INDIAN AND NORTHERN AFFAIRS CANADA

CPT DATA, ARTIFICIAL ISLANDS CANADIAN BEAUFORT SEA

FINAL REPORT
MARCH 1999

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Indian and Northern Affairs Canada
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Mr. Bob Gowan

Dear Mr. Gowan:

CPT Data, Artificial Islands
Canadian Beaufort Sea

We are pleased to submit our report on the CPT data available for selected Artificial Islands in the Canadian Beaufort Sea. This report should be read in conjunction with the June 1997 report on Granular Borrow and Fill Quality which documents the material used to construct these selected artificial islands. Information on the CPT data and the fill quality was documented in a database which is included on the BeauFILL CD Rom.

If you have any questions on the report, please call.

Yours truly,

KLOHN-CRIPPEN CONSULTANTS LTD.

Brian ~~T. Rogers, P.Eng.~~
Manager, Geotechnical Division

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Enclosure

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INDIAN AND NORTHERN AFFAIRS CANADA

CPT DATA, ARTIFICIAL ISLANDS CANADIAN BEAUFORT SEA

FINAL REPORT

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March 1999

Klohn-Crippen

EXECUTIVE SUMMARY

Approximately 40 million cubic metres of granular material have been dredged within the Canadian Beaufort continental shelf to create artificial islands or subsea berms for caisson retained islands and drilling barges. These islands were constructed to provide temporary drilling structures for hydrocarbon exploration. After completing drilling and removing the equipment and consumables, these islands were abandoned to natural erosion, or partially scalped and reused at other exploration sites.

A series of reports by Klohn-Crippen Consultants Ltd. for the Department of Indian and Northern Affairs, Canada, have documented the available sources of good quality granular material located in the Canadian Beaufort Sea that could potentially be used in future developments. Documentation covers the seabed borrow locations which were the original source of the coarser fill material, summarizes the construction history of all the old islands, and documents further fill quality information on eleven of the abandoned islands in which the coarser fill material was placed.

Information on insitu density was monitored by deploying the cone penetration test (CPT). This report provides a summary of the available CPT data for the key abandoned islands that provide the delineated sand and gravel borrow source, and also documents CPT data from some additional sites in the Canadian Beaufort Sea, namely the Nerlerk B-67 berm, the Isserk I-15 Molikpaq deployment site, and at trial berm sites that used Isserk and Ukalerk borrow material. In total the report documents 422 cone penetration tests completed at nine island sites and three trial berm sites.

As part of this project, the CPT data was transferred from the original HP format, to a PC based ASCII format. The combined fill quality and CPT database has been archived on CD Rom. This provides an easily accessible storage of the data.

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1. OVERVIEW

1.1 Background

Since the early 1970's, approximately 40 million cubic metres of granular material have been dredged within the Canadian Beaufort continental shelf to create artificial islands or subsea berms for caisson retained islands (CRI) and drilling barges. These islands were constructed to provide temporary drilling structures for hydrocarbon exploration. Upon completion of exploratory drilling and after removal of equipment and consumables, these islands have generally been abandoned to natural erosion, or partially scalped and reused at other exploration sites.

The purpose of this report is to document the CPT data from abandoned island which contain good quality granular material. The report documents the number of cone penetration tests collated, and provides references to data collected. Other island data and trail berm data are also included. The CPT data have been archived on CD Rom in an PC compatible format for ease of accessibility, together with the information on island construction and fill quality.

1.2 Authorization

This report and database was prepared under Contract A7134-6-0009/001/SS on behalf of the Department of Indian and Northern Affairs, Canada, as part of the Northern Granular Resources Program and the Program of Energy Research and Development, Project 6A4020. Mr. Bob Gowan was the Scientific Authority for the project.

2. PREVIOUS REPORTS

The Klohn-Crippen report titled "Granular Resource Potential of Beaufort Artificial Islands" dated March 1995 documented 37 islands in the Canadian Beaufort Shelf which were believed to represent the total number of offshore islands constructed in the area. A tabular listing of the as-built quantities required for each island, the borrow source and a description of the fill material is included in Table 2.1.

Table 2.1 - As-Built Quantities of Beaufort Artificial Islands

No	Island Name	Island Type	Fill Quantity (m^3)	Borrow Site	Primary Fill Material	Secondary Fill Material
1	Immerk B-48	Sacrificial Beach Island	180 000	Local	Sand	gravel
2	Adgo F-28	Sandbag Retained Island	46 000	Local, Immerk area	Silt	gravel cap
3	Pullen E-17	Gravel Fill Island	65 000	Onshore, Yaya Lakes	Gravel	
4	Unark L-24	Gravel Fill Island	44 000	Onshore, Yaya Lakes	gravel	
5	Pelly B-35	Barge Cored Island	35 000	Local, Yaya Lakes	silt	gravel cap
6	Netserk B-44	Sandbag Retained Island	306 000	Pelly Island	sand	
7	Adgo P-25	Sandbag Retained Island	27 000	Local	silt	gravel cap
8	Adgo C-15	Gravel Fill Island	70 000	Onshore, Yaya Lakes	gravel	
9	Netserk F-40	Sandbag Retained Island	291 000	Pelly Pit, Garry Harbour and Spit	sand	
10	Sarpik B-35	Gravel Fill Island	118 000	Onshore, Adgo C-15 area	gravel	
11	Ikkatok J-17	Sandbag Retained Island	38 000	Local	sand	
12	Kugmallit H-59	Sandbag Retained Island	236 000	Tufts Point	sand	
13	Adgo J-27	Sandbag Retained Island	69 000	Local, Netserk B-44 area	silt	gravel cap
14	Arnak L-30	Sacrificial Beach Island	1 150 000	Local	sand	
15	Kannerk G-42	Sacrificial Beach Island	1 150 000	Local	sand	

No	Island Name	Island Type	Fill Quantity (m^3)	Borrow Site	Primary Fill Material	Secondary Fill Material
16	Isserk E-27	Sacrificial Beach Island	1 908 000	Local, Tufts Point	sand	
17	Issungnak O-61	Sacrificial Beach Island	4 100 000	Local, Tufts Point	sand	
18	Issungnak 2-O-61	Sacrificial Beach Island	4 900 000	Issungnak 0-61	sand	
19	Alerk P-23	Sacrificial Beach Island	2 670 000	Local	sand	
20	North Protection Island	Sacrificial Beach Island	2 000 000	Local	sand	
21	West Atkinson L-17	Sacrificial Beach Island	1 000 000	Local	sand	
22	Tarsiut N-44	Caisson Retained Island	1 800 000	Ukalerk, Issigak, Isserk, Herschel	sand	gravel
23	Uviluk P-66	Single Steel Drilling Caisson	1 900 000	Local, Ukalerk, Isserk, Kadluk, Issigak	sand	gravel cap
24	Itiyok I-27	Sacrificial Beach Island	1 943 000	Local, Ukalerk	sand	
25	Nerlerk B-67	Single Steel Drilling Caisson	4 000 000	Ukalerk, Local	sand	
26	Kogyuk N-67	Single Steel Drilling Caisson	1 450 000	Ukalerk, Uviluk P-66, Banks Island, Rufus	sand	gravel cap
27	Kadluk O-07	Caisson Retained Island	450 000	Ukalerk	sand	
28	Amerk Q-09	Caisson Retained Island	1 700 000	Ukalerk	sand	
29	Adgo H-29	Sandbag Retained Island	75 000	Adgo J-27, Sarpik B-35, Kadluk O-07	sand	gravel cap
30	Nipterk L-19	Sacrificial Beach Island	1 500 000	Issigak, Ukalerk	gravel	sand
31	Tarsiut P-45	Molikpaq	350 000	Ukalerk, Tarsiut N-44, Kogyuk N-67	sand	gravel
32	Minuk I-53	Sacrificial Beach Island	2 000 000	Ukalerk, Issigak, Isserk, Kadluk O-07	gravel	sand
33	Amauligak I-65	Molikpaq	1 410 000	Ukalerk, Kogyuk N-67, Amerk O-09, Issigak	sand	gravel toe
34	Arnak K-06	Sacrificial Beach Island	700 000	Local	sand	
35	Kaubvik I-43	Caisson Retained Island	565 000	Ukalerk, Isserk, Issigak	sand	gravel toe
36	Amauligak F-24	Molikpaq	2 000 000	Ukalerk, Amauligak I-65, Minuk I-53	sand	gravel toe
37	Isserk I-15	Molikpaq	72 000	Amauligak I-65	sand	

The 1995 report provided a summary of the approximate quantity of fill material still remaining in the offshore islands after abandonment following completion of drilling activities. This included approximately fourteen (14) million cubic metres of finer gradation sandfill (D_{50} less than about 280 μm) in place in sandbag retained and sacrificial beach abandoned islands. This material was typically obtained from local borrow sources immediately adjacent to the island site, and is not likely to be transported for use in construction of islands at new exploration or development sites. However the sandfill does represent a valuable base resource for potential development at the site specific exploration site.

The higher quality fill material (D_{50} greater than about 280 μm) identified included over four (4) million cubic metres of a mixture of sands and gravel available as a resource in the abandoned offshore islands, Tarsiut N-44, Nipterk I-19 and Minuk I-53, and close to seven (7) million cubic metres of Ukalerk type sand available in the abandoned berms that were used for the CRI, Molikpaq and SSDC deployments. These islands, which are listed in Table 2.2 and located as shown on Figure 1.1, represent a delineated source of high quality sand and gravel that can be readily used for future construction activities related to exploration or development in the Canadian Beaufort Shelf. These islands have been documented in a second Klohn-Crippen report, data June 1997, titled "Granular Borrow and Fill Quality at Selected Locations in the Canadian Beaufort Sea."

An additional 200,000 m^3 of gravel was used in four older gravel filled islands constructed in the 1970's. Due to the limited size of these individual deposits, this resource has not been included in this database.

Table 2.2 - Canadian Beaufort Artificial Islands with Higher Quality Fill

Island Name	Island Type	Latitude	Longitude	Water Depth (m)	As-built Fill Quantity m^3	Construction Date
Tarsiut N-44	Caisson Retained Island	69.896139	136.193470	21.0	1 800 000	1981-82
Uviluk P-66	Single Steel Drilling Caisson	70.263444	132.313280	29.7	1 900 000	1981-82
Kogyuk N-67	Single Steel Drilling Caisson	70.113722	133.328220	28.1	1 450 000	1982-83
Kadluk O-07	Caisson Retained Island	69.780083	136.021250	14.0	450 000	1989
Amerk P-09	Caisson Retained Island	69.982333	133.514778	27.0	1 700 000	1983-84
Nipterk L-19	Sacrificial Beach Island	69.810583	135.298094	11.7	1 500 000	1983-84
Tarsiut P-45	Molikpaq	69.915444	136.418000	25.5	350 000	1984
Minuk I-53	Sacrificial Beach Island	69.709639	136.458860	14.7	2 000 000	1982-85
Amauligak I-65	Molikpaq	70.077694	133.804556	31.0	1 410 000	1985
Kaubvik I-43	Caisson Retained Island	69.875833	135.422028	17.9	565 000	1983-86
Amauligak F-24	Molikpaq	70.054833	133.630250	32.0	2 000 000	1985-87

Data gathering for this project included accessing the following reports from industry and government sources, to provide the granular borrow source and fill quality data.

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- Gulf Canada Resources Ltd. 1984, Mobile Arctic Caisson Deployment/Berm Construction/Deployment Molikpaq Tarsiut P-45. Internal Gulf Report.
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- Gulf Canada Resources Ltd. 1985, "Molikpaq/Molikpaq at Amauligak I-65 Post-Construction Assessment". Internal Gulf Report.
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- Klohn-Crippen Consultants Ltd. 1997, "Granular Borrow and Fill Quality at Selected Locations in the Canadian Beaufort Sea." Report Submitted to Indian and Northern Affairs Canada, June 1997.
- Lewis Geophysical Consultants 1994, "Review of Granular Resource Potential - South-Central Beaufort Targets". Report submitted to Indian and Northern Affairs, Canada.

- M.J. O'Connor 1984, Geotechnical/Mobile Arctic Caisson 1984 Materials Monitoring Report Construction Tarsiut P-45 Berm/Molikpaq. Report submitted to Gulf Canada Resources Inc.

- Sean Williams FitzPatrick 1986, Insitu State of Beaufort Sea Sandfill Structures, (M.Eng. Thesis 1986) University of Alberta.

Information from the public domain was also reviewed, these sources included the following references:

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3. ARTIFICIAL ISLANDS

3.1 General

This report documents the cone penetration test (CPT) data collated for the Beaufort Sea artificial islands. The report includes data from 422 cone penetration tests, from 9 artificial islands and 3 trial berms. Most of the cone data is associated with construction of the deeper water islands that used higher quality fill. For the purpose of this report, these islands with good quality granular material have been divided into two categories, those consisting almost entirely of Ukalerk size sand obtained either directly from the borrow source or from other islands and those constructed mainly of gravel and sand from Issigak.

The islands are listed in Table 3.1 and Table 3.2, together with a summary of the available CPT data. The location of these islands is shown on Figure 1.1. Data available at each island site will be discussed in the following subsections.

Table 3.1 - Selected Beaufort Sea Artificial Islands and Available CPT Data

Island Name	Island Type	Latitude	Longitude	Water Depth (m)	Number of CPT tests	Construction Date
			Sand Islands			
Uviluk P-66	Single Steel Drilling Caisson	70.263444	132.313280	29.7	16	1981-82
Kogyuk N-67	Single Steel Drilling Caisson	70.113722	133.328220	28.1	55	1982-83
Kadluk O-07	CRI	69.780083	136.021250	14.0	0	1983
Amerk O-09	CRI	69.982333	133.514778	27.0	3	1983-84
Tarsiut P-45	Molikpaq	69.915444	136.418000	25.5	66	1984
Amauligak I-65	Molikpaq	70.077694	133.804556	31.0	44	1985
Amauligak F-24	Molikpaq	70.054833	133.630250	32.0	127	1985-87
			Sand and Gravel Islands			
Tarsiut N-44	Caisson Retained Island	69.896139	136.193470	21.0	27	1981-82
Nipterk L-19	Sacrificial Beach Island	69.810583	135.298194	11.7	0	1983-84
Minuk I-53	Sacrificial Beach Island	69.709639	136.458860	15.3	0	1982-85
Kaubvik I-43	CRI	69.875833	135.422028	17.9	0	1983-86

Table 3.2 - Additional Beaufort Sea CPT Data Sources

Island Name	Island Type	Latitude	Longitude	Water Depth (m)	Number of CPT tests	Construction Date
Isserk Trial Berm	Trial Berm			22.0	8	1982
Ukalerk Trial Berm	Trial Berm				16	1982
Nerlerk B-67		70.433444	133.324556	45.1	43	1982-83
Isserk I-15	Molikpaq	69.912361	134.299222	11.8	17	1989

The CPT data included in this database was originally stored on HP data disks. This data was transferred to PC format, with each CPT data set stored separately as a text file. The header information for all of the CPT data is stored in a database, linked to the text files of the individual CPT data sets.

The CPT data has been set relative to the soil surface, which is either the mudline or the surface of the sand core or berm, depending on the particular test. The data taken from the original digital files contain pore water pressure, skin friction and sometimes lateral stress, in addition to the tip resistance. The skin friction data, where present, has not been adjusted for the friction-bearing offset, which is typical taken as 10 cm. Some of the data files are taken from hard copies which were digitized. These data files contain the tip resistance (Qc) only. All of the available data is included in the BeauFILL CD Rom, which also includes the island construction history, material quality data, the bathymetry contour maps, and the CPT locations maps. Instructions on the use of the BeauFILL CD Rom is included in Appendix I.

3.2 Sand Islands

3.2.1 Uviluk P-66

Dome Petroleum constructed an underwater berm at Uviluk P-66 to accommodate a single steel drilling caisson (SSDC) during the 1981-82 open water season. The berm was built in 29.7 m of water on a sandy seabed. Sand was dredged from local borrow sources, as well as from Isserk, Kaglulik South and the Ukalerk borrow, and hydraulically placed to create a berm 103 m x 212 m. The SSDC was placed on the berm, and under-filled with sand and gravel for erosion protection. Gravel hauled from Issigak, Banks Island, and Herschel Island was used as a cap on the berm, which rose to 8.0 m below sea level. A total of 1.9 million m^3 of fill was used to construct the berm. Drilling activities continued over the winter, and following completion, the SSDC was released and the island abandoned. A cross-section through the island is shown on Figure 3.1.

Cone penetration testing was performed after the construction of the sand berm, and prior to SSDC setdown. A total of 16 tests are available at this site, as noted in Table 3.3.

Table 3.3 - Uviluk P-66 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
UVP6604	Aug. 11 1982	9.10	21.26	vessel - after construction of sand berm
UVP6605	Aug. 11 1982	9.21	21.26	vessel - after construction of sand berm
UVP6606	Aug. 12 1982	9.16	12.75	vessel - after construction of sand berm
UVP6607	Aug. 12 1982	9.11	22.83	vessel - after construction of sand berm
UVP6608	Aug. 12 1982	9.13	23.85	vessel - after construction of sand berm
UVP6609	Aug. 12 1982	9.08	22.76	vessel - after construction of sand berm
UVP6610	Aug. 15 1982	9.00	23.48	vessel - after construction of sand berm
UVP6611	Aug. 15 1982	9.00	23.73	vessel - after construction of sand berm
UVP6612	Aug. 15 1982	9.05	20.88	vessel - after construction of sand berm
UVP6613	Aug. 15 1982	9.05	20.50	vessel - after construction of sand berm
UVP6614	Aug. 16 1982	9.05	11.92	vessel - after construction of sand berm
UVP6615	Aug. 16 1982	9.06	22.99	vessel - after construction of sand berm
UVP6616	Aug. 16 1982	9.04	23.44	vessel - after construction of sand berm
UVP6617	Aug. 16 1982	9.00	22.79	vessel - after construction of sand berm
UVP6618	Aug. 17 1982	9.00	22.74	vessel - after construction of sand berm
UVP6619	Aug. 17 1982	9.49	14.50	vessel - after construction of sand berm

3.2.2 Kogyuk N-67

The Kogyuk N-67 berm was initially constructed in the 1982 open water season in 28.1 m of water by Gulf to support the Molikpaq. The seabed, which consisted of a surficial clay overlying sand was excavated to a depth of 1.5 m over an area of 180 m x 180 m. Approximately 643 000 m^3 of sand from Ukalerk were then placed, primarily through the drag-arms of the Geopotes IX.

In 1983, the berm was raised by Canmar to support the SSDC by the addition of a further 805 000 m^3 of fill material largely from Ukalerk and the abandoned Uviluk island. Most of this material was placed by bottom-dumping but some 66 000 m^3 were placed by the Canmar Constructor around the perimeter to produce 5H:1V slopes. The final berm elevation was -9 m. A total of 1,450,000 m^3 of sand was placed creating a berm surface 103 m x 212 m with 5H:1V and 10H :1V slopes. A cross-section through the island is shown on Figure 3.2.

Drilling activities were conducted over the winter of 1983-84, and the island was abandoned in 1984. A bathymetric survey by Canadian Engineering Surveys Co. Ltd. (CES) in August, 1985 indicated the berm surface was 8 m below sea level.

In 1984 and 1985, material from Kogyuk was used in the construction of Tarsiut P-45 and Amauligak I-65, respectively.

Cone penetration testing at the Kogyuk N-67 site was done in five phases. The first testing was done after the 1982 berm construction, and is described in Stewart et al (1983). Further CPT testing was done in August 1983 to determine the condition of the island prior to the 1983 construction. This was followed by testing at the end of the 1983 construction prior to setdown of the SSDC, and after setdown from the SSDC deck. A final round of CPT testing was performed in 1984 following the removal of the SSDC. A total of 55 CPT records are available from the Kogyuk site, as listed in Table 3.4.

Table 3.4 - Kogyuk N-67 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
TKIC4A	Sept. 1982	19.90	8.89	vessel - after 1982 berm construction
TKIC1C	Sept. 1982	19.60	9.56	vessel - after 1982 berm construction
TKIC3C	Sept. 1982	19.70	8.85	vessel - after 1982 berm construction
TKIC3E	Sept. 1982	20.10	9.40	vessel - after 1982 berm construction
TKIC4B	Sept. 1982	19.80	10.11	vessel - after 1982 berm construction
TKIC4C	Sept. 1982	20.10	9.35	vessel - after 1982 berm construction
TKIC4D	Sept. 1982	20.80	8.41	vessel - after 1982 berm construction
TKIC4E	Sept. 1982	20.10	8.57	vessel - after 1982 berm construction
TKIC5A	Sept. 1982	19.50	9.98	vessel - after 1982 berm construction
TKIC5C	Sept. 1982	19.50	9.78	vessel - after 1982 berm construction
TKIC5E	Sept. 1982	19.50	9.48	vessel - after 1982 berm construction
TKIC3D	Sept. 1982	20.60	9.21	vessel - after 1982 berm construction
TKICX3	Sept. 1982	17.70		vessel - after 1982 berm construction
TKICN2	Sept. 1982	17.70		vessel - after 1982 berm construction
TKICS1	Sept. 1982	17.70		vessel - after 1982 berm construction
TKICS2	Sept. 1982	17.70		vessel - after 1982 berm construction
TKICW1	Sept. 1982	17.70		vessel - after 1982 berm construction
TKICW2	Sept. 1982	17.70		vessel - after 1982 berm construction
TKIC1E	Sept. 1982	20.20	8.53	vessel - after 1982 berm construction
TKICX2	Sept. 1982	17.70		vessel - after 1982 berm construction
TKIC3B	Sept. 1982	20.20	8.93	vessel - after 1982 berm construction
TKIEA1	Sept. 1982	17.70		vessel - after 1982 berm construction
TKIEA2	Sept. 1982	17.70		vessel - after 1982 berm construction
TKIC1A	Sept. 1982	19.50	8.80	vessel - after 1982 berm construction
TKIC1B	Sept. 1982	19.50	7.97	vessel - after 1982 berm construction
TKIC1D	Sept. 1982	19.42	9.34	vessel - after 1982 berm construction
TKIC2B	Sept. 1982	20.40	9.93	vessel - after 1982 berm construction
TKIC2D	Sept. 1982	20.10	8.61	vessel - after 1982 berm construction
TKIC3A	Sept. 1982	20.10	8.66	vessel - after 1982 berm construction
TKICX1	Sept. 1982	17.70		vessel - after 1982 berm construction
KY83C01	Aug. 14 1983	19.90	7.47	vessel - prior to 1983 construction
KY83C03	Aug. 14 1983	19.90	9.11	vessel - prior to 1983 construction
KY83C02	Aug. 14 1983	20.00	8.95	vessel - prior to 1983 construction

DKIK01	Aug. 1983	16.83	12.85	vessel - end of construction, prior to setdown
DKIK04	Aug. 1983	17.51	12.33	vessel - end of construction, prior to setdown
DKIK08	Aug. 1983	15.90	10.92	vessel - end of construction, prior to setdown
DKIK09	Aug. 1983	14.44	14.19	vessel - end of construction, prior to setdown
DKIK11	Aug. 1983	11.09	16.29	vessel - end of construction, prior to setdown
DKIK12	Sept. 1983	10.41	16.85	vessel - end of construction, prior to setdown
DKIK13	Sept. 1983	10.22	12.98	vessel - end of construction, prior to setdown
DKIK14	Sept. 1983	10.72	16.26	vessel - end of construction, prior to setdown
DKIK15	Sept. 1983	10.37	16.00	vessel - end of construction, prior to setdown
DKIBT1	Sept. 1983	9.12	18.48	SSDC deck - after setdown
DKIBT3	Sept. 1983	9.50	18.43	SSDC deck - after setdown
DKIBT5	Sept. 1983	9.60	19.00	SSDC deck - after setdown
DKIBT6	Sept. 1983	9.15	19.07	SSDC deck - after setdown
DKIBT8	Sept. 1983	8.14	8.14	SSDC deck - after setdown
DKIBT9	Sept. 1983	8.00	20.24	SSDC deck - after setdown
KI84C5	Sept. 13 1984	10.20	18.70	vessel - after SSDC removal
KI84C7	Sept. 13 1984	9.30	1.46	vessel - after SSDC removal
KI84C6	Sept. 13 1984	9.90	18.55	vessel - after SSDC removal
KI84C4	Sept. 13 1984	9.45	20.83	vessel - after SSDC removal
KI84C3	Sept. 13 1984	9.60	9.56	vessel - after SSDC removal
KI84C1	Sept. 13 1984	8.10	22.92	vessel - after SSDC removal
KI84C2	Sept. 13 1984	8.40	21.20	vessel - after SSDC removal

3.2.3 Kadluk O-07

Kadluk O-07 was constructed by Esso in the 1983 open water season in 14.0 m of water, on 2 m thick soft clay overlying firm clay in the Ikit Trough. A sand berm was built to within 9 m of the waterline, and Esso's CRI was placed on the berm and in-filled with sand. The sand berm had an average slope of 1V:24H. The placement of this berm was accomplished exclusively by bottom dumping using the trailer suction hopper dredge, the W.D. Gateway. The berm required 450 000 m^3 of sand fill which was dredged from the Ukalerk borrow pit and hopper placed on the site. Drilling activities took place over the winter of 1983-84 and the island was subsequently abandoned, with the removal of the CRI. A cross-section through the island is shown on Figure 3.3.

