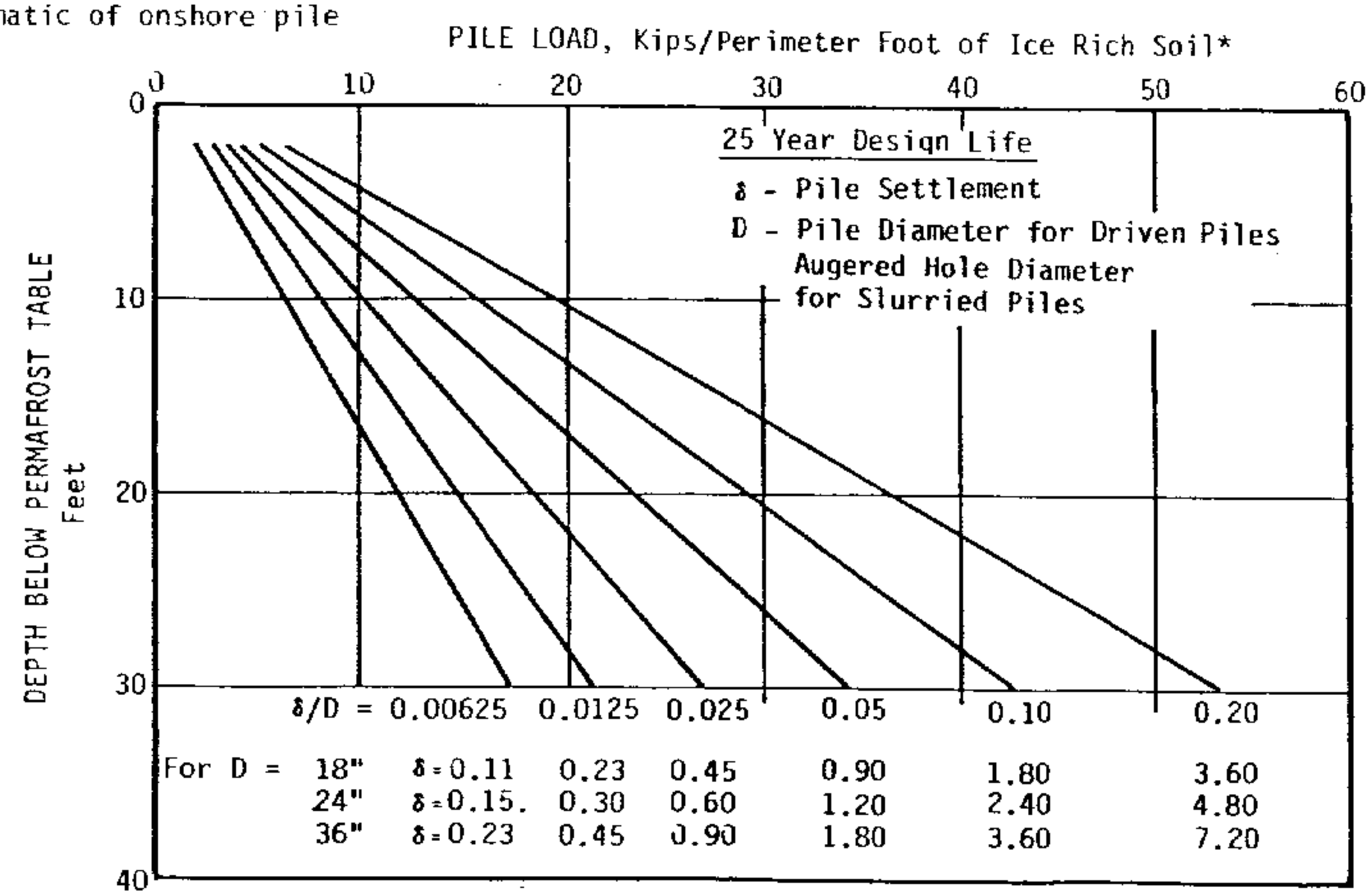


Figure 3. Pile settlement in ice-rich soil (II)

* Perimeter for adfreeze slurry piles is the auger hole size. Perimeter for driven piles is the pile size

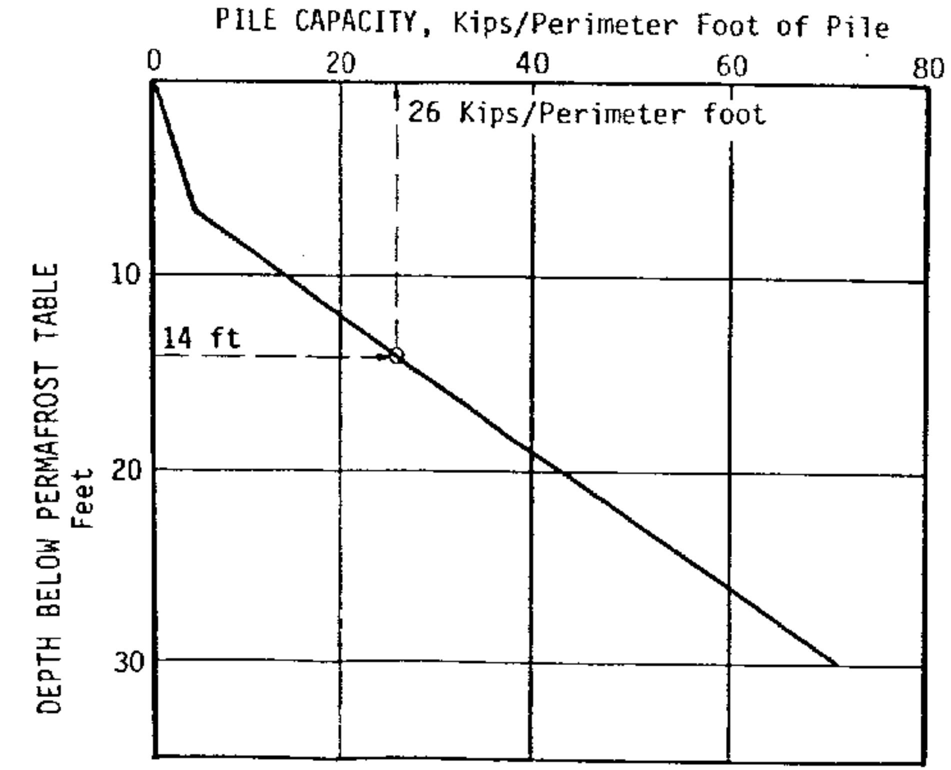


Figure 4. Adfreeze support for ice-poor soil and 7 feet of ice-rich soil over ice-poor soil

Harding Lawson Associates Engineers, Geologists & Geophysicists	Pile Design Criteria Pt. Thomson Development Project, Winter 1982, Geotechnical Study, EXXON Company, U.S.A.	PLATE VII-1

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They conclude that:

> . . . the flow law for ice constitutes an upper bound to these test data. Using this flow law, pile velocities in ice and ice-rich soils have been predicted and the predictions are shown to be in good agreement with available long term creep data for piles in ice and ice-rich soils.

Goughnour and Andersland (1968) have reported a characteristic change in the strength of frozen sand at a sand volume content of 42 percent. Soil with a sand volume concentration of less than 42 percent had a strength comparable to ice, and soil with a sand volume concentration greater than 42 percent was stronger than ice. A 42 percent sand volume concentration corresponds to a water (ice) content of about 50 percent by weight.

Based upon Goughnour and Andersland's work, soils with water contents greater than 50 percent can be considered ice-rich (Type II, as described in Chapter III) and, according to Morgenstern, should be treated as ice in designing pile foundations.

To our knowledge, this distinction between ice-rich and ice-poor soils has not been used within the Prudhoe Bay area. For some foundation installations, such as at Flow Station III, piles have been lengthened when clear ice was encountered in the auger hole. The upper few feet of ice-rich silt have been considered to be creep resistant, allowing for the difference in temperatures, as the underlying ice-poor gravels. Throughout most of the Prudhoe Bay area ice-poor gravel is shallow enough so that piles are generally installed with the bottom several feet of their tips deeper than the ice-rich soil.

Due to the depth of ice-rich soil through much of the Point Thomson area, we believe ice-rich permafrost should be defined as that soil having a water content greater than 50 percent and pile capacity should be analyzed as a creep-settlement problem. We recommend the stress exponents and creep parameters used in Equation (VII-1) based on values for ice from Morgenstern, Roggensack and Weaver (1980).

Morgenstern's flow law for ice can be used to predict the velocity at which a pile will settle if supported completely within ice or ice-rich (Type II) soil. The velocity is dependent upon the pile load, ground temperatures, and ice characteristics. Only fresh water ice has been considered. Saline ice at any given temperature would have higher creep velocities than fresh water ice at the same temperature. Using Morgenstern's flow law, predicted pile settlements for a 25-year design life are presented in Figure 3 on Plate VII-1. The settlement predictions were calculated assuming an elastic pipe pile, a constant uniform shear stress on the ice-rich soil, and the representative ground temperature profile discussed below and shown in Figure 2 on Plate VII-1. No specific safety factors have been applied to the pile load values shown in Figure 3 on Plate VII-1.

Subsurface temperatures fluctuate seasonally and cause variations in pile settlement velocity. To account for this change in settlement velocity throughout the year, a representative soil temperature profile, an average for the full year, was determined using procedures outlined in Appendix E, Volume II and summarized below:

1. The year is divided into eight time periods.
2. The ground temperature variation with depth is determined for each time period assuming a homogeneous soil beneath the active layer and using a damped, sinusoidal oscillation for temperature.

3. For a certain pile embedment depth;
   a) The settlement of the pile is calculated based upon an average temperature along the pile at each time period;
   b) The settlement for all time periods is added to determine the total settlement of the pile for the year; and
   c) This settlement is used to compute a representative ground temperature using Morgenstern's flow law.

4. Step 3 is repeated for other pile embedment depths.

Any negative effect of frost jacking on pile settlement was ignored in our analysis.

The ice-rich soil at the annulus of the auger hole governs the pile creep behavior. When properly constructed, the frozen slurry to pile bond in ice-rich permafrost is generally more creep resistant than the slurry to permafrost bond at the wall of the augered hole. Therefore, for ice-rich soil, the outside diameter of the slurry should be used to calculate pile settlement.

(2) Ice-Poor Soil

Soils with water (ice) contents of less than 50 percent can be considered ice-poor (Types I and III). There are no published data regarding long-term pile load tests in these soils. It is known that ice-poor soils have lower creep velocities than those of ice-rich (Type II) soils. Consequently, piles embedded in an ice-poor soil will settle less than those in an ice-rich soil. To date, slurry piles at Prudhoe Bay have generally been installed with the lower portion of the pile in ice-poor soils; these piles were designed using adfreeze considerations. We understand that the piles at Flow Station III, which are some of the heaviest in the Prudhoe Bay area, have had some minor settlement and heave. In August, 1979 maximum measured displacements of piles at Flow Station III were minus 0.16 inch and plus 0.29 inch.

b. Pile Adfreeze Support Capacity

In ice-poor soils (Type I or III) vertical support can be obtained below the permafrost table by tangential adfreeze between the pile and the soil or frozen slurry backfill. In general, tangential adfreeze strength increases with a decrease in temperatures below 0^oC. Load tests on piles in permafrost have primarily been performed in temperatures above -4^oC. For this reason, pile capacities discussed below have been assumed to be constant at ground temperatures below -4^oC.

