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).

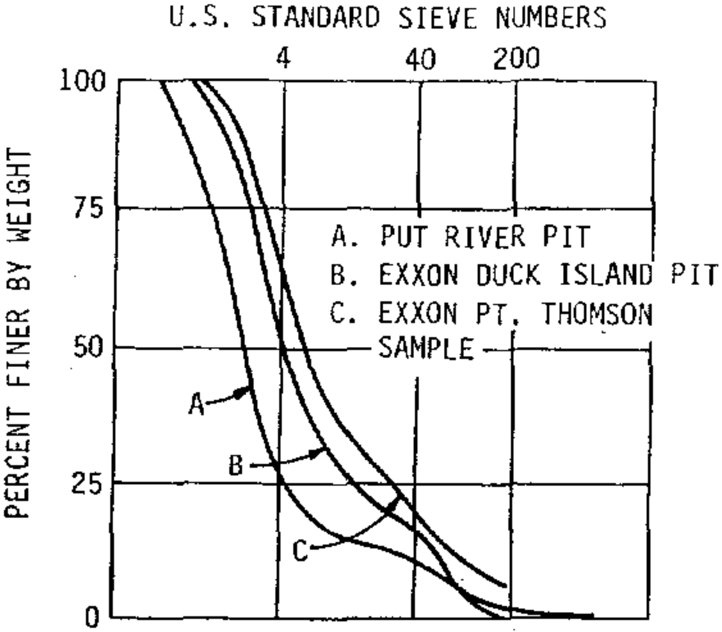

Figure 1 PARTICLE SIZE ANALYSES OF REPRESENTATIVE GRAVEL SAMPLES NORTH SLOPE

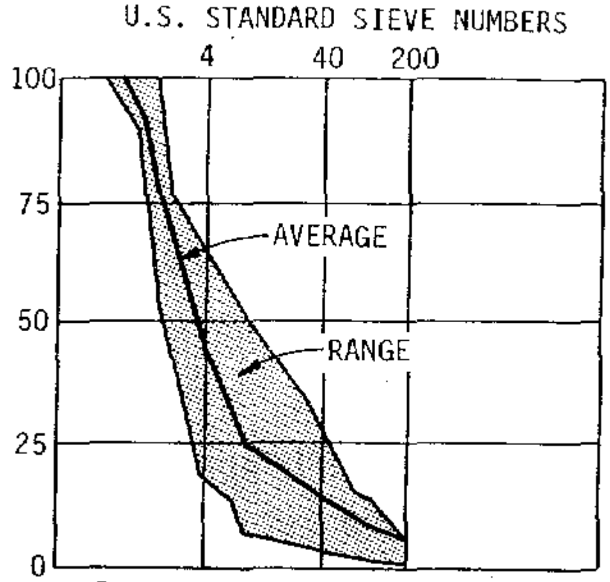

Figure 2 PT. THOMSON OFFSHORE GRAVEL GRADATION

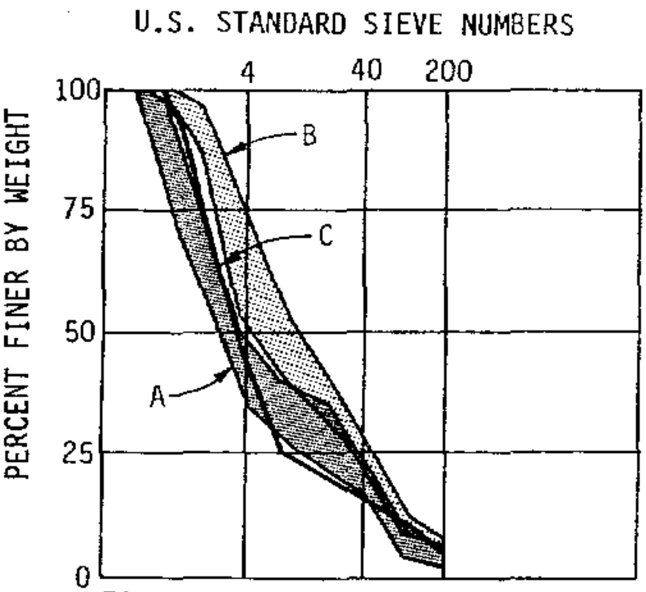

Figure 3 PARTICLE SIZE ANALYSIS OF REPRESENTATIVE GRAVEL SAMPLES POINT THOMSON AREA

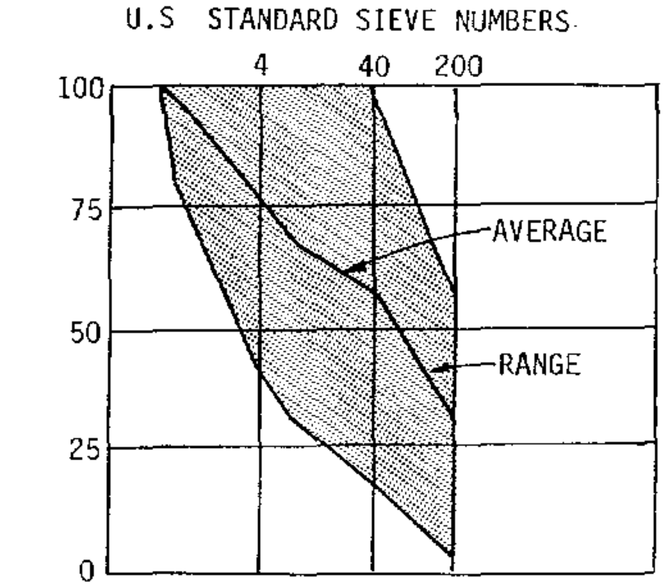

Figure 4 PT. THOMSON OFFSHORE, NEAR SURFACE SILTY SAND/GRAVEL GRADATION

Legend

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

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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
DCR	9612,031.08	DEB	4/82		

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

<u>Category b: Above-Water Placement</u>. 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

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

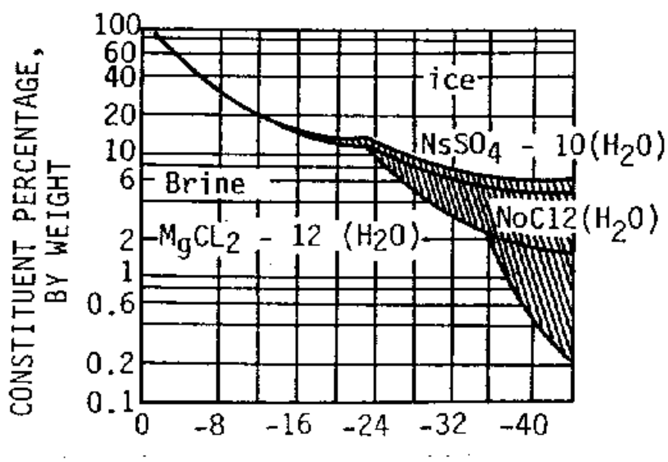
i_i = initial ice content

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

$\nu^{1} = \frac{\text{weight of brine}}{\text{weight of ice and brine}}$ = 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
EXXON Company, U.S.A.

PLATE **V-2**

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JR	9612,031.08	TPB	4/82		

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 e [2]	Dry Density γ_d (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 e [1]	Dry Density γ_d (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).

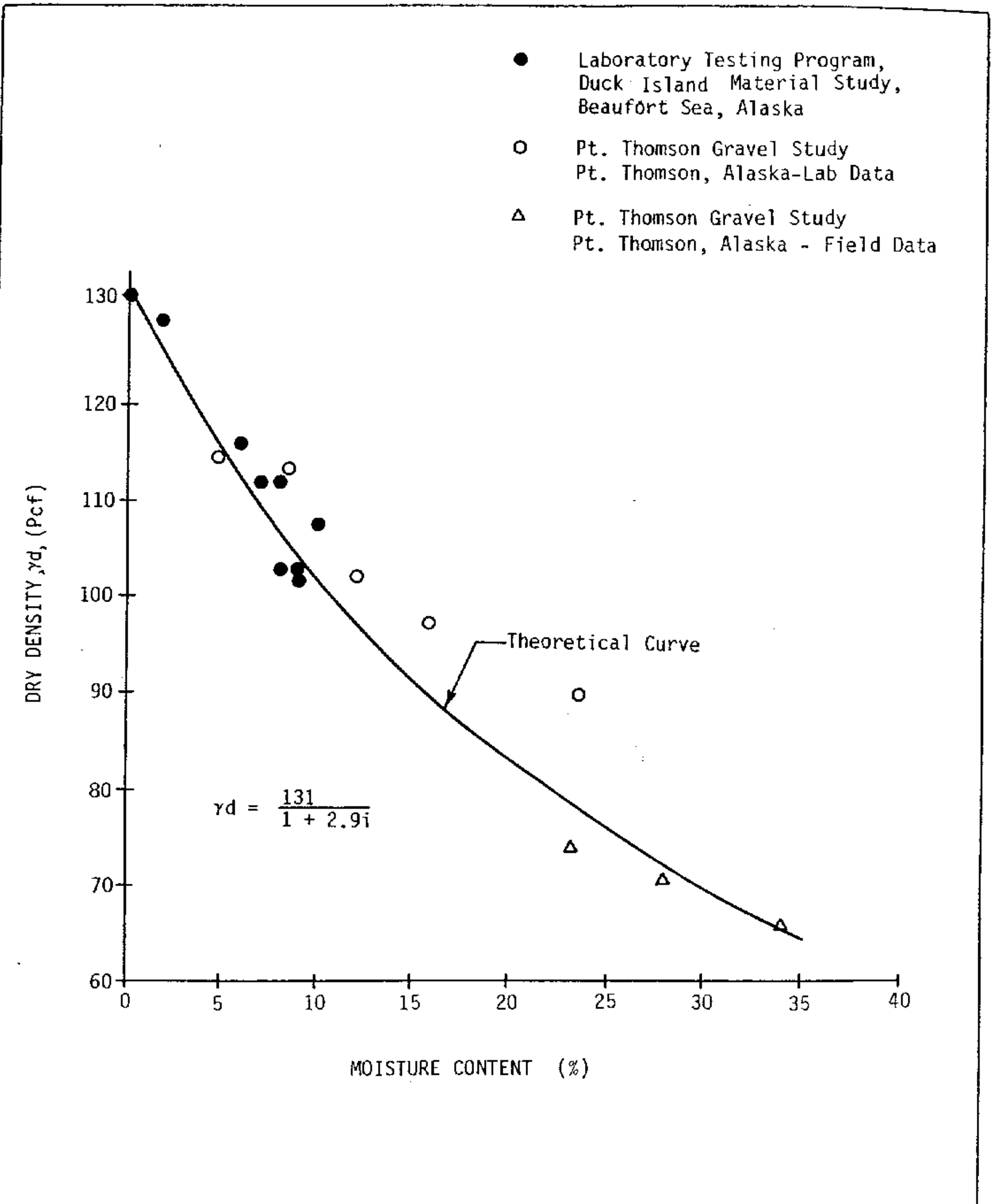
$$\gamma_d = \frac{131}{1 + 2.9i} \qquad \text{(V-2)}$$

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
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

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. <u>Mechanical Properties of Unbonded Fill Materials</u>

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. <u>Mechanical Properties of Ice-Bonded Fill Materials</u>

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[1]		Poisson's Ratio μ	Shear Strength		[2] One-Dimensional Compression Modulus	
	k	n		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	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.

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

$\dot{\varepsilon} = \dot{\varepsilon}_c \left(\frac{\sigma}{\sigma_c}\right)^n$ 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) (@ $\dot{\varepsilon}_c$ = 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 Ø (degrees)	26	10	5	18	22
($N_Ø$ - 1)	1.56	0.42	0.19	0.89	1.20
Adjusted Creep Modulus, σ_c	21.0+1.56 σ_3	②(45.0+0.42 σ_3)	48.0+0.19 σ_3	2.98+0.89σ_3	3.76+1.20σ_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)
   $\dot{\varepsilon}$ = strain rate resulting from applied stress, σ
   $\dot{\varepsilon}_c$ = 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
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Winter 1982, Geotechnical Study
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PLATE **V-7**

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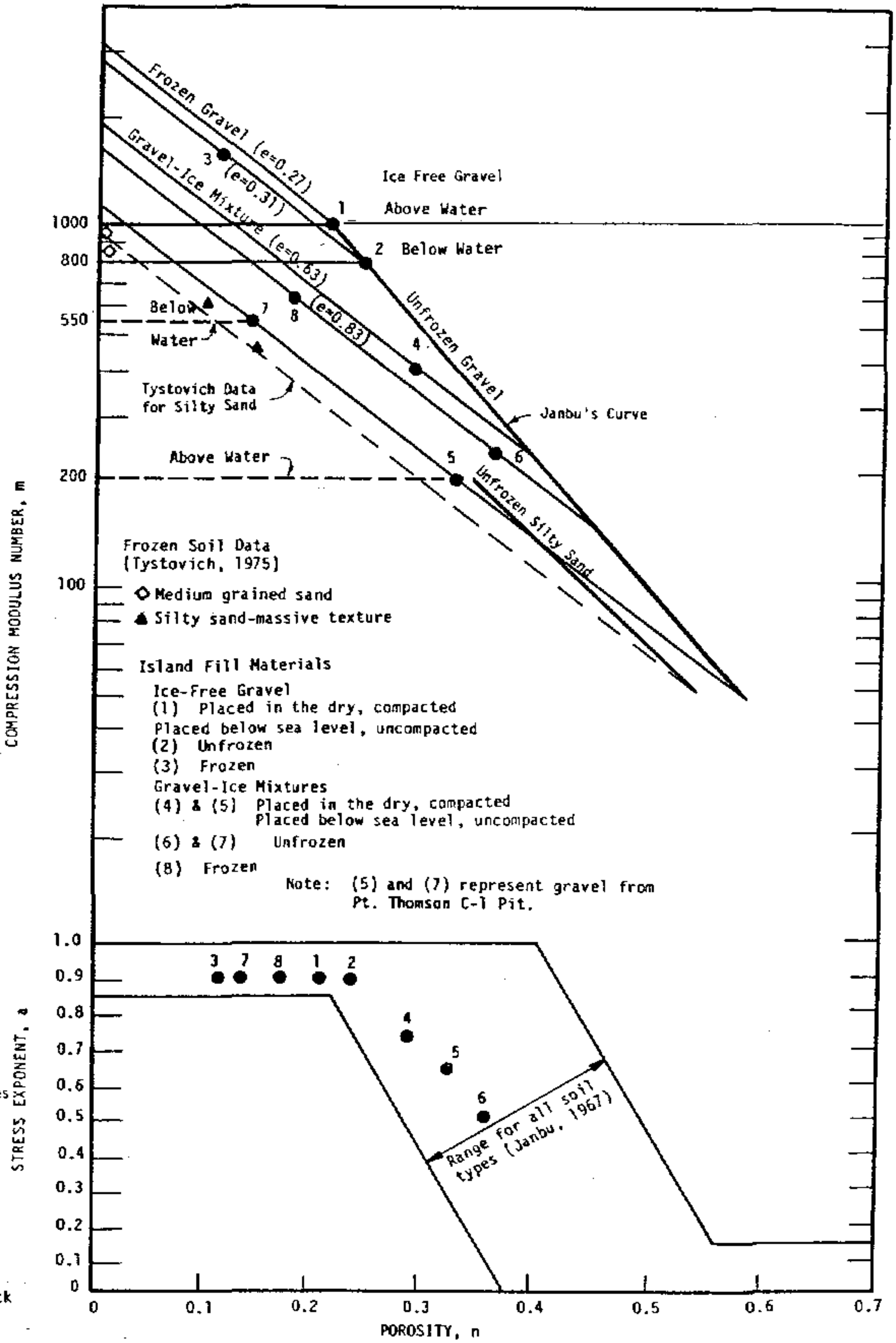

NOTES:
(1) See Appendix E for explanation of procedures used to develope this Plate
(2) $M=mPa[\frac{\sigma_0}{P_a}]^{1-a}$

where:
P_a=atmospheric pressure

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

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.