No CPT data is available from the Kadluk O-07 site.

3.2.4 Amerk O-09

The Amerk well site was located 40 km offshore in the Kugmallit Channel in a water depth of approximately 27 m. The seabed at Amerk consists of a 6 m thick soft clay layer underlain by a further 12 m of unfrozen soft to firm clay. Below this 18 m depth, the foundation soils are competent and frozen.

The original design required the removal of the top 18 m of material over an area approximately 150 m x 150 m centrally located directly beneath the island surface. This excavation was commenced in 1983 but only advanced to an average depth of 6 m, with local excavations to 9 m, as the dredging equipment could not excavate the firm clays beyond that depth.

Construction of the berm took place over the 1983-84 open water seasons, during which 1.5 million m^3 of sand fill was excavated from the Ukalerk borrow area transported and placed on the site by hopper suction dredge. The berm rose to within 9 m of sea level and had a top surface 128 m in diameter. The CRI was placed on the berm, and the centre was filled with sand. After drilling over the 1984-85 winter season, the rig was released in March 1985, and the island was subsequently abandoned with the removal of the CRI. A cross-section through the island is shown on Figure 3.4.

Data from three cone penetration tests are available from the Amerk O-09 site as listed in

Table 3.5. CPT testing was performed from the surface following construction in 1984.

Table 3.5 - Amerk O-09 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
AME84C2a	Aug. 1984	0.02	43.43	surface
AME84C3	Aug. 1984	0.07	19.76	surface
AME84C2	Aug. 21 1984	0.04	19.96	surface

3.2.5 Tarsiut P-45

Tarsiut P-45 was the first deployment site for Gulf's Molikpaq drilling structure. The Molikpaq, a reusable 110 m octagonal steel caisson, was in-filled with sand on a pre-constructed berm The berm was built in 1984 on the Kringalik Plateau in a water depth of 25.5 m. The original seabed conditions consisted of stiff clay and silt overlain by soft clay. A total of 113 830 m^3 of the soft clay was subcut from the foundation area by trailer suction hopper dredges. A total of 400 000 m^3 of sandfill was bottom discharged to fill the subcut, to create the 140 m square berm and to in-fill the Molikpaq core. The berm surface was 19.5 m below sea level and had side slopes ranging from 7H:1V to 15H:1V. Ukalerk and North Ukalerk borrow areas and the abandoned islands, Tarsiut N-44 and Kogyuk N-67, were dredged for sand fill. The island was used for drilling over the 1984 winter, and was abandoned in September 1985 when the Molikpaq was removed and towed to the newly constructed berm at Amauligak I-65. A cross-section through the island is shown on Figure 3.5.

CPT testing was undertaken as part of the site investigation for Tarsiut P-45, and 13 CPT records are available from this data set. Testing of the berm at 19.5 m below sea level was performed in mid September 1984, with 17 CPT records available. The locations of these tests are shown on Figure 3.6. The sand core verification CPT tests are shown on Figure 3.7, with 31 CPT records available from the October 1984 testing. A further 5 CPT tests were performed in the sand core in June 1985. The locations of these tests are shown on Figure 3.8. A total of 66 CPT records are available from Tarsiut P-45 as listed in Table 3.6. The core test data were described in Jefferies et al (1985).

Table 3.6 - Tarsiut P-45 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
TD84CI06	Aug. 01 1984	24.95	43.97	vessel - site investigation
TD84CI04	Aug. 01 1984	26.35	39.65	vessel - site investigation
TD84CI03	Aug. 01 1984	25.30	31.27	vessel - site investigation
TD84CI02	Aug. 01 1984	25.00	33.49	vessel - site investigation
TD84CI07	Aug. 01 1984	24.20	40.35	vessel - site investigation
TD84CI05	Aug. 01 1984	25.00	43.97	vessel - site investigation
TD84CI01	Aug. 02 1984	25.10	35.03	vessel - site investigation
TD84CI09	Aug. 03 1984	25.20	33.86	vessel - site investigation
TD84CI10	Aug. 04 1984	25.00	38.10	vessel - site investigation
TD84CI11	Aug. 04 1984	24.60	16.46	vessel - site investigation
TD84CI08	Aug. 06 1984	24.30	40.35	vessel - site investigation
TD84CI15	Aug. 06 1984	24.70	35.79	vessel - site investigation
TD84CI14	Aug. 06 1984	25.60	41.24	vessel - site investigation
TD84CV04	Sept. 15 1984	18.90	9.84	vessel - berm verification
TD84CV25	Sept. 15 1984	19.30	9.43	vessel - berm verification
TD84CV21	Sept. 15 1984	19.20	9.90	vessel - berm verification
TD84CV18	Sept. 15 1984	19.30	9.53	vessel - berm verification
TD84CV11	Sept. 15 1984	19.40	8.90	vessel - berm verification
TD84CV07	Sept. 15 1984	19.10	8.15	vessel - berm verification
TD84CV13	Sept. 16 1984	19.30	11.22	vessel - berm verification
TD84CV24	Sept. 16 1984	18.90	10.20	vessel - berm verification
TD84CV20	Sept. 16 1984	19.20	11.55	vessel - berm verification
TD84CV17	Sept. 16 1984	19.10	11.16	vessel - berm verification
TD84CV06	Sept. 16 1984	19.10	11.13	vessel - berm verification
TD84CV15	Sept. 16 1984	19.20	11.60	vessel - berm verification
TD84CV10	Sept. 16 1984	19.00	11.41	vessel - berm verification
TD84CV09	Sept. 16 1984	19.30	11.39	vessel - berm verification
TD84CV08	Sept. 16 1984	19.30	10.82	vessel - berm verification
TD84CV03	Sept. 16 1984	19.40	11.34	vessel - berm verification
TD84CV16	Sept. 16 1984	19.10	11.43	vessel - berm verification
TD84CC02	Oct. 04 1984	-2.80	17.04	surface - sand core verification
TD84CC01	Oct. 04 1984	-2.90	9.68	surface - sand core verification
TD84CC03	Oct. 05 1984	-2.70	18.41	surface - sand core verification
TD84CC07	Oct. 06 1984	-2.60	26.42	surface - sand core verification
TD84CC06	Oct. 06 1984	-2.60	26.82	surface - sand core verification

TD84CC05	Oct. 06 1984	-2.60	31.71	surface - sand core verification
TD84CC04	Oct. 06 1984	-2.70	20.42	surface - sand core verification
TD84CC09	Oct. 06 1984	-2.50	25.08	surface - sand core verification
TD84CC08	Oct. 06 1984	-2.60	36.74	surface - sand core verification
TD84CC11	Oct. 06 1984	-2.70	15.43	surface - sand core verification
TD84CC10	Oct. 06 1984	-2.70	22.33	surface - sand core verification
TD84CC12	Oct. 07 1984	-2.60	33.49	surface - sand core verification
TD84CC14	Oct. 07 1984	-2.50	25.34	surface - sand core verification
TD84CC17	Oct. 07 1984	-2.90	32.85	surface - sand core verification
TD84CC15	Oct. 07 1984	-2.60	25.68	surface - sand core verification
TD84CC13	Oct. 07 1984	-2.60	31.66	surface - sand core verification
TD84CC16	Oct. 07 1984	-2.70	25.60	surface - sand core verification
TD84CC19	Oct. 08 1984	-2.90	32.69	surface - sand core verification
TD84CC26	Oct. 08 1984	-2.70	31.84	surface - sand core verification
TD84CC24	Oct. 08 1984	-2.90	16.65	surface - sand core verification
TD84CC18	Oct. 08 1984	-2.90	24.66	surface - sand core verification
TD84CC20	Oct. 08 1984	-2.60	24.43	surface - sand core verification
TD84CC21	Oct. 08 1984	-2.70	21.64	surface - sand core verification
TD84CC22	Oct. 08 1984	-2.90	17.89	surface - sand core verification
TD84CC23	Oct. 08 1984	-2.90	20.82	surface - sand core verification
TD84CC25	Oct. 08 1984	-2.80	21.17	surface - sand core verification
TD84CC27	Oct. 09 1984	-2.70	21.28	surface - sand core verification
TD84CC28	Oct. 09 1984	-2.70	25.33	surface - sand core verification
TD84CC30	Oct. 29 1984	-2.70	19.29	surface - sand core verification
TD84CC31	Oct. 30 1984	-2.70	21.67	surface - sand core verification
TD84CC33	Oct. 31 1984	-2.70	21.76	surface - sand core verification
MACRES01	Jun. 12 1985	-1.60	22.31	surface - sand core research program
MACRES02	Jun. 13 1985	-1.60	27.06	surface - sand core research program
MACRES03	Jun. 15 1985	-1.50	32.60	surface - sand core research program
MACRES04	Jun. 18 1985	-1.50	30.91	surface - sand core research program
MACRES05	Jun. 18 1985	-1.40	29.48	surface - sand core research program

3.2.6 Amauligak I-65

The berm at Amauligak I-65 was built by Gulf in the 1985 open water season to support the Molikpaq drilling structure. The island was built in 31.0 m of water in the Kugmallit Channel physiographic region. Site conditions consisted of surficial soft clay overlying fine sand. A total of 440 000 m^3 of soft clay was subcut from the foundation area using trailer suction hopper dredges. A total of 1 408 000 m^3 of fill was used to fill the subcut, build the berm and in-fill the core. Sand was dredged from the Ukalerk borrow area and the abandoned Kogyuk N-67 and Amerk P-09 islands. The completed berm had a 140 m square surface 19.5 m below the waterline sloping away at about 7H:1V to the seabed. The Molikpaq was placed on the berm and the core in-filled with sand to 1.5 m above sea level. Gravel excavated from the Issigak borrow area was used as toe protection. Due to adverse ice conditions and accelerated schedule, four dredges, the Geopotes X, IX, WD Gateway, and Cornelis Zanen were used. Several wells were drilled and tested from this platform in 1985 and 1986 before abandonment. The Molikpaq was released from Amauligak I-65 in September 1986. The abandoned Amauligak I-65 berm was used as a borrow source for Molikpaq core fill at Amauligak F-24 and Isserk I-15. A cross-section through the island is shown on Figure 3.9.

Cone penetration test data was obtained in four phases from the Amauligak I-65 artificial island. Site investigation and berm verification testing was performed from a vessel in September 1985. The 9 berm verification CPT tests are shown on Figure 3.10. The 21 core verification CPT tests performed from the core surface in October 1985 are shown on Figure 3.11. An additional 10 tests performed to assess the insitu density of the sand core following the April 12 1986 ice loading event are shown on Figure 3.12. A total of 44 tests are available for Amauligak I-65, as listed in Table 3.7.

Table 3.7 - Amauligak I-65 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
AM85C01	Aug. 31 1985	33.50	24.19	vessel - site investigation
AM85C01C	Sept. 01 1985	33.50	35.99	vessel - site investigation
AM85C01d	Sept. 01 1985	33.50	40.11	vessel - site investigation
AM85C01e	Sept. 01 1985	33.50	58.44	vessel - site investigation
AM85CV14	Sept. 18 1985	20.20	24.78	vessel - berm verification
AM85CV10	Sept. 18 1985	19.70	19.80	vessel - berm verification
AM85CV13	Sept. 18 1985	19.60	18.95	vessel - berm verification
AM85CV18	Sept. 18 1985	20.10	25.88	vessel - berm verification
AM85CV15	Sept. 18 1985	21.00	24.87	vessel - berm verification

AM85CV20	Sept. 19 1985	20.10		vessel - berm verification
AM85CV09	Sept. 19 1985	19.00	25.97	vessel - berm verification
AM85CV06	Sept. 19 1985	19.00	23.25	vessel - berm verification
AM85CV22	Sept. 19 1985	19.40	25.09	vessel - berm verification
AM85CC01	Oct. 01 1985	-3.30	43.71	surface - core verification
AM85CC04	Oct. 01 1985	-3.40	26.83	surface - core verification
AM85CC03	Oct. 01 1985	-3.40	25.92	surface - core verification
AM85CC05	Oct. 01 1985	-3.10	43.74	surface - core verification
AM85CC10	Oct. 02 1985	-3.20	40.76	surface - core verification
AM85CC06	Oct. 02 1985	-3.10	26.80	surface - core verification
AM85CC02	Oct. 02 1985	-3.30	43.82	surface - core verification
AM85CC07	Oct. 02 1985	-3.10	43.76	surface - core verification
AM85CC08	Oct. 02 1985	-3.10	30.66	surface - core verification
AM85CC09	Oct. 02 1985	-3.10	43.69	surface - core verification
AM85CC12	Oct. 03 1985	-3.20	43.67	surface - core verification
AM85CC11	Oct. 03 1985	-3.20	30.69	surface - core verification
AM85CC16	Oct. 05 1985	-2.90	30.71	surface - core verification
AM85CC15	Oct. 05 1985	-3.10		surface - core verification
AM85CC14	Oct. 05 1985	-3.10	35.71	surface - core verification
AM85CC13	Oct. 05 1985	-3.20	30.75	surface - core verification
AM85CC18	Oct. 06 1985	-3.00	49.94	surface - core verification
AM85CC17	Oct. 06 1985	-3.00	44.19	surface - core verification
AM85CC19	Oct. 07 1985	-2.80	47.49	surface - core verification
AM85CC20	Oct. 17 1985	-3.00	39.97	surface - core verification
AM85CC21	Oct. 20 1985	-3.00	49.92	surface - core verification
AM86C02	Apr. 15 1986	-2.43	21.88	core surface - post April 12 ice event
AM86C01	Apr. 15 1986	-2.62	17.76	core surface - post April 12 ice event
AM86C03	Apr. 16 1986	-1.52	40.20	core surface - post April 12 ice event
AM86C04	Apr. 16 1986	-1.96	26.92	core surface - post April 12 ice event
AM86C05	Apr. 16 1986	-2.00	21.82	core surface - post April 12 ice event
AM86C08	Apr. 17 1986	-2.27	43.67	core surface - post April 12 ice event
AM86C07	Apr. 17 1986	-2.61	22.95	core surface - post April 12 ice event
AM86C09	Apr. 17 1986	-2.25	20.85	core surface - post April 12 ice event
AM86C06	Apr. 17 1986	-2.74	25.80	core surface - post April 12 ice event
AM86C10	Apr. 18 1986	-1.84	41.87	core surface - post April 12 ice event

3.2.7 Amauligak F-24

The berm at Amauligak F-24 was built in 32.0 m of water to support the Molikpaq structure. Located in the Kugmallit Channel, foundation conditions consisted of soft clay overlying fine sand. A 12 m deep subcut with 5H:1V side slopes was required to remove

806 605 m^3 of soft clay from the foundation area which was subsequently in-filled with sand from the Ukalerk borrow area in 1987 and the berm was built using material from Ukalerk and the abandoned Minuk Island. The Molikpaq was placed on the berm at a draft of 15.8 m and in-filled with sand dredged from the Amauligak I-65 abandoned island. The sand in the core was 4.8 m above sea level after fill densification. Two million cubic metres of sand was required for the subcut, berm and core. Gravel from Minuk I-53 was used in the shoulders of the berm and as toe protection. The rig was released in September 1988 and the island was abandoned by removing sand from the core and refloating the Molikpaq. A survey on August 17, 1989 indicated that the abandoned berm surface was at 10.2 m below sea level. A cross-section through the island is shown on Figure 3.13.

CPT data was taken at the site of the Amauligak F-24 artificial island in 7 stages, with a total of 127 CPT records available from this location. In September 1987, testing was done from a vessel as part of the site investigation, at the locations shown on Figure 3.14, and then for verification of the berm, as shown on Figure 3.15. Figure 3.16 shows the locations of the 17 CPT records from the test series taken from the core surface after the core filling was complete, but prior to the densification of the core by blasting, the 9 CPT tests that were performed during the first blast series are available, and the 18 tests from the second blast series. Figure 3.17 shows the locations of the 29 CPT tests done to evaluate the density of the sand core following the program of explosive densification, and shows the 23 CPT tests performed during the vibroprobe densification of the sand near the caisson walls from December 1987 to January 1988. Test results from the densification are described in Rogers et al (1990). A total of 127 tests are available for Amauligak F-24, as listed in Table 3.8

Table 3.8 - Amauligak F-24 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
AF85C05	Sept. 03 1985	32.60	12.07	vessel - site investigation
AF85C05b	Sept. 03 1985	32.60	32.24	vessel - site investigation
AF85C05c	Sept. 03 1985	32.60	51.59	vessel - site investigation
AF85C03a	Sept. 04 1985	30.70	13.09	vessel - site investigation
AF85C03b	Sept. 04 1985	30.70	13.68	vessel - site investigation
AF85C03c	Sept. 06 1985	31.80	52.27	vessel - site investigation
AF85C03d	Sept. 06 1985	31.80	12.27	vessel - site investigation
AF85C06a	Sept. 06 1985	32.20	12.11	vessel - site investigation
AF85C06b	Sept. 07 1985	31.50	28.42	vessel - site investigation
AF85C06c	Sept. 08 1985	31.50	39.42	vessel - site investigation
AF85C06d	Sept. 08 1985	31.50	53.21	vessel - site investigation
AF85C11	Sept. 10 1985	32.10	11.41	vessel - site investigation
AF85C12	Sept. 10 1985	30.90	12.78	vessel - site investigation
AF85C09	Sept. 10 1985	31.30	12.77	vessel - site investigation
AF85C08	Sept. 10 1985	31.70	12.20	vessel - site investigation
AF85C04	Sept. 10 1985	32.00	12.06	vessel - site investigation
AF85C02	Sept. 10 1985	31.60	13.98	vessel - site investigation
AF85C07	Sept. 10 1985	32.50	11.50	vessel - site investigation
AF85C10	Sept. 11 1985	32.30	53.33	vessel - site investigation
AF85C01	Sept. 11 1985	32.60	12.24	vessel - site investigation
AF87CV07	Sept. 15 1987	16.00	22.40	vessel - berm verification
AF87CV13	Sept. 15 1987	15.90	23.90	vessel - berm verification
AF87CV10	Sept. 15 1987	15.80	13.90	vessel - berm verification
AF87CV01	Sept. 15 1987	15.70	21.00	vessel - berm verification
AF87CV08	Sept. 15 1987	15.90	18.70	vessel - berm verification
AF87CV05	Sept. 15 1987	16.00	22.90	vessel - berm verification
AF87CV04	Sept. 15 1987	16.00	23.70	vessel - berm verification
AF87CV09	Sept. 15 1987	15.60	20.20	vessel - berm verification
AF87CV03	Sept. 16 1987	16.00	23.60	vessel - berm verification
AF87CV14	Sept. 16 1987	16.10	19.40	vessel - berm verification
AF87CV17	Sept. 16 1987	15.80	20.60	vessel - berm verification
AF87CC01	Sept. 24 1987	-5.00	20.54	core surface - prior to blasting
AF87CC02	Sept. 24 1987	-5.00	12.35	core surface - prior to blasting
AF87CC08	Sept. 25 1987	-5.00	32.96	core surface - prior to blasting
AF87CC05	Sept. 25 1987	-5.00	31.22	core surface - prior to blasting
AF87CC04	Sept. 25 1987	-5.00	30.45	core surface - prior to blasting
AF87CC06	Sept. 25 1987	-5.00	30.98	core surface - prior to blasting
AF87CC07	Sept. 25 1987	-5.00	31.34	core surface - prior to blasting
AF87CC03	Sept. 25 1987	-5.00	26.28	core surface - prior to blasting
AF87CC09	Sept. 26 1987	-5.00	30.72	core surface - prior to blasting

AF87CC10	Sept. 26 1987	-5.00	32.07	core surface - prior to blasting
AF87CC11	Sept. 26 1987	-5.00	32.98	core surface - prior to blasting
AF87CC12	Sept. 27 1987	-5.00	30.93	core surface - prior to blasting
AF87CC13	Sept. 27 1987	-5.00	30.95	core surface - prior to blasting
AF87CC14	Sept. 27 1987	-5.00	30.91	core surface - prior to blasting
AF87CC15	Sept. 27 1987	-5.00	30.91	core surface - prior to blasting
AF87CC16	Sept. 27 1987	-5.00	29.83	core surface - prior to blasting
AF87CC17	Sept. 27 1987	-5.00	31.65	core surface - prior to blasting
AF87CC18	Oct. 04 1987	-4.70	25.18	core surface - during blast "A"
AF87CC20	Oct. 05 1987	-5.00	27.56	core surface - during blast "A"
AF87CC22	Oct. 07 1987	-4.50	31.00	core surface - during blast "A"
AF87CC23	Oct. 07 1987	-4.50	28.92	core surface - during blast "A"
AF87CC21	Oct. 07 1987	-4.50	11.97	core surface - during blast "A"
AF87CC26	Oct. 08 1987	-4.40	21.22	core surface - during blast "A"
AF87CC25	Oct. 08 1987	-4.50	29.97	core surface - during blast "A"
AF87CC24	Oct. 08 1987	-4.40	30.75	core surface - during blast "A"
AF87CC28	Oct. 09 1987	-5.00	26.02	core surface - during blast "B"
AF87CC27	Oct. 09 1987	-4.70	28.86	core surface - during blast "A"
AF87CC30	Oct. 11 1987	-4.90	35.82	core surface - during blast "B"
AF87CC29	Oct. 11 1987	-4.70	30.68	core surface - during blast "B"
AF87CC32	Oct. 11 1987	-4.70	30.03	core surface - during blast "B"
AF87CC31	Oct. 11 1987	-4.90	39.99	core surface - during blast "B"
AF87CC34	Oct. 12 1987	-4.60	27.00	core surface - during blast "B"
AF87CC35	Oct. 12 1987	-4.60	28.86	core surface - during blast "B"
AF87CC33	Oct. 12 1987	-4.60	31.08	core surface - during blast "B"
AF87CC36	Oct. 13 1987	-4.50	30.49	core surface - during blast "B"
AF87CC37	Oct. 13 1987	-4.50	29.68	core surface - during blast "B"
AF87CC38	Oct. 13 1987	-4.50	26.70	core surface - during blast "B"
AF87CC39	Oct. 13 1987	-4.50	26.80	core surface - during blast "B"
AF87CC41	Oct. 14 1987	-4.50	26.76	core surface - during blast "B"
AF87CC42	Oct. 14 1987	-4.70	26.92	core surface - during blast "B"
AF87CC40	Oct. 14 1987	-4.50	26.77	core surface - during blast "B"
AF87CC43	Oct. 22 1987	-4.70	31.00	core surface - during blast "B"
AF87CC44	Oct. 22 1987	-4.70	28.17	core surface - during blast "B"
AF87CC45	Oct. 22 1987	-4.80	30.95	core surface - during blast "B"
AF87CF20	Oct. 23 1987	-5.00	30.95	core surface - post blasting
AF87CF19a	Oct. 23 1987	-5.00	30.83	core surface - post blasting
AF87CF19	Oct. 23 1987	-5.00	30.88	core surface - post blasting
AF87CF15a	Oct. 25 1987	-5.00	62.96	core surface - post blasting
AF87CF15	Oct. 25 1987	-4.47	30.91	core surface - post blasting
AF87CF17	Oct. 25 1987	-5.00	30.96	core surface - post blasting
AF87CF18	Oct. 25 1987	-5.00	30.97	core surface - post blasting
AF87CF18a	Oct. 25 1987	-5.00	30.94	core surface - post blasting
AF87CF13	Oct. 26 1987	-4.98	24.94	core surface - post blasting