The lowest adfreeze strengths occur in early winter when ground temperatures are the warmest. In Prudhoe Bay these warm temperatures usually occur in late September or early October. The subsurface temperatures measured in our borings on April 17, 1982 are close to the coldest temperatures.

Compilation of temperature data from previous studies in Prudhoe Bay indicates that the subsurface temperatures generally fluctuate around an average ground temperature of -8^oC (17.6^oF) with a surface amplitude of $\pm 10^oC$ as discussed in Appendix E. The April 17, 1982 temperature readings agree well with this generalization. Using these fluctuations the maximum subsurface temperatures were calculated for several depths and for various soil conditions. The maximum temperatures were used in our adfreeze design calculations and are shown in Figure 2 on Plate VII-1.

Linell and Lobacz (1980) have published recommendations for slurry adfreeze pile design in permafrost. These recommendations have been used to determine pile adfreeze support capacity in ice-poor soils. Adfreeze strengths used in our calculations were average values at ultimate bearing capacity for sand slurry to steel piles and divided by a safety factor of three.

Within the project area, 5 to 15 feet of ice-rich soil overlies ice-poor soil. The pile adfreeze support capacity for this case is presented in Figure 4 on Plate VII-1 and assumes the average thickness of the ice-rich soil is seven feet. An average adfreeze strength of 5 psi was assumed for the ice-rich soil.

This is more conservative than the 1979 design procedure recommended by ARCO for the Prudhoe Bay Unit. We understand that the current Prudhoe Bay Unit design assumes no adfreeze strength above a temperature of $25^{o}F$ ($-3.9^{o}C$). Further, if a layer of ice-rich soil or massive ice more than 7 feet thick is encountered during pile installation, the auger hole depth and pile length can be increased on a foot-for-foot basis or until a length is achieved that satisfies the settlement criteria for ice-rich soil.

The capacity of driven piles is determined by the adfreeze bond between the natural soil and the pile. This bond depends on soil gradation, ice content, and skin thaw occurring during placement. ARCO has recently performed pile load tests of driven piles at Prudhoe Bay. One set of driven piles was installed with a vibratory hammer and another set was installed through a small hole warmed with hot water. A formal report of these tests has not been issued and additional analysis is needed. We understand that pile capacities varied considerably and in some cases the capacity of driven piles was less than half of that for slurried piles.

Linell and Lobacz (1980) recommended that the allowable load-bearing capacity of a conventionally driven steel pipe pile should be reduced to 75 percent of that for a slurried pile in which the slurry is made from the same foundation soil mixed with fresh water. For a driven 10-inch by 10-inch steel H-pile, Linell recommends the pile capacity be reduced to 87 percent of that for a slurried pile.

To determine the design vertical load capacity of driven piles, we recommend that the support values presented on Plate VII-1 be reduced to the percentages in Table VII-1. These values should be reviewed when analysis of the ARCO pile load test data is completed.

TABLE VII-1. PILE CAPACITY REDUCTION FACTORS FOR DRIVEN PILES

Foundation Soil Type	Steel Pipe Piles	Steel H-piles
Ice-poor sand	75%	85%
Ice-poor silt	50%	60%

c. Frost Jacking

During the summer season in Prudhoe Bay, air temperatures rise above $0^{o}C$, and based on our field investigation, fine-grained saturated soils commonly thaw to a depth of between one and three feet. This thawed layer is known as the active zone. At the onset of winter when the thawed soil refreezes, ice lenses tend to form in these frost-susceptible soils. The growth of the ice lenses causes an uplift force known as frost heave. When the soil next to a pile heaves, the adfreeze bond exerts an upward force on the pile called frost jacking. To prevent vertical pile movement, frost jacking forces must be resisted by the adfreeze bond in the underlying permafrost. Piles should be embedded to the minimum depth at which the design adfreeze capacity equals the estimated jacking force. If the piles are installed through an insulated work pad which reduces the depth of thaw or are equipped with thermal tubes which change the direction of freezing, the frost heave forces could be substantially lessened.

In design calculations, the estimated depth of the active layer should be increased by one foot to account for possible increased skin thaw around a pile caused by the warming effects of solar radiation on unshaded steel piles.

Frost jacking forces in our design were estimated using the procedures suggested by Linell and Lobacz (1980). The analysis was performed assuming a three-foot active layer in the fine-grained, frost-susceptible soils and a 60 psi frost jacking force acting on the surface of the pile in the active zone; the resulting frost jacking force is 26 kips per perimeter foot of pile. The 60 psi frost jacking force is based on an experimentally determined value of 40 psi for piles in Fairbanks silt, increased by 50 percent to account for the greater bond that develops with a sand slurry. Embedment depths required to prevent frost jacking are given in Table 1 on Plate VII-1.

The Fairbanks frost heave tests had one pile that showed a frost heave force as large as 40 psi. Because of the relative thinness of the North Slope active layer and the limited amount of available water to cause frost heave, the 40 psi value should be conservative for the PTD project. Since pile design for lightly loaded buildings and pipelines will probably be based on the minimum length to resist frost jacking, and since no significant frost jacking problems have been observed at Prudhoe Bay, a pile jacking field test may be warranted to optimize the design.

4. Design For Lateral Loads

Piles will be subject to lateral loads caused by dead loads and by live loads from operations, winds, and ice in river crossings. The factors which have a major influence on lateral support capacity in frozen soils include:

- Pile type, shape, size, and embedment depth;
- Existing soil profile;
- The depth of the active zone and the subsurface temperature profile; and
- Duration of lateral load.

We recommend that the analysis of laterally loaded piles be performed using the methods developed by Meyer and Reese (1979). Computations can be performed on a digital computer using "beam-on-elastic foundations" types of programs. Data required as input for these programs are a series of "p-y" curves for the soil strata within which the pile is embedded. The p-y curve shown in Figure 1 on Plate VII-2, represents the load-deformation behavior of the soil at a certain depth in the form of horizontal soil reaction per unit pile length (p) versus horizontal pile deflection (y). Values of p and y depend upon the ultimate soil resistance, P_{ult}, and the lateral pile deflection, Y_{50}, at 50 percent of P_{ult}. Both P_{ult} and Y_{50} vary with soil depth. A detailed discussion on the methods used to develop the p-y curves is presented in Appendix E, Volume II.