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

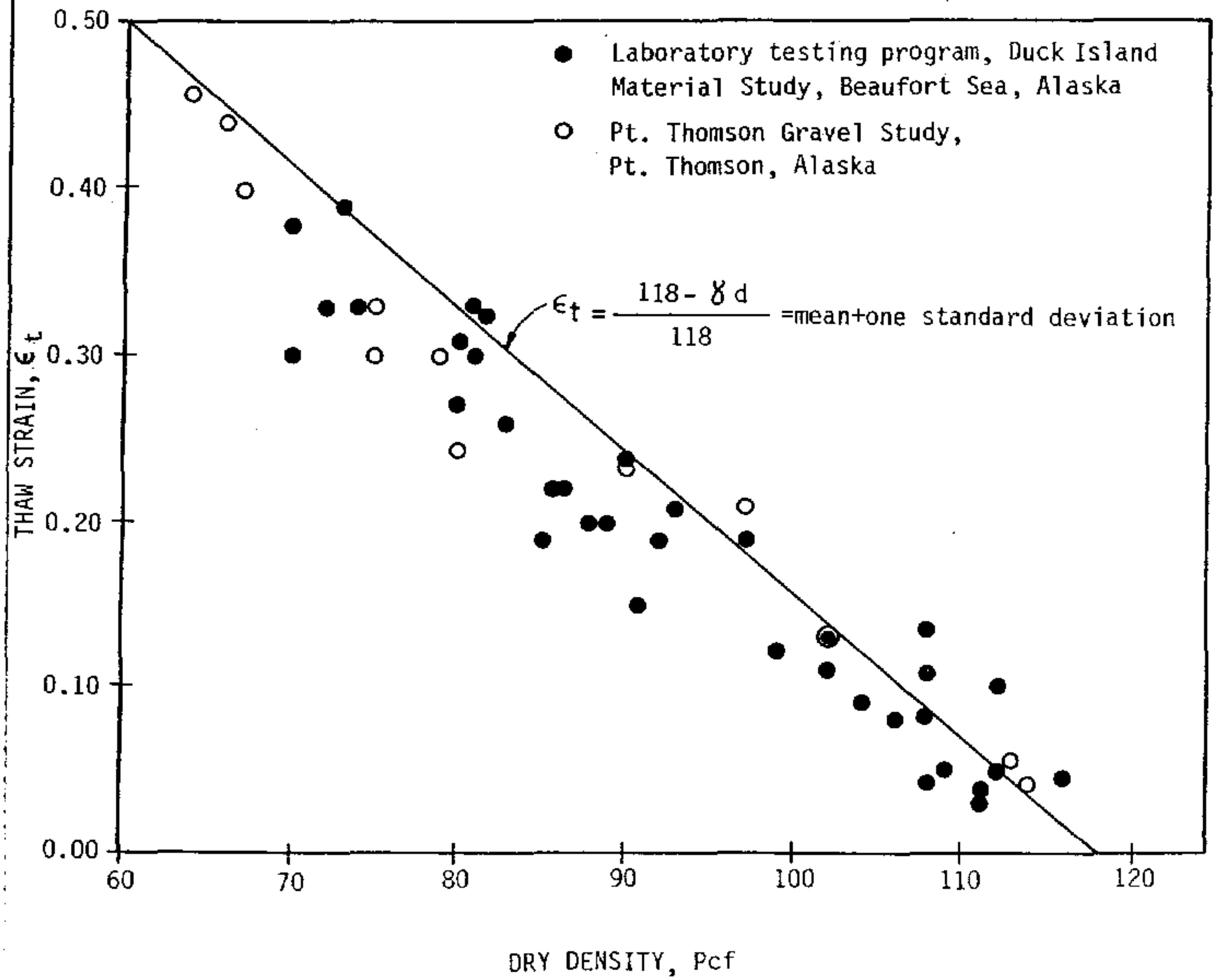
Thaw-strain data for gravel-ice mixtures compacted to various dry densities are presented in Plate V-9. Equation V-3 represents the mean plus one standard deviation of the test data. The thaw-strain tests were performed with surcharge loads of 500 and 1000 psf, and no significant difference in thaw-strain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

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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)

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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

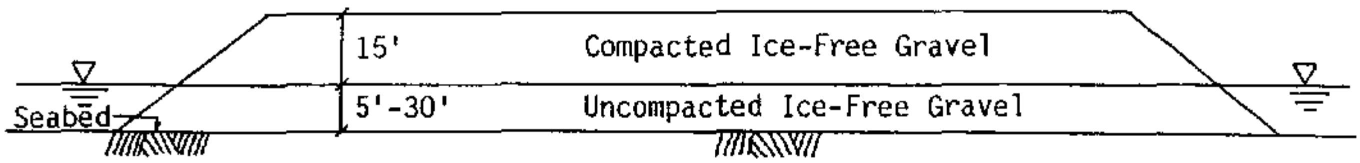
Because of the availability of gravel in the area, the islands will probably be constructed of ice-free gravel or gravel-ice mixtures. The islands will become frozen with time. The freeze front will advance from the surface and progressively penetrate deeper in subsequent winters, eventually extending 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.

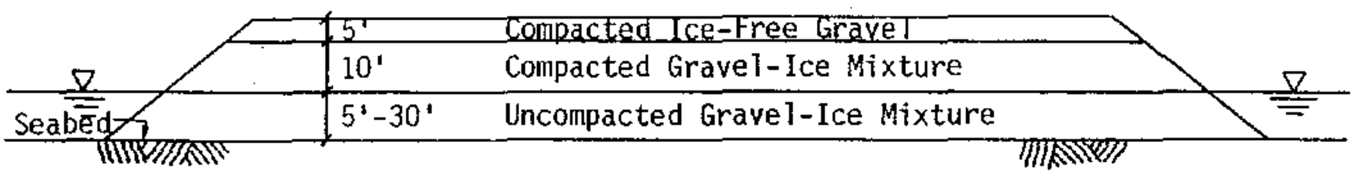
2. Slope Stability

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

Case "a" Summer Construction



Case "b" Winter Construction



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Probable Island Fill Sections
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
EXXON Company, U.S.A.

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

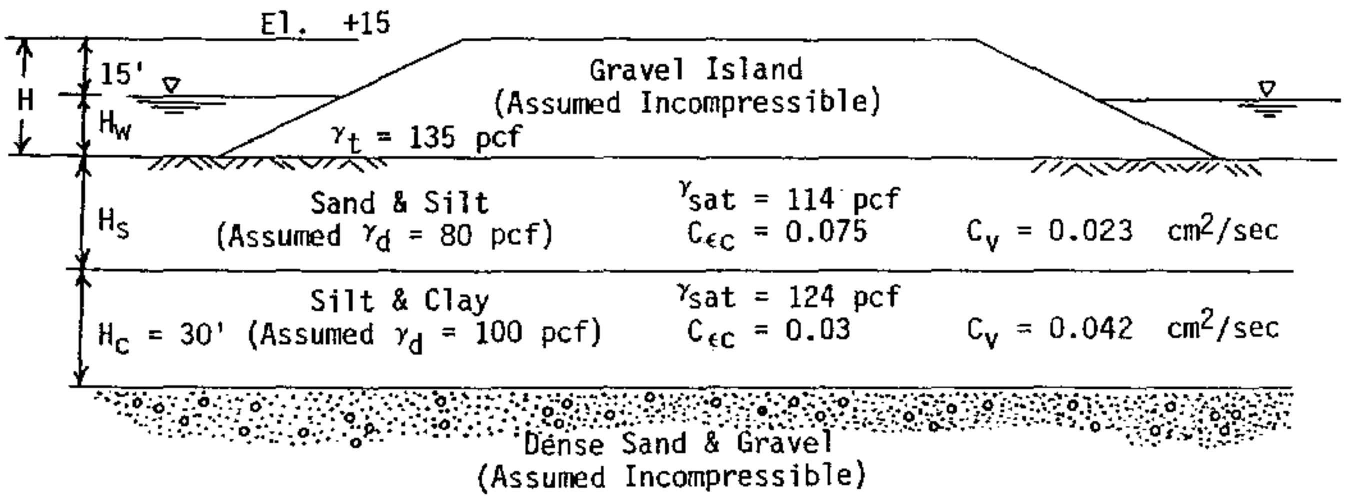
	Factor of Safety					
	2 to 1 Slope		3 to 1 Slope		4 to 1 Slope	
Case	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$	$\emptyset' = 35^{\circ}$	$\emptyset' = 40^{\circ}$
End-of-Construction	1.4	1.7	2.1	2.5	2.8	3.4
Seepage Parallel to Slope	0.7	0.9	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
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **VI-2**

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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. <u>Thaw-strain Settlement of Natural Sediments at the Barrier Islands</u>

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.

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

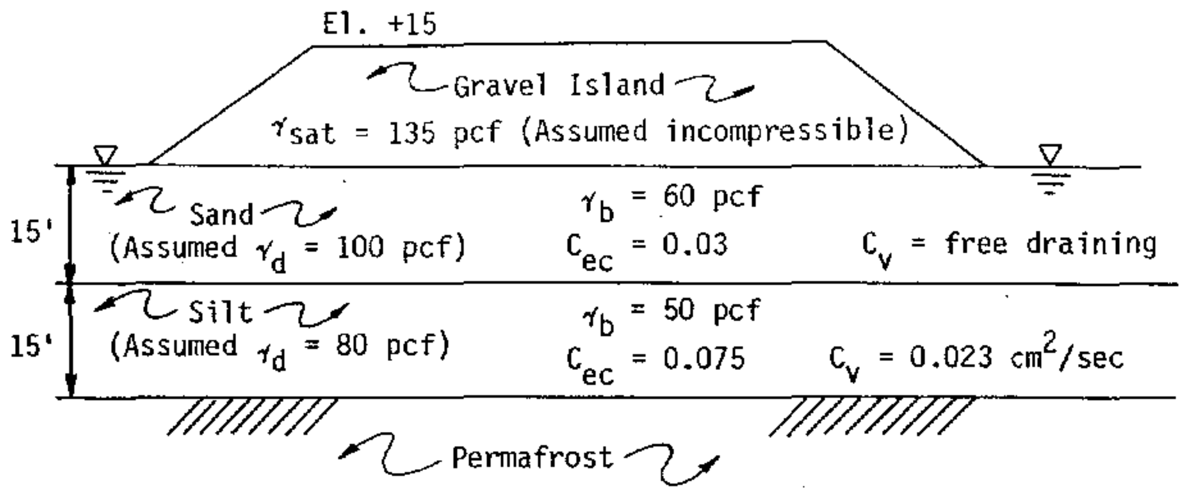
Where TS = Total thaw-strain settlement in feet

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.

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Settlement of Gravel Pads on Barrier Islands
Pt. Thomson Development Project, Winter 1982
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PLATE **VI-3**

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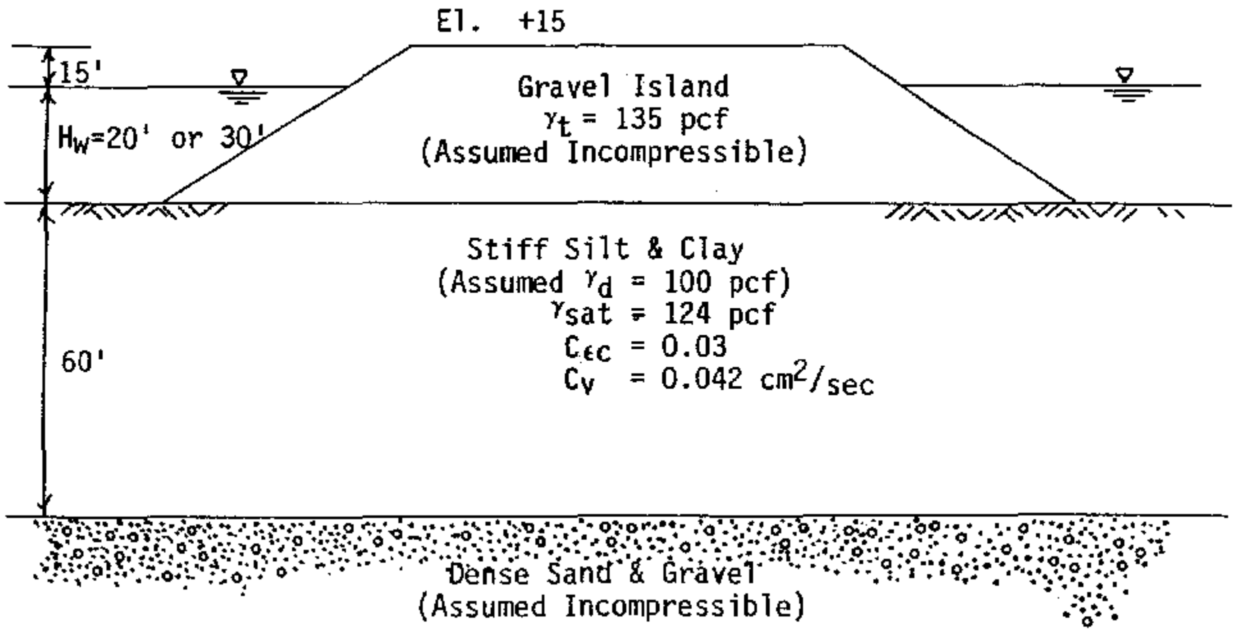

Water Depth (ft)	Total Settlement (ft)	Time at 90% Consolidation (months)
20	0.9	7
30	1.0	7

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Settlement of Gravel Island North of the Barrier Islands
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PLATE **VI-4**