AF87CF14	Oct. 26 1987	-4.95	30.99	core surface - post blasting
AF87CF13a	Oct. 26 1987	-5.00	26.93	core surface - post blasting
AF87CF16	Oct. 26 1987	-4.50	31.08	core surface - post blasting
AF87CF08	Oct. 27 1987	-4.92	30.85	core surface - post blasting
AF87CF06	Oct. 27 1987	-4.40	19.56	core surface - post blasting
AF87CF07	Oct. 27 1987	-4.60	25.53	core surface - post blasting
AF87CF09	Oct. 27 1987	-4.70	30.92	core surface - post blasting
AF87CF10	Oct. 27 1987	-4.75	22.96	core surface - post blasting
AF87CF12	Oct. 27 1987	-4.84	20.85	core surface - post blasting
AF87CF11	Oct. 27 1987	-4.88	26.15	core surface - post blasting
AF87CF02	Oct. 28 1987	-4.50	30.99	core surface - post blasting
AF87CF02a	Oct. 28 1987	-4.44	30.96	core surface - post blasting
AF87CF05	Oct. 28 1987	-4.30	21.96	core surface - post blasting
AF87CF06a	Oct. 28 1987	-4.44	18.02	core surface - post blasting
AF87CF14a	Oct. 29 1987	-4.92	32.03	core surface - post blasting
AF87CF14b	Oct. 29 1987	-4.85	30.92	core surface - post blasting
AF87CF14c	Oct. 29 1987	-4.84	30.93	core surface - post blasting
AF87CF04	Oct. 31 1987	-4.50	43.00	core surface - post blasting
AF87CF03	Oct. 31 1987	-4.60	43.04	core surface - post blasting
AF87CF01	Nov. 02 1987	-4.40	29.13	core surface - post blasting
AF87C01	Dec. 13 1987	-4.80	24.60	core surface - during vibroprobe densification
AF87C05	Dec. 13 1987	-4.80	20.95	core surface - during vibroprobe densification
AF87C04	Dec. 13 1987	-4.90	21.35	core surface - during vibroprobe densification
AF87C02	Dec. 13 1987	-4.80	22.45	core surface - during vibroprobe densification
AF87C08	Dec. 14 1987	-5.00	21.95	core surface - during vibroprobe densification
AF87C09	Dec. 14 1987	-4.80	21.80	core surface - during vibroprobe densification
AF87C11	Dec. 15 1987	-4.80	22.75	core surface - during vibroprobe densification
AF87C10	Dec. 15 1987	-4.80	21.85	core surface - during vibroprobe densification
AF87C13	Dec. 17 1987	-4.80	22.80	core surface - during vibroprobe densification
AF87C12	Dec. 17 1987	-4.80	22.75	core surface - during vibroprobe densification
AF87C14	Dec. 17 1987	-4.80	14.00	core surface - during vibroprobe densification
AF88C17	Jan. 04 1988	-4.40	22.67	core surface - during vibroprobe densification
AF88C16	Jan. 04 1988	-4.50	16.87	core surface - during vibroprobe densification
AF88C15	Jan. 04 1988	-4.60	17.31	core surface - during vibroprobe densification
AF88C18	Jan. 16 1988	-4.50	24.40	core surface - during vibroprobe densification
AF88C19	Jan. 16 1988	-4.40	18.57	core surface - during vibroprobe densification
AF88C24	Jan. 16 1988	-4.50	24.64	core surface - during vibroprobe densification
AF88C25	Jan. 16 1988	-4.30	24.66	core surface - during vibroprobe densification
AF88C20	Jan. 16 1988	-5.00	21.04	core surface - during vibroprobe densification
AF88C23	Jan. 17 1988	-4.50	17.00	core surface - during vibroprobe densification
AF88C26	Jan. 17 1988	-4.50	23.60	core surface - during vibroprobe densification
AF88C22	Jan. 17 1988	-4.40	35.39	core surface - during vibroprobe densification
AF88C21	Jan. 17 1988	-5.00	24.40	core surface - during vibroprobe densification

3.3 Sand and Gravel Islands

3.3.1 Tarsiut N-44

Tarsiut N-44 was the first sub-sea berm in the Beaufort Sea, built by Dome Petroleum Ltd. in 1981. The berm was built in 21.0 m of water on Kringalik Plateau. The local seabed consisted of soft clay overlying stiff clay and silt. A three metre subcut was required to remove the soft clay from the foundation area prior to fill placement. The berm was constructed using 1.8 million cubic metres of both sand and gravel excavated from the Issigak (South Tarsiut) borrow area and sand from the Ukalerk borrow pit, bottom dumped on the site. Gravel from Herschel Island was placed on the 170 m diameter berm crest as a one metre cap. The finished berm was 6.5 m below sea level. Concrete caissons were placed on the berm to retain a core zone of sand and provide erosion protection and a working surface for drilling operations. The caissons were in-filled with sand hauled from the Ukalerk borrow pit, creating the working surface. The berm was upgraded in 1982, by adding 10 000 m^3 of riprap scour rock with an average size of 1 m diameter around the berm to protect it from wave action. The island was abandoned in 1984. A cross-section through the island is shown on Figure 3.18.

A total of 27 CPT records as listed in Table 3.9 are available from Tarsiut N-44, taken in August 1981 as part of the verification of the berm, and from the core surface in September 1981 to verify the density of the sand core. The location of the CPT holes are shown on Figure 3.19.

Table 3.9 - Tarsiut N-44 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
DTR81C05	Aug. 1981	6.48	10.96	vessel - sand and gravel berm verification
DTR81C13	Aug. 1981	6.52	16.41	vessel - sand and gravel berm verification
DTR81C22	Aug. 1981	6.51	15.82	vessel - sand and gravel berm verification
DTR81C11	Aug. 1981	6.49	16.48	vessel - sand and gravel berm verification
DTR81C09	Aug. 1981	6.49	14.48	vessel - sand and gravel berm verification
DTR81C16	Aug. 1981	6.52	14.92	vessel - sand and gravel berm verification
DTR81C17	Aug. 1981	6.52	15.91	vessel - sand and gravel berm verification
DTR81C18	Aug. 1981	6.49	17.88	vessel - sand and gravel berm verification
DTR81C19	Aug. 1981	6.54	18.47	vessel - sand and gravel berm verification
DTR81C20	Aug. 1981	6.50	16.18	vessel - sand and gravel berm verification
DTR81C15	Aug. 1981	6.51	15.49	vessel - sand and gravel berm verification
DTR81C06	Aug. 1981	6.53	16.44	vessel - sand and gravel berm verification
DTR81C01	Aug. 1981	6.47	10.85	vessel - sand and gravel berm verification
DTR81C02	Aug. 1981	6.47	11.00	vessel - sand and gravel berm verification
DTR81C03	Aug. 1981	6.49	10.49	vessel - sand and gravel berm verification
DTR81C04	Aug. 1981	6.53	10.67	vessel - sand and gravel berm verification
DTR81C07	Aug. 1981	6.54	15.87	vessel - sand and gravel berm verification
DTR81C08	Aug. 1981	6.53	16.41	vessel - sand and gravel berm verification
DTR81C23	Sept. 1981	-7.04	30.37	surface - sand core verification
DTR81C24	Sept. 1981	-6.94	29.34	surface - sand core verification
DTR81C25	Sept. 1981	-6.60	29.21	surface - sand core verification
DTR81C26	Sept. 1981	-6.98	10.64	surface - sand core verification
DTR81C27	Sept. 1981	-7.04	31.05	surface - sand core verification
DTR81C28	Sept. 1981	-6.98	30.40	surface - sand core verification
DTR81C29	Sept. 1981	-7.06	31.21	surface - sand core verification
DTR81C30	Sept. 1981	-6.86	29.54	surface - sand core verification
DTR81C31	Sept. 1981	-6.96	24.28	surface - sand core verification

3.3.2 Nipterk L-19

Nipterk L-19 was a sacrificial beach island built by Esso in the 1983 and 1984 open water seasons. Located in 11.7 m of water in the Ikit Trough, the seabed consisted of soft clay overlying firm clay. A total of 1.5 million m^3 of Ukalerk sand and Issigak sand and gravel was hopper placed to create an island with a 110 m diameter working surface at a 5 m freeboard. Island dimensions included a 170 m diameter at waterline and 370 m diameter base area. The material was bottom dumped in 1983 to an elevation of 8 m below mean sea level using material from Ukalerk and Issigak. In 1984, a bow spreader method was used to bring the island to sea level and a nozzle was used to complete construction using mainly Issigak gravel. Slopes of 3H:1V were maintained above waterline, while the beach and base had 8H:1V and 4H:1V slopes, respectively. Two wells were drilled from this platform over the 1984-85 winter and spring seasons. The island was originally named Nipterk I-19, but was renamed to L-19 based on re-evaluated seismic data. A cross-section through the island is shown on Figure 3.20.

No CPT data is available from Nipterk L-19.

3.3.3 Minuk I-53

When construction at Minuk I-53 started in September 1982, it was Esso's intention to use it as the foundation for the first use of its steel caisson retained island (CRI) and, during 1982 and 1983, Ukalerk sand was bottom-dumped to produce a sand berm to Elevation -9 m. When the CRI was subsequently located at Kadluk O-07, Minuk I-53 was converted to a sacrificial beach island. In 1984, 914 500 m^3 of material was placed on the island to bring it to Elevation +1 m. The majority (71%) of this material was obtained from Issigak. The abandoned Kadluk O-07 provided 11% of the material; 9% came direct from Ukalerk and 9% from Isserk. The island was topped off in 1985 using nozzle discharge of Issigak gravel. However, the island was severely damaged by a major storm in the latter part of that open water season, resulting in the loss of the drilling rig and associated equipment. Repair was effected in 1985 and the drilling operations were carried out over the ensuing winter season.

Minuk I-53, in 14.7 m of water, is founded on a soft clay overlying a firm clay on the Kringalik Plateau. Construction of the island required approximately 2 million m^3 of fill.

It had a 110 m diameter working surface at a 5 m freeboard. The freeboard slope was 2H:1V with a surrounding gravel beach at 15H:1V. A cross-section through the island is shown on Figure 3.21.

No CPT data is available from Minuk I-53.

3.3.4 Kaubvik I-43

Esso constructed the caisson retained island at Kaubvik I-43 during the 1983 to 1986 open water seasons in 17.9 m of water. Located in the Ikit Trough, the local seabed consists of soft, silty clay overlying firm clay. A glory hole was excavated in the seabed prior to berm construction. Sand and gravel totalling 566 000 m3 was dredged from the Ukalerk, Issigak and Isserk borrow pits using trailing suction hopper dredges and bottom dumped on the site. The berm was built to within 9 m of the waterline with a top surface 90 m in diameter. Slopes of 12H:1V were maintained. The caisson toe protection material was pipeline placed Issigak material.

The location was hit by a summer storm when the geotechnical instrumentation program was underway. No ponding of water was observed due to wave overtopping indicating free draining nature of the fill material in the core of the CRI. The storm lasted for approximately 10 hours. The island was used for drilling over the fall of 1986, and abandoned in 1987. During deballasting, the sand core was partially removed, leaving a berm 1 m below the waterline. A cross-section through the island is shown on Figure 3.22.

The majority of the material used in the construction was obtained from the Issigak pit, though some material was obtained from Ukalerk, Isserk and the abandoned Kadluk O-07 berm.

No CPT data is available from Kaubvik I-43.

3.4 Other Islands with CPT Data

3.4.1 Isserk Trial Berm

A trail berm was built in 1984 using fine sand from the Isserk Plateau to test the insitu density of dredge transported material. The insitu density was monitored using the cone penetration test.

Data from 8 CPT tests are available from the Isserk Trial berm, as listed in Table 3.10.

Table 3.10 - Isserk Trial Berm CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
IBCI01	Oct. 09 1984	18.00	5.39	vessel - trial berm
IBCI02	Oct. 09 1984	18.80	7.49	vessel - trial berm
IBCI03	Oct. 09 1984	19.20	6.39	vessel - trial berm
IBCI08	Oct. 09 1984	18.70	6.74	vessel - trial berm
IBCI06	Oct. 09 1984	18.40	7.51	vessel - trial berm
IBCI05	Oct. 09 1984	18.20	8.06	vessel - trial berm
IBCI04	Oct. 09 1984	18.40	7.44	vessel - trial berm
IBCI07	Oct. 09 1984	18.80	6.59	vessel - trial berm

3.4.2 Ukalerk Trial Berms

Similar trials had been completed in 1982 to test the insitu density of dredge transported Ukalerk sand. Two small trial berms were constructed, first using dredge bottom valve discharge, and secondly using pumpout through the drag arm.

Data from 6 CPT tests are available from the Ukalerk bottom valve trial berm, as listed in Table 3.11. 10 CPT tests are available from the Ukalerk pump out trial berm, as listed in

Table 3.12. The test results indicated that material placed by bottom valve discharge achieved a higher density than pump-out placed material. No further work was carried out at these trial berm locations.

Table 3.11 - Ukalerk Bottom Valve Trial Berm CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
UKB82C01	Sept. 01 1982	18.00	5.57	vessel - bottom valve berm verification
UKB82C06	Sept. 01 1982	18.60	4.39	vessel - bottom valve berm verification
UKB82C02	Sept. 01 1982	18.50	4.30	vessel - bottom valve berm verification
UKB82C04	Sept. 01 1982	19.50	4.40	vessel - bottom valve berm verification
UKB82C05	Sept. 01 1982	18.60	5.30	vessel - bottom valve berm verification
UKB82C03	Sept. 01 1982	18.80	5.30	vessel - bottom valve berm verification

Table 3.12 - Ukalerk Pump Out Trial Berm CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
UKP82C13	Sept. 01 1982	23.20	7.39	vessel - pumpout berm verification
UKP82C04	Sept. 01 1982	23.20	7.67	vessel - pumpout berm verification
UKP82C05	Sept. 01 1982	23.20	6.68	vessel - pumpout berm verification
UKP82C06	Sept. 01 1982	23.00	6.44	vessel - pumpout berm verification
UKP82C07	Sept. 01 1982	23.00	5.29	vessel - pumpout berm verification
UKP82C08	Sept. 01 1982	24.50	4.28	vessel - pumpout berm verification
UKP82C11	Sept. 01 1982	23.50	7.42	vessel - pumpout berm verification
UKP82C01	Sept. 01 1982	23.70	6.69	vessel - pumpout berm verification
UKP82C12	Sept. 01 1982	23.50	7.38	vessel - pumpout berm verification
UKP82C14	Sept. 01 1982	23.50	4.98	vessel - pumpout berm verification

3.4.3 Nerlerk B-67

The Nerlerk berm has been included in this documentation as it provides a good data set of cone tests in sandfill that subsequently failed during island construction. The Nerlerk B-67 berm was constructed in 1982 and 1983 to support Canmar's SSDC oil exploration structure. The berm was to be a 36 m high sand berm founded on the seabed in 44 m of water. Foundation conditions at the site consist of a relatively thin veneer of Holocene clay underlain by apparently dense sand deposits. The clay thickness is a minimum (about 1 m) at the centre of the island and generally increases to about 1.5 m at the toe of the berm. The clay was not stripped prior to island construction.

Fill placement was by hopper dredge dumping sand onto the berm, or by stationary suction dredge placing slurry by pipeline onto the berm. Construction commenced in 1982 with placement of sand dredged from the Ukalerk borrow area and transported in hopper dredges. Later in the season the suction dredge Aquarius started fill placement of the local borrow material through a floating pipeline and chain nozzle system. Approximately 3 million m^3 of material was placed in 1982. In 1983, fill placement was almost exclusively by the Aquarius, but in this case using an umbrella shaped nozzle discharge. Approximately 2 million m^3 was placed in 1983.

When the sand berm reached a height of about 26 m, several large failures of the berm occurred. The first failure occurred at the end of the 1982 construction season, and 5 other failures occurred in July and August 1983. The project was abandoned in 1983. A cross-section through the island is shown on Figure 3.23.

A total of 43 CPT records are available from Nerlerk B-67 as listed in Table 3.13. The locations of these CPT tests are shown on Figure 3.24. 7 tests were taken prior to the July 20 1983 failure, 3 prior to the July 28 failure and 15 prior to the August 4 failure. An additional 18 CPT tests were performed on the berm in 1988. The location of these tests are shown on Figure 3.25.

Table 3.13 - Nerlerk B-67 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
D01NERLERK	Jul. 14 1983	25.73	14.70	vessel - berm prior to July 20 failure
D02NERLERK	Jul. 17 1983	20.98	17.06	vessel - berm prior to July 20 failure
D03NERLERK	Jul. 17 1983	20.98	7.50	vessel - berm prior to July 20 failure
D05NERLERK	Jul. 18 1983	27.18	18.75	vessel - berm prior to July 20 failure
D04NERLERK	Jul. 18 1983	27.55	9.20	vessel - berm prior to July 20 failure
D07NERLERK	Jul. 19 1983	25.05	9.77	vessel - berm prior to July 20 failure
D06NERLERK	Jul. 19 1983	25.68	10.01	vessel - berm prior to July 20 failure
D09NERLERK	Jul. 25 1983	20.82	9.73	vessel - berm prior to July 28 failure
D10NERLERK	Jul. 26 1983	17.94	10.62	vessel - berm prior to July 28 failure
D11NERLERK	Jul. 27 1983	18.73	14.80	vessel - berm prior to July 28 failure
D12NERLERK	Aug. 06 1983	25.82	18.67	vessel - berm after August 4 failure
D13NERLERK	Aug. 06 1983	23.46	15.39	vessel - berm after August 4 failure
D14NERLERK	Aug. 06 1983	25.00	13.25	vessel - berm after August 4 failure
D15NERLERK	Aug. 06 1983	22.38	9.87	vessel - berm after August 4 failure
D16NERLERK	Aug. 06 1983	20.87	11.85	vessel - berm after August 4 failure
D20NERLERK	Aug. 07 1983	23.04	14.51	vessel - berm after August 4 failure
D22NERLERK	Aug. 07 1983	19.38	17.19	vessel - berm after August 4 failure

D21NERLERK	Aug. 07 1983	21.87	16.40	vessel - berm after August 4 failure
D19NERLERK	Aug. 07 1983	20.14	3.04	vessel - berm after August 4 failure
D18NERLERK	Aug. 07 1983	19.10	11.16	vessel - berm after August 4 failure
D17NERLERK	Aug. 07 1983	19.38	10.20	vessel - berm after August 4 failure
D23NERLERK	Aug. 11 1983	18.52	12.38	vessel - berm after August 4 failure
D24NERLERK	Aug. 11 1983	17.70	13.52	vessel - berm after August 4 failure
D25NERLERK	Aug. 11 1983	21.43	14.22	vessel - berm after August 4 failure
D26NERLERK	Aug. 11 1983	23.08	15.13	vessel - berm after August 4 failure
B6788C04	Aug. 18 1988	20.40	19.70	vessel - berm
B6788C05	Aug. 18 1988	19.70	18.13	vessel - berm
B6788C11	Aug. 18 1988	27.20	14.87	vessel - berm
B6788C12	Aug. 18 1988	26.70	18.29	vessel - berm
B6788C09	Aug. 18 1988	20.10	23.15	vessel - berm
B6788C13	Aug. 18 1988	27.70	17.35	vessel - berm
B6788C03	Aug. 18 1988	21.30	17.56	vessel - berm
B6788C06	Aug. 21 1988	21.70	23.85	vessel - berm
B6788C05C	Aug. 21 1988	43.10	1.88	vessel - berm
B6788C05B	Aug. 21 1988	20.70	24.28	vessel - berm
B6788C08	Aug. 22 1988	20.10	25.00	vessel - berm
B6788C06B	Aug. 22 1988	42.80	2.75	vessel - berm
B6788C20	Aug. 22 1988	20.50	24.00	vessel - berm
B6788C20B	Aug. 23 1988	42.70	1.80	vessel - berm
B6788C21	Aug. 23 1988	20.40	18.52	vessel - berm
B6788C07	Aug. 24 1988	21.50	18.74	vessel - berm
B6788C14	Aug. 24 1988	34.95	11.93	vessel - berm
B6788C01	Aug. 25 1988	21.40	8.91	vessel - berm

3.4.4 Isserk I-15

The Isserk Molikpaq deployment provides the last set of cone tests documented in this project. Isserk I-15 was drilled from the Molikpaq structure during the 1989-1990 winter season for Chevron et al. A 2 m deep subcut was excavated in the surficial soft clay at the site prior to setdown of the Molikpaq. The caisson was partially filled with 72,000 m^3 of sand fill from the abandoned Amauligak I-65 berm. The rig was released in January 1990, and was subsequently removed from site in the open water season. A cross-section through the island is shown on Figure 3.26.

Figure 3.27 shows the locations of the 17 CPT tests performed from the Molikpaq deck after setdown on the berm and before, during, and after the core filling. Table 3.14 lists the data set, which provides useful information in the insitu density of the sandfill in the core of the Molikpaq. It also provides useful data on the effect of load from the corefill on the foundation cone tip resistance.

Table 3.14 - Isserk I-15 CPT Data

Test	Test Date	Depth to Soil Surface From MSL (m)	Maximum CPT Depth (m)	Testing Mode
89ISSI08	Sept. 21 1989	13.40	14.60	Molikpaq deck - after core filling
89ISSI03	Sept. 21 1989	13.56	9.72	Molikpaq deck - prior to core filling
89ISSI01	Sept. 22 1989	13.04	5.44	Molikpaq deck - prior to core filling
89ISSI02	Sept. 22 1989	14.56	8.74	Molikpaq deck - prior to core filling
89ISSI05	Sept. 24 1989	12.00	8.15	Molikpaq deck - during core filling
89ISSI04A	Sept. 24 1989	13.04	2.40	Molikpaq deck - during core filling
89ISSI06A	Sept. 26 1989	8.20	13.90	Molikpaq deck - during core filling
89ISSI07	Sept. 27 1989	9.40	13.50	Molikpaq deck - during core filling
89ISSI10	Sept. 29 1989	11.90	13.90	Molikpaq deck - after core filling
89ISSI09	Sept. 29 1989	11.60	6.70	Molikpaq deck - after core filling
89ISSI11	Oct. 01 1989	11.90	17.60	Molikpaq deck - after core filling
89ISSI12	Oct. 01 1989	12.50	18.90	Molikpaq deck - after core filling
89ISSI13	Oct. 01 1989	12.40	17.60	Molikpaq deck - after core filling
89ISSI14	Oct. 02 1989	14.30	12.80	Molikpaq deck - after core filling
89ISSI15	Oct. 03 1989	13.70	13.10	Molikpaq deck - after core filling
89ISSI16A	Oct. 03 1989	9.60	13.80	Molikpaq deck - after core filling
89ISSI17	Oct. 03 1989	10.00	7.80	Molikpaq deck - after core filling

4. SUITABILITY OF ARTIFICIAL ISLAND MATERIAL FOR FUTURE CONSTRUCTION

4.1 Material Available on Abandoned Islands

The approximate quantities of sand and gravel material remaining in the selected abandoned artificial islands was collated in the Klohn-Crippen 1997 report, and is summarized in Table 4.1. This information is based on the most recently available bathymetric surveys. Shown on the table are the date of the latest survey, the water depths, the average fill gradation and notes on constraints which may limit the usefulness of these islands as sources of borrow material.