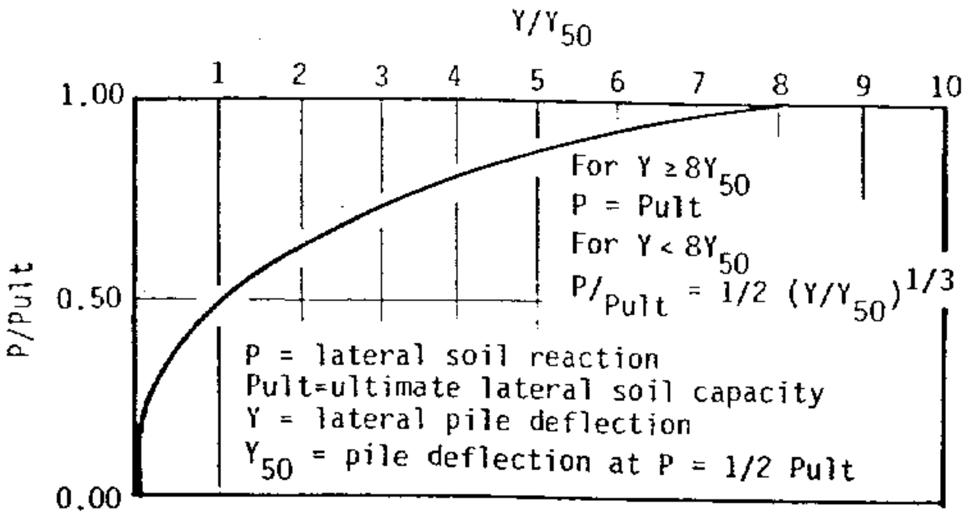


Figure 1. P-Y Curve

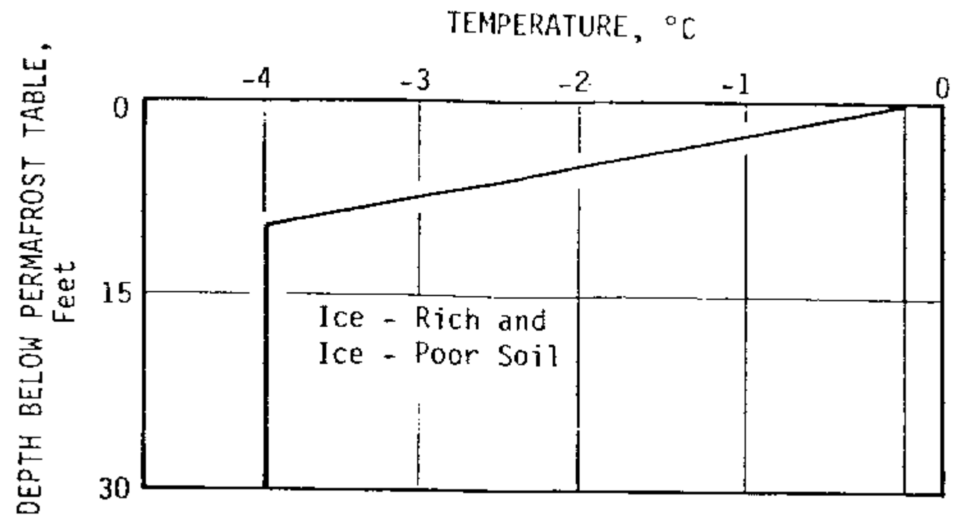


Figure 2. Temperature profile used in analysis

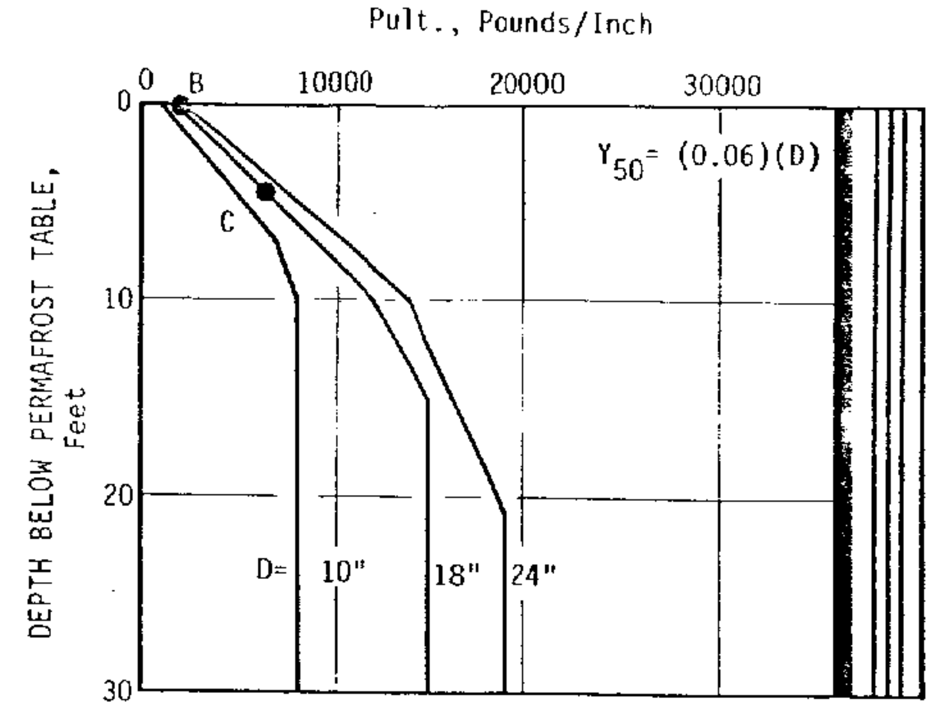
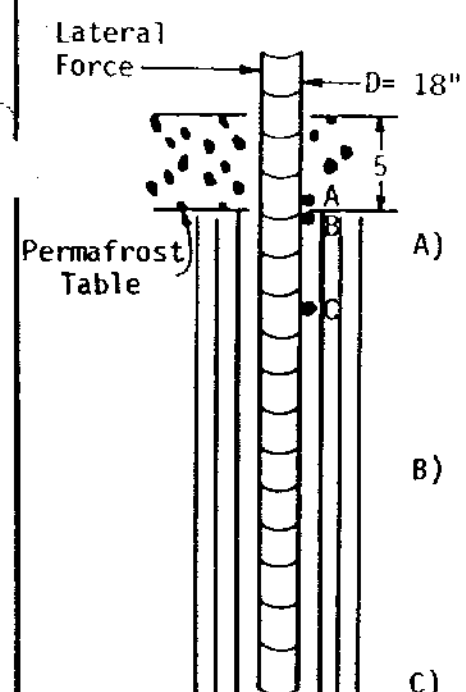


Figure 3. Ice-rich soil



EXAMPLE

Determine P-Y relationship at the points A,B and C as on the adjacent drawing

A) At 5 Ft. into unfrozen gravel from Figure 5 Point A, P_{ult} = 800lbs/inch Y_{50}= (0.02)(18")= 0.36 inches
(P/800lbs/inch)=1/2(Y/0.36")$^{1/3}$

B) At top of ice-rich soil from Figure 3 Point B, P_{ult}=1600lbs /inch Y_{50}= (0.06)(18")= 1.08 inches
(P/1600lbs/inch)=1/2(Y/1.08")$^{1/3}$

C) At 5 Ft. into ice-rich soil from Figure 2 Point C, P_{ult}=7000lbs/ inch Y_{50} = (0.06)(18") = 1.08 inches
(P/7000lbs/inch)=1/2(Y/1.08")$^{1/3}$

These equations correspond to the P-Y curve shown on Figure 1

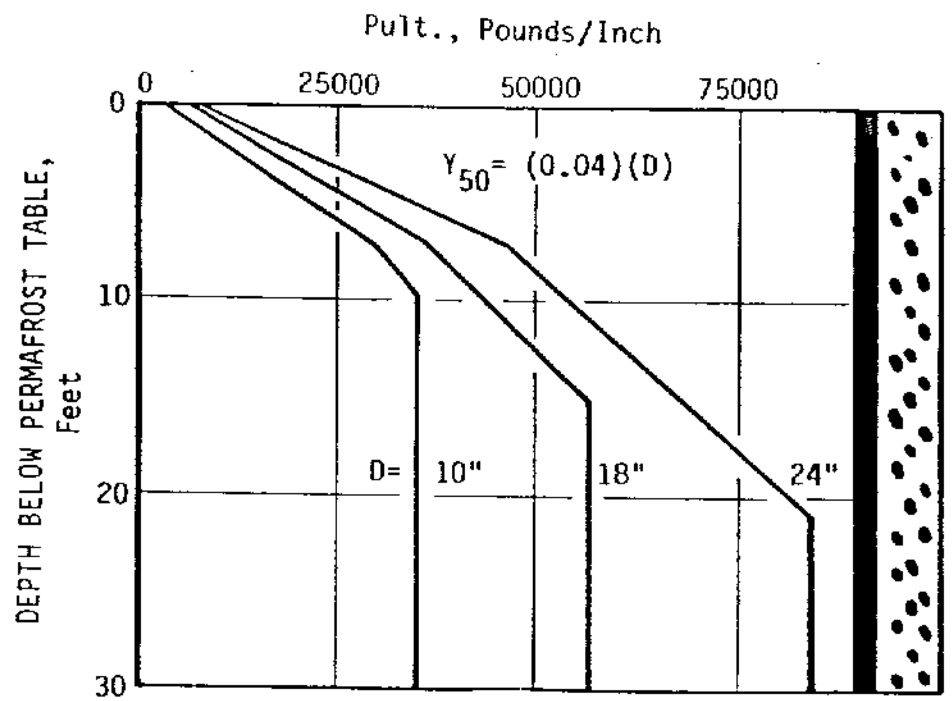


Figure 4. Ice-poor soil

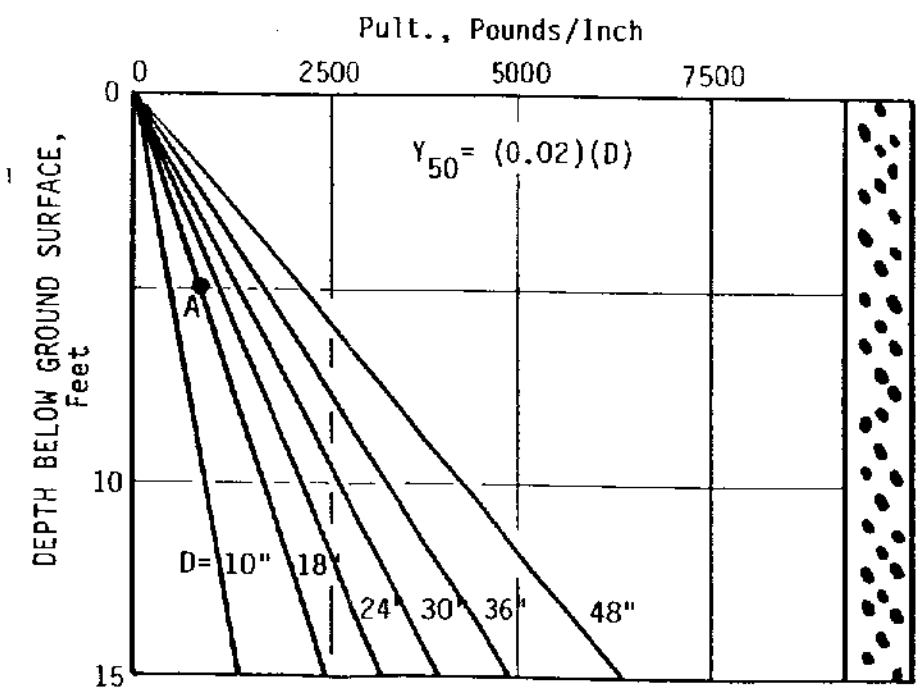


Figure 5. Unfrozen Gravel

Ice-Rich Soil
Unfrozen Gravel
Ice-Poor Soil

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Lateral Load-Deflection of Piles for 24 Hours of Transient Loading
Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE **VII-2**

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When loaded at a stress less than their strength, frozen soils experience a strain deformation process known as creep. In general, the longer the load duration, the greater the decrease in the shear strength of the soil. We have predicted the strength of ice-poor (Type I or III) soils after 24 hours of loading for several design temperatures using data published for Ottawa sand by Sayles (1968). Vialov's strain equation (1962) was used to predict the creep strain that occurs under a stress equal to the 24-hour strength. These data were used to calculate P_{ult} and Y_{50} values for various depths in ice-poor Type I and III soils. These values are presented in Figure 4 on Plate VII-2.

To account for the effects of creep in ice-rich Type II soils, we have used a modification of Vialov's method (Sayles and Haines, 1974). This method requires the determination of instantaneous unconfined compressive strength and the short-term creep behavior of the soil for at least two different stress levels at a given temperature. These data are used to calculate creep parameters which can be used to calculate the creep strength at the given temperature for any duration of loading. The temperature dependence of the creep stress-strain behavior can be evaluated for a range of temperatures.

The data used to calculate the creep parameters for ice-rich soils were obtained from the results of laboratory strength and creep tests on undisturbed samples. The tests were performed for the Geotechnical Investigation of the Gas Conditioning Facility at Prudhoe Bay (HLA, 1980). This analysis is described in detail in Appendix E, Volume II. The data from these tests were used to calculate P_{ult} and Y_{50} for various depths in ice-rich soil for load durations of 24 hours. These values are presented in Figure 3 on Plate VII-2.

The presence of a frozen slurry annulus separating the piles from the in situ soils will also influence the lateral load behavior. However, in our opinion, this influence is likely to be minor for the following reasons:

1. The critical season for lateral deformation is late summer, when the active layer is at its maximum depth.
2. The largest lateral deflections occur within a few feet of the ground surface, where ground temperatures are warmest (in late summer).
3. At warmer temperatures (below, but near freezing), the strength and creep behavior of frozen soils or slurry or even pure, fresh-water ice do not differ significantly (Linell and Lobacz, 1980).

Accordingly, our analysis of lateral load-deflection behavior is based on the properties of the in situ soil mass and the actual diameter of the pile. We believe that this approach is appropriate although it is conservative in that (1) a warm soil temperature profile was used to determine creep parameters, (2) the laboratory test results are probably conservative, and (3) the additional resisting strength due to adfreeze bond on the pile was neglected.

The lateral capacity for unfrozen gravel was calculated using procedures from Hansen (1966). The gravel pad is assumed to be saturated. An angle of internal friction of 40 degrees and a unit weight of 70 pcf was used in the analysis. The results of the analysis are shown in Figure 5 on Plate VII-2.

P_{ult} and Y_{50} values for 24 hours of static lateral pile loading and the designated soil conditions are presented in Figures 3, 4 and 5 on Plate VII-2. These values were developed for late summer conditions when ground temperatures are the warmest and the greatest lateral load deflections will occur. The P_{ult} values presented are for the anticipated on-shore soil profiles. The allowable lateral deflections can be computed using these relationships for specific pile shapes and loads.

5. Typical Design Case

a. Ice-Rich over Ice-Poor Soil

The design case for ice-rich over ice-poor soil is represented by an average of about seven feet of ice-rich Type II fine-grained soil over ice-poor Type III sand. The subsurface temperature profile assumed for this case is shown in Figure 2 on Plates VII-1 and VII-2.

The vertical pile capacity may be calculated using the data found in Figure 4 on Plate VII-1. The capacities include a 5 psi adfreeze value in the ice-rich soil. From Table 1 on Plate VII-1, piles in these soils should be embedded to a minimum depth of 14 feet below the active zone. From Figure 4, at this depth they would resist a frost-jacking force of 26 kips per perimeter foot.