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

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

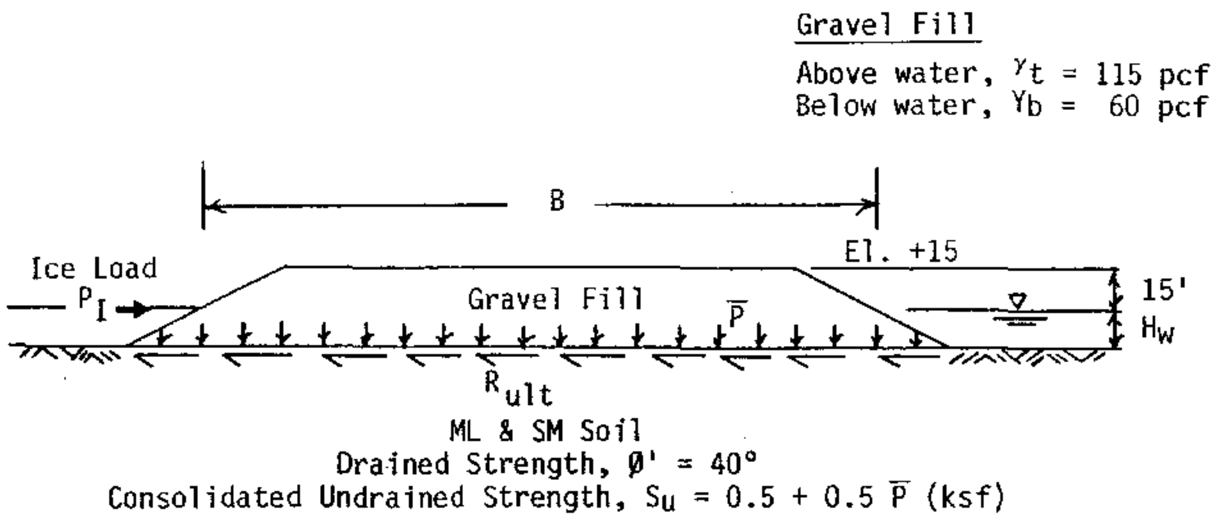
The resistance against base sliding is dependent on the thermal regime of the island. The freeze front will advance from the surface and eventually penetrate into the natural sediments. At the end of construction when the gravel fill is at its weakest, a large lateral ice load impinging on the island 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.

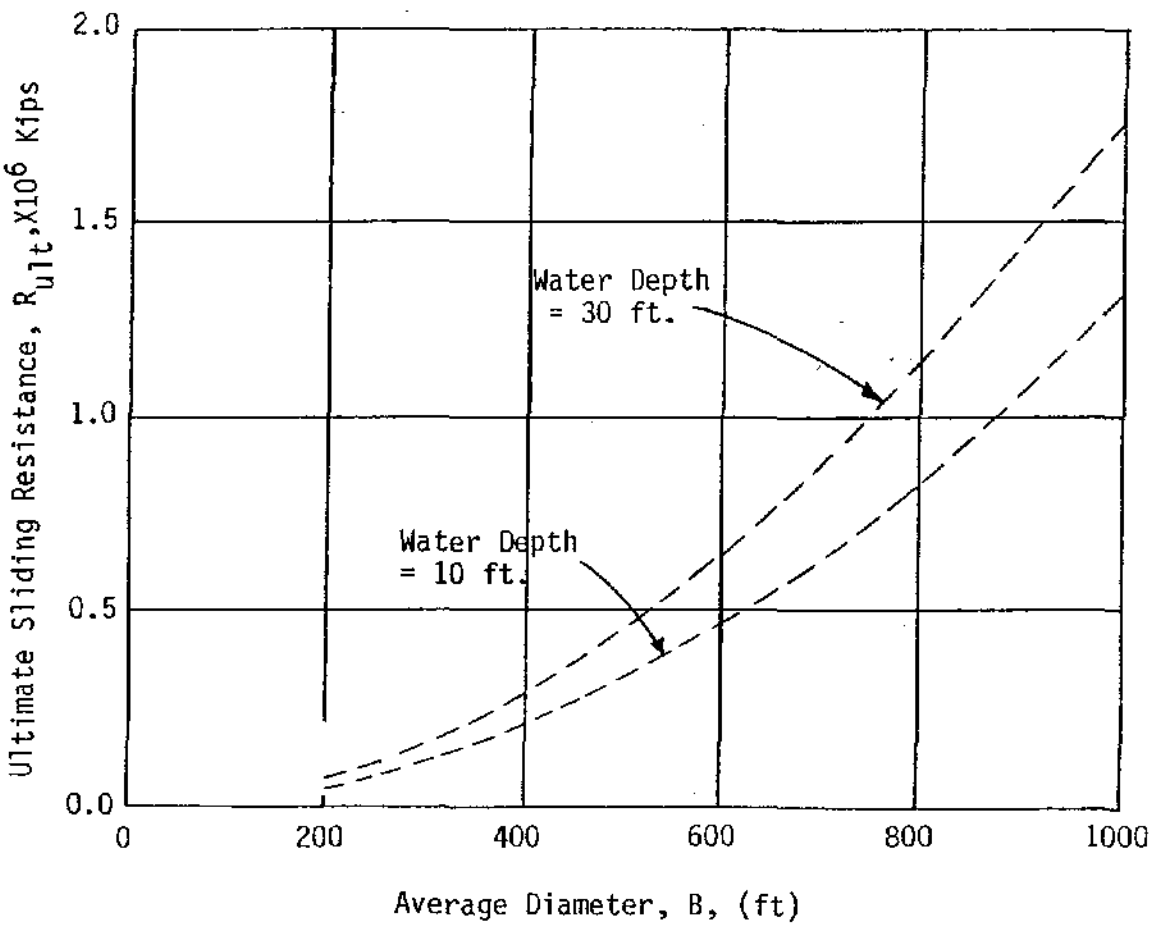
When a lateral ice load impinges on a frozen island, two things occur simultaneously: (1) the island compresses elastically in the direction of the load, and (2) shear stresses are induced into the underlying sediments which cause shear deformation. Therefore, the initial island movement is the result 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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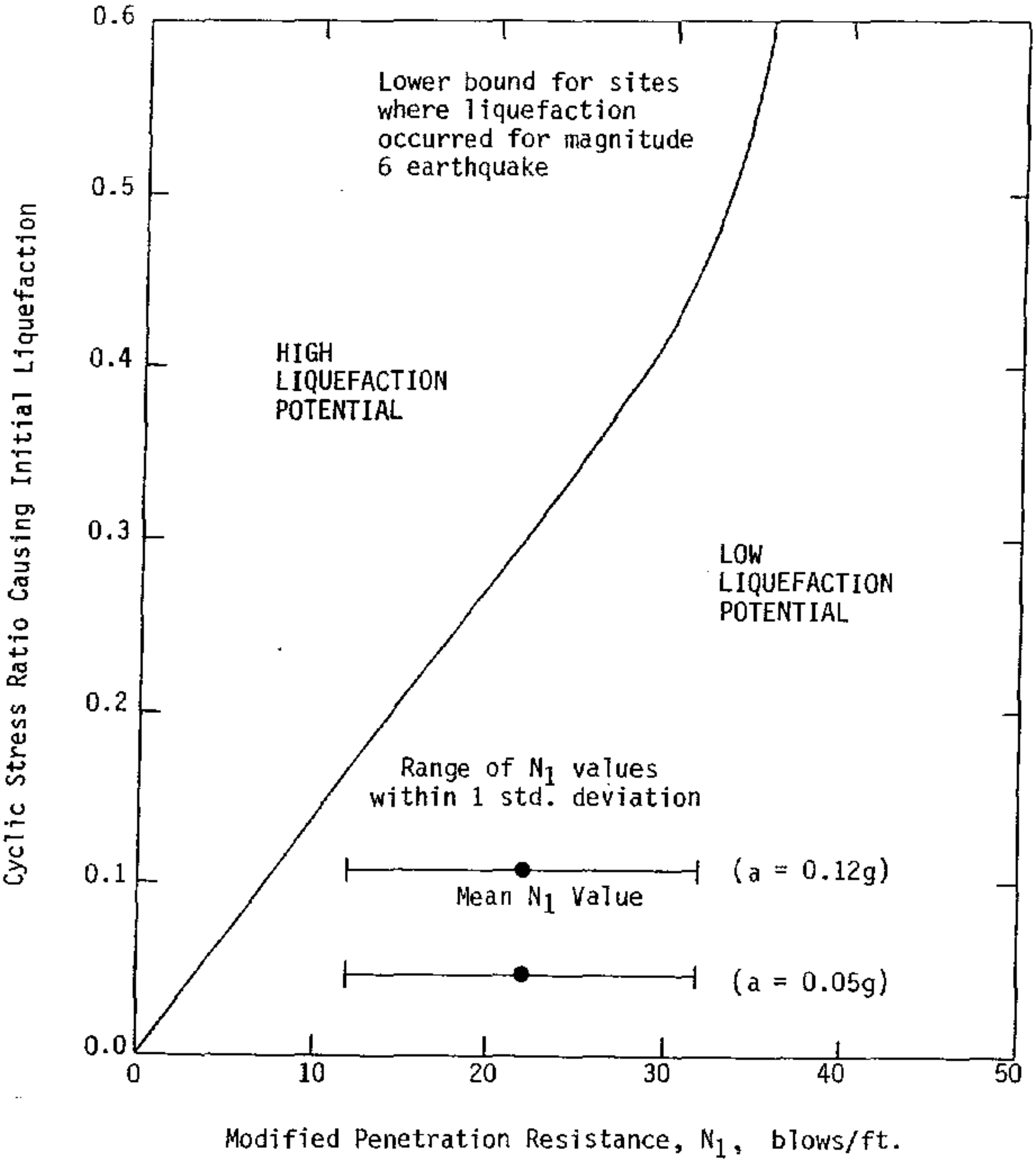
design earthquake of Richter magnitude 5.5 with an effective duration of 5 seconds and a horizontal ground acceleration of 0.12 g, as developed for the Alyeska Pipeline Service Company (Donovon and Singh, 1978); and (2) a ground acceleration of 0.05 g for Zone 1 seismicity in accordance with the 1976 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 ϕ', or undrained shear strength. These other methods are felt to be applicable during site-specific design.

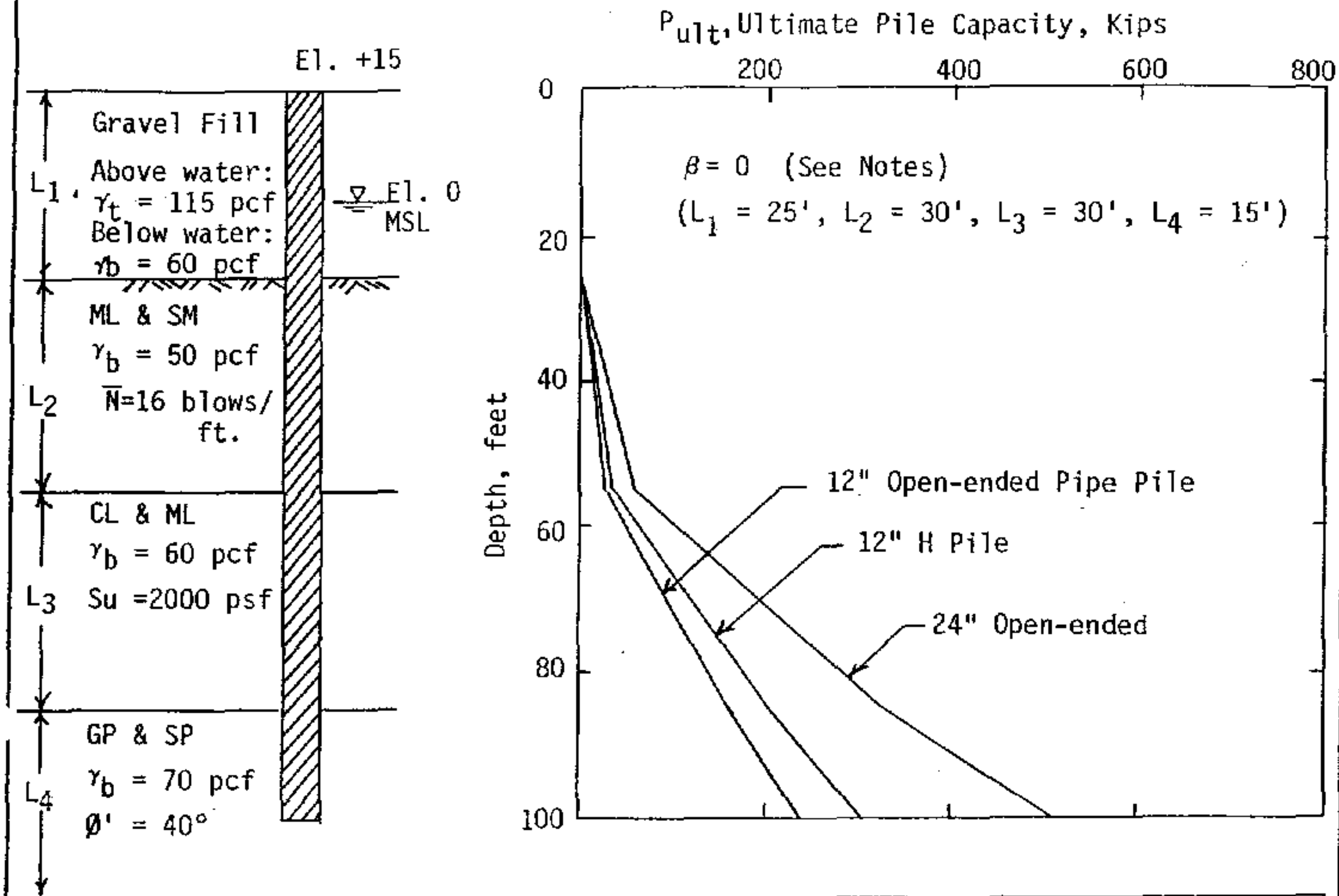
Side friction in the gravel fill depends on the quality of the fill, the thermal regime and the method of pile installation. If a gravel-ice mixture is used, excessive creep settlement will occur under a sustained load, eliminating any side friction support within the fill. If the gravel-ice fill thaws, 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 = β = 0 if fill remains frozen
β = -0.2 if fill becomes thawed
For fill with low ice content = β = 0.5 if pile was predrilled thru fill
β = 1.0 if no predrilling nor jetting

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Pile Capacity in Lagoon Areas
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **VI-7**