The quality of fill material varied from site to site, therefore the islands with similar material types have been grouped together. Fill quality in the sand islands is very consistent, with a D_{50} in the range of 310 μm to 360 μm, and typically less than 1.5% fines. Fill gradation in the sand and gravel islands varies considerably due to the different borrow sources used, but typically includes fill with a D_{50} between 300 μm and 3000 μm, with less than 2.5% fines.

Table 4.1 - Material in Abandoned Islands

Island Name	Island Type	Water Depth (m)	Date of Survey	Depth to Island (m)	Approximate Fill Quantity (m)	Fill Gradation		Constraints to Utilization
						D_{50} µm	Fines %	
Sand Islands								
Uviluk P-66	Single Steel Drilling Caisson	29.7	-	>9	1 500 000	330 - 370	(a)	Minimal
Kogyuk N-67	Single Steel Drilling Caisson	28.1	08/29/85	8.0	1 000 000	320	1.4	Minimal
Kadluk O-07	Caisson Retained Island	14.0	-	?	300 000	290 - 360	2.4	Minor Drilling Debris
Amerk P-09	Caisson Retained Island	27.0	19/10/85	9.0	1 000 000	315	(a)	Minor Drilling Debris
Tarsiut P-45	Molikpaq	25.5	1985	>10	300 000	315 - 400	1 - 1.5	Minimal
Amauligak I-65	Molikpaq	31.0	18/17/89	16.0	900 000	310 - 350	1 - 1.5	Minimal
Amauligak F-24	Molikpaq	32.0	08/17/90	10	1 600 000	310 - 360	0.5 - 1.5	Minimal
TOTAL					6 600 000			
Sand and Gravel Islands								
Tarsiut N-44	Caisson Retained Island	21.0	09/14/84	6.5	1 500 000	300 - 780	1 - 5	Piles/Clay and Rock Content
Nipterk L-19	Sacrificial Beach Island	11.7	07/90	2.0	1 000 000	300 - 1500	0.5 - 2	Minor Drilling Debris
Minuk I-53	Sacrificial Beach Island	14.7	07/90	awash	1 500 000	240 - 3100	0.7 - 2.4	Drilling Debris and Potential Contaminants
Kaubvik I-43	Caisson Retained Island	17.9	07/90	4.5	400 000	(a)	(a)	Minor Drilling Debris
TOTAL					4 400 000			

(a) No records found.

4.2 Expected Density

The database of CPT records provides a means of assessing the expected densities of hydraulic fill islands constructed using material from the Ukalerk borrow area. Placement methods have a major influence on insitu density. This has been reported by a number of authors. Two relevant publications are included in Appendix II to introduce the reader to the issues that influence insitu density. These two publications are:

- Jefferies, M.G., Rogers, B.T., Stewart, H.R., Shinde, S., James, D. and Williams-Fitzpatrick, S. 1988. Island Construction in the Canadian Beaufort Sea. Proceedings, Conference on Hydraulic Fill Structures, ASCE, Fort Collins, CO, pp. 816-883.

- Sladen, J.A. and Hewitt, K.J., 1989. Influence of placement method on the insitu density of hydraulic sand fills. Canadian Geotechnical Journal, vol. 26, no. 3, August 1989, pp. 453-466.

The insitu density of hydraulic fill is influenced by material grain size. The material from the Ukalerk pit with D_{50} in the range 310 mm to 360 mm, and typically less than 1.5% fines, has been found to result in acceptable densities when the material was bottom dumped from dredges. Material placed by pumpout through the dredge drag arms results in a lower density, but not a low as for fill placed from a single spiggot pipe in the core of the Molikpaq.

To achieve a density that is not subject to pore water pressure rise during cyclic ice loading, the density of the spiggot placed material at Amauligak F-24 was increased by densification, using explosives. The density achieved post densification was approximately the same as the density that was achieved from bottom dumped dredge operations.

Material from the Issigak pit contains gravel and cobble size material. Hence the cone penetration test is not suitable as a means of assessing insitu density. However this coarser material has a higher insitu permeability, and as such is less susceptible to liquefaction.

All the material remaining in the islands summarized in Table 4.1 is considered a future resources for construction of islands in the Canadian Beaufort Sea. The precedent from previous construction operations allows for predictions of expected insitu density, dependent of the construction methods used for deposition.

5. CLOSURE

This report archives information on the cone penetration testing performed at selected artificial islands in the Canadian Beaufort Sea. These islands contain approximately eleven (11) million cubic metres of delineated sand and gravel that are available as a granular resource in eleven (11) abandoned island sites. The mean D_{50} for this resource is 0.3 mm or coarser, with fines contents in the range of 0.5% to 2%.

The CPT data from 422 cone profiles and the fill quality data, together with a brief listing of the construction activities related to island construction, have been archived to CD Rom. This data is easily accessed using the program BeauFILL.

Respectfully submitted,

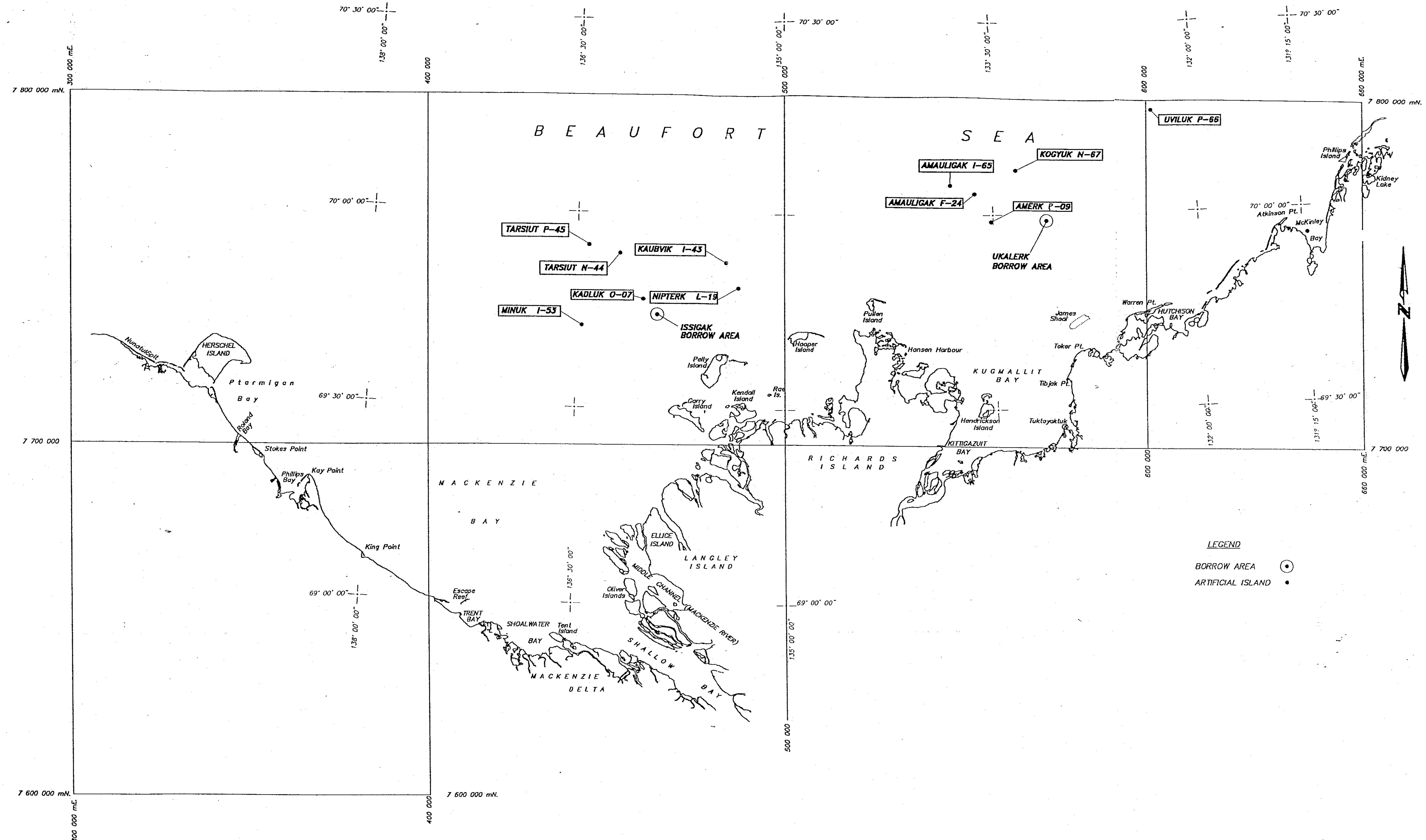
KLOHN-CRIPPEN CONSULTANTS LTD.

for Scott N. Martens, P.Eng.
Geotechnical Engineer

Brian T. Rogers, P.Eng.
Manager, Geotechnical Division

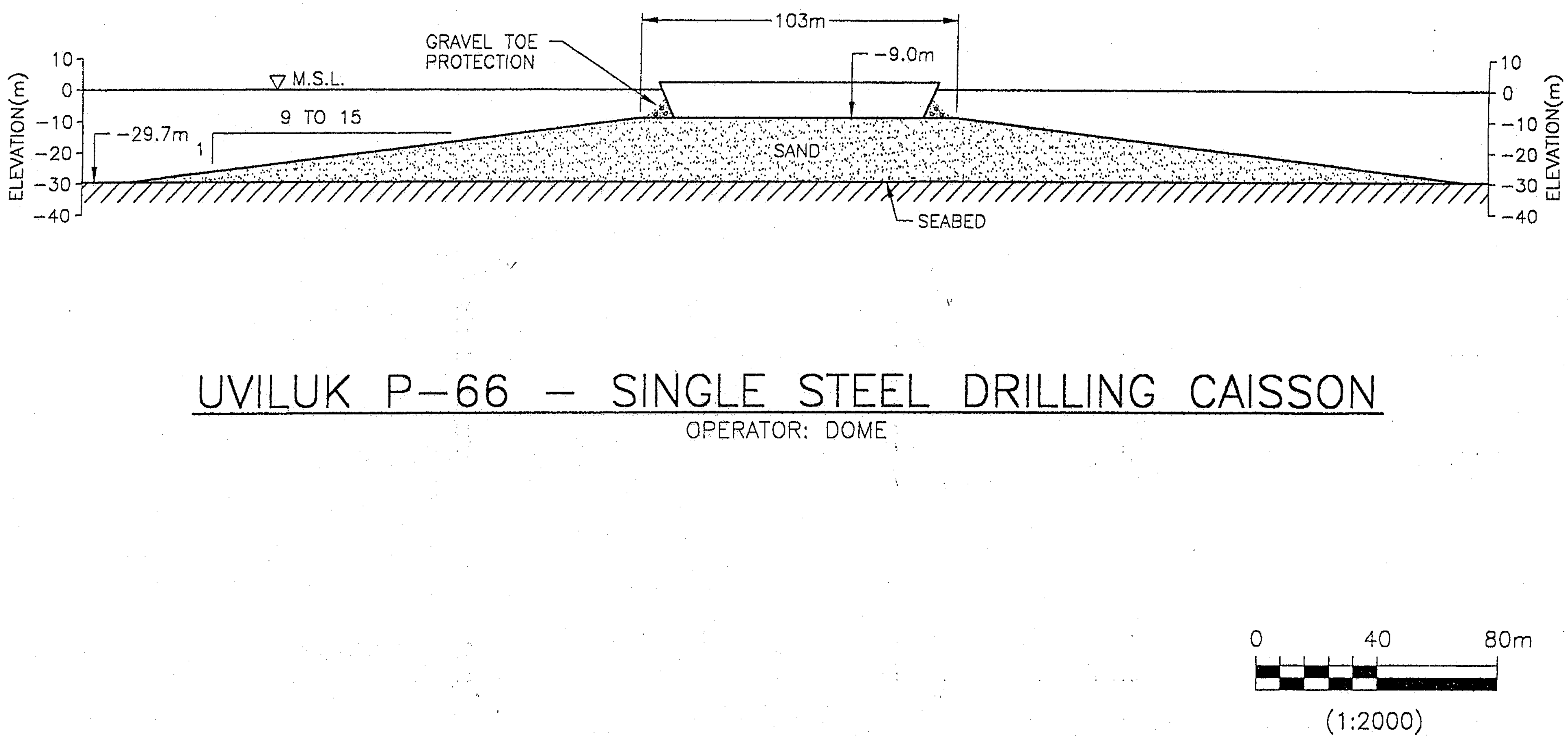
BTR/SNM
attachments

FIGURES



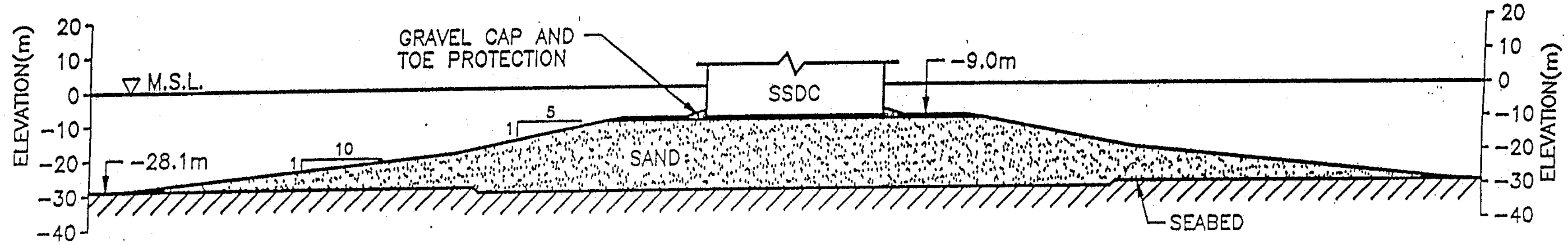
KLOHN-CRIPPEN

Figure 1.1: Location Plan



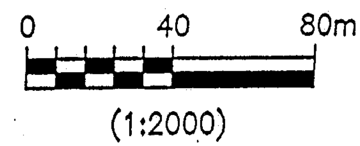
KLOHN-CRIPPEN

Figure 3.1: Uviluk P-66 - Cross-Section



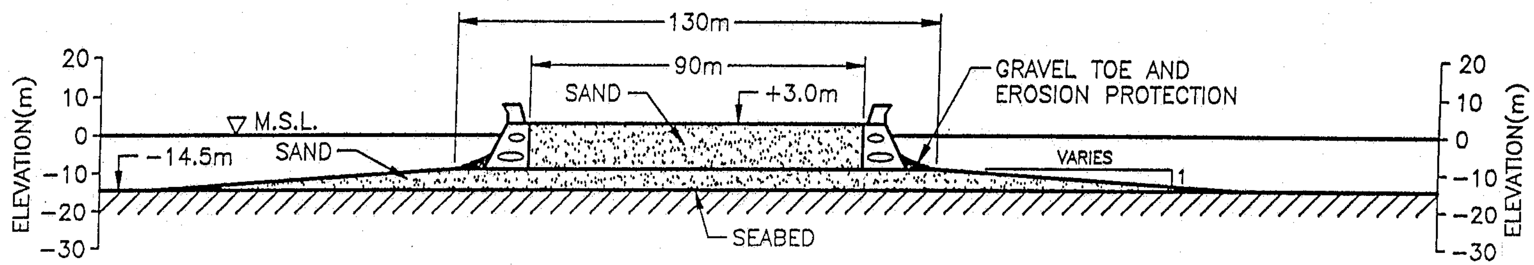
KOGYUK N-67 – SINGLE STEEL DRILLING CAISSON

OPERATOR: GULF



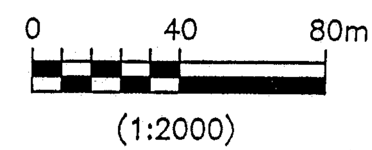
KLOHN-CRIPPEN

Figure 3.2: Kogyuk N-67 - Cross-Section



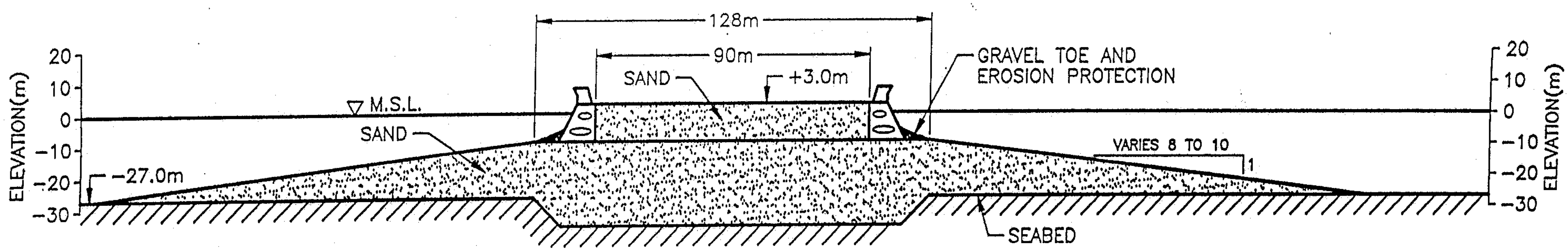
KADLUK O–07 – CAISSON RETAINED ISLAND

OPERATOR: ESSO



KLOHN-CRIPPEN

Figure 3. 3 : Kadluk O-07 - Cross-Section



AMERK O-09 - CAISSON RETAINED ISLAND

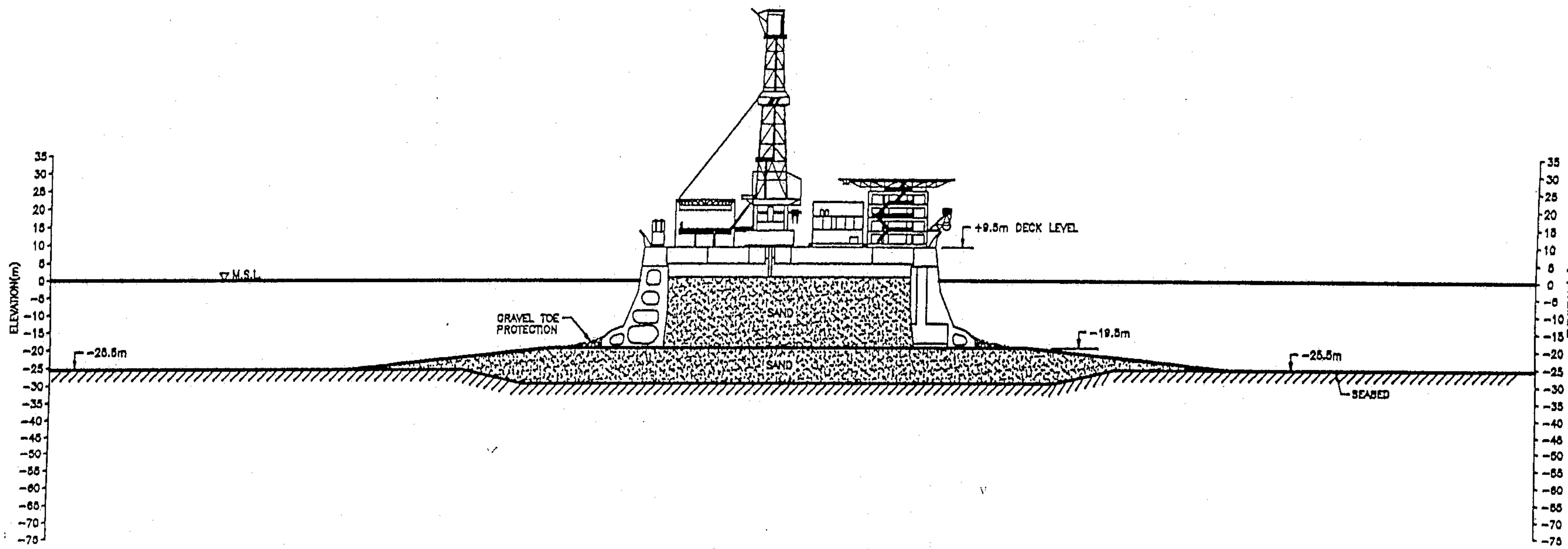
OPERATOR: ESSO

0 40 80m

(1:2000)

KLOHN-CRIPPEN

Figure 3. 4 : Amerk P-09 - Cross-Section



TARSIUT P–45 – MOLIKPAQ

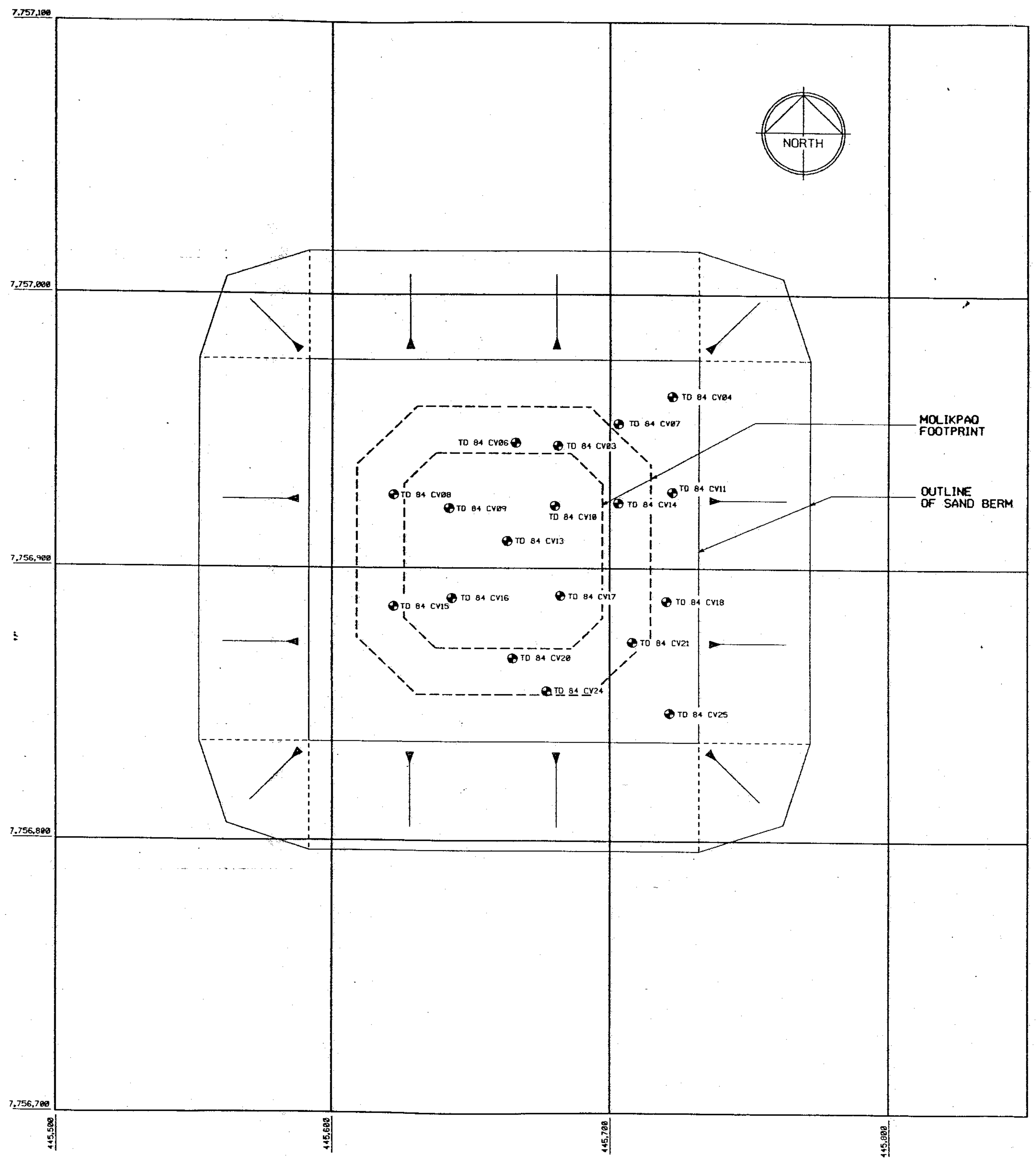
OPERATOR: GULF

0 40 80m

(1:2000)

KLOHN-CRIPPEN

Figure 3.5 : Tarsiut P-45 - Cross-Section

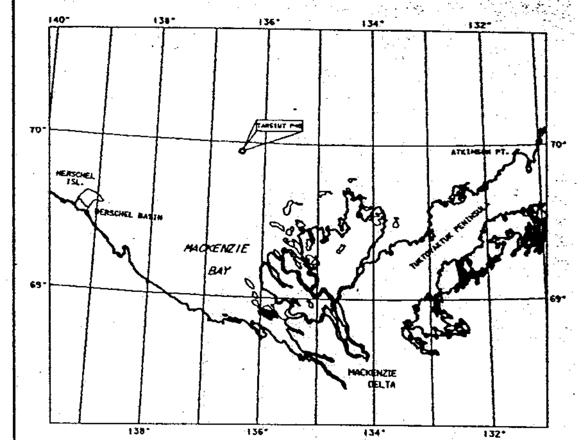


PLAN

SCALE (metres) 1:1000

TABLE OF VERIFICATION COREHOLES

CORE HOLE LABEL	UTM (ZONE 8)		DEPTH (METERS)
	N (METERS)	E (METERS)	
TD 84 CV03	7,756,945	445,681	
TD 84 CV04	7,756,963	445,722	
TD 84 CV06	7,756,946	445,666	
TD 84 CV07	7,756,953	445,703	
TD 84 CV08	7,756,927	445,622	
TD 84 CV09	7,756,922	445,642	
TD 84 CV10	7,756,923	445,680	
TD 84 CV11	7,756,928	445,722	
TD 84 CV13	7,756,910	445,663	
TD 84 CV14	7,756,924	445,703	
TD 84 CV15	7,756,886	445,622	
TD 84 CV16	7,756,889	445,643	
TD 84 CV17	7,756,890	445,682	
TD 84 CV18	7,756,888	445,720	
TD 84 CV20	7,756,867	445,665	
TD 84 CV21	7,756,873	445,708	
TD 84 CV24	7,756,855	445,677	
TD 84 CV25	7,756,847	445,721	



KEY PLAN

NOTES:

1. MOLIKPAQ CENTRE COORDINATES

   UTM (ZONE 8) 7,756,906.3 N. 445,661.6 E.