In the unfrozen active zone, lateral capacity in the soft fine grained soils is minor and should be ignored in design. The largest pile deflection will occur within a few feet of the permafrost table. Therefore, the lateral pile capacity criteria presented in Figure 3 on Plate VII-2 for ice-rich soil should be used for this case.

b. Ice-Rich Soil

This design case covers piles installed in on-shore areas having ice-rich soil and massive ice. The allowable vertical load on piles embedded in these soils should be based upon the permissible settlement. After choosing the permissible settlement, pile dimensions should be selected using the information presented in Figure 3 on Plate VII-1.

Lateral capacities of piles embedded in ice-rich soils should be calculated using the criteria presented in Figure 3 on Plate VII-2. Piles in these soils should be embedded to a minimum depth of 14 feet below the active zone to resist a frost jacking force of 26 kips per perimeter foot of pile.

c. Ice-Poor Soil

Ice-poor soil is generally associated with low ice content sand and gravel deposits (Type I and Type III), which are considered to be non-frost susceptible. Piles placed in these soils are not likely to be subject to frost jacking; however, to account for pockets of moderately frost-susceptible material, piles should be designed for a frost heave force of 20 kips per perimeter foot of pile.

Vertical pile capacities based upon the adfreeze bond are presented in Figure 4 on Plate VII-1. As indicated in Table 1 on Plate VII-1, piles installed in the ice-poor soils should be embedded to a minimum depth of 11 feet below the active zone to resist frost heave.

Lateral pile capacity criteria for the upper eight feet of unfrozen gravel are presented in Figure 5 on Plate VII-2; similar criteria for the underlying ice-poor gravels are given in Figure 4 on Plate VII-2.

6. Other Considerations

a. General

For the lateral and vertical pile capacity designs, we assumed that the natural tundra surface would not be altered during construction. The subsurface temperature profile will be altered from that assumed in design if (1) either the surface vegetation or organic material is removed, (2) gravel is placed on the ground surface, or (3) water is ponded. Pile capacities should be selected from the figures (shown on Plates VII-1 and VII-2) for the thermal conditions.

Construction equipment can disturb the vegetative and organic ground cover, which could result in degradation of the underlying permafrost. To avoid such disturbance, a fill pad for equipment traffic should be placed over the undisturbed tundra prior to other construction activity. This fill pad can be constructed of either snow or gravel. Along pipelines, it should be placed adjacent to but not on the pipeline alignment. This fill pad could have the effect of increasing the depth of the active layer.

Batter piles could be used to resist large lateral loads when associated deflections are not acceptable. Batter piles have been used on some pipeline support systems in the Prudhoe Bay area. As these systems should be designed on a site specific basis, no design recommendations are included at this time.

b. Pile Installation

(1) Slurry Adfreeze Piles

Slurry piles have been installed at Prudhoe Bay during both summer (Flow Station III) and winter (Flow Stations I and II and the Central Compressor Plant). There are advantages and disadvantages to pile installation during either season. Summer conditions contribute to greater construction efficiency; equipment and personnel both perform better in warmer temperatures. Although the ambient air temperature is usually above freezing, the cold permafrost temperatures prevent thawing of the augered pile hole walls. A gravel work pad is necessary during the summer. Surface water can cause problems if it migrates down the hole walls and forms ice; consequently, the active zone must be cased to keep out the water. Casing is usually required to prevent the thawed gravel fill from sloughing into augered holes.

If piles are installed during the winter, after the active layer has refrozen, the problem of surface water infiltration is eliminated and casing normally is not necessary. Either gravel or snow pads can be used to protect the tundra. If gravel pads are used, casing is not needed when the gravel is frozen. Because there is usually a time lapse as much as several days between drilling the pile hole and topping off the final lift of slurry, the hole must be tightly covered to keep out blowing snow and construction debris. The construction crews must plug the annulus after every slurry pour. An additional concern is that steel piles stored outside in very cold temperatures tend to "flash freeze" the slurry water on contact and form a film of ice between the pile and slurry soil. This is not considered a problem for properly mixed slurry; however, if excess water is present so that ice forms, adfreeze values could be lower than normally expected.

(2) Driven Piles

Driving piles with conventional diesel or steam hammers in permafrost as cold as that at Prudhoe Bay will require drilling pilot holes to prevent damage to the pile tip. Even with predrilling, the pile tip will probably need reinforcement. Before driving in granular soils, it may be necessary to temporarily warm the permafrost by adding hot water or steam into the pilot hole. Pile driving with vibratory hammers may be feasible but this method should be tested at the site prior to production driving. It should be noted that it is not possible to log the subsurface soils during driven pile installation unless a pilot hole is drilled. Consequently, if a pilot hole is not drilled, it is difficult to adjust pile lengths in the field to accommodate ice conditions.

c. Pile Corrosion

Our electrical conductivity measurements of the onshore pore fluids indicate negligible salt concentrations in the soil. Even if present, salt concentrations in the pore ice should not affect untreated steel piles. The low electrical conductivity of the frozen soils, coupled with the cold temperatures, reduces the rate of corrosion of steel (or decay of wood) to a relatively low level. The annular zone of sand/fresh water slurry in pre-augered piles will probably keep the pile and saline pore ice separated.

If the influence of the active zone pore water on the pile is a concern, the portion of the pile in the active zone could be coated with a corrosion resistant material. However, if any surface treatment were applied to the portion of the pile placed below the active zone, the pile capacity would be substantially reduced. To preserve their adfreeze capacity, steel or timber piles should remain uncoated below the active zone.

d. Thaw-Strain Settlement

Structures will settle if the ice-bonded fill material placed during the winter and in situ frozen soil subsequently thaws. Thawing could be caused by heat from structures placed at-grade, warm buried utilidors, or the well area.

Table VII-2 presents estimated maximum thaw-strain settlements calculated for a five-foot gravel pad constructed at different ice contents. The thaw-strain properties of the fill are discussed in Chapter V.

TABLE VII-2. THAW-STRAIN SETTLEMENT: FIVE-FOOT GRAVEL PAD

Fill Condition		Thaw Strain (%)	Settlement (feet)
Summer placed gravel		0	0
Winter placed gravel: moisture content	5%	2	0.1
	10%	14	0.7
	15%	23	1.2
	20%	30	1.5
	25%	36	1.8
	30%	41	2.1

Notes: Thaw strain estimated from Plate V-9. Summer-placed gravel assumed compacted to 95 percent of maximum dry density as determined by ASTM 1557.

The relative high thaw-strain settlements calculated for gravel-ice mixtures placed in winter could be reduced by using a lower ice content gravel. For example, if gravel with an average ice content of 5 percent instead of 10 percent is used, thaw-strain settlements will be reduced by about 85 percent.

An average thaw strain for in situ ice-rich and ice-poor soils is presented in Table VII-3. Thaw-strain settlement will be variable because of the presence of ice lenses and/or wedges. For example, the massive ice 15 feet thick encountered in Boring 13 would cause large thaw-strain settlements locally if it melted.

TABLE VII-3. THAW-STRAIN SETTLEMENT: IN SITU SOIL

Material Type	Average Ice Content (%)	Typical Thaw Strain Settlement (feet)
Ice-Poor Soil	25	($0.10\ Z_t$)
Ice-Rich Soil	50	($0.30\ Z_t$)

Notes: Z_t = Total depth of thaw of in situ soil, feet.

C. Roadways

1. General

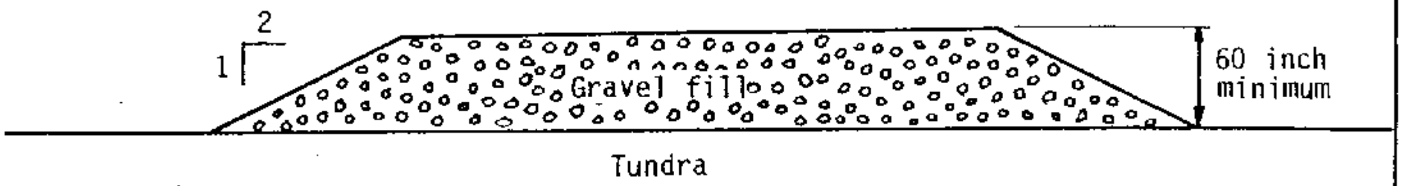
To construct and service onshore facilities and pipelines, roadways will be built within the unit. The roadways will be designed to support conventional construction equipment, lightweight service vehicles, and heavy module transporters.

Three considerations in Arctic road design are: (1) limiting the depth of summer thawing below the road to maintain the permafrost table; (2) providing a stable foundation for traffic loads; and (3) reducing snow drift across the road surface.

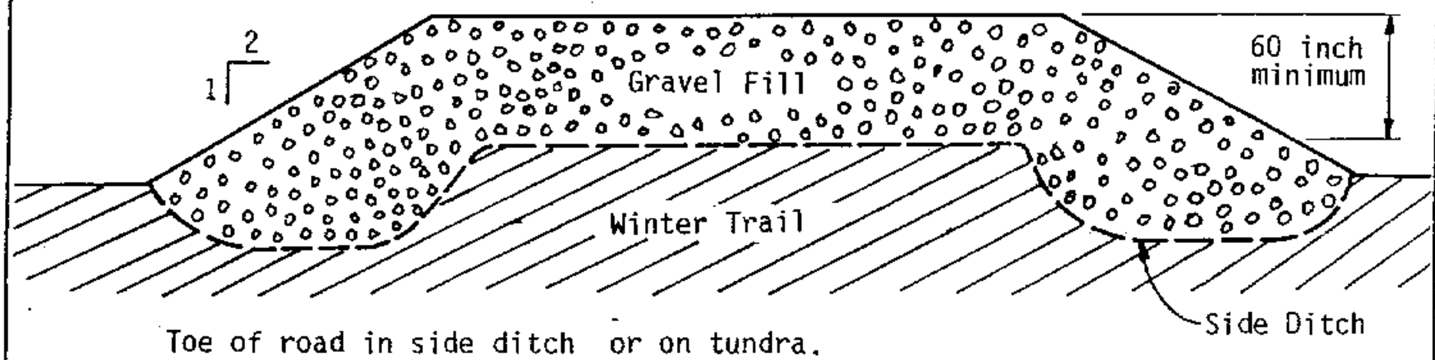
The standard Prudhoe Bay road cross section over tundra areas consists of five feet of gravel fill as shown on Plate VII-3.

2. Winter Trail Conditions

As shown on the Regional Study Area Plate II-1, a winter trail traverses through the project area. The trail was built several years ago by bulldozers shaping the top 1 or 2 feet of tundra vegetation and soil into a road. In the Duck Island area, the trail is 1 to 4 feet higher than the adjacent tundra and is 18 to 32 feet wide. There are ditches up to 2 feet

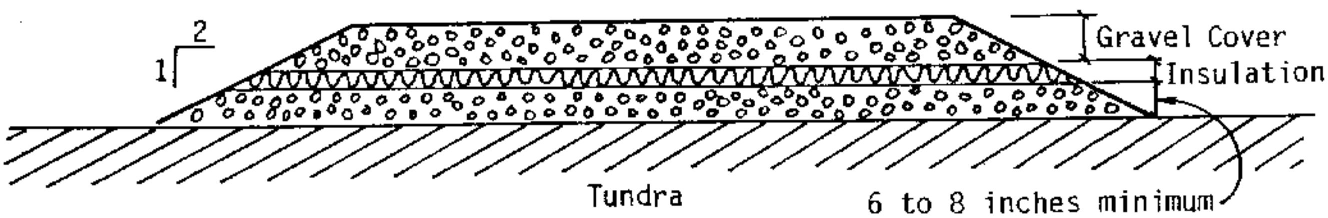


STANDARD PRUDHOE BAY ROAD



Toe of road in side ditch or on tundra.
Existing ditch should be filled to prevent ponding.