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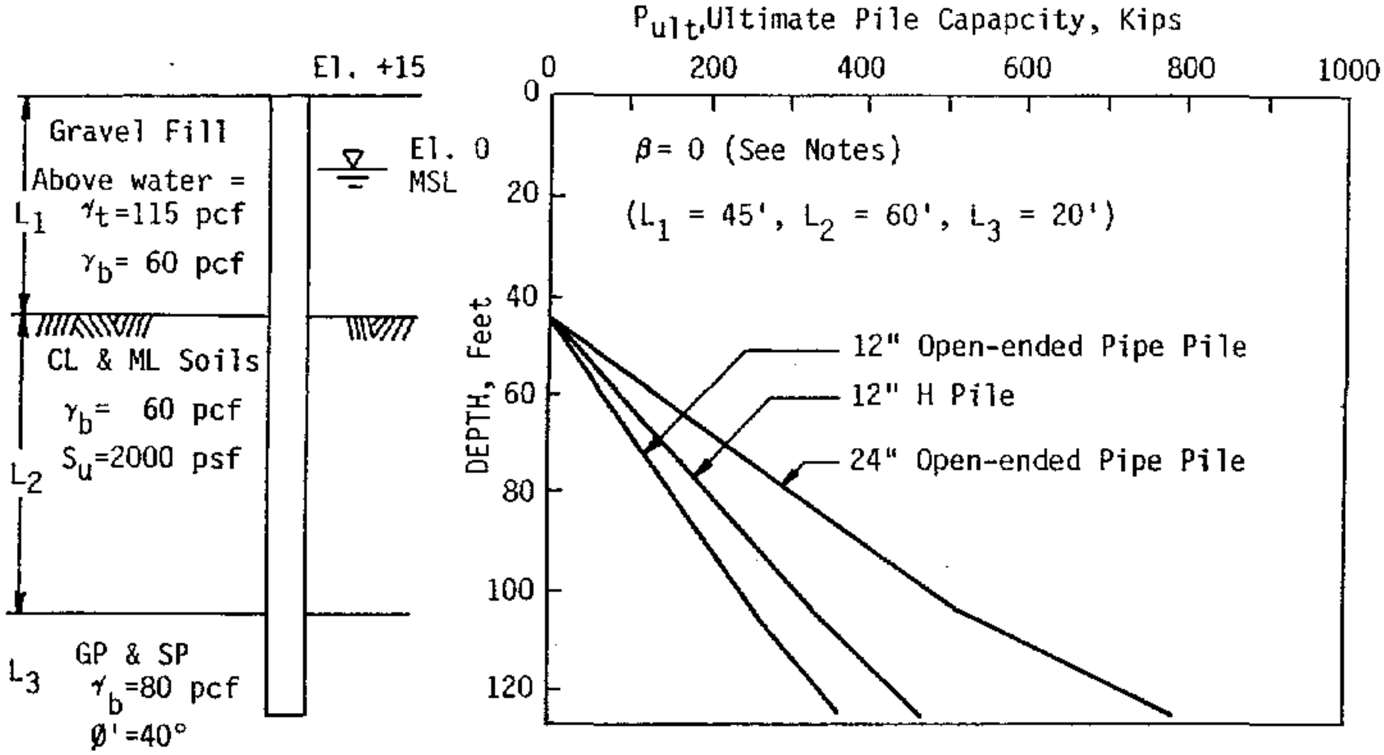
Purpose = Pile Capacity for Areas North of Barrier Islands



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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PLATE **VI-8**

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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.

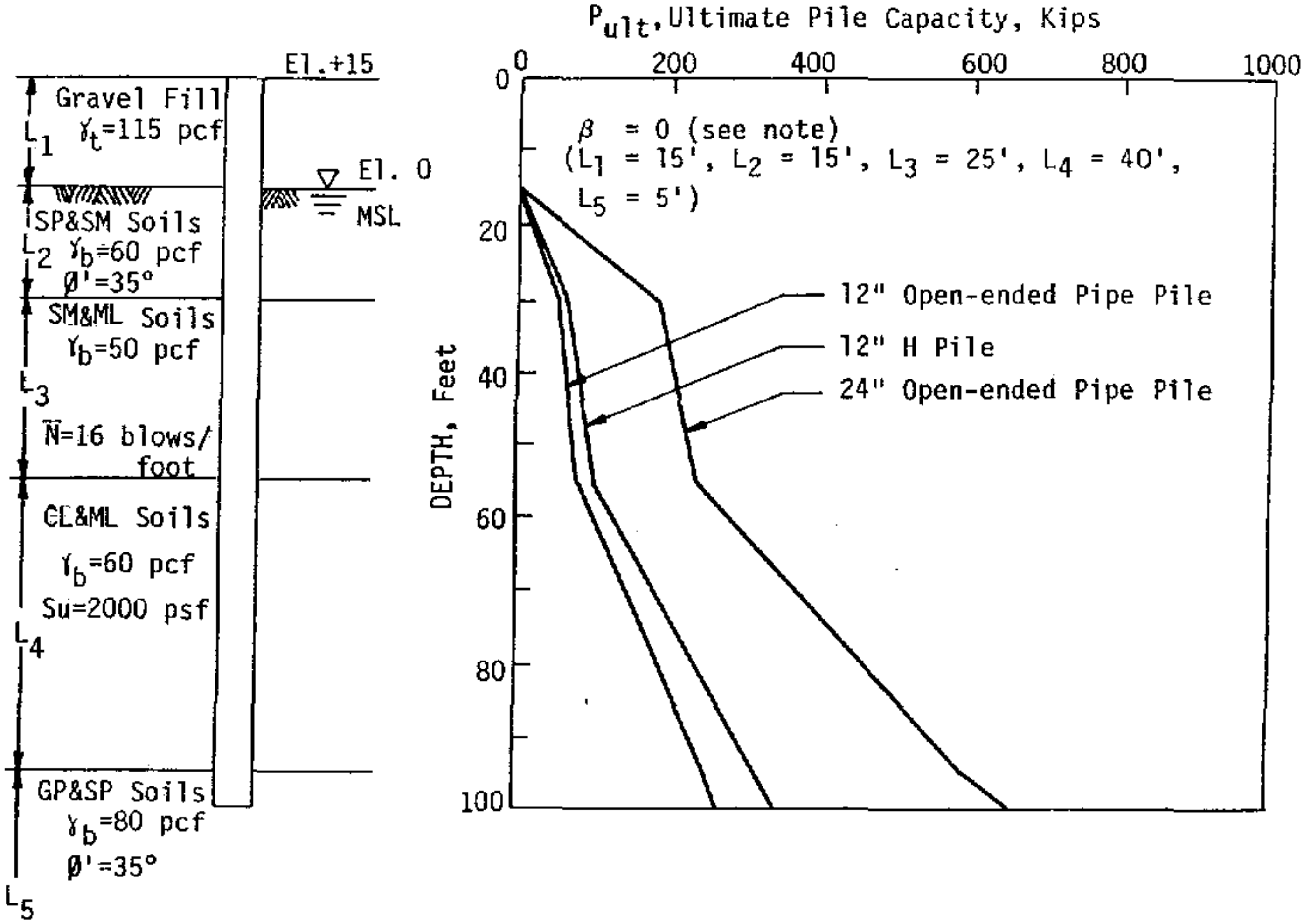
For pile groups, we recommend a center-to-center spacing of at least three pile diameters. A group efficiency of 1.0 can be used for preliminary design (Morgenstern, 1980).

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 = $R\left[13\beta + 1.13\ L_2 + 0.32\ L_3 + 1.4\ L_4 + 1.13\ L_5\right]$ Kips

where R = pile perimeter in feet

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

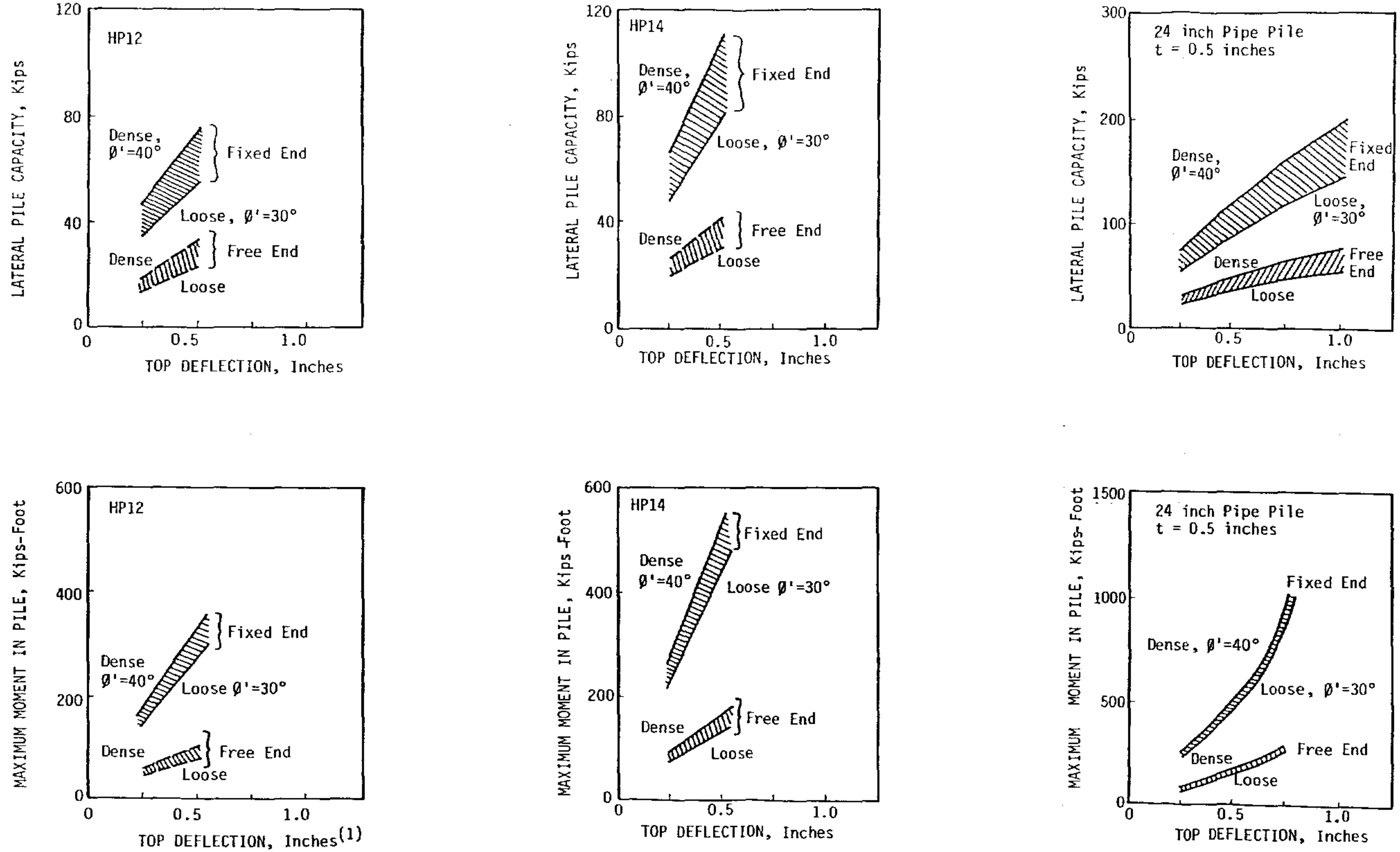
3. If the gravel fill and permafrost become thaw, the pile capacity and thaw-strain settlement should be evaluated on individual basis.

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^{0} 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 ($\emptyset' = 40^{0}$) and a loose sand ($\emptyset' = 30^{0}$), 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.

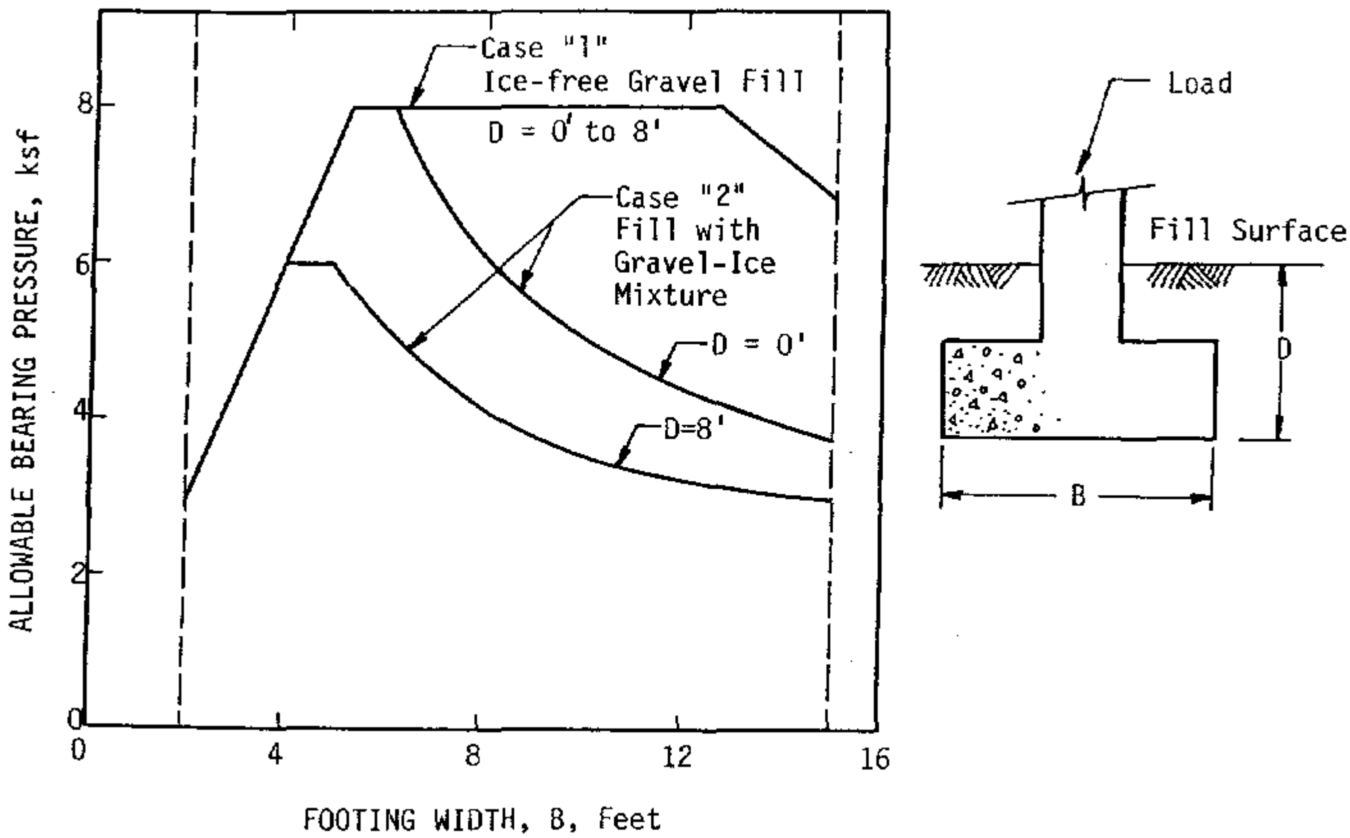
Settlement analyses were made for both bonded and unbonded conditions. For the bonded condition, the settlements are based on creep deformation theory using the cavity expansion method proposed by Ladanyi (1975). An average bonded fill temperature of -5^{0}C and a creep period of 25 years were assumed 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