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 175°W LONG

3. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH DRAWING TD84003 SITE PLAN AND REGIONAL DATA

LEGEND

- CV INDICATES CONE PENETRATION TEST
- SV INDICATES DRILLED AND SAMPLED COREHOLE
- PV INDICATES SELF BORED PRESSUREMETER PROFILE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
85/02/25	1	AS BUILT DRAWING		
84/08/27	0	ISSUED FOR CONSTRUCTION		
84/08/15	0	ISSUED FOR COGLA		
84/07/20	0	ISSUED FOR COMMENTS		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

ACCEPTED FOR CONSTRUCTION

BEAUDRIL PROJECT MANAGER — DATE

	BY	DATE		BY	DATE
DESIGNED			CHECKED		
DRAWN	JLM		APPROVED		

GULF CANADA RESOURCES INC.
FRONTIER DEVELOPMENT

TITLE

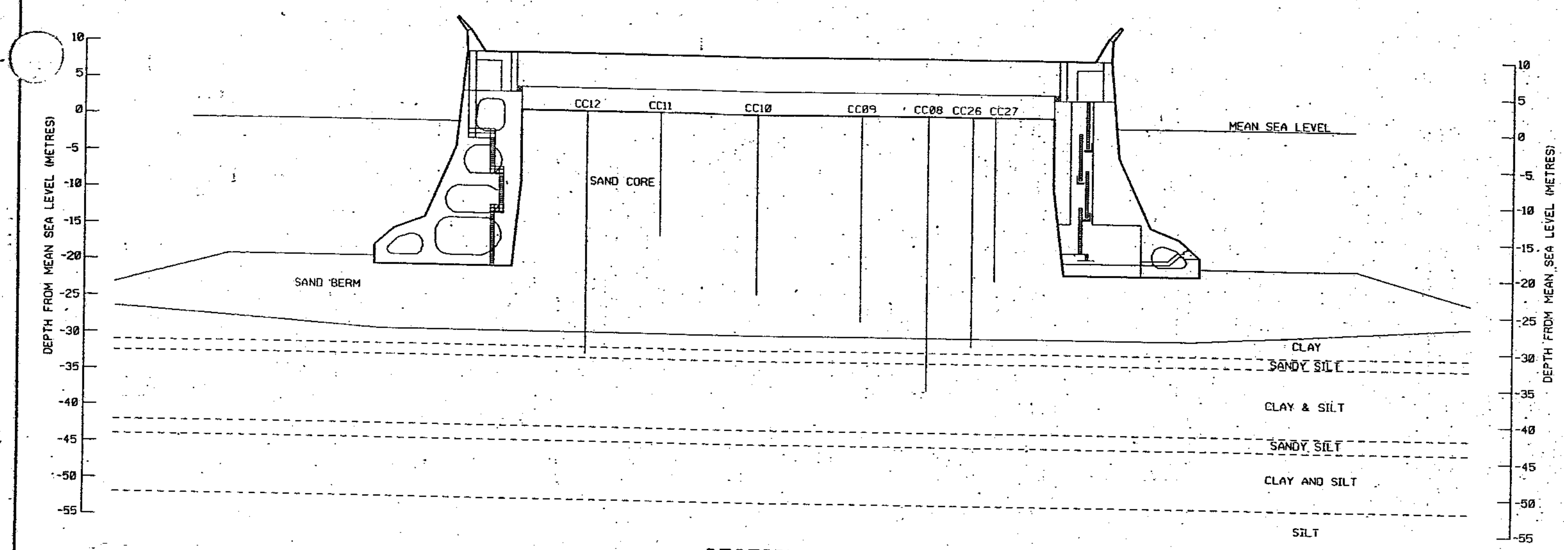
MOLIKPAQ
TARSIUT DELINEATION - 1984/85 SEASON

BERM VERIFICATION

Figure 3.6

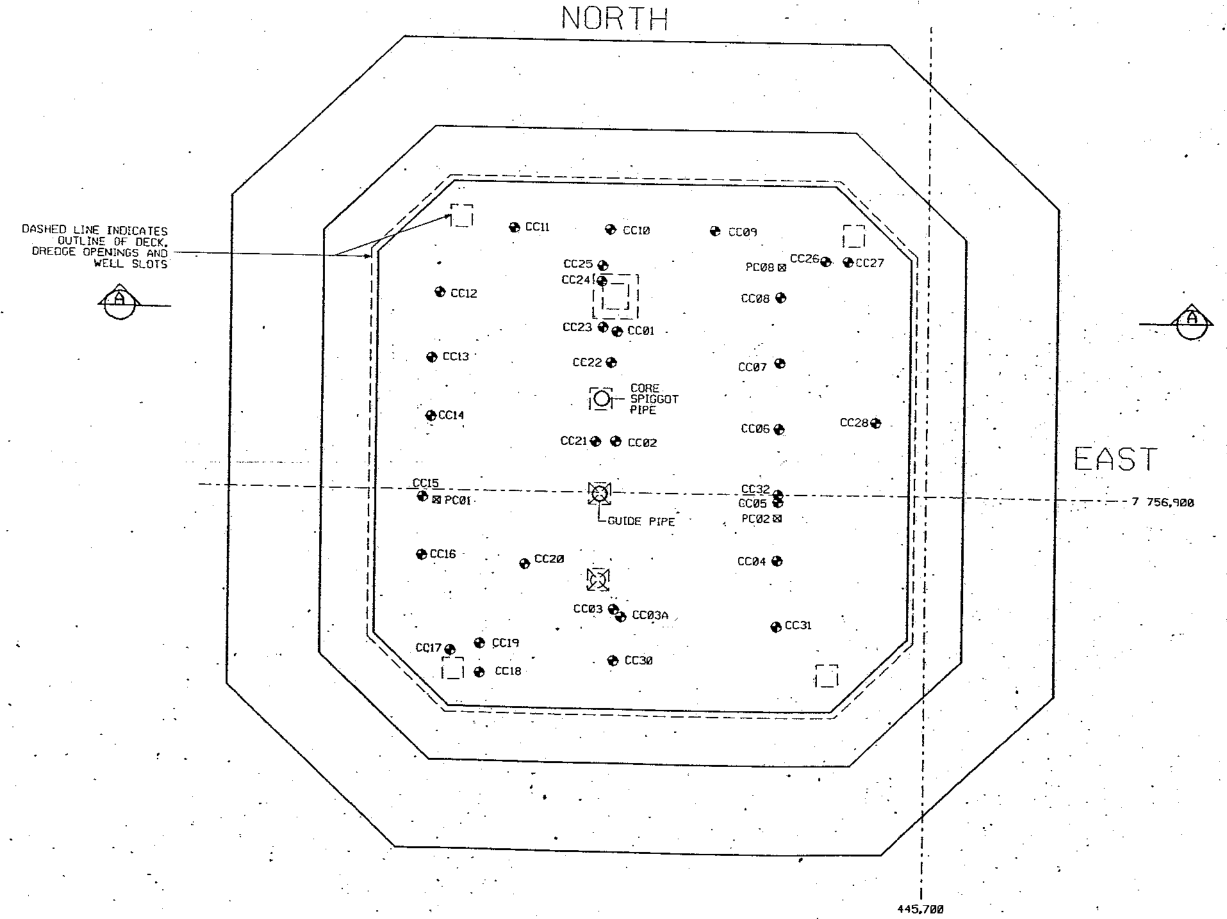
DATE: 84/06/26 — DRAWING No. TD-84-009

FILE NAME: OSA2:[163,24]TD84009.; 85/02/25 BCM



SECTION A-A

SCALE 1:500

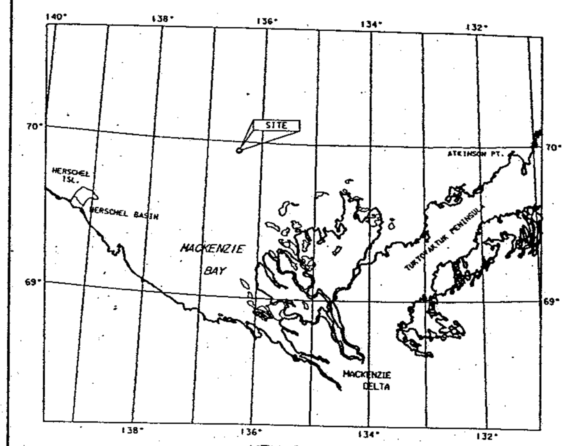


PLAN

SCALE 1:500

TABLE OF COORDINATES

CORE HOLE LABEL	UTM (ZONE 8) N (metres)	UTM (ZONE 8) E (metres)	SURFACE ELEVATION	MAXIMUM DEPTH BLEOW MSL	DEPTH TO CLAY BELOW MSL
TD 84 CC01	7 756 922	445 658	1.9	8.1	
TD 84 CC02	7 756 907	445 658	1.8	15.2	
TD 84 CC03	7 756 884	445 658	1.7	16.7	
TD 84 CC03A	7 756 883	445 659	1.7	17.5	
TD 84 CC04	7 756 891	445 680	1.7	18.7	
TD 84 CC05	7 756 899	445 680	1.6	31.2	28.9
TD 84 CC06	7 756 909	445 680	1.6	25	
TD 84 CC07	7 756 918	445 680	1.6	24.8	
TD 84 CC08	7 756 927	445 680	1.6	35.4	29.1
TD 84 CC09	7 756 936	445 671	1.5	25.5	
TD 84 CC10	7 756 936	445 657	1.7	22.3	
TD 84 CC11	7 756 936	445 644	1.7	14.3	
TD 84 CC12	7 756 927	445 634	1.6	31.4	29.2
TD 84 CC13	7 756 918	445 633	1.6	30.4	29.5
TD 84 CC14	7 756 910	445 633	1.5	24.5	
TD 84 CC15	7 756 899	445 632	1.6	24.4	
TD 84 CC16	7 756 891	445 632	1.7	24.3	
TD 84 CC17	7 756 878	445 636	1.9	31.1	29.8
TD 84 CC18	7 756 875	445 640	1.9	24.1	
TD 84 CC19	7 756 879	445 640	1.9	31.1	29.9
TD 84 CC20	7 756 890	445 646	1.6	22.4	
TD 84 CC21	7 756 907	445 656	1.7	19.3	
TD 84 CC22	7 756 917	745 654	1.9	16.1	
TD 84 CC23	7 756 922	445 656	1.9	20.1	
TD 84 CC24	7 756 929	445 656	1.9	14.1	
TD 84 CC25	7 756 931	445 656	1.8	20.2	
TD 84 CC26	7 756 932	445 686	1.7	30.3	28.9
TD 84 CC27	7 756 932	445 689	1.7	20.3	
TD 84 CC28	7 756 910	445 693	1.7	24.3	
TD 84 CC30	7 756 877	445 658	1.7	16.8	
TD 84 CC31	7 756 882	445 680	1.7	20.0	
TD 84 CC32	7 756 900	445 680	1.7	17.9	
TD 84 CC33	7 756 928	445 680	1.7	19.3	
TD 84 P01	7 756 899	445 634	1.6	8.6	
TD 84 P02	7 756 897	445 680	1.6	14.9	
TD 84 P03	7 756 930	445 680	1.6	14.9	



KEY PLAN

NOTES:

1. MOLIKPAQ CENTER COORDINATES
   UTM (ZONE 8) 7,756,906.3 N. 445,661.6 E.

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 175°W LONG

LEGEND

- CC INDICATES CONE PENETRATION TEST
- PC INDICATES SELF-BORING PRESSUREMETER PROFILE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
85/02/26	1	AS BUILT DRAWING	BTR	BTR
84/08/27	0	ISSUED FOR CONSTRUCTION		
84/08/15	0	ISSUED FOR COGLA		
84/07/20	0	ISSUED FOR COMMENTS		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

ACCEPTED FOR CONSTRUCTION

BEAUDRIL PROJECT MANAGER

DATE

	BY	DATE		BY	DATE
DESIGNED	BTR		CHECKED	HRS	
DRAWN	JLM		APPROVED	MGJ	

GULF CANADA RESOURCES INC.

FRONTIER DEVELOPMENT

TITLE

MOLIKPAQ

TARSIUT DELINEATION - 1984/85 SEASON

VERIFICATION AFTER CORE FILLING

Figure: 3.7

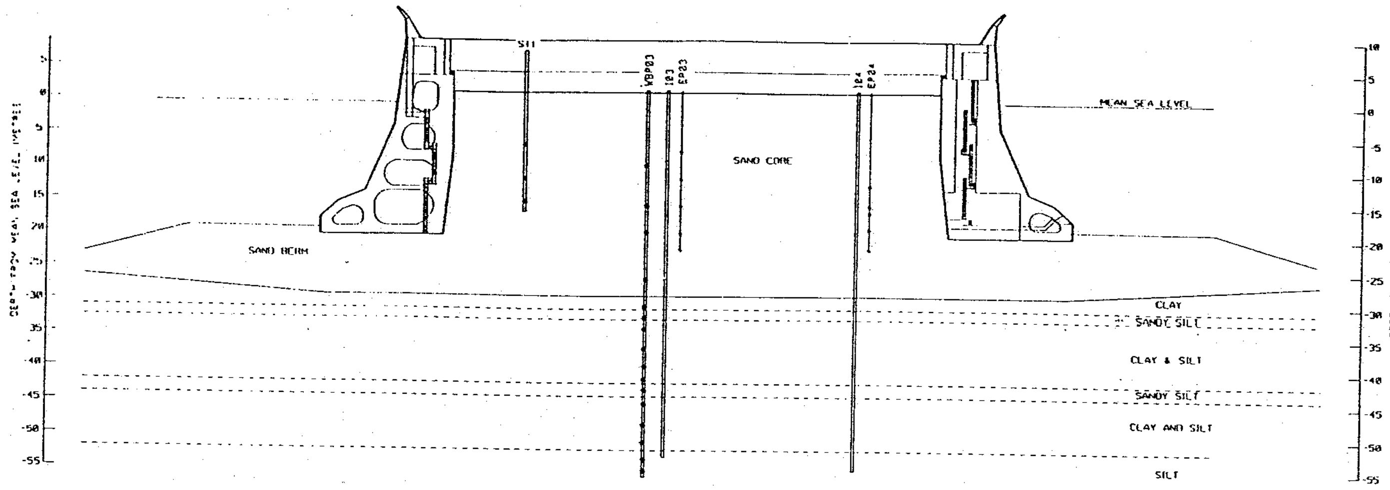
DATE: 84/07/17

DRAWING No. TD-84-013

PROJECT 84F012

FILE NAME 05A2(183,24)TD84013,1 85/04/24 RBC

SITE LOCATION PLAN
(1985 PROGRAM)



SECTION A-A

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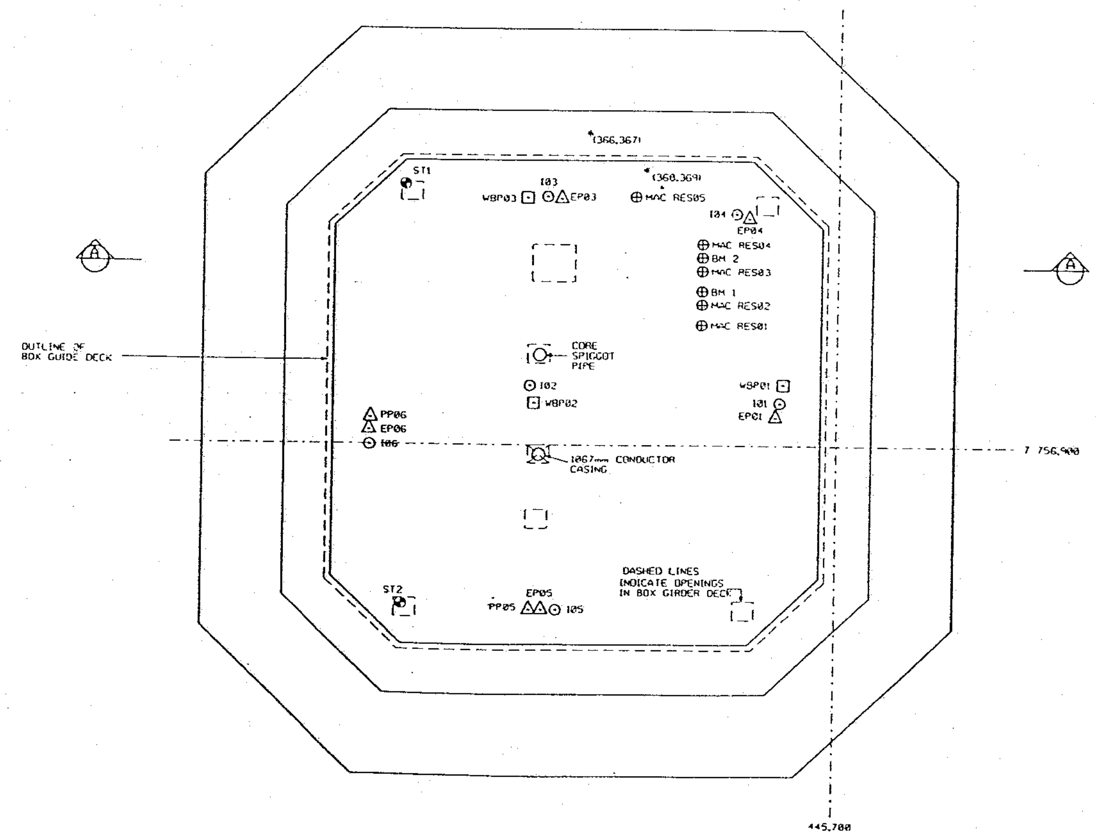


PLAN

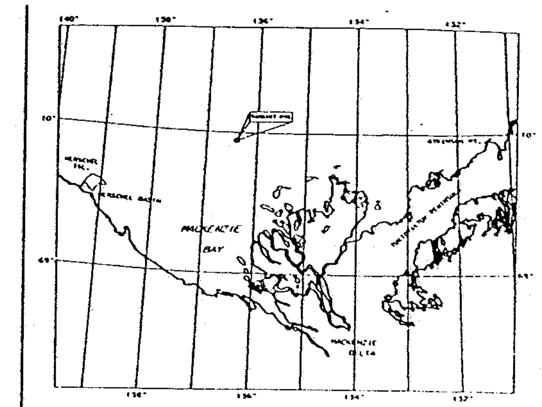
NORTH

TABLE OF COORDINATES

HOLE NO.	UTM (ZONE 8) COORDINATES N (metres)	E (metres)	ELEVATION ABOVE MSL TOP OF CASING	SAND SURFACE	DEPTH BELOW MSL
ST1	7 756 938	445 638	9	2	15.5
ST2	7 756 877	445 638	9	1.5	18.5
WBP01	7 756 909.5	445 692.5	2.03	1.7	54
101	7 756 907	445 692	2.24	1.7	55
EP01	7 756 905	445 691.5	1.96	1.7	22.5
102	7 756 909	445 654.5	1.84	1.7	55
WBP02	7 756 906.5	445 655	1.93	1.7	57
WBP03	7 756,937	445,654	2.03	1.9	55
103	7 756,937	445,657	2.42	1.9	53
EP03	7 756, 937	445,659	2.06	1.9	22
104	7 756, 934.5	445,685	1.94	1.7	55
EP04	7 756,934	445,687	1.88	1.7	22.5
PP05	7 756,876	445,654.5	1.82	1.6	44.5
EP05	7 756,876	445,656.5	1.98	1.6	21
105	7 756,876	445,658.5	1.67	1.6	55
PP06	7 756,904.5	445,631	1.97	1.7	46.5
EP06	7 756,902.5	445,631	1.97	1.7	19.9
106	7 756,900	445,631	1.99	1.7	53
MAC RES01	7 756,918.1	445,680	—	1.62	20
MAC RES02	7 756,921.3	445,680	—	1.56	25.5
MAC RES03	7 756,927.5	445,680	—	1.49	30.5
MAC RES04	7 756,930	445,680	—	1.48	28.5
MAC RES05	7 756,936	445,671	—	1.39	27.5
BH1	7 756,923.1	445,680	1.6	1.57	27.5
BH2	7 756,928.7	445,680	—	1.47	30.7

TABLE OF WESTBAY PIEZOMETER PORT LOCATIONS

BOREHOLE	PIEZOMETER COUPLING NUMBER	ELEVATION (DATUM-MSL)	BOREHOLE	PIEZOMETER COUPLING NUMBER	ELEVATION (DATUM-MSL)	BOREHOLE	PIEZOMETER COUPLING NUMBER	ELEVATION (DATUM-MSL)
WBP01	30	-13	WBP02	32	-13.6	WBP03	34	-11.5
	26	-19		28	-19.6		31	-16
	21	-26.5		22	-26.6		28	-19.5
	18	-30.5		19	-30.7		23	-26
	17	-32		18	-32.3		20	-30.5
	16	-33.5		17	-33.8		19	-32
	15	-35		16	-35.3		17	-34
	13	-37		14	-37.3		14	-36.5
	12	-38.5		13	-38.8		12	-39.5
	11	-40		12	-40.3		10	-41.5
	10	-41.5		11	-41.8		9	-43
	9	-43		10	-43.3		8	-44.5
	8	-44.5		8	-45.3		6	-47.5
	7	-46		6	-48.3		4	-50.5
	5	-48		4	-51.3		2	-52.5
	3	-50		3	-52.8		1	-53.9
	1,	-53		2	-54.3			
				1	-55.8			



KEY PLAN

NOTES:

1. MOLIKPAQ CENTER COORDINATES
   UTM (ZONE 8) 7,756,906.3 N. 445,661.6 E.
2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN GEODESIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG.
3. SOIL SAMPLES WERE EXTRACTED AT 3 METRE DEPTH INCREMENTS DURING INCLINOMETER INSTALLATION.
4. MEAN SEA LEVEL ASSUMED 19.5m ABOVE CAISSON BASE
5. TWO ACCELEROMETERS WERE INSTALLED 0.5 METERS BELOW SAND SURFACE NEAR NORTH CAISSON FACE;
   TWO ACCELEROMETERS WERE INSTALLED IN BASE OF NORTH PUMP ROOM.