ROAD OVER WINTER TRAIL



INSULATION THICKNESS (Inches)	REQUIRED GRAVEL COVER (Inches)
1.5	24
2.0	18

ROAD WITH INSULATION

Harding Lawson Associates
Engineers, Geologists
& Geophysicists

Roadway Design Criteria
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE
VII-3

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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deep on both sides. Due to snow and ice cover, the trail was not observed during this study.

In late summer, the active zone along the center of the trail is about two to three feet deep. The subgrade is relatively firm when dry but soft when wet. As much as two feet of standing water has been observed in the ditches alongside the trail. When thawed, the soils below the water are soft.

3. Road Design

The winter trail and natural tundra surfaces are suitable for a road foundation. The roadway fill should be constructed of non-frost-susceptible sand and gravel, as described in Chapter V. If the road is placed over the winter trail, as shown on Plate VII-3, its toe should be in a side ditch or on the flanking tundra. The height of the trail varies each winter, but the road section should have a gravel fill thickness of at least five feet over the trail and the flanking tundra. This thickness should limit seasonal thaw to a depth below the fill no greater than the natural active layer prior to fill placement.

The required gravel fill thickness over the tundra can be reduced if a layer of insulation is placed within the road, as shown on Plate VII-3. Such roads should be built by placing: (1) a layer of gravel over the tundra and grading it to a level surface; (2) a layer of insulation over the level surface; and (3) the required additional fill. The insulation used for road construction should be expanded polystyrene, not polyurethane, and should have:

1. A compressive strength of at least 30 psi;

2. A closed-cell structure resulting in a low water absorption potential; and

3. A thermal conductivity of less than 0.25 BTU in/hr Ft^2 ^oF

Some insulation products that meet these specifications include DOW Chemical's HI-35 and ARCO Chemical Company's GEOFORM Series.

We understand that during the construction of insulated work pads for the Alyeska project the insulation was laid directly on a leveled tundra surface. The Alyeska design consisted of a 2-inch-thick layer of insulation with an 18-inch-thick surface layer of gravel fill. If the surface is irregular the rigid insulation boards will break and lose some thermal effectiveness. Consequently, we recommend that the gravel subbase thickness can vary from 6 to 18 inches thick depending on the evenness of the tundra surface. At least an 18-inch thickness of fill should be placed over the insulation to reduce the wheel load effect on the insulation. To provide a thermal barrier, two layers of board stock should be used with overlapping joints.

Typical sections for roads built over the tundra are summarized in Table VII-4. These sections were developed to reduce the potential of thermal degradation of the underlying permafrost.

TABLE VII-4. INSULATED ROAD DESIGN

Gravel Subbase Thickness (in.)	Insulation Thickness (in.)	Gravel Surface Thickness (in.)
0	0	60
18	1.5	24
18	2.0	18

Snow drifts form across a road when flanking objects are above its surface. A road with an insulating layer will be closer to the tundra than the standard road and snow drifts are more likely to form across its surface. Snow drifting may be reduced if the level of the road surface is kept above the flanking objects.

4. Fill Placement and Compaction

If frozen gravel fill material is placed at its natural ice content, typically between 5 and 15 percent but possibly as high as 35 percent, relatively low densities will result regardless of the compactive effort applied. For example, gravel containing 3 percent ice usually cannot be compacted to more than about 90 percent relative compaction. Gravel fills from the Point Thomson C-1 material site had measured relative compaction of from 40 to 50 percent when compacted frozen. Depending on its initial ice content, compacted frozen gravel can have thaw-strains of 10 to 35 percent. For this reason, gravel placed during the winter normally requires regrading and compaction during the summer to improve its supporting capacity and to compensate for thickness lost due to thaw settlement.

It is best to place gravel fill when it is thawed. At the onset of road construction, a working pad 18 inches thick should be placed directly on the undisturbed tundra. Subsequent fill should be placed in lifts no greater than 12 inches thick. The fill should be compacted to at least 95 percent relative compaction as determined by ASTM Test Method D1557.

Roads with an insulation layer have normally been constructed during the winter. If the insulated pad is constructed in the summer, the thawed subgrade will probably remain unfrozen through most of the following winter.

With the thin gravel surface, traffic loads would have to be restricted to protect the gravel fill and insulation from damage until the subgrade freezes.

If frozen gravels are used, fill slopes should be no steeper than 3 horizontal to 1 vertical (3:1) to reduce slumping during the summer. Fill slopes for thawed gravels can be as steep as 2:1.

5. Road Culverts

At culvert and ponded water locations, a depression can develop in the permafrost table as deep as 10 feet below the roadbed surface. The depression may be caused by thinner gravel fill, warm ambient air and water flowing through the culvert, water ponded next to the road, and/or deeper snow drifts. This depression may lead to thaw consolidation of the foundation soils and operational problems with the culvert. The culvert locations should be selected to reduce the adverse thermal effects, and even if the road is not insulated, insulation should be placed beneath culverts.

D. Onshore Drill Pads

The standard onshore Prudhoe Bay drill pad over tundra areas consists of at least 5 feet of gravel fill. We understand that typical spacing between production wells is generally 150 feet (Exxon Communication). Additional gravel fill is required over the years due to thaw consolidation of the underlying ice-rich soil. Gravel for the pad should be placed and compacted as described in the previous section.

E. Onshore Base Camp

A base camp will be required to support development of the Point Thomson area. As previously discussed, the onshore area is underlain by ice-rich frozen soils that extended to the depths explored. Structures should be constructed above grade so that air circulation beneath the buildings will prevent heat transfer to the frozen ground. Access roads in the camp area should be constructed of gravel pads thick enough to limit or prevent thawing of the tundra surface. Utility lines should be layed on gravel pads, placed above ground on piles, or buried. If buried, the lines should be far enough from foundations so that utilidor heat does not degrade the permafrost. For some foundations, only slight increases in temperature can result in loss of support. The lines should be insulated to reduce thawing of the deeper soil.

Details of the camp design, layout, and loads are not known at this time. Preliminary recommendations and soil engineering design criteria for fill pads and foundations are presented in the following paragraphs.

1. Fill Embankments

As discussed, gravel embankments are required for roads where year-round traffic is expected. Gravel fills placed around buildings should extend beneath the buildings to prevent snow drifting that could occur if depressions were present.

The gravel thickness required to achieve a stable fill depends on the properties of the near surface soils and fill materials and the freezing and thawing indexes. For the camp site, a thickness of five feet of gravel fill should be used for building pads and access roads. The fill pad should be constructed in accordance with the recommendations for roadway fill presented in the preceding sections.

2. Foundations

The camp buildings should be supported above the surface of the fill so that heat from the buildings does not affect the ground temperature. The simplest means of implementing substructure ventilation is by supporting the structure on piles embedded in permafrost. Light structures can be supported on posts extending from footings founded in a gravel pad. A clear space of at least 36 inches should be provided between the base of the structure and the ground surface. A larger separation should be provided if snow can accumulate or where deep fronting members, pipe trays, or very wide structures are planned.

Either augered-slurried piles or driven piles can be installed. The piles can be designed for vertical and lateral loads using the criteria presented in the preceding sections.

Relatively light or temporary structures can be supported on post and pad type foundations bottomed on a gravel pad. The pads can consist of a timber or steel grillage with wood or steel columns extending above ground to support the floor and provide a ventilated airspace. Some vertical movement of this type of foundation should be expected as the gravel fill freezes and thaws. If pads are underlain by at least four feet of gravel fill, bearing pressures of 3000 psf can be used for total design loads.

Design of an above-grade structure might not be economical for large heated buildings with heavy floor loads and vehicle access requirements, such as garages or warehouses. For these cases, the permafrost can be maintained using an insulated floor with a cooling system below the insulation. The cooling system can consist of ventilation ducts or passive liquid-vapor heat transfer systems buried in the fill. Design of the cooling and insulating system depends on building size and heat.

3. Water Supply

Water use for the proposed PTD project will include water for personal needs such as drinking, washing, laundering, and food preparation, and for industrial needs such as camp maintenance and drilling. Water use in camps in the Deadhorse area averages about 85 gallons per capita per day. The current total use is approximately 800,000 gallons per day (U.S. Army Corps of Engineers, 1980).

The North Slope is underlain by continuous permafrost which extends to a depth of about 1800 feet in the Prudhoe Bay area. Limited supplies of ground water may occur in thawed areas beneath the permafrost, but water from these sources tends to be of poor quality. As a result, ground water is not used for domestic and industrial purposes.

We anticipate that surface water supplies in the areas of proposed campsites will be limited to deep lakes in the vicinity of the proposed campsite, possibly an active channel of the Staines River or man made reservoirs.

The water quality in lakes and ponds is generally good during the open water months and remains good until freeze-up approaches. The major problems associated with lake water are objectionable color, odor, and iron content. Generally, lakes and ponds within one mile of the coast are characterized by higher levels of salt and higher alkalinity than inland waters.

Lakes generally begin to freeze over in mid to late September and attain a maximum ice thickness of between six and seven feet by late March. Ice break up on lakes normally occurs in late June or early July. Lakes that do not freeze completely to the bottom in winter often exhibit deteriorating water quality as the winter progresses. Dissolved oxygen levels may decrease to near zero and dissolved salts may become concentrated in the unfrozen waters beneath the ice (Hobbie, 1978).

The pore water (ice) of gravelly soils may have significant salt concentrations that could affect reservoir water quality. If such salts are present, a reservoir lining may be necessary. Some unlined reservoirs have apparently become contaminated by salts released during degradation of the permafrost under the reservoir.

4. Waste Disposal

a. Existing Facilities

The North Slope Borough (NSB) currently operates a plant at Deadhorse capable of processing both liquid and solid wastes. Liquid wastes consist of sludge produced in primary treatment plants operated by private camps in the area and raw sewage pumped directly from holding tanks at those camps which do not have primary treatment. Raw sewage is processed at the NSB plant to produce sludge, which is incinerated, and waste water, which is stored in a lagoon. Solid wastes are separated into burnable and non-burnable materials. Burnable waste is incinerated at the NSB treatment plant. The ashes and non-burnable materials are then trucked to the landfill that NSB operates in an abandoned gravel pit located in an oxbow of the Putuligayuk River.

Table VII-5 presents rates in effect in November 1981 for waste disposal at the NSB plant (HLA, 1981). The rates quoted do not include trucking to the treatment plant. Trucking waste from the PTD area would involve transporting the waste a distance over 60 miles.