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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

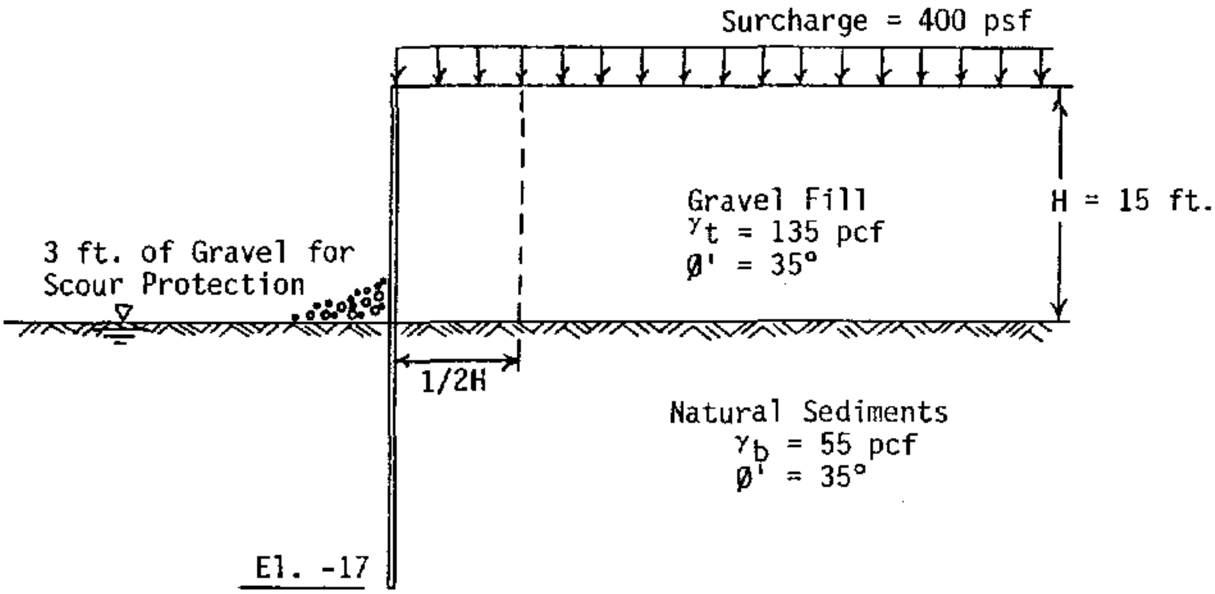
A dock structure may be constructed at an artificial island, at the end of a causeway extending from the shore or at the barrier islands.

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

PLATE
VI-12

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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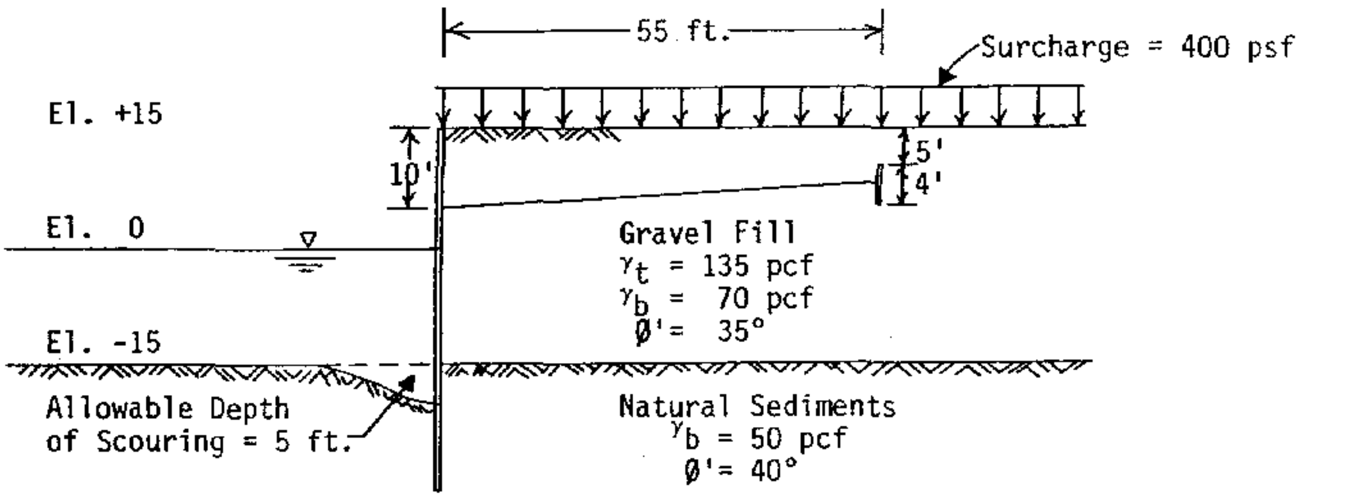

Depth of Penetration = 16 ft.

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
EXXON Company, U.S.A.

PLATE
VI-13

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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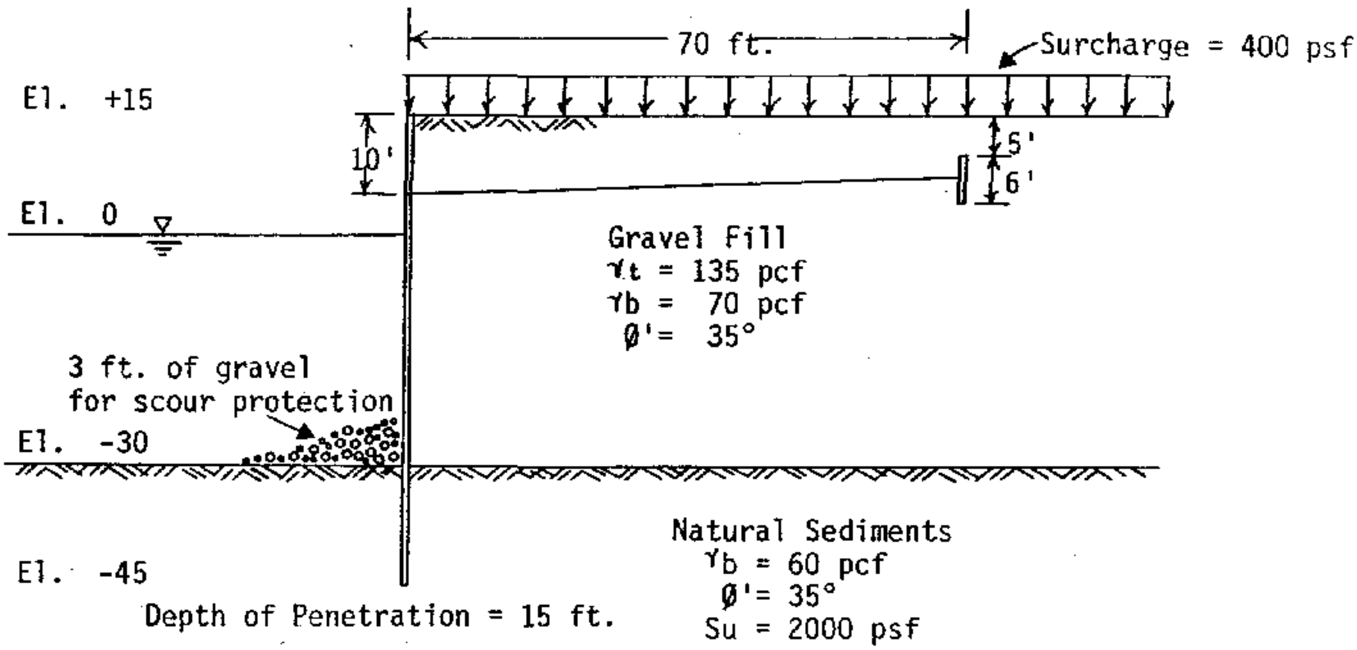

Method of Analysis = Free Earth Support Method with Rowe's Moment Reduction

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

PLATE **VI-14**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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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

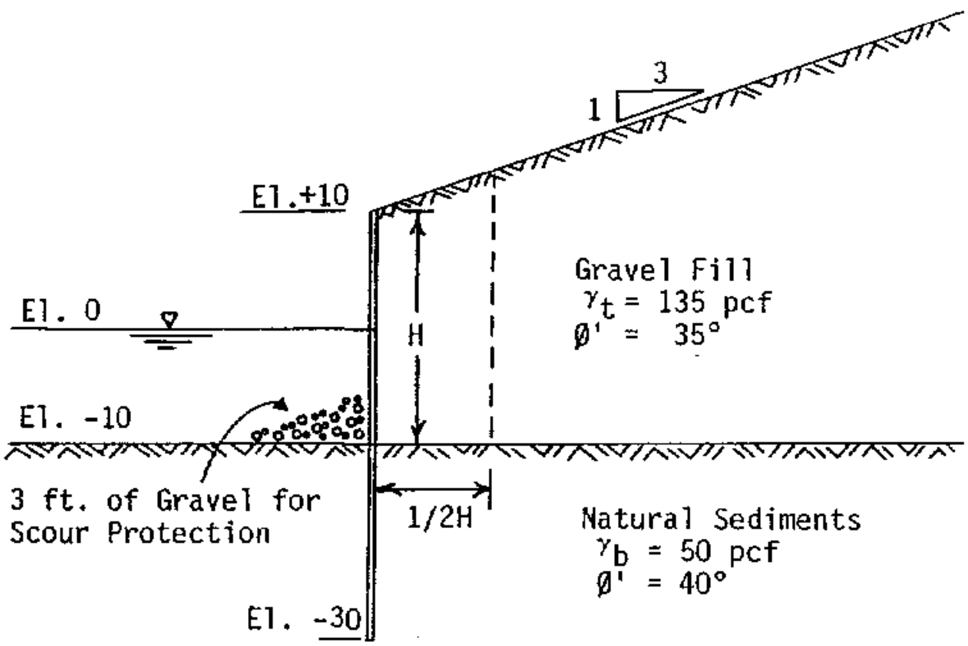
The top of the cantilever sheet pile wall for the abutments is assumed to be at Elevation +10 with backfil slopes of 3:1. The required pile length is 40 feet, and the depth of embedment below mudline is 20 feet. The required pile section is PZ-38 (ASTM A690 steel). A gravel blanket (3 feet thick) should 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

Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE **VI-15**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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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^{o}F (93^{o}C) for the produced fluid lines. The waterflood intake line will be cool, approximately 50^{o}F (10^{o}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^{0}F (-2^{0}C) in winter to about 36^{0}F ($+2^{0}$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^{0}F (93^{0}C). Longitudinal thermal expansion of the pipeline steel will impose substantial forces on the surrounding soil.

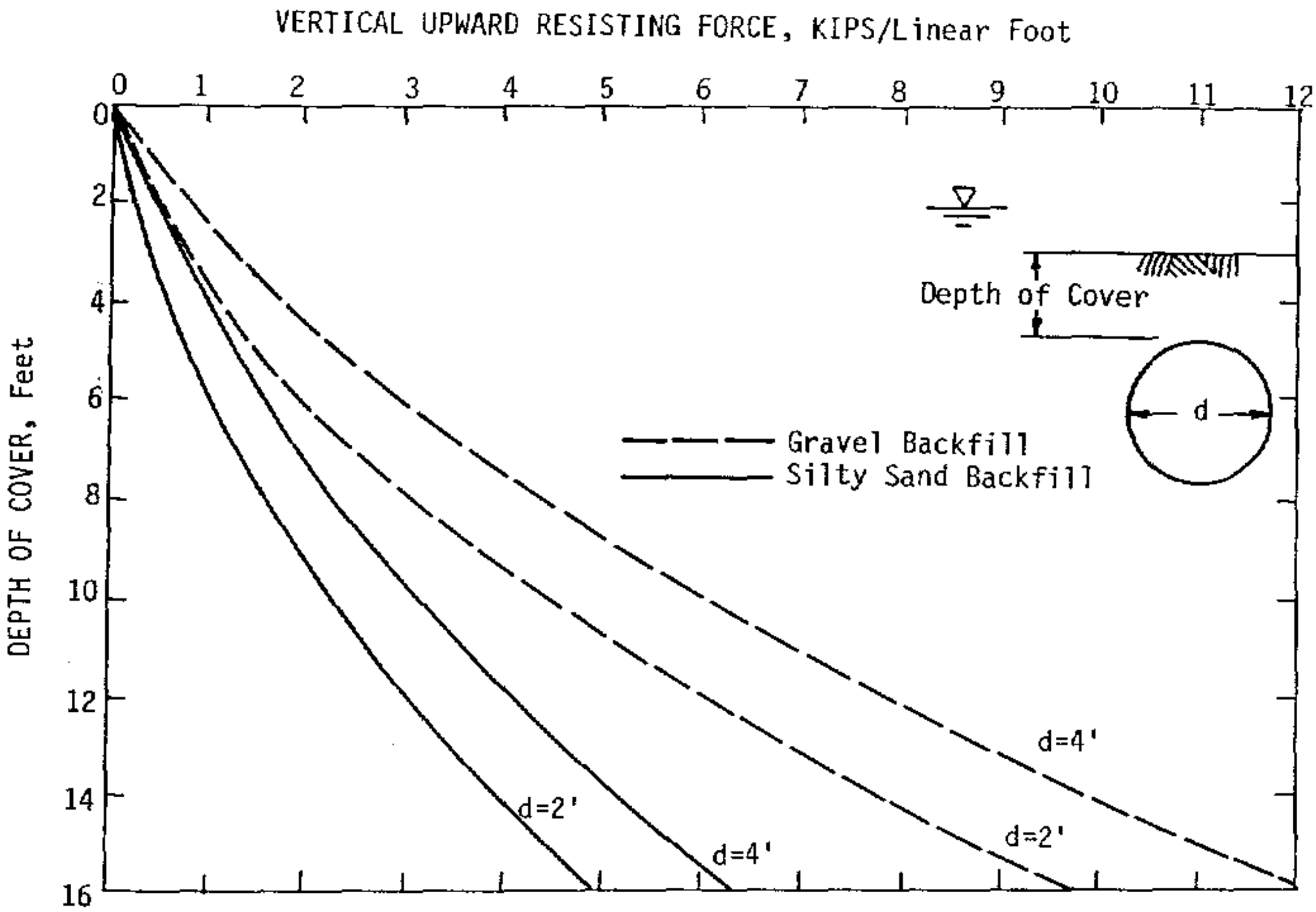
2. The pipeline will impose lateral forces on the adjacent soil resulting from surges and momentum changes at pipeline bends.