LEGEND

- ST1 - STANDPIPE
- 101 - SINCO INCLINOMETER CASING
- WBP02 - WESTBAY MP CASING
- EP01 OR PP05 - SCHAEVITZ AND PETUR PIEZOMETER STRING
- PIEZOMETER LOCATIONS
- (366) - SCHAEVITZ ACCELEROMETER (CHANNEL NUMBER)
- JUNE 1985 RESEARCH PROJECT HOLES

DATE	REV.	ISSUED	CHK'D BY	APP'D BY
85/02/26	1	AS BUILT DRAWING	BTH	DTR
84/08/22	0	ISSUED FOR CONSTRUCTION		
84/08/15	0	ISSUED FOR COGLA		
84/07/20	0	ISSUED FOR COMMENTS		

DRAWING RECORD

GULF CANADA RESOURCES INC.
FRONTIER DEVELOPMENT

TITLE

MOLIKPAQ
TARSIUT DELINEATION - 1984/85 SEASON

GEOTECHNICAL INSTRUMENTATION AND RESEARCH PROJECT HOLES

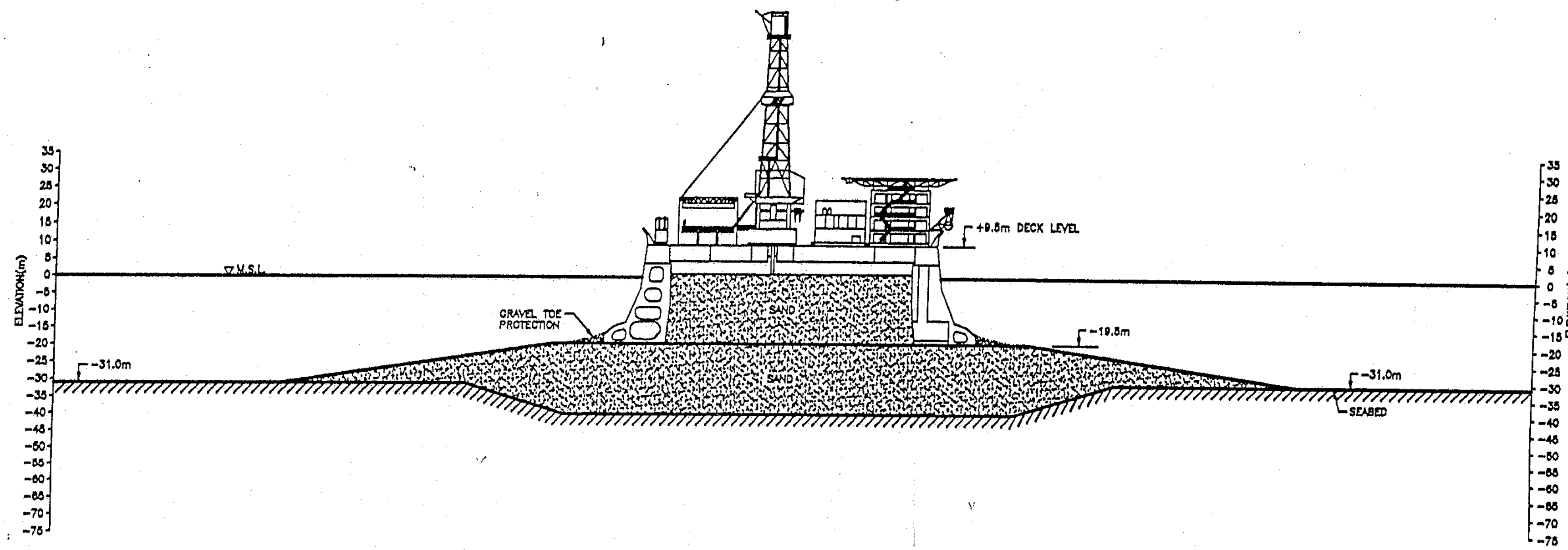
DATE: 85/06/25 DRAWING No. TD-84-014B

PROJECT 84F012

Golder Associates

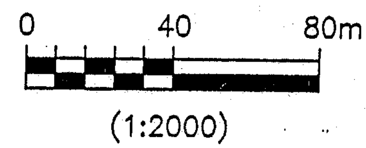
Drawn: ______
Reviewed: ______
Date: ______

Figure: 3.8



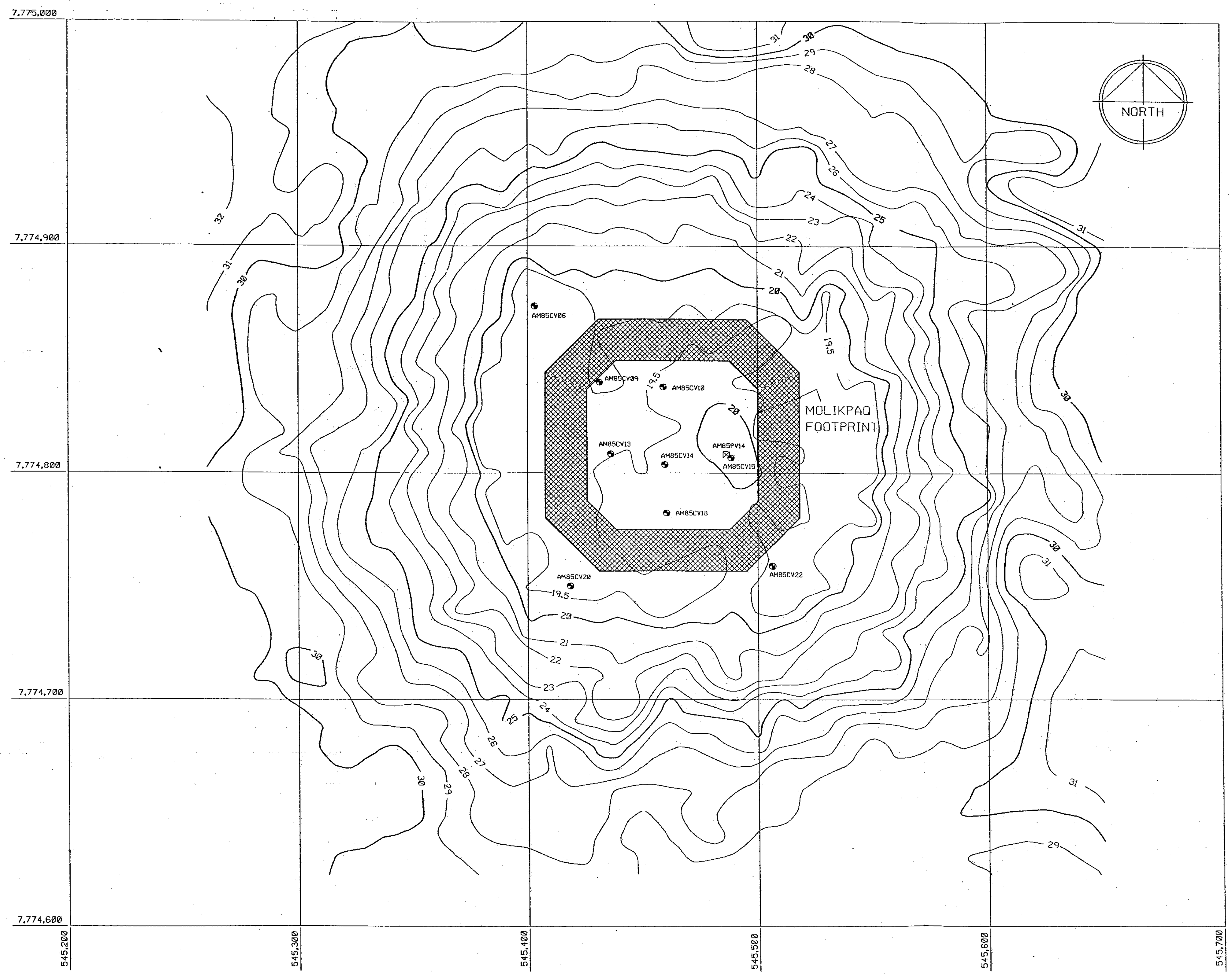
AMAULIGAK I–65 – MOLIKPAQ

OPERATOR: GULF



KLOHN-CRIPPEN

Figure 3.9 : Amauligak I-65 - Cross-Section

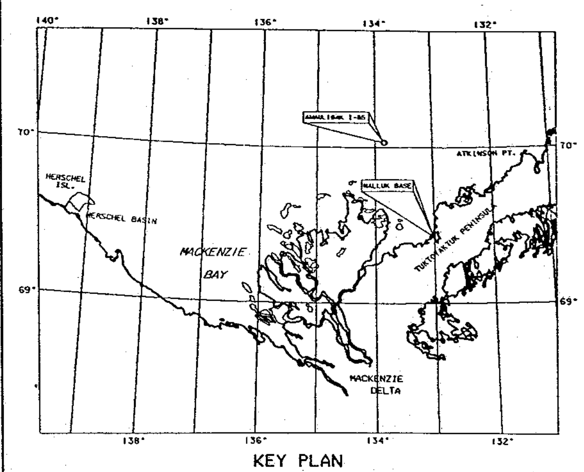


PLAN

SCALE (metres) 10 0 10 20 30 40 50
1:1000

TABLE OF VERIFICATION COREHOLES

CORE HOLE LABEL	UTM (ZONE 8) N (METERS)	UTM (ZONE 8) E (METERS)	BERM SURFACE ELEVATION (m)	DEPTH BELOW BERM SURFACE (m)
AM85CV06	7774876	545399	19.5	22.7
AM85CV09	7774842	545428	19.5	25.1
AM85CV10	7774840	545457	19.9	19.8
AM85CV13	7774811	545434	19.3	19.0
AM85CV14	7774806	545458	19.6	24.8
AM85CV15	7774809	545488	20.6	24.9
AM85CV18	7774785	545459	19.9	26.9
AM85CV20	7774752	545415	19.0	18.8
AM85CV22	7774761	545507	19.2	25.2
AM85PV14	7774810	545485	20.6	10.8



KEY PLAN

NOTES:

1. BERM CENTRE COORDINATES

   GEOGRAPHIC 70°04'39.8" N. 133°48'16.3" W.
   UTM (ZONE 8) 7,774,812 N. 545,462 E.

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG

3. BERM VERIFICATION CONDUCTED FROM THE A.KIGGIAK ON SEPT.18 AND 19,1985

LEGEND

CV INDICATES CONE PENETRATION TEST

PV INDICATES SELF BORED PRESSUREMETER PROFILE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
85/12/11	2	AS BUILT DRAWINGS	GPc	BR
85/06/25	1	ISSUED FOR CONSTRUCTION		
85/06/25		ISSUED TO COGLA		
85/05/02	0	ISSUED FOR COMMENTS		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY
1	85/12/11	ADD ACTUAL LOCATIONS	JLM		

ACCEPTED FOR CONSTRUCTION

BEAUDRIL PROJECT MANAGER — DATE

	BY	DATE		BY	DATE
DESIGNED	Gary Carbone	85/04/25	CHECKED	Brian Rogers	24 Feb 86
DRAWN	VRD		APPROVED	Brian Rogers	24 Feb 86

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

MOLIKPAQ
AMAULIGAK I-65 - AS BUILT

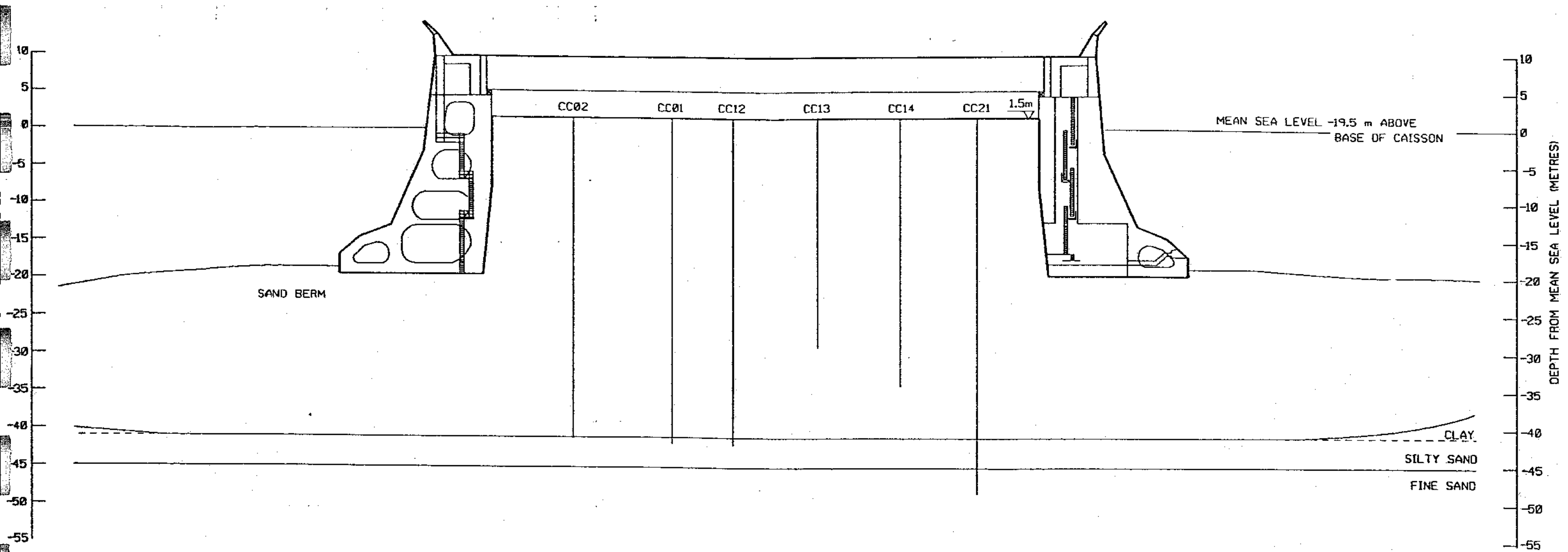
BERM VERIFICATION

Figure: 3.10

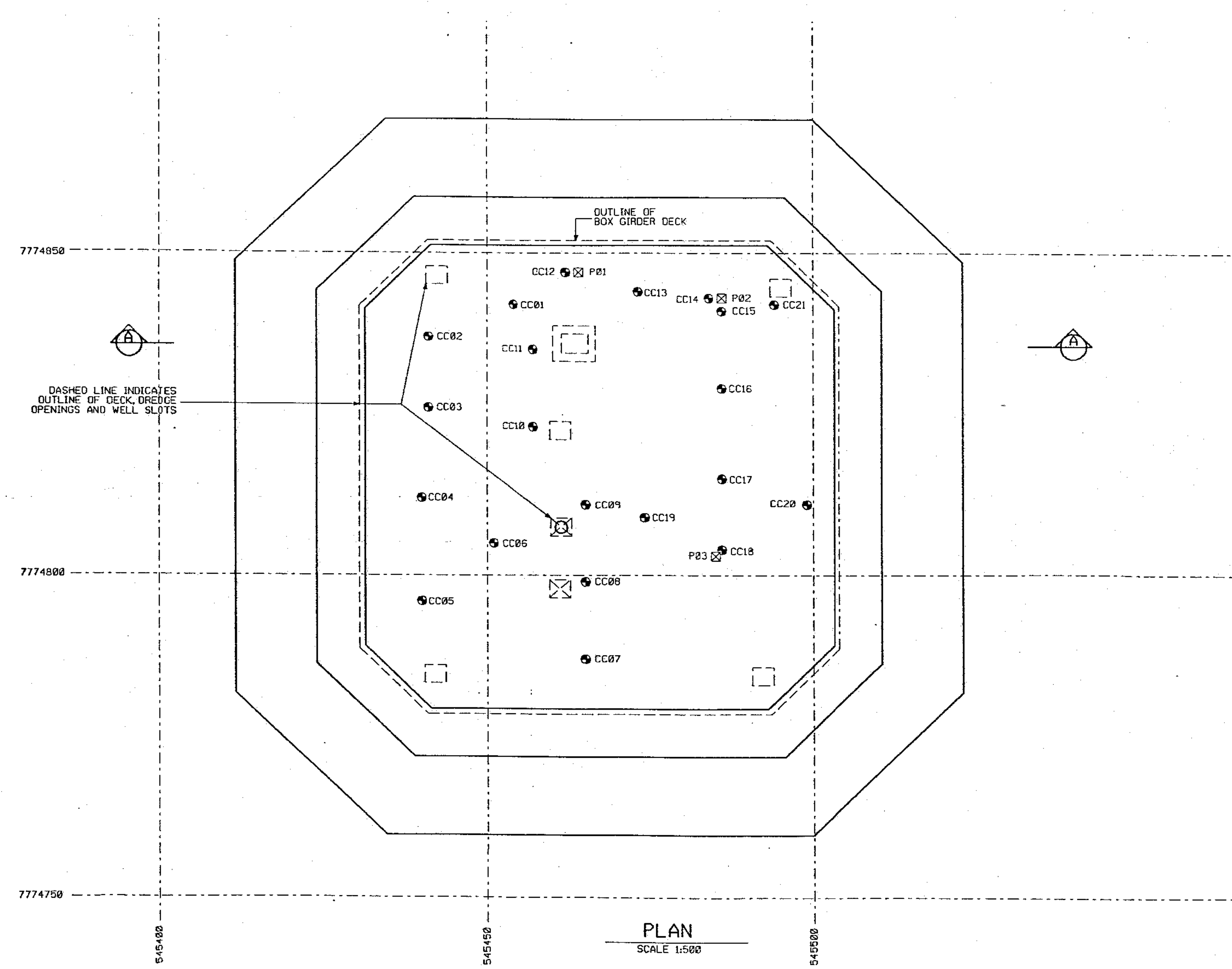
DATE: 85/12/11 | DRAWING No. AM-85-010

PROJECT 84F012

FILE NAME: QSA2:[163,24]AD8501.0;1 86/01/22 JLM



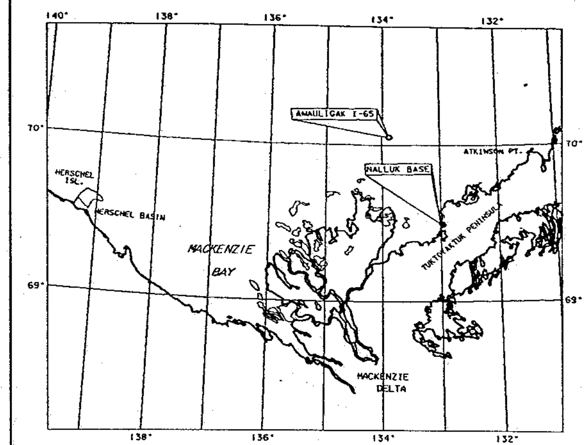
SECTION A-A
SCALE 1:500



PLAN
SCALE 1:500

TABLE OF COORDINATES

INSTRUMENT LABEL	UTM (ZONE 8)COORDINATES N (metres)	E (metres)	SURFACE ELEVATION	MAXIMUM DEPTH BELOW MSL
CC01	7774842	545454	+1.8	41.9
CC02	7774837	545441	+1.8	41.2
CC03	7774826	545441	+1.9	25.04
CC04	7774812	545440	+1.9	24.8
CC05	7774796	545440	+1.6	42.15
CC06	7774805	545451	+1.6	25.3
CC07	7774787	545465	+1.6	42.2
CC08	7774799	545465	+1.6	29.1
CC09	7774811	545465	+1.6	42.1
CC10	7774823	545457	+1.7	39.0
CC11	7774835	545457	+1.7	29.0
CC12	7774847	545462	+1.7	42.0
CC13	7774844	545473	+1.7	29.0
CC14	7774843	545484	+1.6	34.1
CC15	7774841	545486	+1.6	11.4
CC16	7774829	545486	+1.4	29.3
CC17	7774815	545486	+1.5	43.1
CC18	7774804	545486	+1.5	48.5
CC19	7774809	545474	+1.3	46.9
CC20	7774811	545499	+1.5	38.5
CC21	7774842	545494	+1.5	48.4
P01	7774847	545464	+1.7	27.6
P02	7774843	545486	+1.6	13.6
P03	7774803	545485	+1.5	14.7



KEY PLAN

NOTES:

1. MOLIKPAQ CENTER COORDINATES

UTM (ZONE 8): 7774815N 545467E

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG

3. DRAFT OF MOLIPAQ AT MEAN SEA LEVEL TAKEN AS 19.5 METRES.

LEGEND

CC INDICATES CONE PENETRATION TEST

PV INDICATES SELF BORED PRESSUREMETER PROFILE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
85/11/25	2	AS BUILT DRAWING	GPC	
85/06/25	1	ISSUED FOR CONSTRUCTION		
85/06/25		ISSUED TO COGLA		
85/05/03	0	ISSUED FOR COMMENTS		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY
1	85/11/25	ADD ACTUAL LOCATIONS			

ACCEPTED FOR CONSTRUCTION

BEAUDRIL PROJECT MANAGER

DATE

	BY	DATE		BY	DATE
DESIGNED		24/2/86	CHECKED	Brian Rogers	24 Feb 86
DRAWN	JLM		APPROVED	Brian Rogers	26 Feb 86