TABLE VII-5. WASTE DISPOSAL RATES FOR NORTH SLOPE TREATMENT PLANT

Material	Rate
Liquid sewage	$ 0.21/gallon
Sewage sludge	$ 0.28/gallon
Burnable solids	$31.00/cubic yard
Non-burnable solids	$20.00/cubic yard

ARCO and Sohio have secondary treatment plants installed at their base camps. Both systems use a centrifuge to separate the liquids and dewater the solid wastes. The sludge produced by the dewatering process is then burned in on-site incinerators. The effluent is treated and discharged to holding lagoons. Solid wastes are delivered to the North Slope Borough and processed as described above.

b. Waste Disposal Alternatives

Use of the NSB treatment plant will require transporting wastes approximately 60 miles. Provisions would need to be made to store wastes at the camp site during those times when truck transport is not possible due to weather conditions or flooding.

The impermeable nature of the ice-bonded soils beneath the proposed camp site would allow construction of containment lagoons similar to those employed by ARCO and Sohio. It may be possible to use a shallow lake adjacent to the camp as an effluent storage lagoon.

Landfill operations are also feasible from a soil engineering standpoint. It will be necessary to route surface water away from the excavation to avoid flooding of the landfill pit. A NSB permit will be required before constructing and operating an effluent lagoon or a landfill pit.

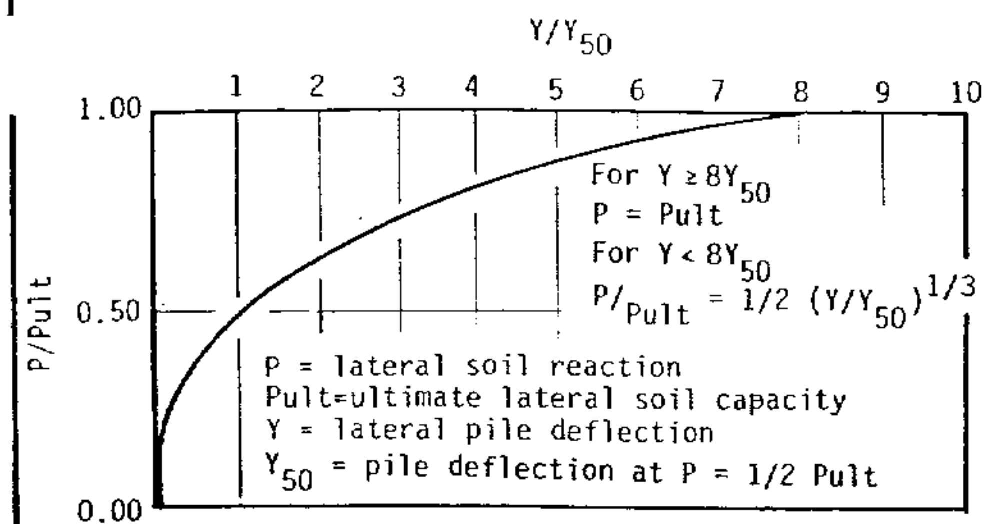


Figure 1. P-Y Curve

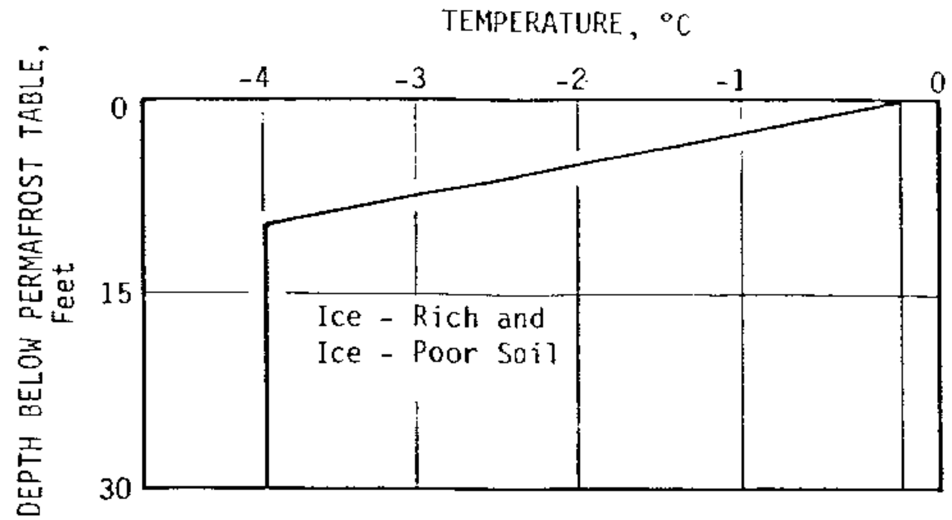


Figure 2. Temperature profile used in analysis

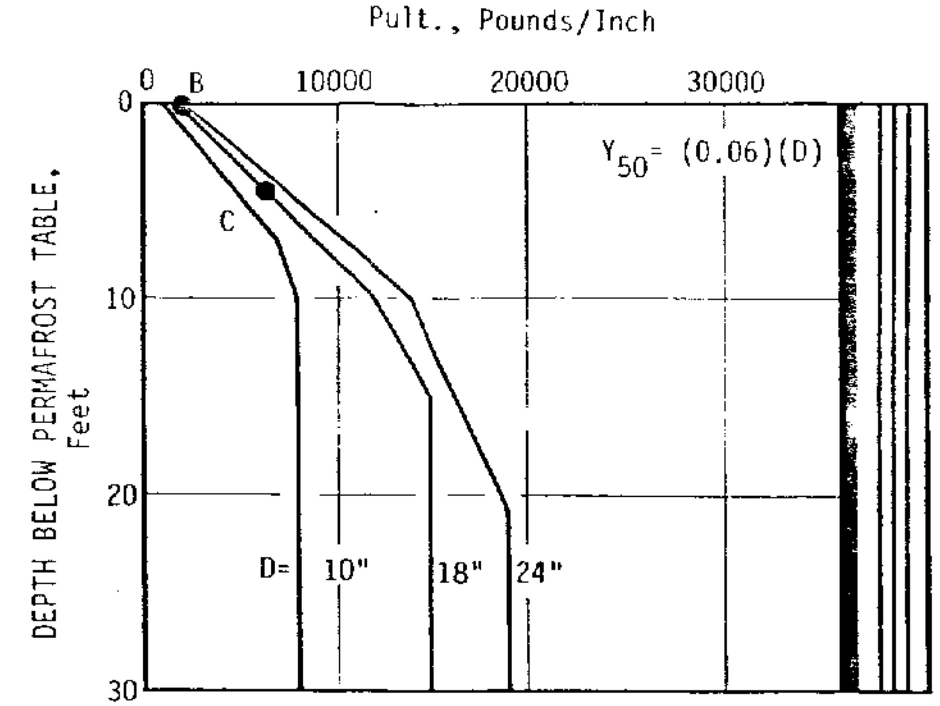
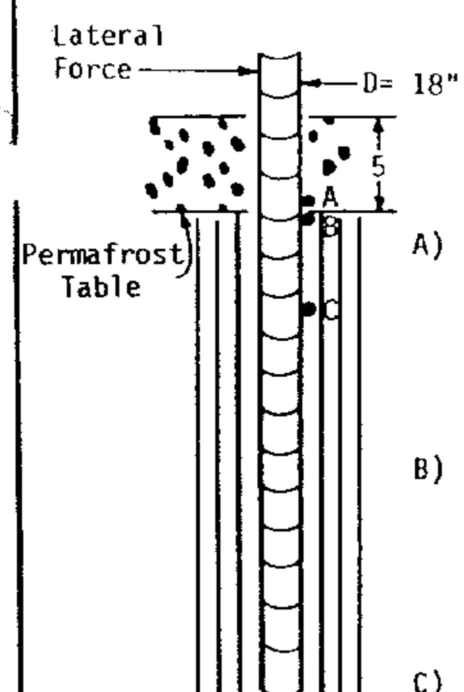


Figure 3. Ice-rich soil



EXAMPLE

Determine P-Y relationship at the points A,B and C as on the adjacent drawing

A) At 5 Ft. into unfrozen gravel from Figure 5 Point A, P_{ult} = 800lbs/inch $Y_{50} = (0.02)(18") = 0.36$ inches
$(P/800lbs/inch) = 1/2(Y/0.36")^{1/3}$

B) At top of ice-rich soil from Figure 3 Point B, P_{ult} = 1600lbs/inch $Y_{50} = (0.06)(18") = 1.08$ inches
$(P/1600lbs/inch) = 1/2(Y/1.08")^{1/3}$

C) At 5 Ft. into ice-rich soil from Figure 2 Point C, P_{ult} = 7000lbs/inch $Y_{50} = (0.06)(18") = 1.08$ inches
$(P/7000lbs/inch) = 1/2(Y/1.08")^{1/3}$