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
Geotechnical Study, EXXON Company, U.S.A.

PLATE **VI-16**

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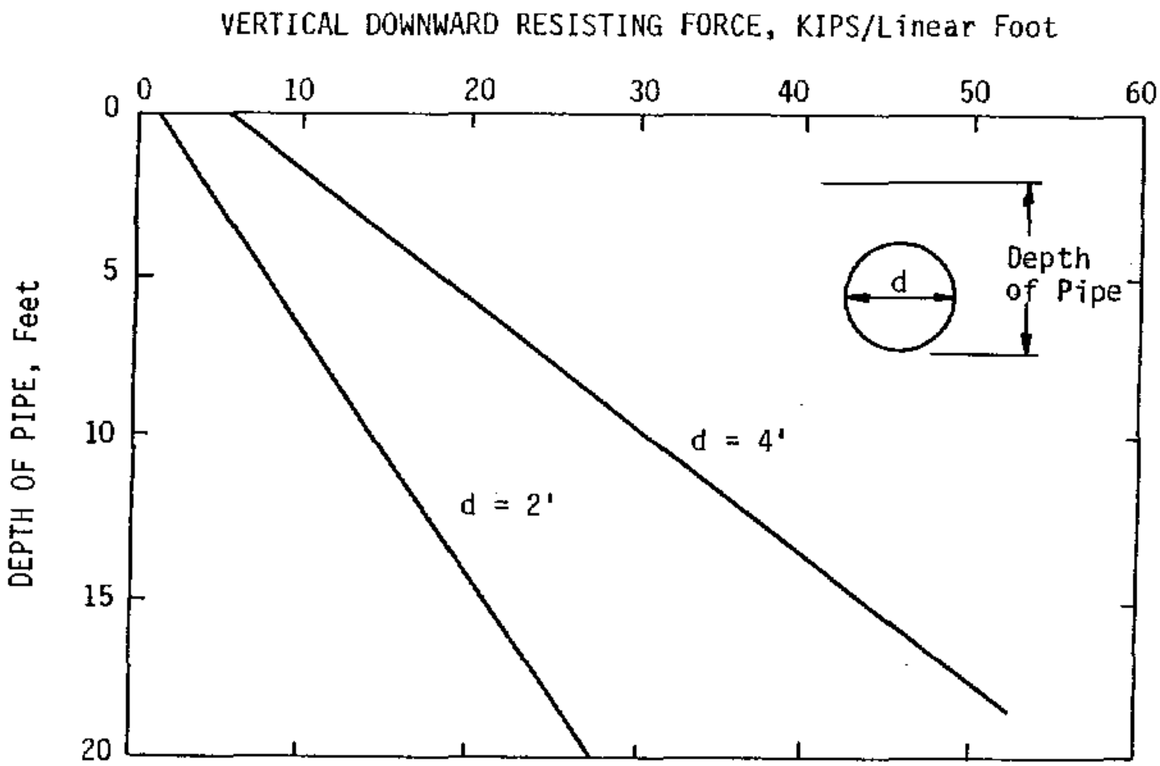

NOTES:

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
Geotechnical Study, EXXON Company, U.S.A.

PLATE **VI-1**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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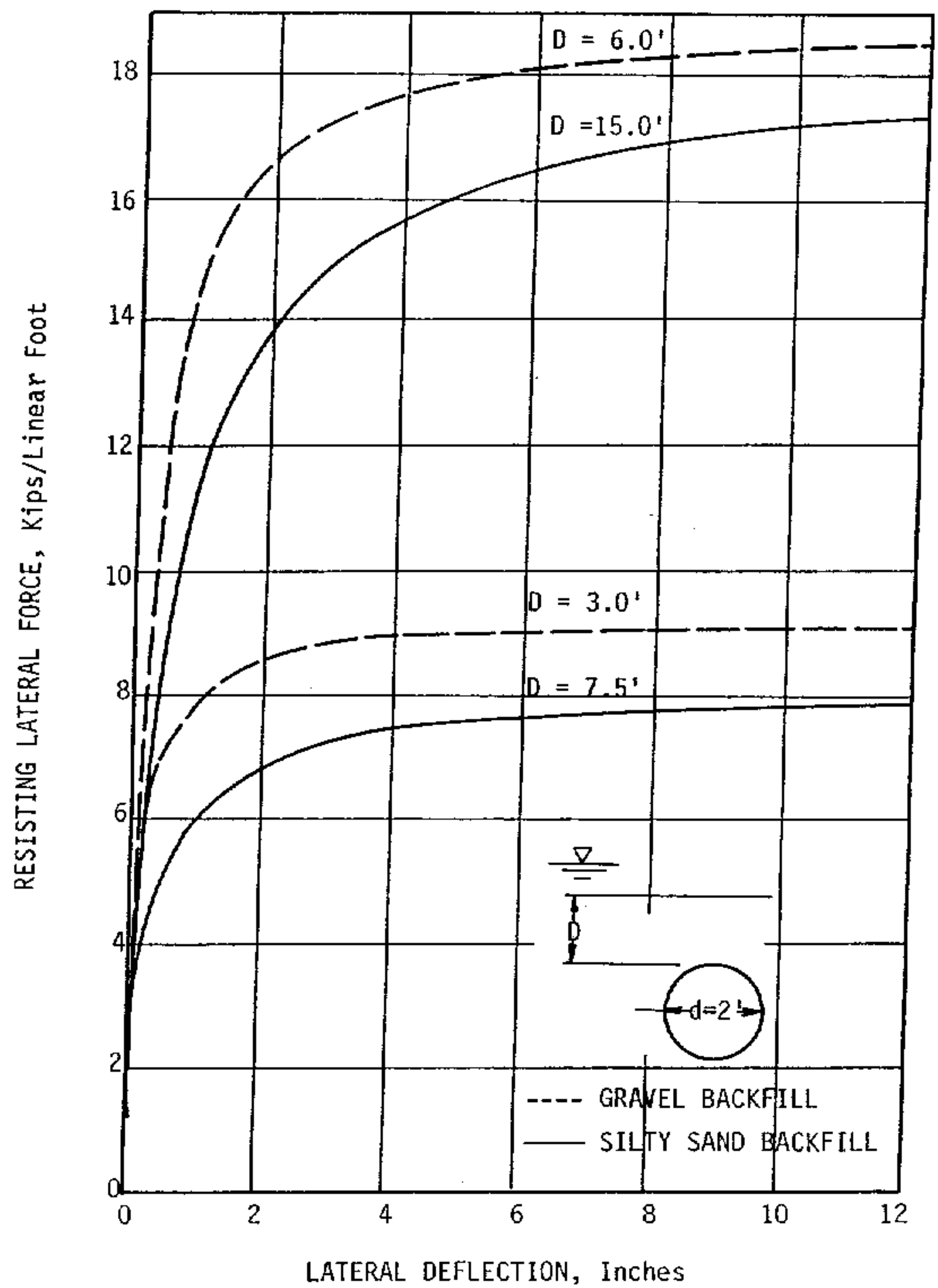
soil resistance was analyzed using methods developed by Audibert and Nyman (1977) to determine the variation in resisting lateral force with pipeline strain at various burial depths. The analysis was performed for gravel backfill and sand backfill cover depths of 3 to 15 feet using a two-foot-diameter 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. <u>Thaw Strain</u>

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
Pt. Thomson Development Project
Winter 1982, Geotechnical Study
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PLATE
VI-18

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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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

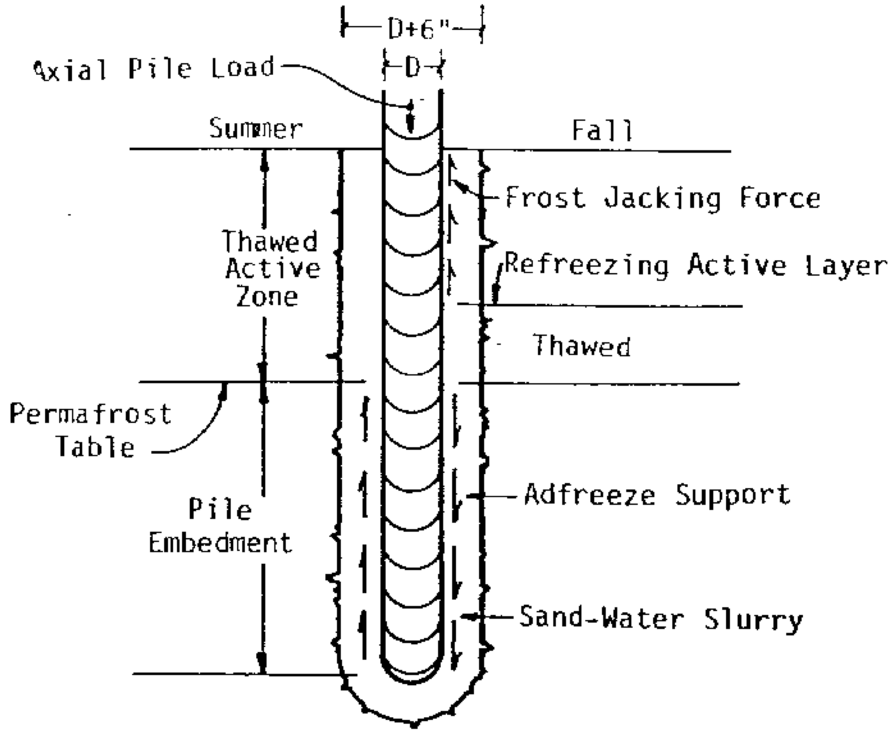
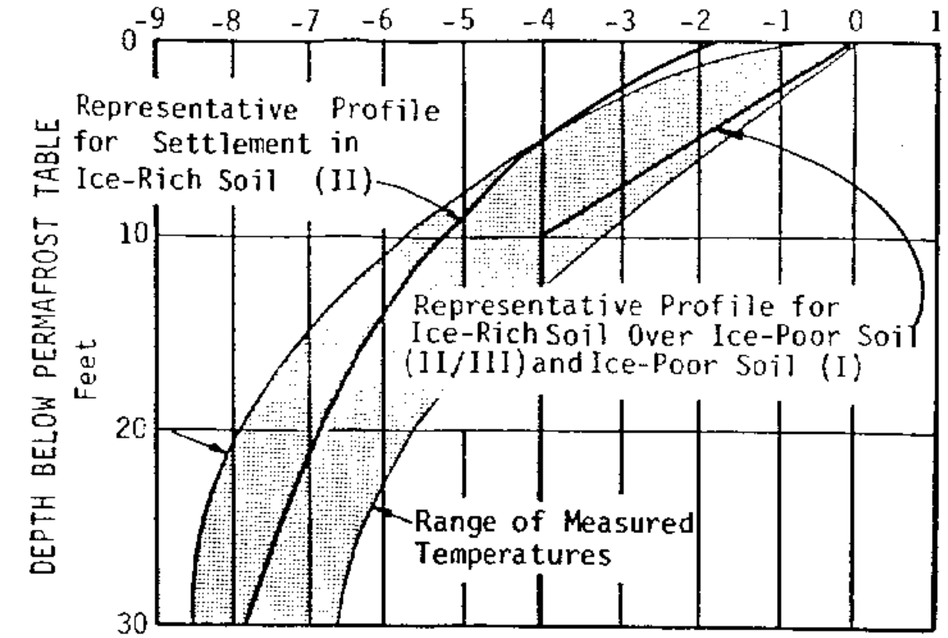


Figure 1. Schematic of onshore pile

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

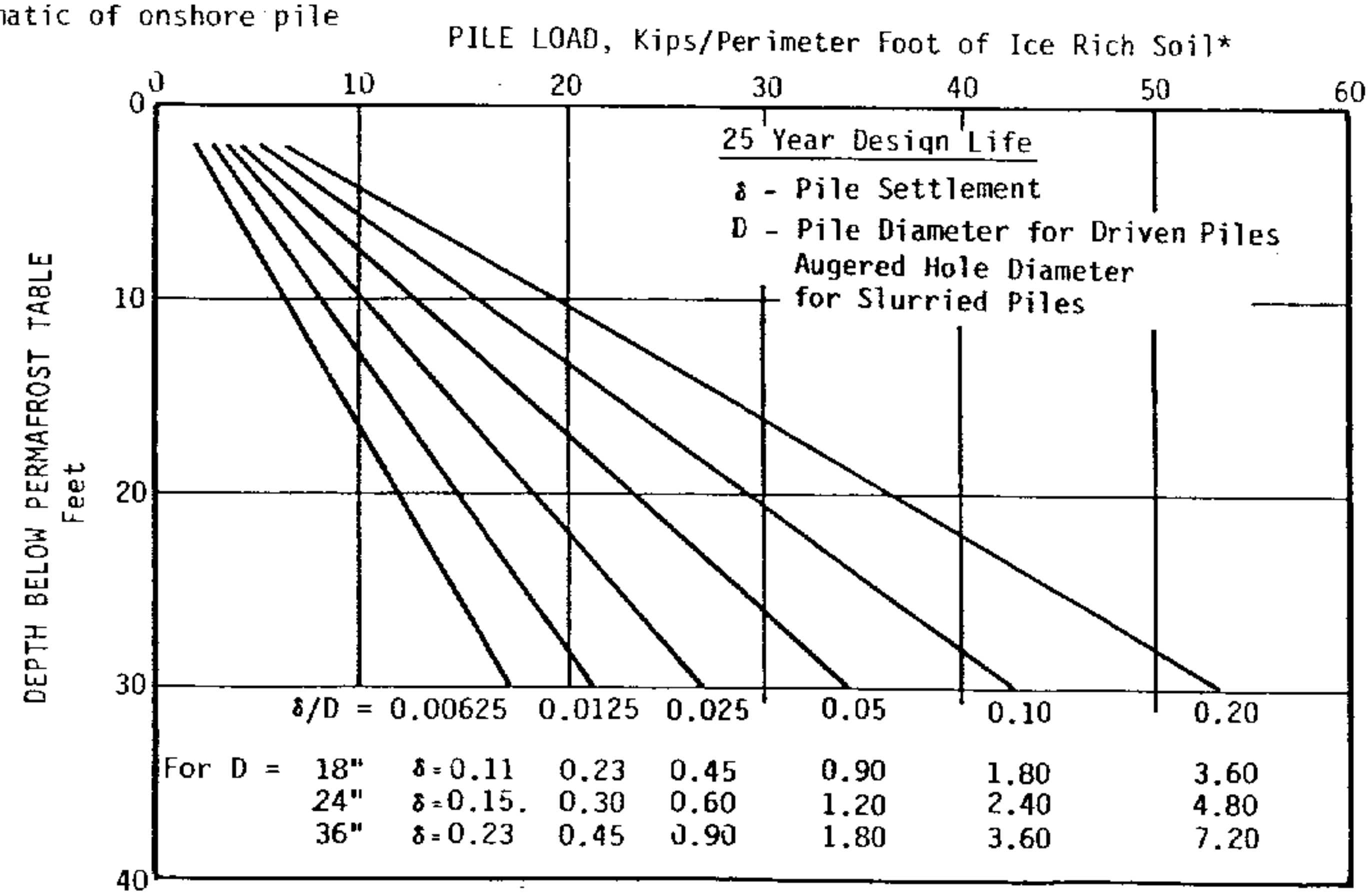


Figure 3. Pile settlement in ice-rich soil (II)