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

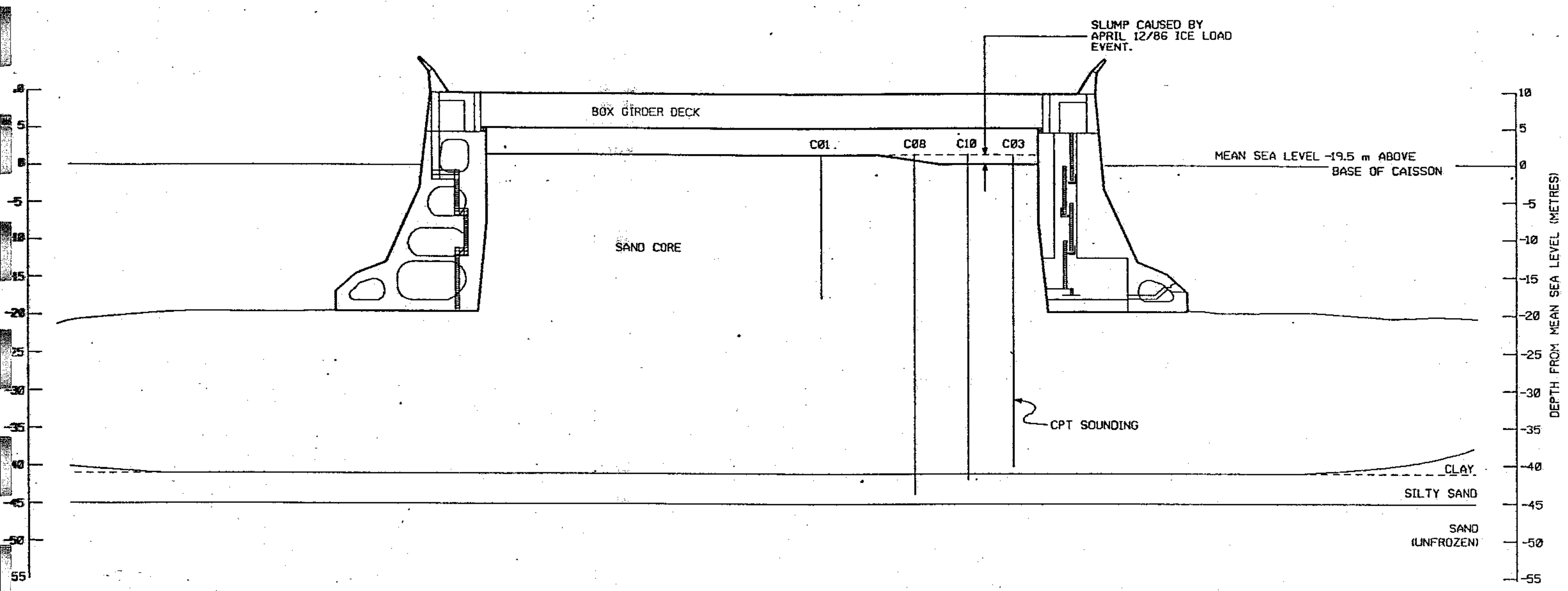
MOLIKPAQ
AMAULIGAK I-65 - AS BUILT

VERIFICATION AFTER CORE FILLING

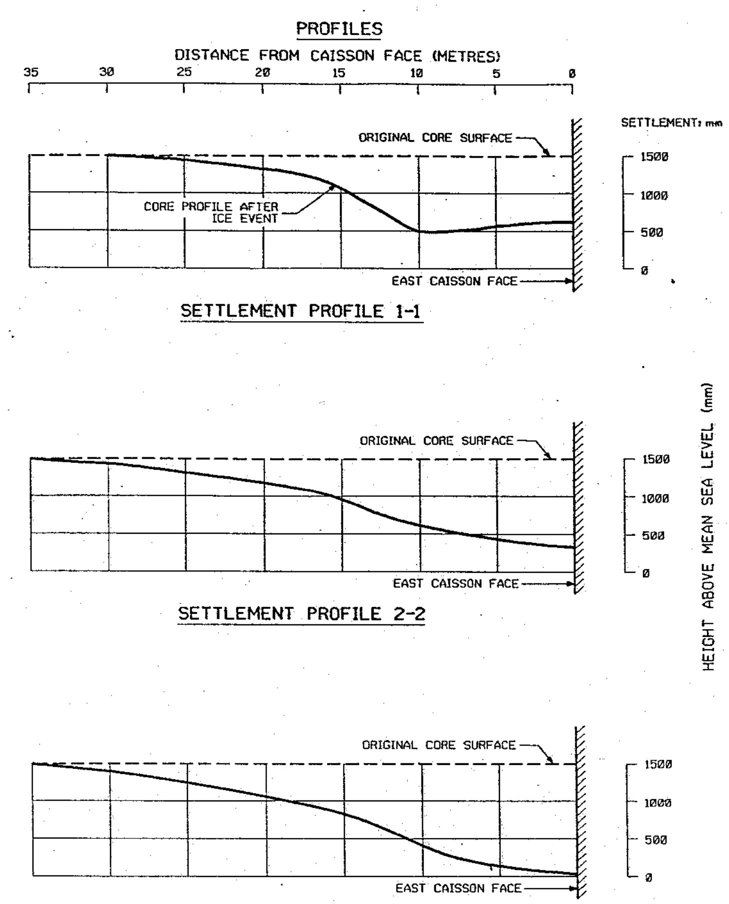
Figure: 3.11

DATE: 85/11/25 DRAWING No. AM-85-012

PROJECT 84F012



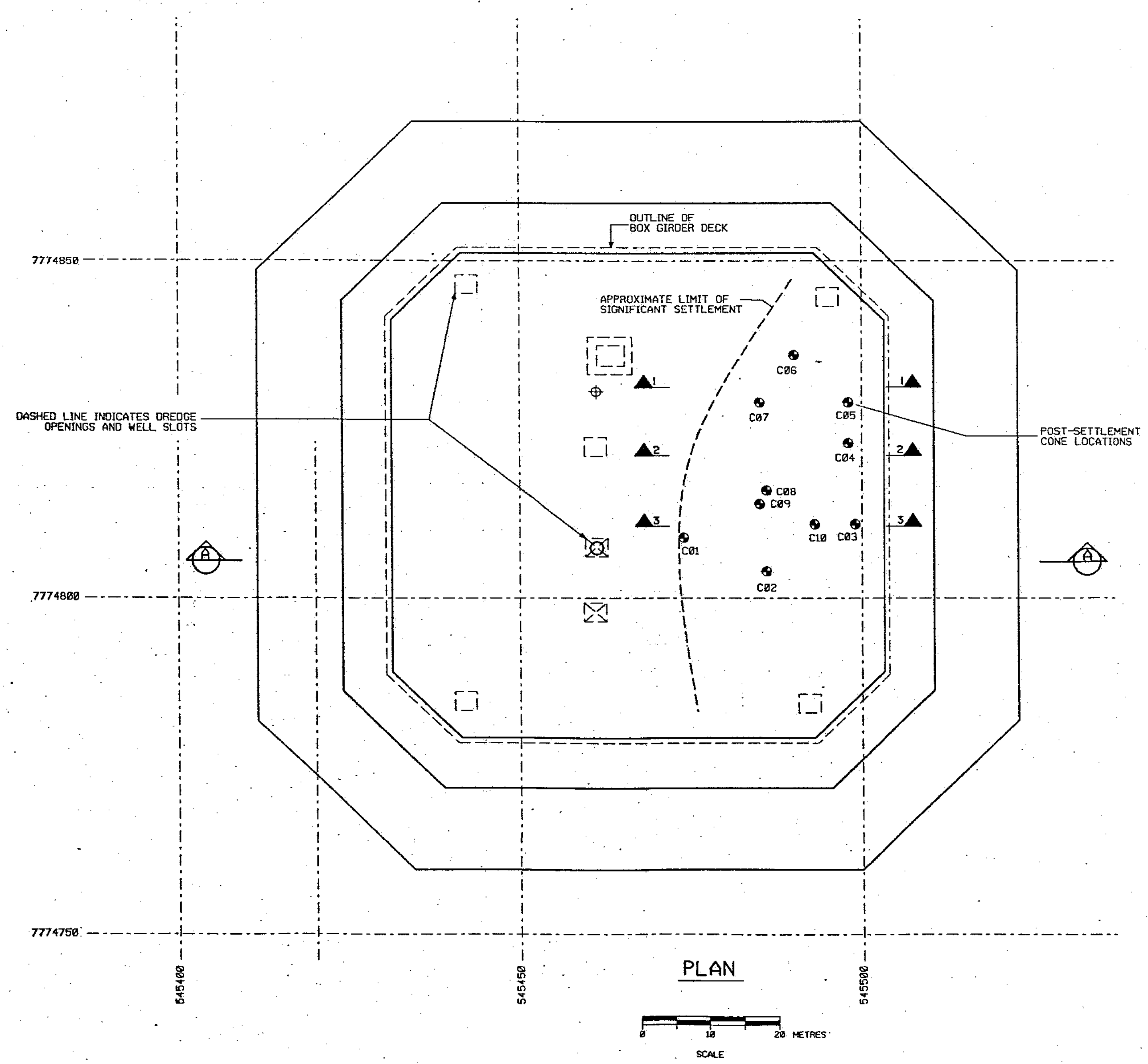
SECTION A-A



SETTLEMENT PROFILE 1-1

SETTLEMENT PROFILE 2-2

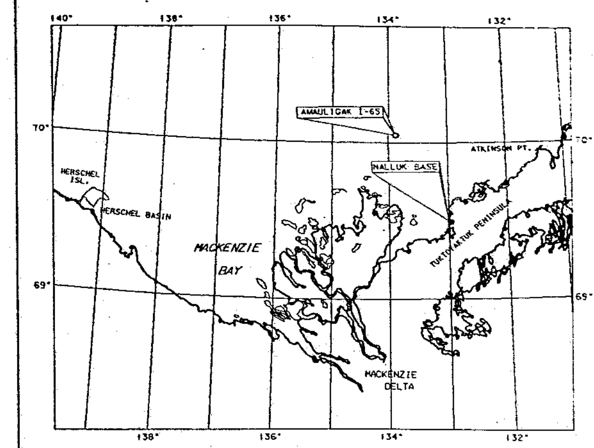
SETTLEMENT PROFILE 3-3



PLAN

TABLE OF COORDINATES & ELEVATIONS

TEST NUMBER	UTM (ZONE 8) COORDINATES N (metres)	E (metres)	SURFACE ELEVATION BEFORE SLUMP	AFTER SLUMP	MAXIMUM DEPTH OF SOUNDING	REMARKS	PRE-SLUMP CPT FOR REFERENCE
AM86C01	7 774 809	545 474	+1.30	+1.12	17.8	COMPUTER POWER SUPPLY FAILURE @18m.	AM85CC19
AM86C02	7 774 804	545 486	+1.50	+0.93	22.0		AM85CC18
AM86C03	7 774 811	545 499	+1.50	+0.02	40.2	REFUSAL	AM85CC20
AM86C04	7 774 823	545 498	+1.5 (est)	+0.46	27.0		-
AM86C05	7 774 829	545 498	+1.5 (est)	+0.50	21.7	TRANSDUCERS OVER-STRESSED; DATA SHOULD BE DISREGARDED.	-
AM86C06	7 774 840	545 486	+1.60	+1.24	25.6		AM85CC15
AM86C07	7 774 829	545 485	+1.40	+1.11	22.8	LATERAL STRESS CHANNEL NOT WORKING	AM85CC16
AM86C08	7 774 816	545 486	+1.50	+0.77	43.8	LATERAL STRESS ZERO SHIFTED @23m	AM85CC17
AM86C09	7 774 814	545 485	+1.50	+0.75	20.9	LATERAL STRESS SENSOR IN CENTRAL LOCATION	AM85CC17
AM86C10	7 774 811	545 493	+1.5 (est)	+0.34	41.9	REFUSAL	-



KEY PLAN

NOTES:

1. MOLIKPAQ CENTER COORDINATES
   GEOGRAPHIC: 70°04'39"N 133°48'16.3"W
   UTM (ZONE 8): 7774812N 545462E
2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG
3. MEAN SEA LEVEL TAKEN AS 19.5 METRES ABOVE CAISSON BASE.
4. CORE SURFACE ELEVATIONS ARE MEASURED WITH REGARD TO MEAN SEA LEVEL.

LEGEND

● INDICATES CONE PENETRATION TEST

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

	BY	DATE		BY	DATE
DESIGNED	MGJ	86/10/20	CHECKED	BTR	86/10/22
DRAWN	VRD	86/10/20	APPROVED	BTR	86/10/20

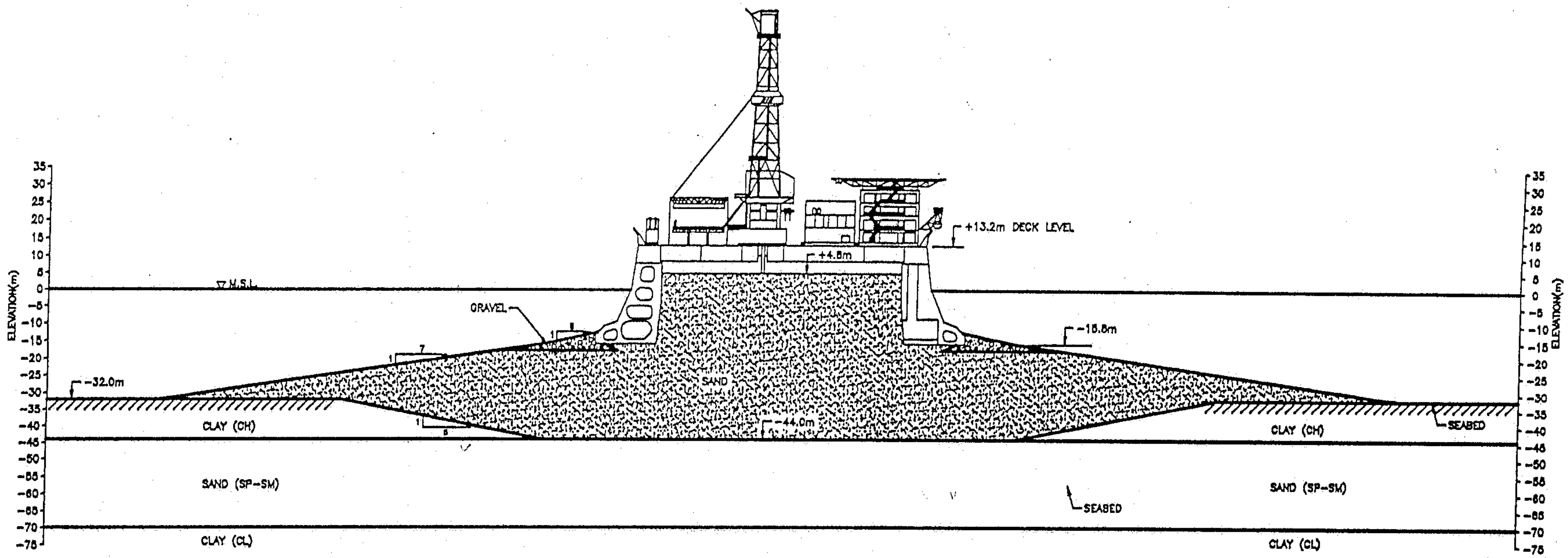
GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

MOLIKPAQ
AMAULIGAK I-65 AS-BUILT
CORE DENSITY VERIFICATION AFTER
APRIL 12,1986 ICE LOAD EVENT

Figure: 3.12

DATE: 86/05/07 DRAWING No. AM-85-017



AMAULIGAK F–24 – MOLIKPAQ

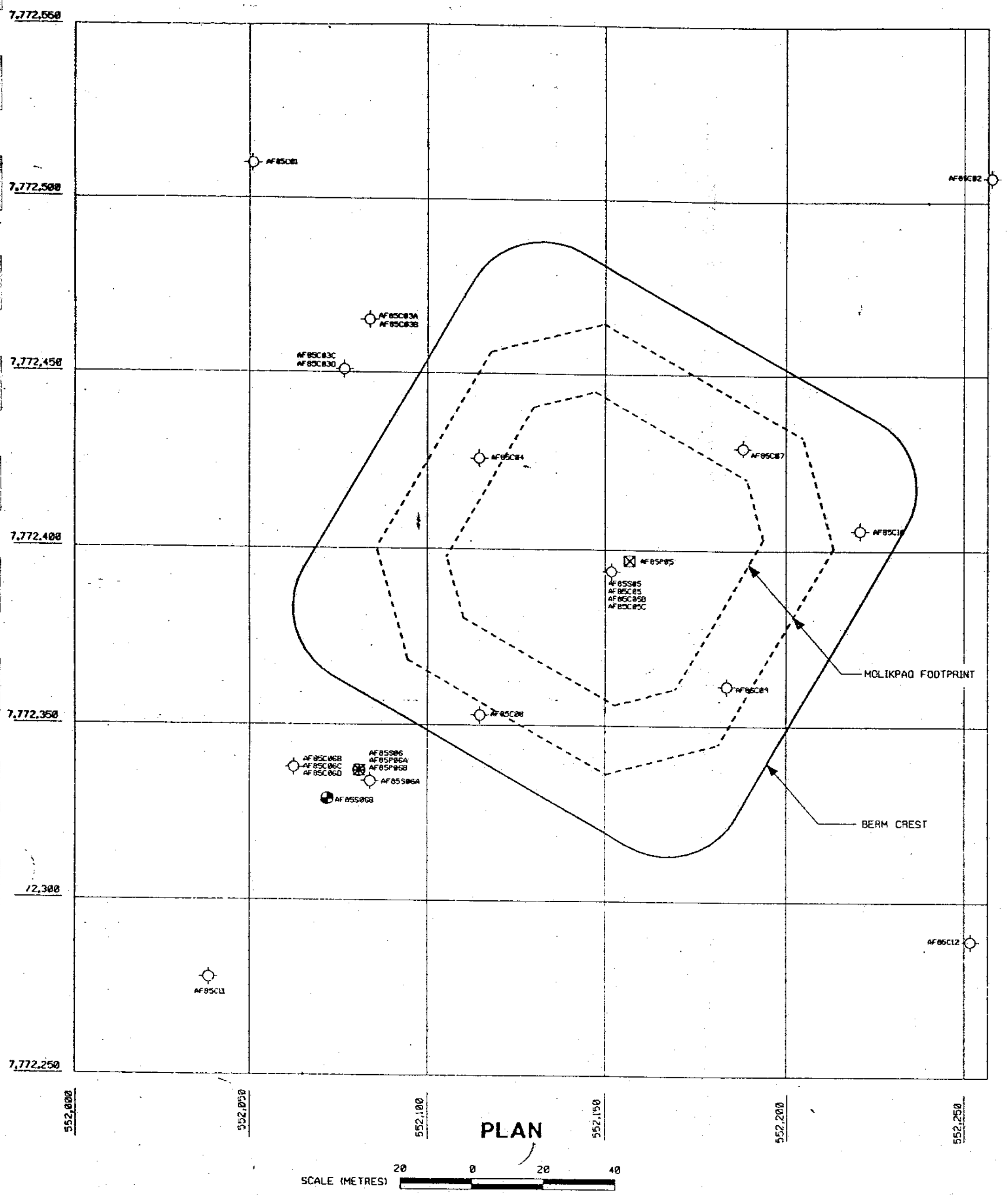
OPERATOR: GULF

0 40 80m

(1:2000)

KLOHN-CRIPPEN

Figure 3.13: Amauliguk F-24 - Cross-Section



PLAN

SCALE (METRES) 20 0 20 40

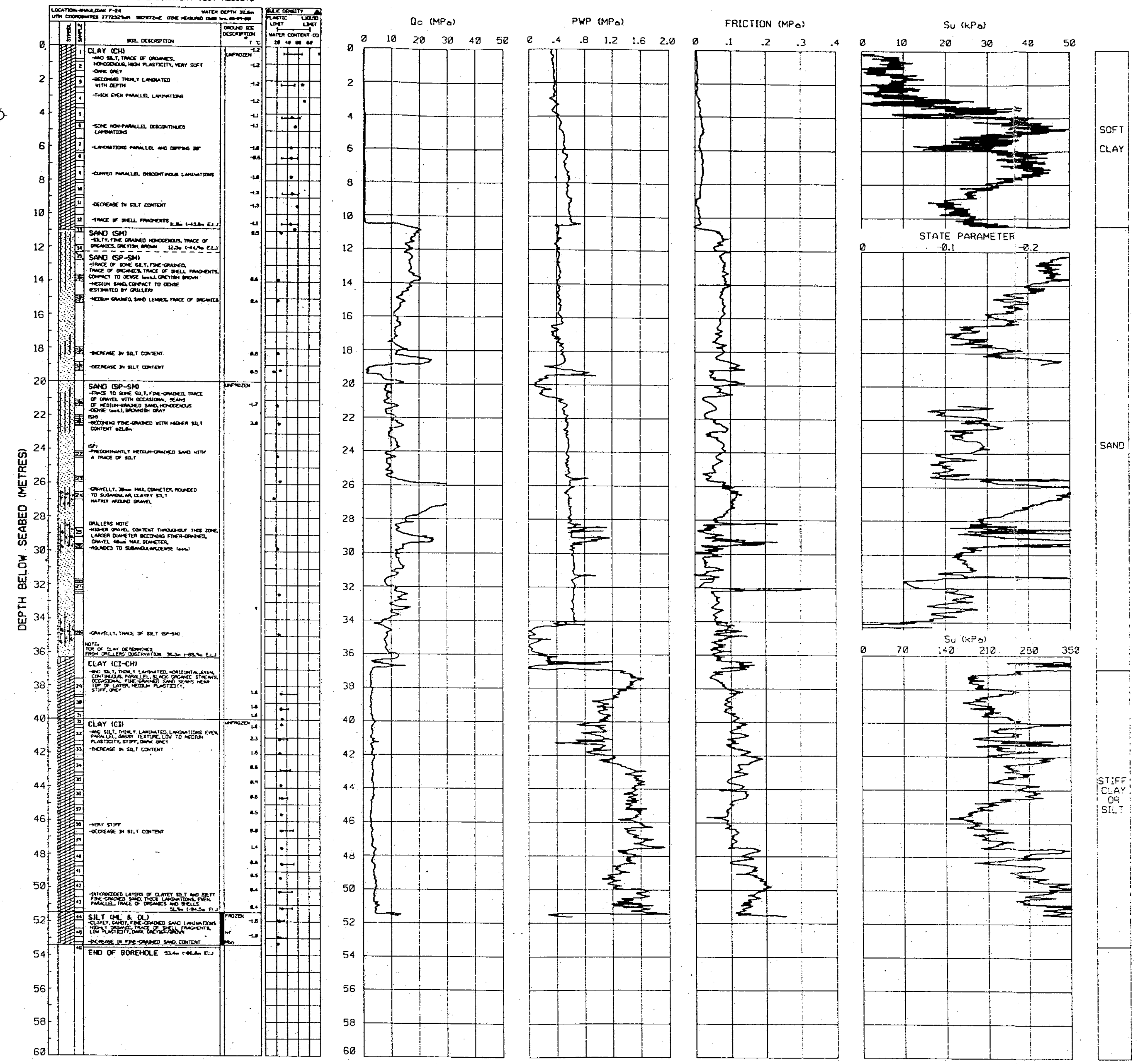
TABLE OF COREHOLES

BOREHOLE OR PROBEHOLE	UTM COORDINATES (ZONE 8)	GEOGRAPHIC COORDINATES LATITUDE LONGITUDE	DATE (COMPLETED)	SEABED PENETRATION (METRES)
AF85S05	N7772394 E552152	70°03'17.26" 133°37'48.40"	85/09/04	53.0 - 68.6
AF85S06	N7772337 E552081	70°03'15.44" 133°37'55.16"	85/09/07	38.3 - 41.6
AF85S06B	N7772329 E552072	70°03'15.44" 133°38'55.16"	85/09/09	0.0 - 53.4
AF85C05	N7772394 E552152	70°03'17.26" 133°37'48.40"	85/09/03	0.0 - 12.2
AF85C05B	N7772394 E552152	70°03'17.26" 133°27'48.40"	85/09/03	10.9 - 32.0
AF85C05C	N7772394 E552152	70°03'17.26" 133°27'48.40"	85/09/03	32.0 - 52.8
AF85C03A	N7772465 E552084	70°03'19.59" 133°37'54.62"	85/09/04	0.0 - 13.2
AF85C03B	N7772465 E552084	70°03'19.59" 133°37'54.62"	85/09/04	13.2 - 37.2
AF85C03C	N7772451 E552077	70°03'19.15" 133°37'55.35"	85/09/06	34.1 - 52.7
AF85C03D	N7772451 E552077	70°03'19.15" 133°37'55.35"	85/09/06	0.0 - 12.3
AF85C06A	N7772334 E552084	70°03'15.34" 133°37'54.90"	85/09/06	0.0 - 12.2
AF85C06B	N7772338 E552063	70°03'15.51" 133°37'56.93"	85/09/08	10.0 - 28.4
AF85C06C	N7772338 E552063	70°03'15.51" 133°37'56.93"	85/09/09	29.4 - 39.5
AF85C06D	N7772338 E552063	70°03'15.51" 133°37'56.93"	85/09/08	39.5 - 53.3
AF86C09	N7772361 E552184	70°03'16.14" 133°37'45.37"	85/09/10	0.0 - 12.8
AF85C07	N7772429 E552189	70°03'18.35" 133°37'44.77"	85/09/10	0.0 - 11.2
F85C12	N7772289 E552252	70°03'13.77" 133°37'39.16"	85/09/10	0.0 - 12.9
F85C11	N7772278 E552038	70°03'13.59" 133°37'59.35"	85/09/10	0.0 - 11.5
AF85C08	N7772353 E552115	70°03'15.96" 133°37'51.96"	85/09/10	0.0 - 12.2
AF85C04	N7772426 E552115	70°03'18.30" 133°37'51.74"	85/09/10	0.0 - 12.1
AF85C02	N7772507 E552258	70°03'20.80" 133°37'39.11"	85/09/11	0.0 - 14.0
AF85C01	N7772510 E552051	70°03'21.07" 133°37'57.68"	85/09/11	0.0 - 13.3
AF85C10	N7772406 E552221	70°03'17.57" 133°37'41.84"	85/09/11	0.0 - 53.4
AF85P06A	N7772337 E552061	70°03'15.44" 133°37'55.16"	85/09/07	0.0 - 11.0
AF85P06B	N7772337 E552081	70°03'15.44" 133°37'55.16"	85/09/07	42.0 - 51.8
AF85P05	N7772397 E552157	70°03'17.32" 133°37'47.92"	85/09/11	15.6 - 30.8