These equations correspond to the P-Y curve shown on Figure 1

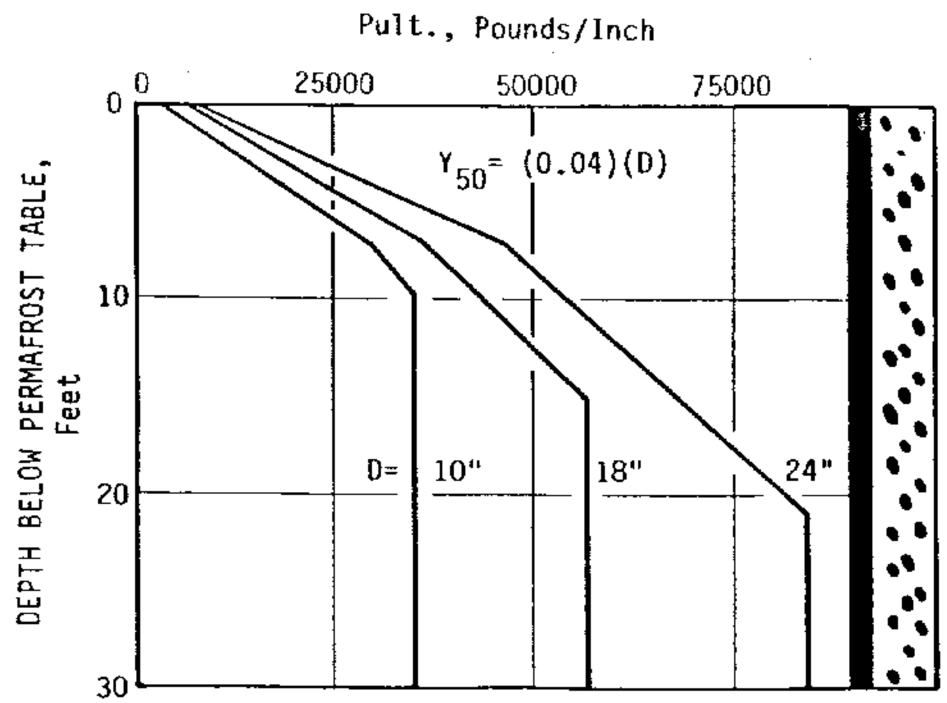


Figure 4. Ice-poor soil

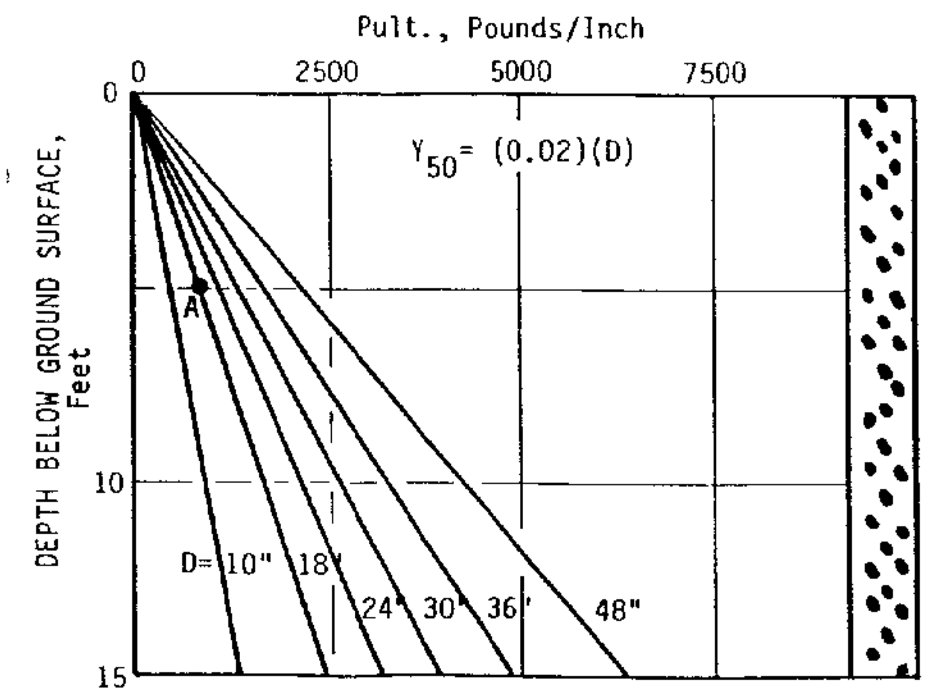


Figure 5. Unfrozen Gravel

Ice-Rich Soil

Unfrozen Gravel

Ice-Poor Soil

Harding Lawson Associates
Engineers, Geologists & Geophysicists

Lateral Load-Deflection of Piles for 24 Hours of Transient Loading
Pt. Thomson Development Project, Winter 1982
Geotechnical Study, EXXON Company, U.S.A.

PLATE **VII-2**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	9612,031.08	DEB	4/82		

VIII GLOSSARY

ACTIVE ZONE. The surface layer of soil that experiences annual freezing and thawing cycles. An unfrozen zone beneath the active zone is a talik.

ALBEDO. The percentage of the incoming radiation that is reflected by a natural surface such as the ground, ice, snow or water.

ARCUATE. Bowed, curved or arched.

ANOMALOUS ZONES. Zones that do not fit the general pattern.

BARRIER ISLAND. A long, low, narrow, wave-built island whose surface elevation is sufficiently above high tide that it protects or shelters the coastline.

BASAL TRANSGRESSIVE DEPOSIT. The first deposit created during a rise in sea level or as land subsidence occurs.

BERGY BITS. Small pieces of ice less than 100 feet across that originate from the break up of ice bergs.

BIOGENIC GAS. Hydrocarbon gas produced by recent biological activity.

BONDED. In permafrost, the condition where soil particles are cemented by ice. Permafrost can be unbonded at below 0°C due to a depressed freezing point, a common occurrence in subsea permafrost.

BOTTOM-FAST ICE. Sea ice that freezes to the sea floor; this ice is generally immobile and undeformed.

CONDUCTIVITY. A measure of the ability of a material to conduct an electrical current.

CONSTRUCTIONAL ISLANDS. Islands experiencing active deposition, reworking, erosion and reshaping by waves and sea ice. Challenge Island, Alaska Island, Duchess Island and North Star Island are examples in the project area.

COPPICE SAND DUNES. Sand dunes stabilized by a thin cover of vegetation.

CRYOTURBATION. The churning and mixing of soil by freezing and thawing cycles.

EOLIAN. Pertaining to the wind; said especially of rocks, soils, and deposits such as dune sands whose constituents were transported and deposited by atmospheric currents.

GLOSSARY
(continued)

EROSIONAL REMNANT ISLANDS. Islands representing the erosional remains of a submerging coastline. Flaxman Island is an example in the project area.

FACIES. Part of a geologic unit differentiated from other parts by appearance or composition.

FLOATING FAST ICE. Sea ice that is immobile and undeformed but has unfrozen sea water between it and the sea bed.

FLUVIAL. Of or pertaining to a river or rivers; produced by the action of a stream or river.

FREEZING INDEX. The sum of the differences between freezing and the average daily temperature for all days in the year when the average daily temperature is below freezing, expressed in degree-days.

GLACIOFLUVIAL. Pertaining to the melt water streams flowing from glacier ice and especially to the deposits and land forms produced by such streams.

GYRE. A circular motion. In the Arctic Ocean the clockwise or western current around the North Pole.

HORIZON. The surface separating two geologic units.

ICE-BONDED PERMAFROST. Permafrost in which the interstitial water has frozen and cemented the mass. See bonded.

ICE GOUGING. A plowing of the sea floor by a wind, current, or ice sheet driven ice keel or ice foot.

ICE-POOR. A frozen soil with a dry density that is not less than what is expected for the same soil when unfrozen. In this report, ice-poor was used for onshore, coarse and fine-grained soil when the moisture content was less than 50 percent.

ICE-RICH. A frozen soil containing sufficient ice so that the dry density is less than the minimum thawed density. In this report, ice-rich onshore permafrost has a moisture content greater than 50 percent, or clean gravel has a moisture content greater than 25 percent.

ICE-WEDGE POLYGONS. A polygon-shaped ground pattern from a few feet to tens of feet across produced by the cyclic cooling and associated contraction of the upper portion of the permafrost.

GLOSSARY
(continued)

ICE WEDGES. Ground ice that develops from water filling vertical cracks that develop in the surface of the permafrost when the ground contracts due to winter cold. The wedges are associated with fine-grained soils and can extend tens of feet deep and be several feet wide at the top.

ISOBATH. A line on a map or chart that connects points of equal depth.

ISOPACH. A line drawn on a map through points of equal thickness of a designated stratigraphic unit or group of stratigraphic units.

ISOSTATIC DEPRESSION. The lowering of the ground surface due to weight of material such as ice or sediments.

JIGGER MARKS. Shallow punch marks in the soil formed by wave-lifted sea ice vertically striking the sea floor.

LAG DEPOSITS. A residual accumulation of coarse, hard rock fragments remaining on a surface after the finer material has been blown away by winds or eroded away by water.

LITHOLOGY. The study of the physical characteristics of rock, especially sedimentary clastics.

LITTORAL ZONE. The portion of the ocean near the coastline.

MORPHOLOGY. The study of the form and structure of the land or ice.

PACK ICE. Seasonal ice which moves with the generally westward circulation of the Arctic Ocean.

PALEOVALLEY. A valley that has since been filled by recent sediments.

PATTERN GROUND. Ground affected by temperature cycles so that patterns such as polygons or stone rings develop.

PERMAFROST. Any soil, bedrock, or ice contained therein, that remains below $0^{o}C$ for two years or longer. However, when used to describe offshore soils in this report, permafrost simply means ice-bonded.

PETROGENIC GAS. Hydrocarbon gas originating from petroleum sources.

PINGO. A relatively large mound raised by frost action above the permafrost and generally persisting for more than a single season.

GLOSSARY
(continued)

POLYGONAL GROUND. A ground pattern caused by the orientations of ice wedges in a polygon shape.

PRO-DELTA. The area in front of a delta.

PROGRADATION. The building forward or outward toward the sea of a shoreline or coastline by near-shore deposition of sediment.

RELICT PERMAFROST. Permafrost that formed during a previous age and is not presently at thermal or chemical equilibrium.

RESOLUTION. The sharpness of an image or the accuracy with which it can be defined.

SASTRUGI PATTERNS. Wind-sculpted patterns in snow or eolian deposits.

SEA ICE ZONATION. The arrangement of sea ice into different types of zones. In the southern Beaufort Sea, the sea ice has been divided into four broad zones: (1) bottom fast ice; (2) floating fast ice; (3) stamukhi or shear zone; and (4) seasonal pack ice.

SEA LEVEL STANDSTILL. A period between glacial epochs during which sea levels remain nearly constant for an extended time.

SHEAR ZONE. The boundary between the shore-fast ice and the pack ice which moves with the Arctic gyre.

STAMUKHI ZONE. A zone of slippage between the moving pack ice and the floating fast ice. See shear zone.

STRATIGRAPHY. The study of rock strata, especially of their distribution, deposition, and age.

STRUDEL. Holes or cracks in the sea ice through which fresh river water drains during ice break up. From the German for whirlpool which describes the flow of water through the ice.

STRUDEL SCOUR. Sea floor depressions created by fresh river water draining through strudel.

TALIKS. Zones which are not bonded but are below the permafrost table.

THAWING INDEX. The sum of the differences between freezing and the average daily temperature for all days in the year when the average daily temperature is above freezing, expressed in degree-days.

GLOSSARY
(continued)

THAW LAKE. A lake or pond in a permafrost area whose basin is formed by thawing of ground ice.

THERMO-EROSION. The process of losing material when thawing occurs.

THERMO-EROSIONAL NICHING. The undercutting of river banks or bluffs by the thermal action of flowing water.

THERMOKARST. A highly irregular surface that develops from differential subsidence of the ground when underlying permafrost thaws.

TRANSGRESSION. The spread or extension of the sea over land areas or the evidence of such an advance; a rise in sea level or subsidence of land.

VIBROCORE. A vibrating coring device used to obtain core samples of the seabed.

WIND-ORIENTED. Surface features such as lakes or dunes with a predominant dimension that has a direction related to the prevailing wind.

WINNOWING. Separation of fine particles from coarser ones by action of the wind.