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

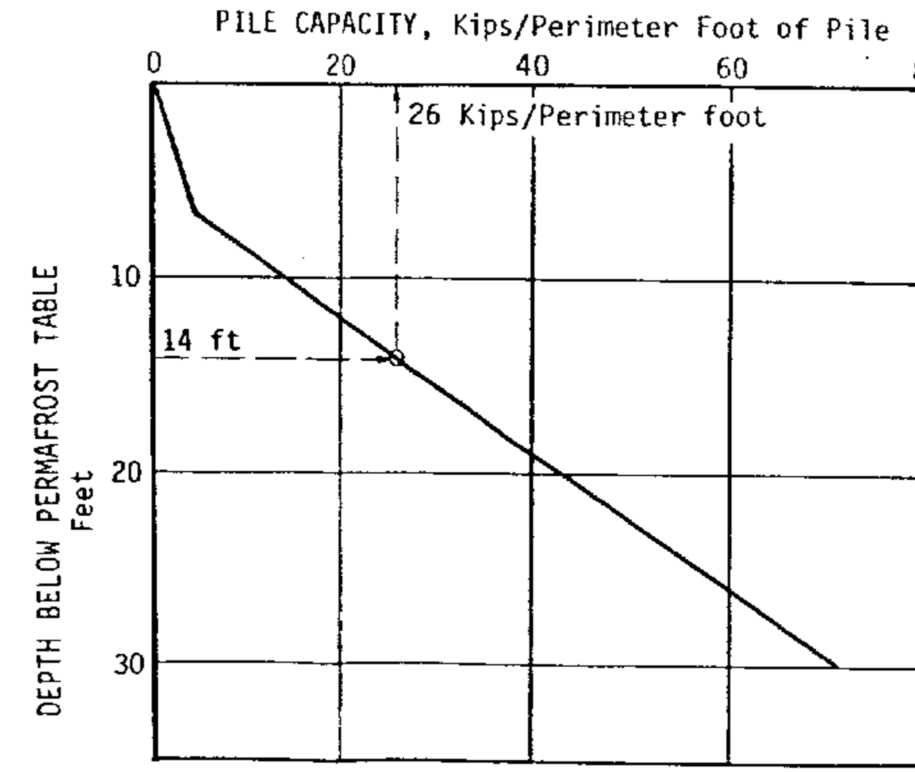


Figure 4. Adfreeze support for ice-poor soil and 7 feet of ice-rich soil over ice-poor soil

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

PLATE **VII-1**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
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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. <u>Pile Adfreeze Support Capacity</u>

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 $0^{\circ}C$. Load tests on piles in permafrost have primarily been performed in temperatures above $-4^{\circ}C$. For this reason, pile capacities discussed below have been assumed to be constant at ground temperatures below $-4^{\circ}C$.

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 $-8^{\circ}C$ ($17.6^{\circ}F$) with a surface amplitude of $\pm 10^{\circ}C$ 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.

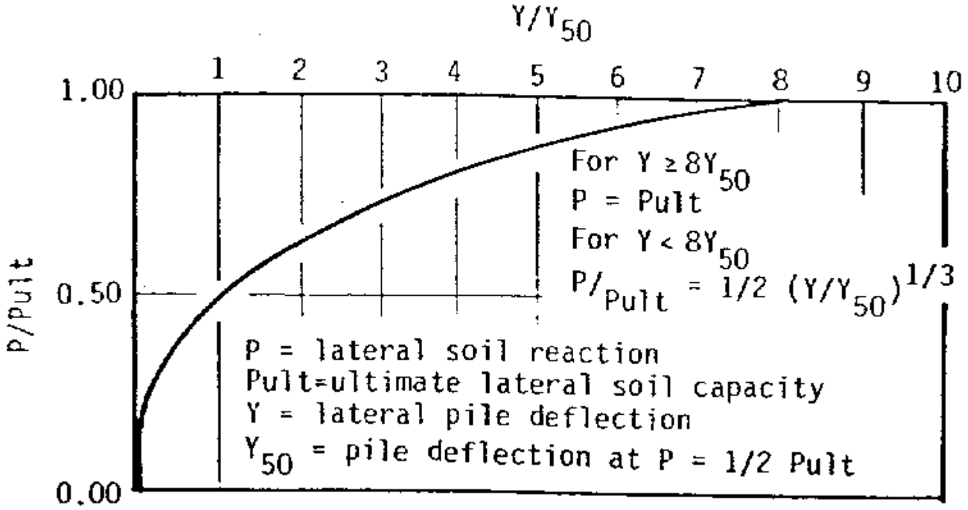


Figure 1. P-Y Curve

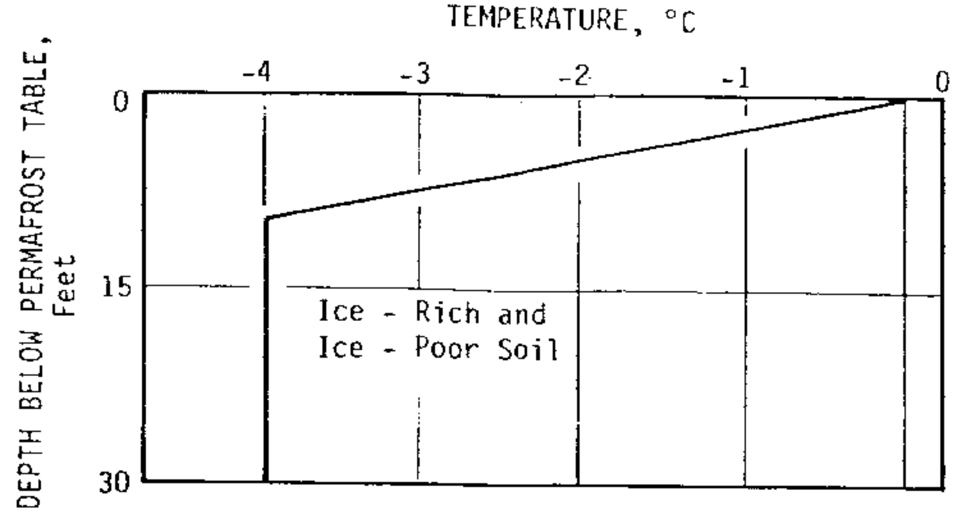


Figure 2. Temperature profile used in analysis

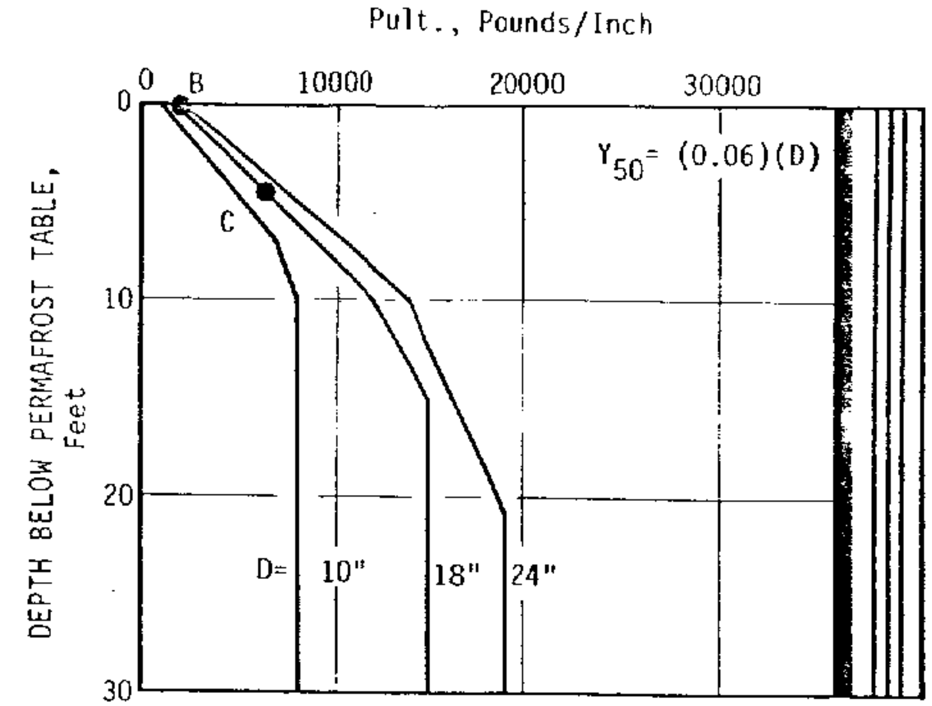
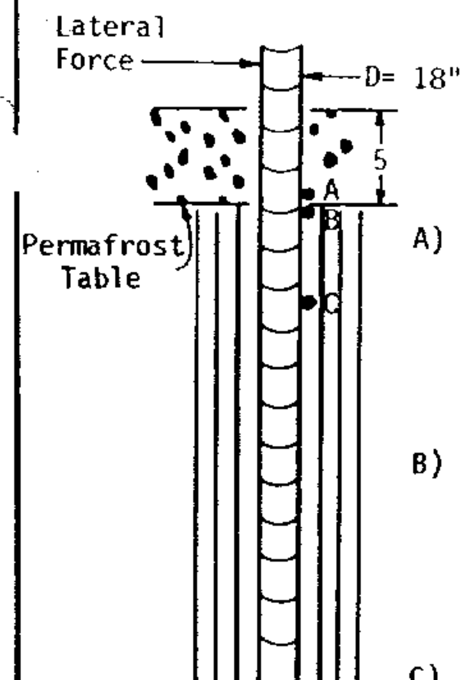


Figure 3. Ice-rich soil



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

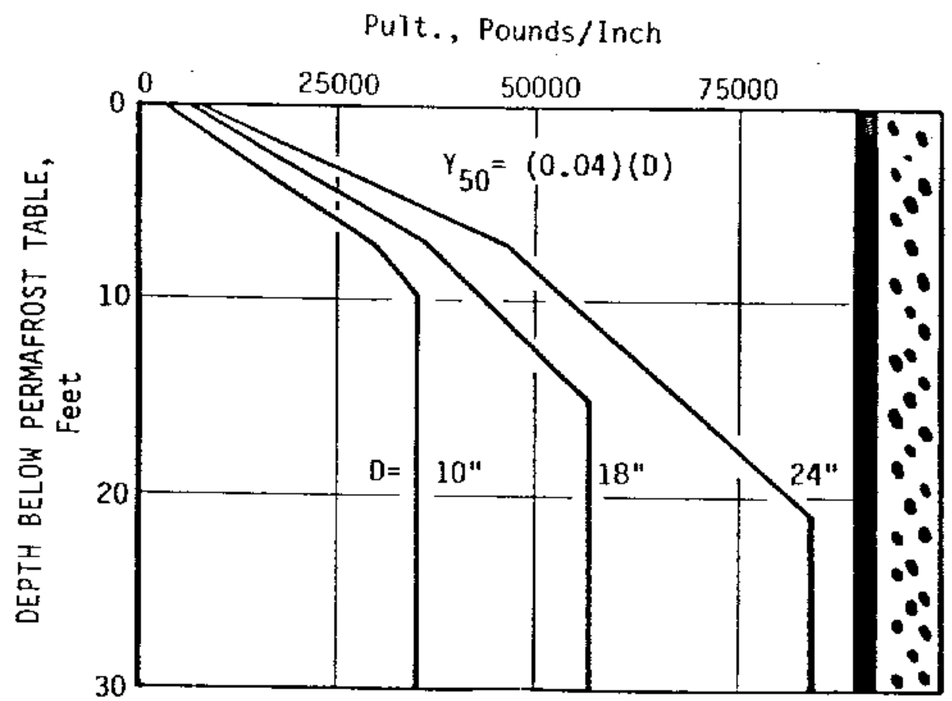


Figure 4. Ice-poor soil

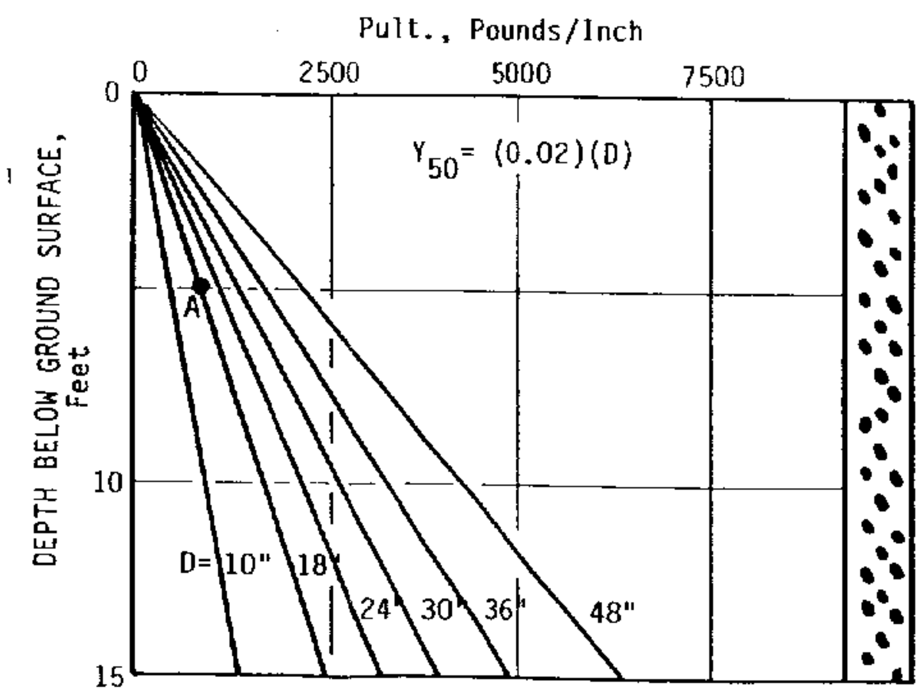


Figure 5. Unfrozen Gravel

Ice-Rich Soil

Unfrozen Gravel

Ice-Poor Soil

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	DGB	4/82		

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

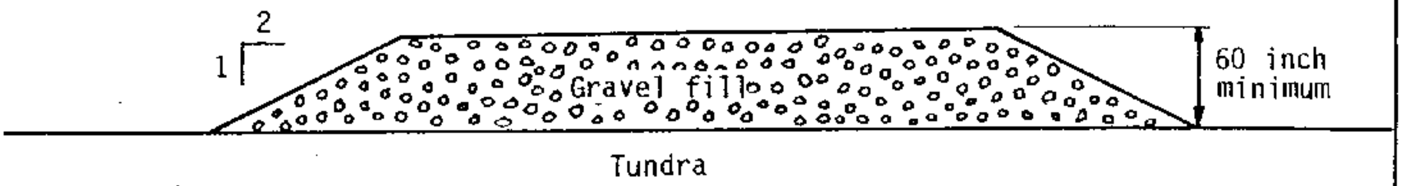
To construct and service onshore facilities and pipelines, roadways will be built within the unit. The roadways will be designed to support conventional construction equipment, lightweight service vehicles, and heavy module transporters.

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.

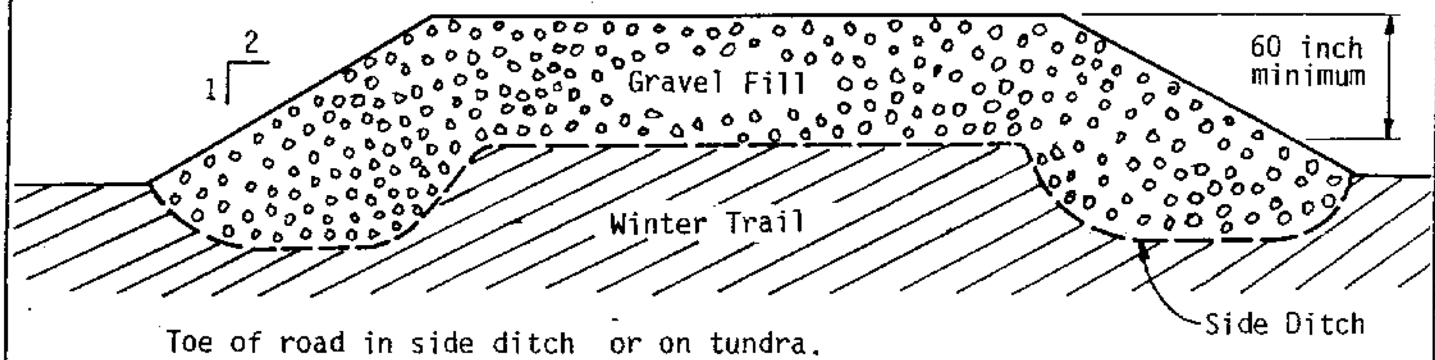
The standard Prudhoe Bay road cross section over tundra areas consists of five feet of gravel fill as shown on Plate VII-3.

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

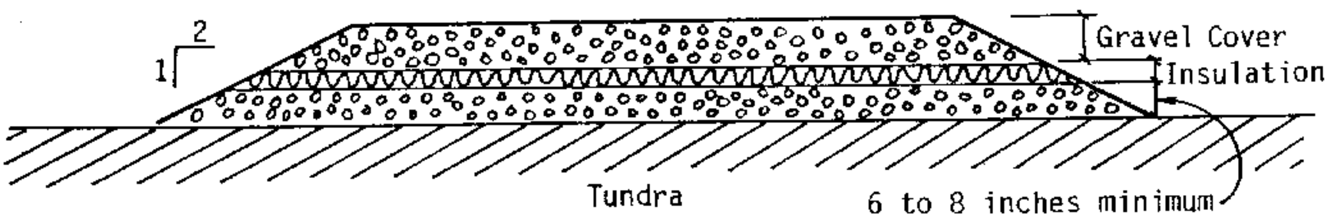


STANDARD PRUDHOE BAY ROAD



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.

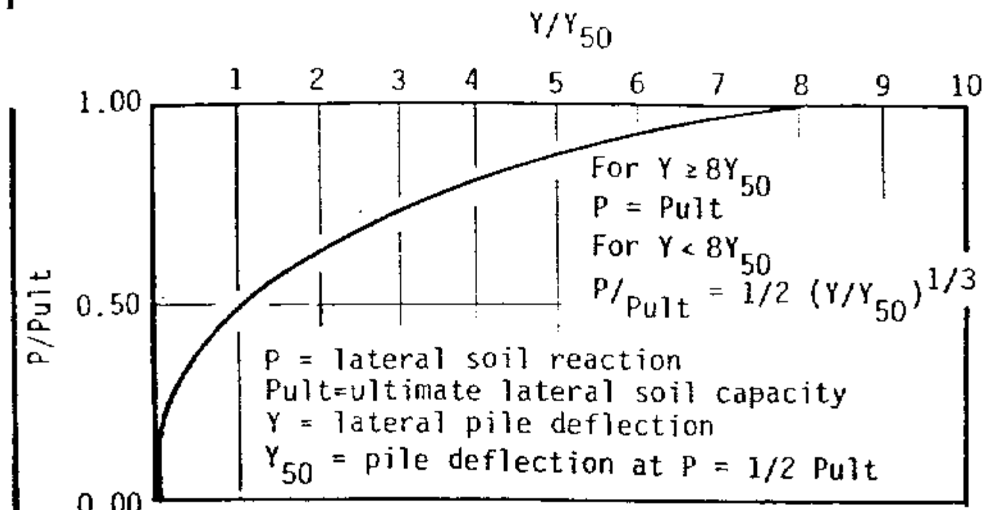


Figure 1. P-Y Curve

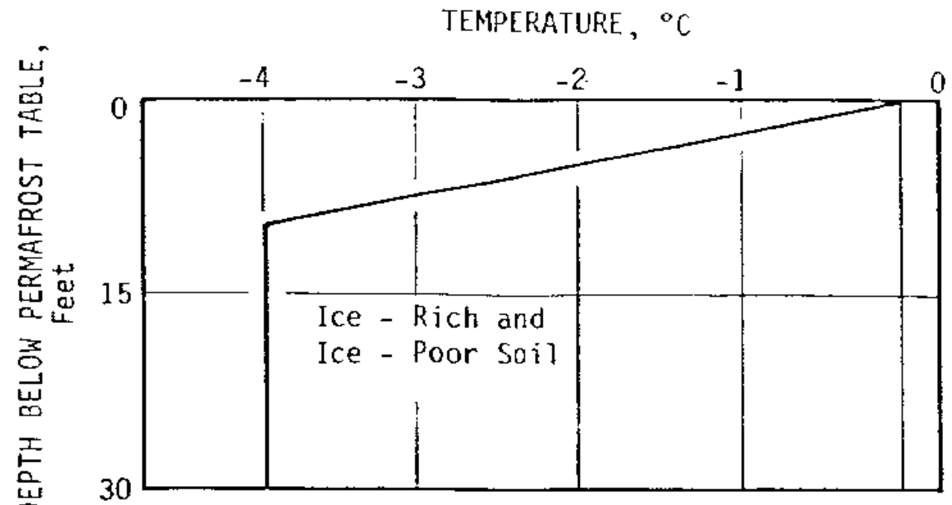


Figure 2. Temperature profile used in analysis

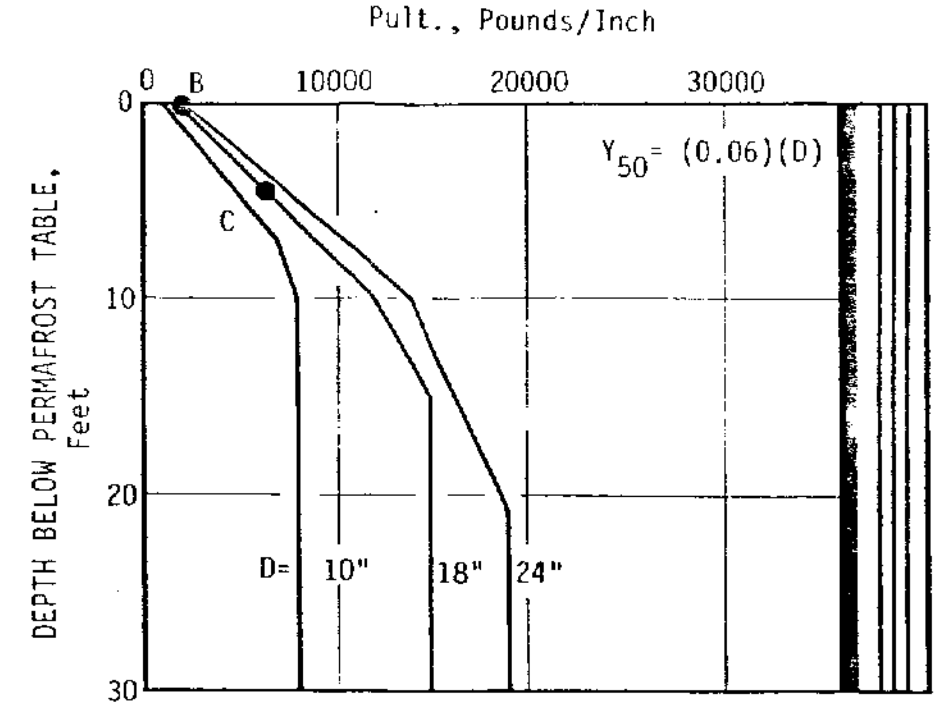
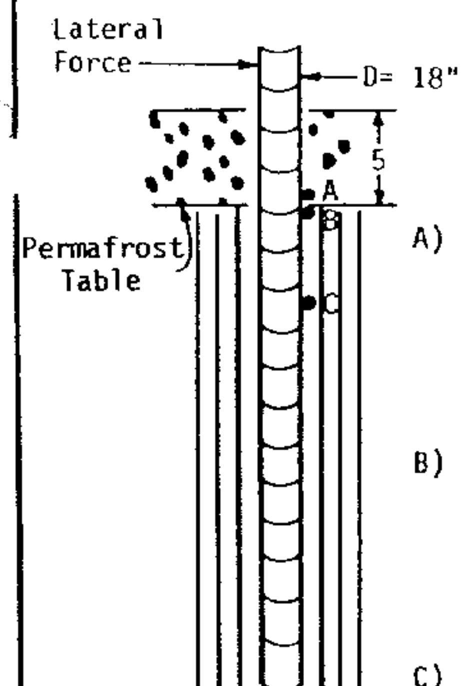


Figure 3. Ice-rich soil



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 $(P/800\text{lbs/inch}) = 1/2(Y/0.36")^{1/3}$

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

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 $(P/7000\text{lbs/inch}) = 1/2(Y/1.08")^{1/3}$

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

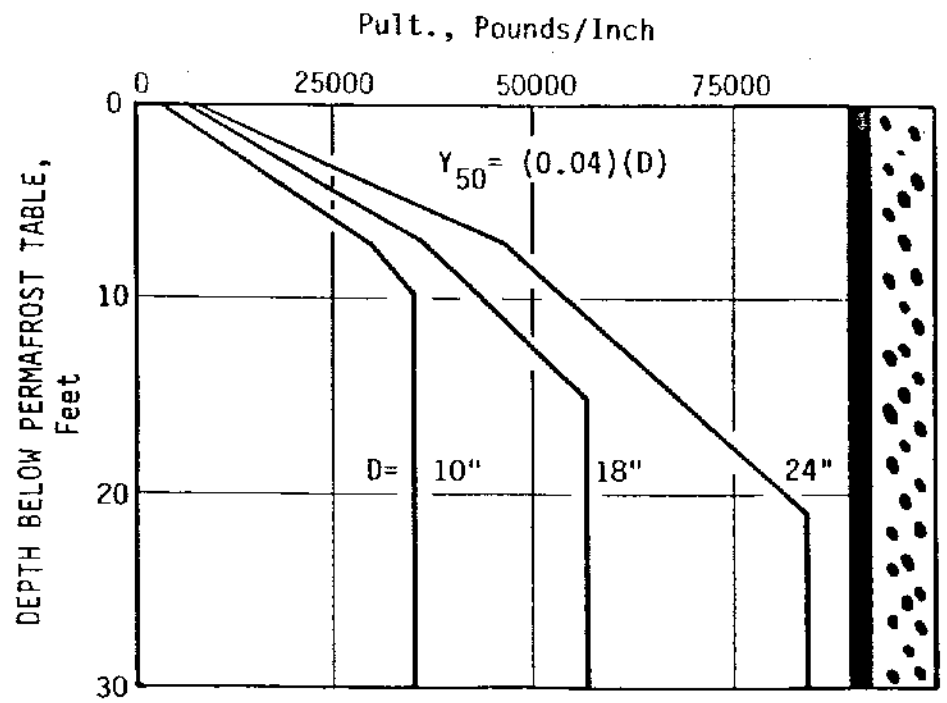


Figure 4. Ice-poor soil

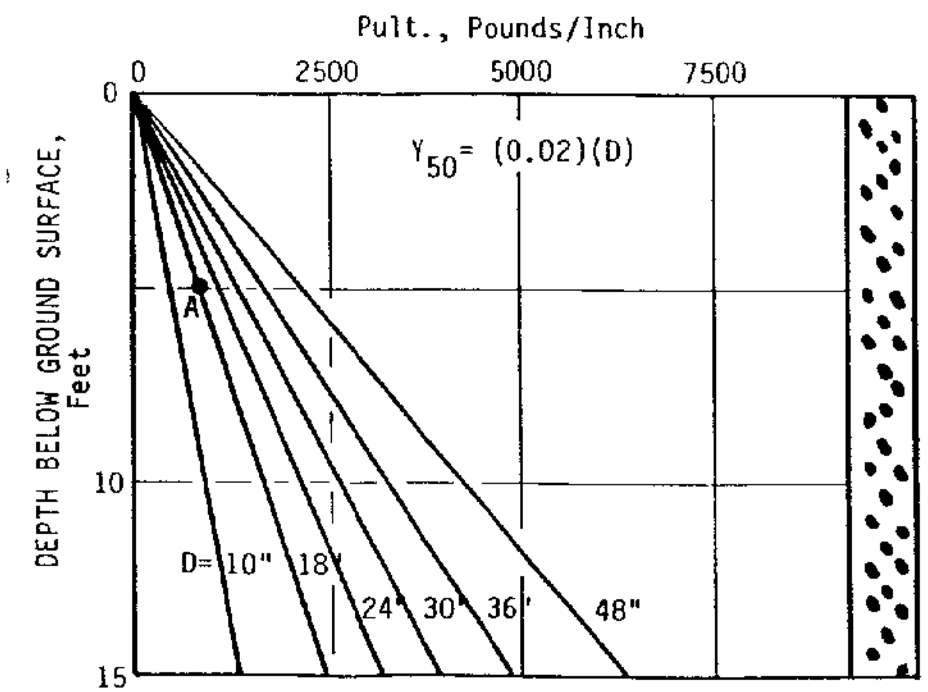


Figure 5. Unfrozen Gravel

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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