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Harding Lawson Associates
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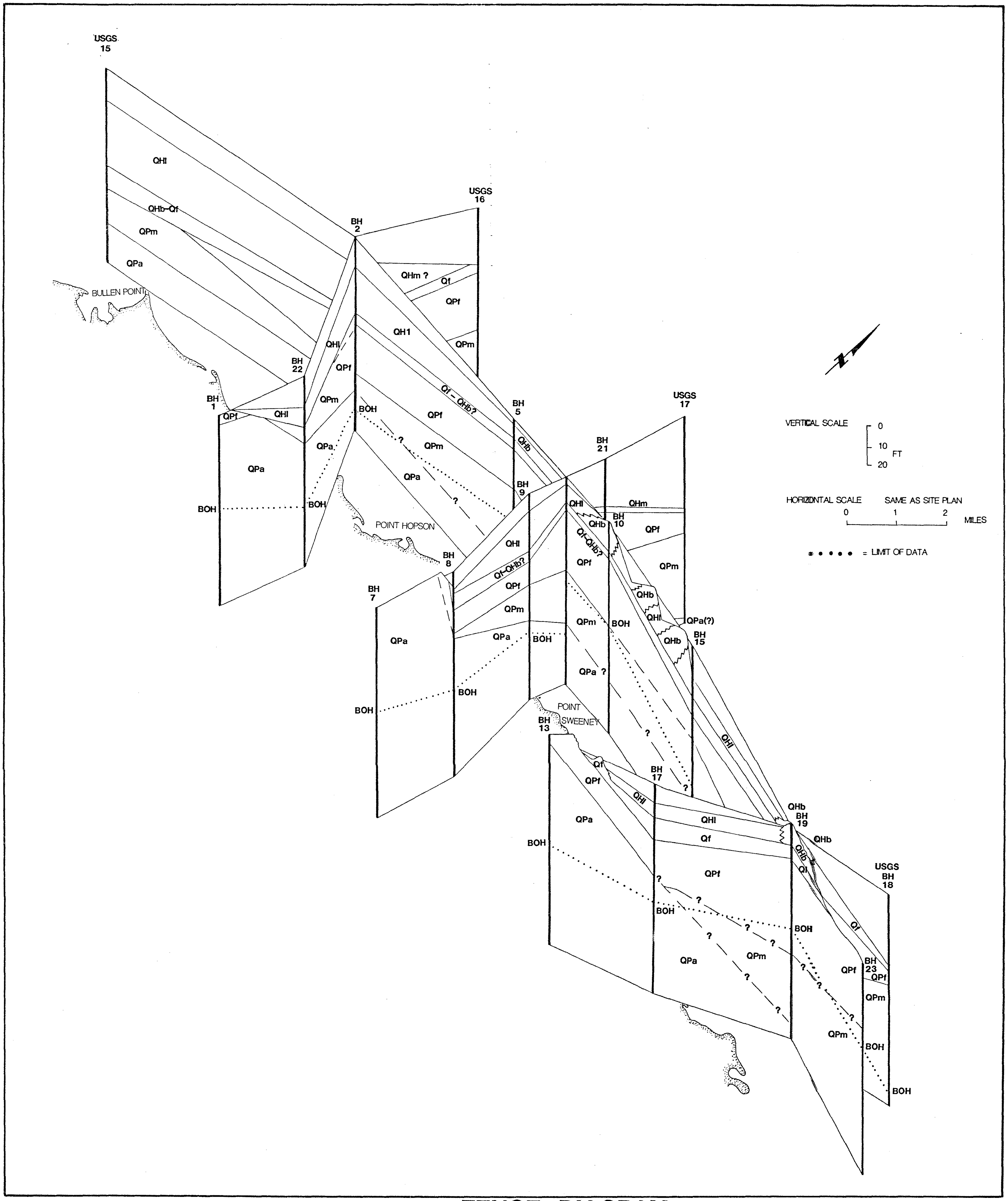
SURFICIAL GEOLOGIC MAP

POINT THOMSON DEVELOPMENT PROJECT
WINTER 1982 GEOTECHNICAL STUDY
EXXON COMPANY USA

PLATE III-4

DRAWN	JOB NUMBER	APPROVED	DATE
DCR	9612,031.08	DEB	5/82

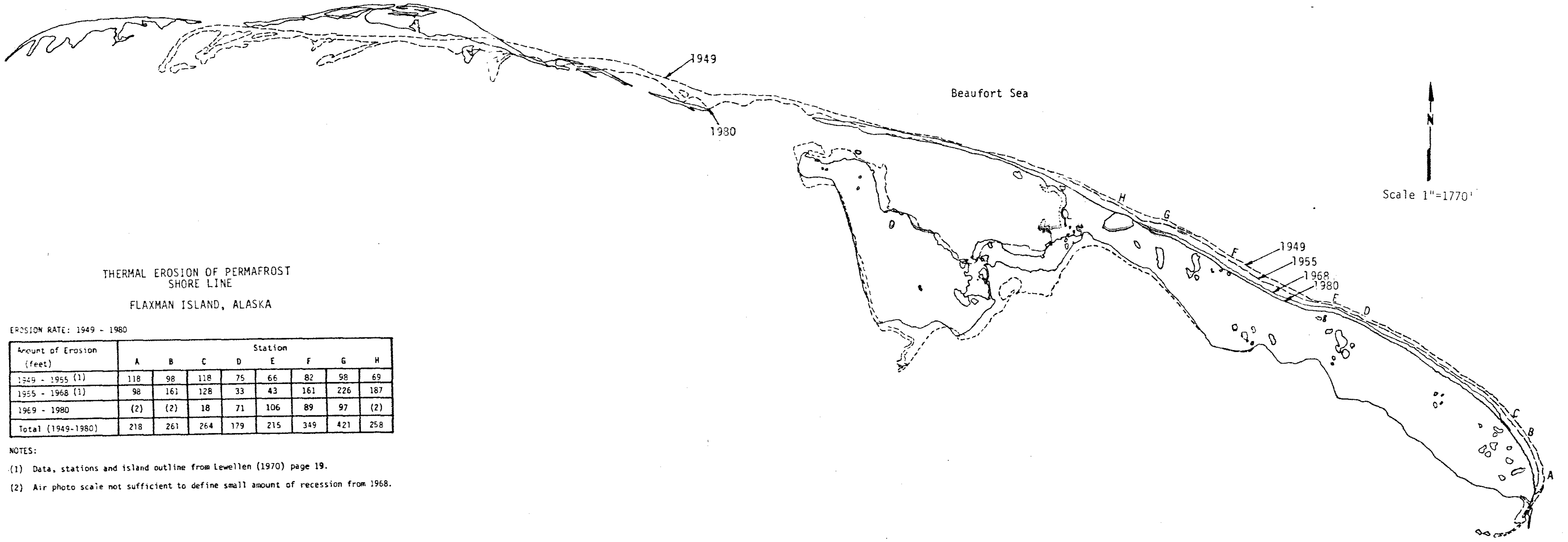


Harding Lawson Associates
Engineers Geologists
& Geophysicists

FENCE DIAGRAM
Pt. Thomson Development Project
Winter 1982 Geotechnical Study
EXXON Company, USA

PLATE
III-5

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
JP	9612,031,08		4/82		



THERMAL EROSION OF PERMAFROST
SHORE LINE

FLAXMAN ISLAND, ALASKA

EROSION RATE: 1949 - 1980

Amount of Erosion (feet)	Station A	B	C	D	E	F	G	H
1949 - 1955 (1)	118	98	118	75	66	82	98	69
1955 - 1968 (1)	98	161	128	33	43	161	226	187
1969 - 1980	(2)	(2)	18	71	106	89	97	(2)
Total (1949-1980)	218	261	264	179	215	349	421	258

NOTES:

(1) Data, stations and island outline from Lewellen (1970) page 19.

(2) Air photo scale not sufficient to define small amount of recession from 1968.

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Engineers Geologists
& Geophysicists

Flaxman Island Erosion, 1949-1980
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

III-9

9612,031.08 4/82

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TERRAIN UNIT MAP

POINT THOMSON DEVELOPMENT PROJECT
WINTER 1982 GEOTECHNICAL STUDY
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III-10

9612,031.08 DEB 4/82

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REGIONAL SOILS MAP

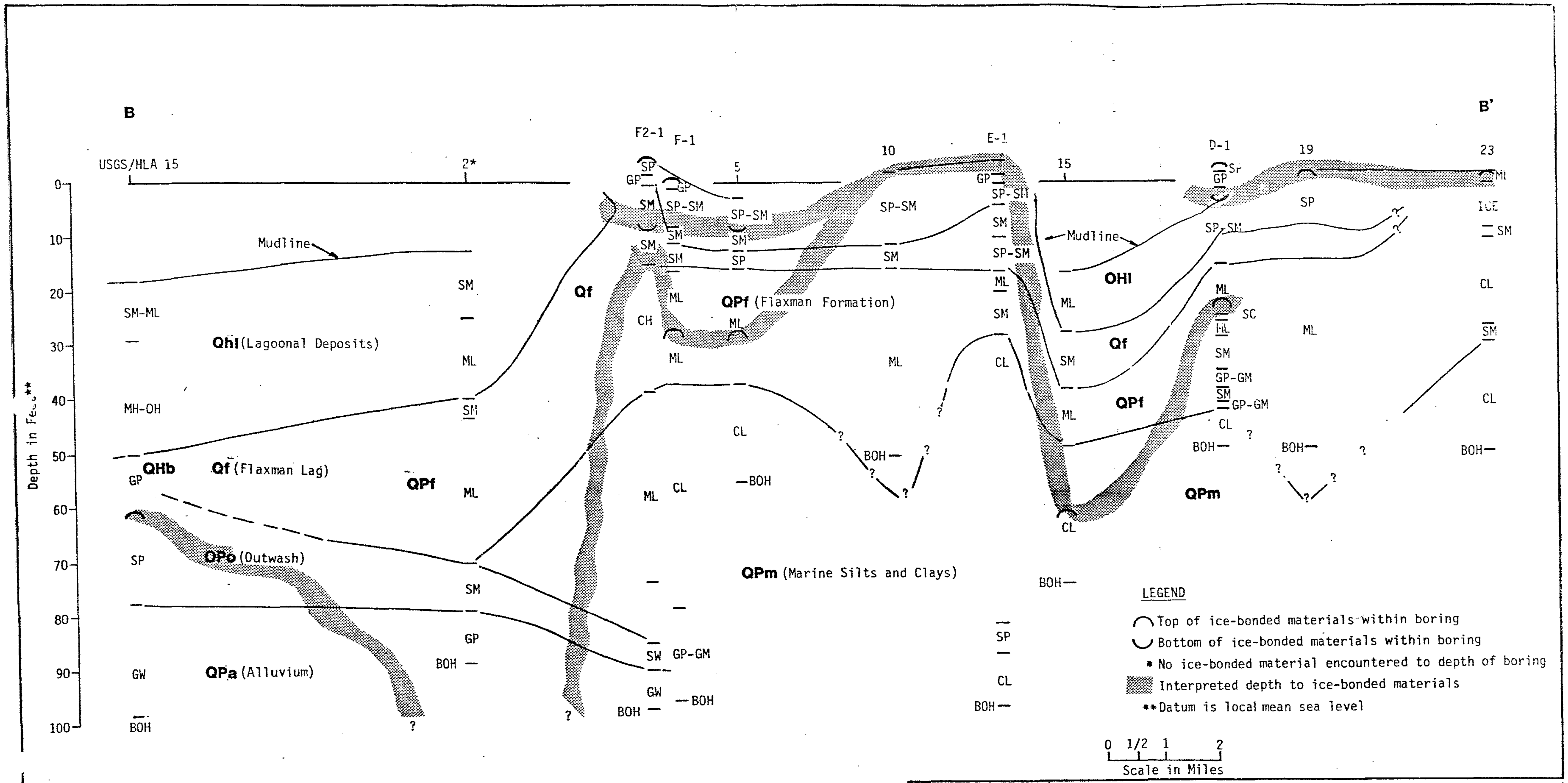
POINT THOMSON DEVELOPMENT PROJECT
WINTER 1982 GEOTECHNICAL STUDY
EXXON COMPANY USA

IV1

9612,031.08

APPROVED DEB

5/82



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Geologic Cross-Section B-B'
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

PLATE **IV-4**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
SP	9612,031.08	DEB	4/82		