FILE NAME: ZF1:[163.53]AF88008.;1 88/06/16 WHL

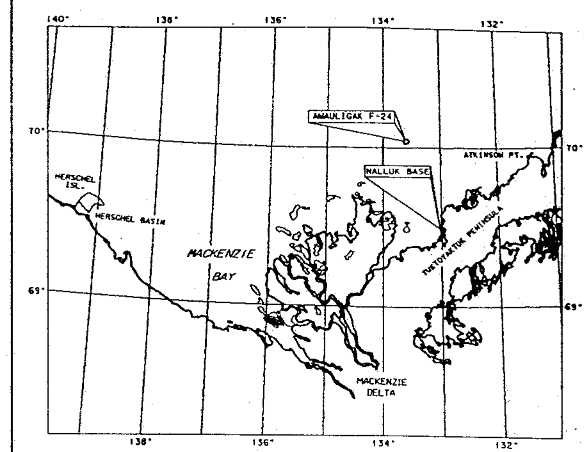
AF85S06B
BOREHOLE LOG AND LABORATORY TEST RESULTS

LOCATION: AMAULIGAK F-24
WATER DEPTH (m): 32.6



DRILLING COMPLETED: 85/09/09
TERMINATION DEPTH: 86.0m
DRILLING RIG: SIMCO 5000 (KIGGIAK)
LOG COMPILED: TRM/MDW
BOREHOLE NO.: AF85S06B

SOIL SYMBOLS
SAND
SILT
CLAY



KEY PLAN

NOTES:

1. MOLIKPAQ CENTRE COORDINATES

   GEOGRAPHIC 70°03'17.5"N 133°37'48.2"W
   UTM (ZONE 8) 7 772 401 N; 552 154 E

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG

3. COMPLETE GEOTECHNICAL DATA FOR EXISTING AF85S06B BOREHOLE PRESENTED IN EBA REPORT TO GCR DATED NOV. 1985.

4. SEE ALSO DWG. AF-87-012 FOR STRATIGRAPHIC SECTIONS

INDICATES CONE PENETRATION TEST
INDICATES DRILLED AND SAMPLED COREHOLE
INDICATES SELF-BORED PRESSUREMETER TEST

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
88/03/01	0	AS-BUILT		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

	BY	DATE		BY	DATE
DESIGNED	GO'N	86/08/03	CHECKED	BYR	88/06/01
DRAWN	JLM	86/08/03	APPROVED	BTR	88/06/01

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

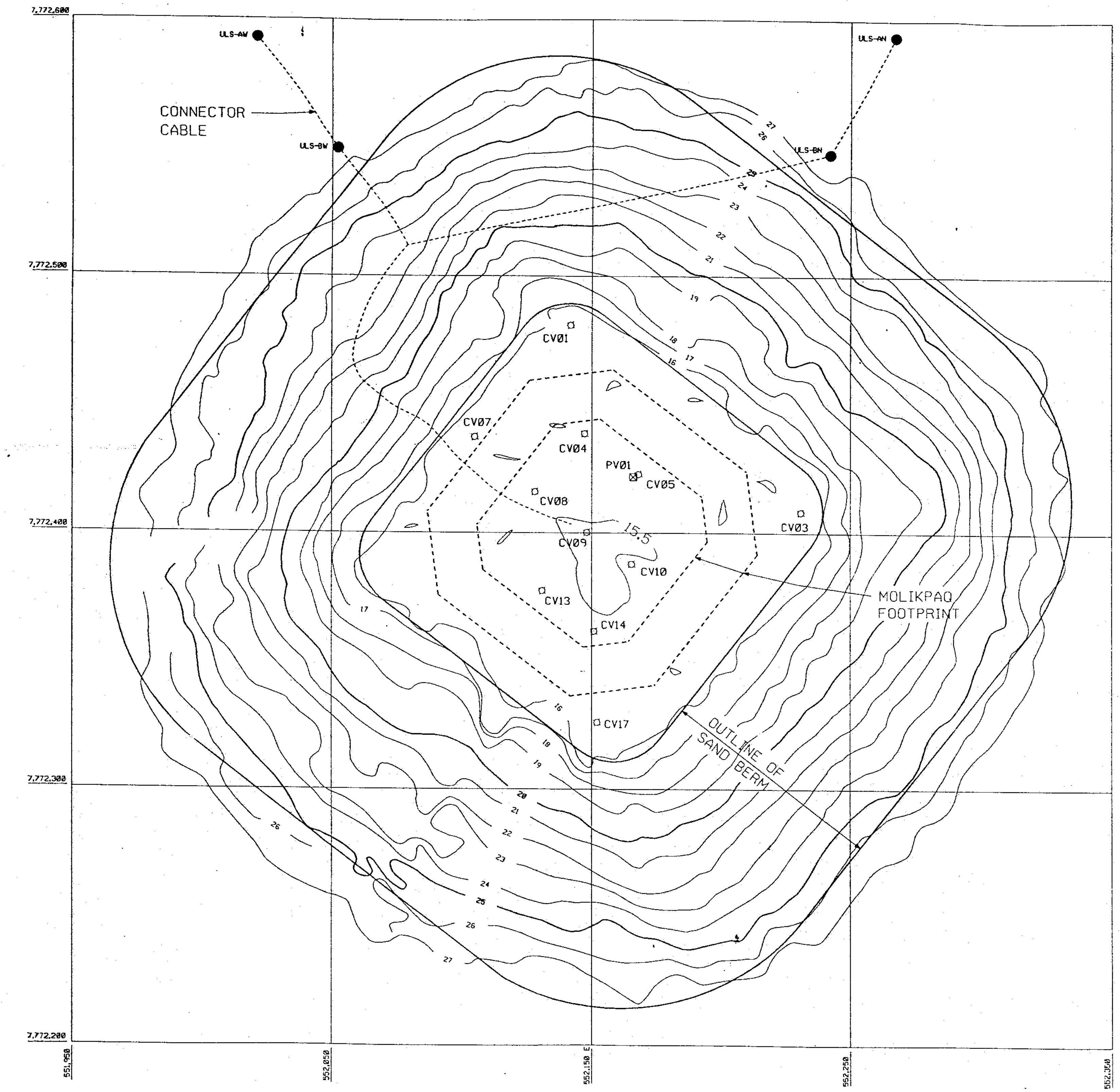
TITLE

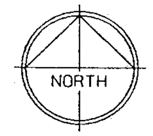
MOLIKPAQ AT AMAULIGAK F-24
AS-BUILT DRAWINGS

BERM SITE INVESTIGATION

Figure: 3.14

DATE: 88/03/01 DRAWING No. AF-88-008





CPT SITES: BERM VERIFICATION

SCALE (metres) 10 0 10 20 30 40 50

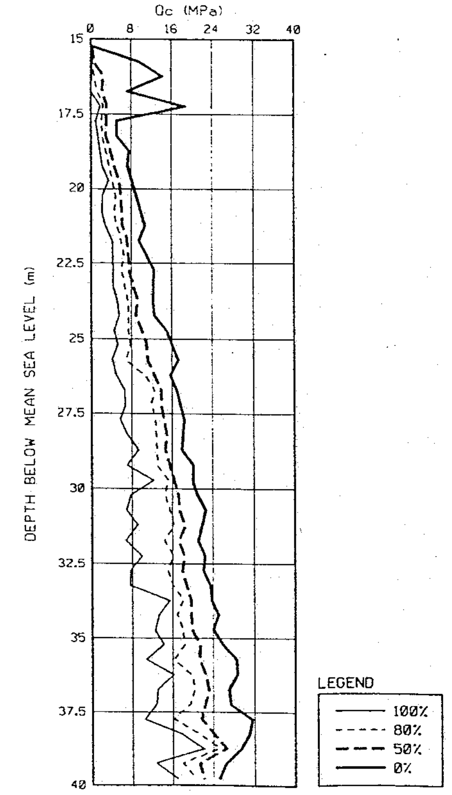
BERM VERIFICATION TABLE OF COORDINATES

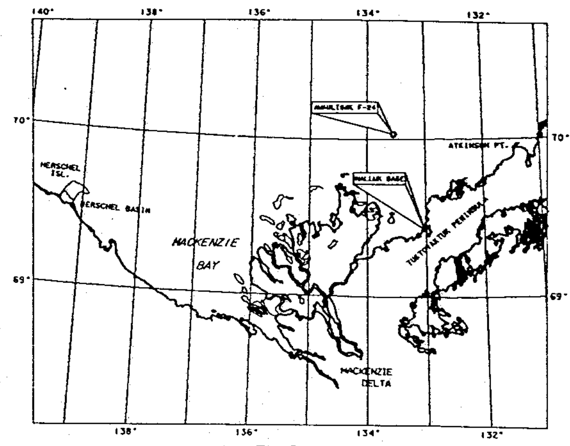
INSTRUMENT LABEL	NORTHING N (METERS)	EASTING (E METERS)	HOLE DEPTH (METERS)
AF87CV01	7 772 479	552 131	20.0
AF87CV03	7 772 419	552 229	23.7
AF87CV04	7 772 438	552 142	23.8
AF87CV05	7 772 425	552 165	30.8
AF87CV07	7 772 431	552 100	20.0
AF87CV08	7 772 413	552 126	19.0
AF87CV09	7 772 400	552 148	20.0
AF87CV10	7 772 390	552 167	20.2
AF87CV13	7 772 375	552 134	22.0
AF87CV14	7 772 362	552 156	19.5
AF87CV17	7 772 327	552 162	20.3
AF87PV01	7 772 424	552 163	24.0

DEPLOYED SONAR LOCATIONS AMAULIGAK F-24

LOCATION	NORTHING (N METERS)	EASTING (E METERS)
ULS - AW	7 772 593	552 021
ULS - BW	7 772 550	552 052
ULS - AN	7 772 594	552 267
ULS - BN	7 772 548	552 242

SUMMARY OF TIP RESISTANCE





KEY PLAN

NOTES:

1. BERM CENTRE COORDINATES

   GEOGRAPHIC 70°03'17.4" N. 133°37'48.5" W.
   UTM (ZONE 8) 7,772,400 N. 552,150 E.

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG

3. THE BERM VERIFICATION CARRIED OUT UPON COMPLETION OF CONSTRUCTION PRIOR TO SET DOWN.

4. CONTOURS FROM CHALLENGER SURVEY NO. 53, POST LEVELLING

LEGEND

- ◇ CV INDICATES CONE PENETRATION TEST
- ⊠ PV INDICATES SELF BORED PRESSUREMETER PROFILE
- ● ULS DENOTES UPWARD LOOKING SONAR
- AW DENOTES A UNIT, WEST SIDE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
88/03/01	0	AS-BUILT		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

	BY	DATE		BY	DATE
DESIGNED	MVH	87/07/07	CHECKED	BTR	88/06/01
DRAWN	VRD	86/08/23	APPROVED	BTR	88/06/01

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

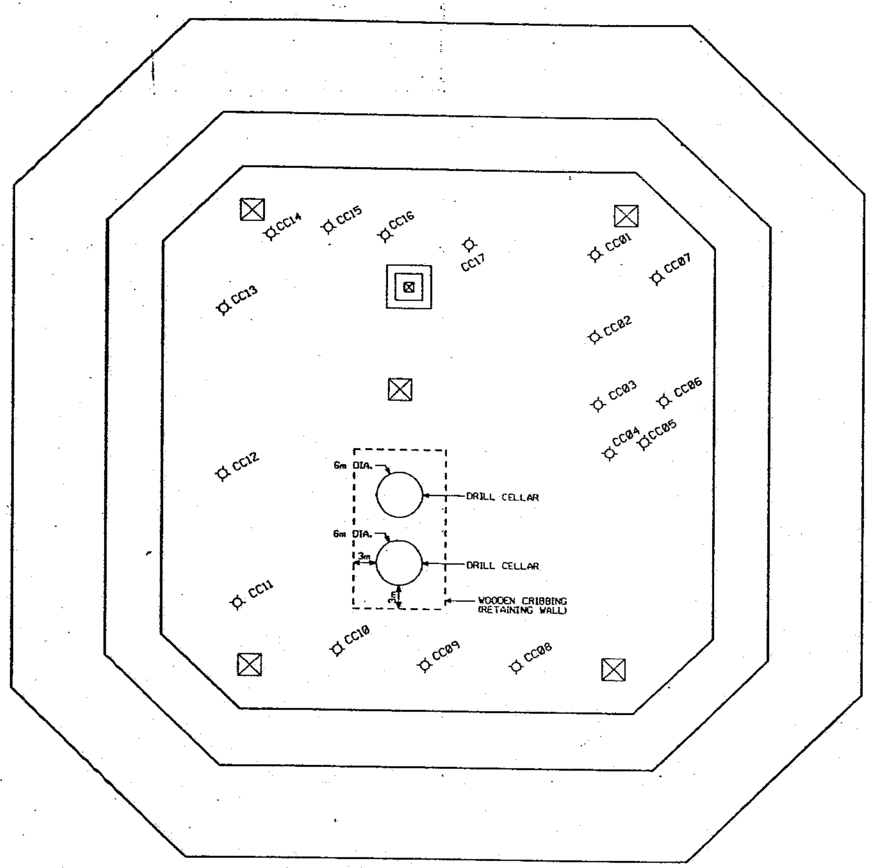
MOLIKPAQ AT AMAULIGAK F-24
AS-BUILT DRAWINGS

BERM VERIFICATION & MESOTECH SONAR UNITS

Figure: 3.15

DATE: 88/03/01 | DRAWING No. AF-88-013

FILE NAME: ZF1:[163,53]AF88013.;1 88-06/14 WHL

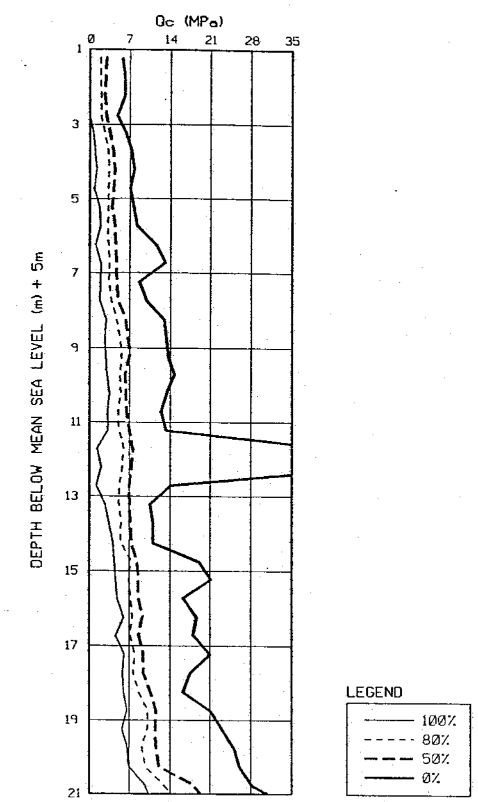


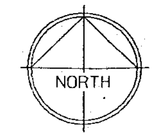
CPT SITES: BEFORE DENSIFICATION

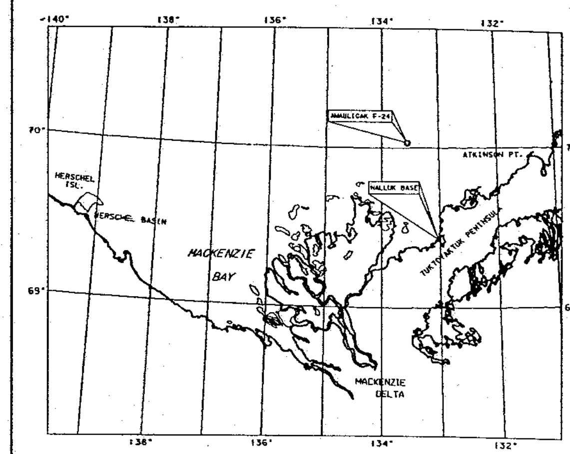
CORE BEFORE DENSIFICATION TABLE OF COORDINATES

INSTRUMENT LABEL	FROM NORTH (m)	FROM EAST (m)	SURFACE ELEVATION (m)	MAXIMUM DEPTH BELOW MSL
AF87CC01	11.0	15.5	+ 5.0	15.1
AF87CC02	22.0	15.5	+ 5.0	10.1
AF87CC03	31.0	15.0	+ 5.0	20.8
AF87CC04	37.5	13.5	+ 5.0	25.0
AF87CC05	36.0	9.0	+ 5.0	25.7
AF87CC06	30.5	6.5	+ 5.0	25.7
AF87CC07	14.0	7.5	+ 5.0	25.8
AF87CC08	66.0	25.5	+ 5.0	27.4
AF87CC09	66.0	37.5	+ 5.0	25.2
AF87CC10	64.0	49.0	+ 5.0	26.5
AF87CC11	58.0	62.0	+ 5.0	27.5
AF87CC12	41.0	64.0	+ 5.0	25.0
AF87CC13	19.0	64.0	+ 5.0	25.5
AF87CC14	9.0	58.0	+ 5.0	25.5
AF87CC15	8.0	50.5	+ 5.0	25.0
AF87CC16	9.0	43.0	+ 5.0	24.3
AF87CC17	10.0	32.0	+ 5.0	26.2

SUMMARY PLOT OF TIP RESISTANCE OF CORE BEFORE DENSIFICATION







KEY PLAN

NOTES:

1. MOLIKPAQ CENTRE COORDINATES

   GEOGRAPHIC 70°03'17.5" N. 133°37'48.2" W.
   UTM (ZONE 8) 7,772,401 N. 552,154 E.

2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM
   WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972)
   COORDINATES SHOWN ARE UTM ZONE 8. CENTRAL MERIDIAN 135°W LONG

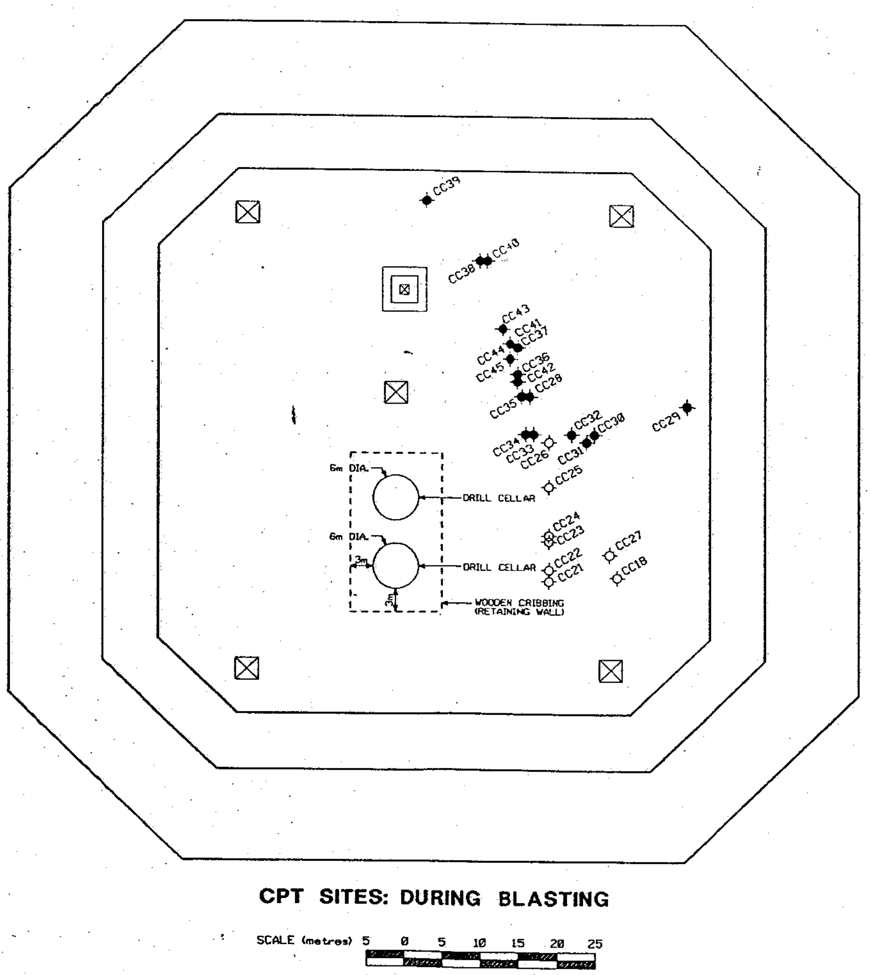
3. MOLIKPAQ BEARING

   GRID NORTH ~ 36°25'21"
   TRUE NORTH - 37°42'36"

LEGEND

CC INDICATES CONE PENETRATION TEST

INDICATES BLAST B TESTS

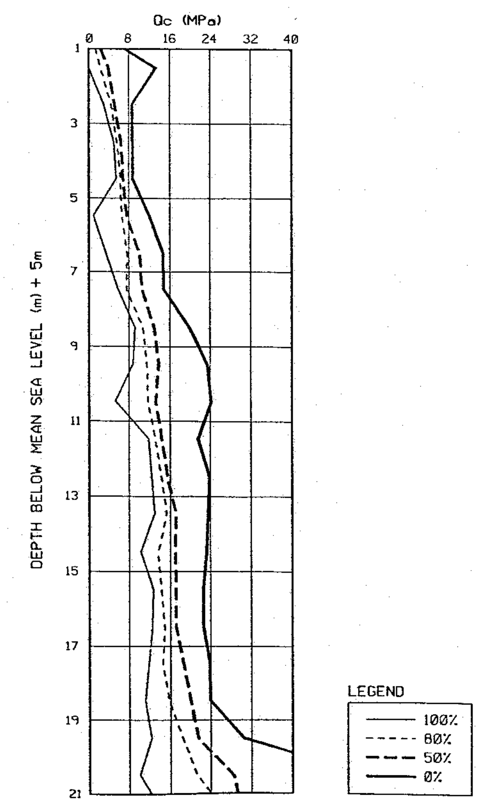


CPT SITES: DURING BLASTING

CORE DURING BLASTING TABLE OF COORDINATES

	INSTRUMENT LABEL	FROM NORTH (m)	FROM EAST (m)	SURFACE ELEVATION (m)	MAXIMUM DEPTH BELOW MSL
BLAST A	AF87CC18	54.0	12.0	+ 4.7	20.0
BLAST A	AF87CC21	54.5	21.0	+ 4.5	7.5
BLAST A	AF87CC22	53.0	21.0	+ 4.5	26.0
BLAST A	AF87CC23	49.3	21.0	+ 4.5	24.0
BLAST A	AF87CC24	48.5	21.0	+ 4.4	26.0
BLAST A	AF87CC25	42.0	21.0	+ 4.5	25.7
BLAST A	AF87CC26	36.0	21.0	+ 4.4	16.0
BLAST A	AF87CC27	51.0	13.0	+ 4.7	24.0
BLAST B	AF87CC28	30.0	23.5	+ 5.0	20.7
BLAST B	AF87CC29	31.2	3.0	+ 4.7	25.5
BLAST B	AF87CC30	35.0	15.0	+ 4.9	30.0
BLAST B	AF87CC31	36.0	16.0	+ 4.9	30.0
BLAST B	AF87CC32	35.0	18.0	+ 4.7	25.5
BLAST B	AF87CC33	35.0	23.0	+ 4.6	26.5
BLAST B	AF87CC34	35.0	24.0	+ 4.6	21.8
BLAST B	AF87CC35	30.0	24.5	+ 4.6	24.2
BLAST B	AF87CC36	27.0	25.0	+ 4.5	25.0
BLAST B	AF87CC37	23.5	25.0	+ 4.5	25.0
BLAST B	AF87CC38	12.0	30.0	+ 4.5	21.8
BLAST B	AF87CC39	4.0	37.0	+ 4.5	21.8
BLAST B	AF87CC40	12.0	29.0	+ 4.5	21.8
BLAST B	AF87CC41	23.0	26.0	+ 4.5	21.8
BLAST B	AF87CC42	28.0	25.0	+ 4.7	21.8
BLAST B	AF87CC43	21.0	27.0	+ 4.7	26.5
BLAST B	AF87CC44	23.0	26.0	+ 4.7	23.1
BLAST B	AF87CC45	25.0	26.0	+ 4.8	25.0

SUMMARY PLOT OF TIP RESISTANCE AFTER "A" BLAST



DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
88/03/04	1	AS-BUILT		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

	BY	DATE		BY	DATE
DESIGNED	MJVH	88/03/04	CHECKED	BTR	88/06/01
DRAWN	VRD	88/03/04	APPROVED	BTR	88/06/01

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

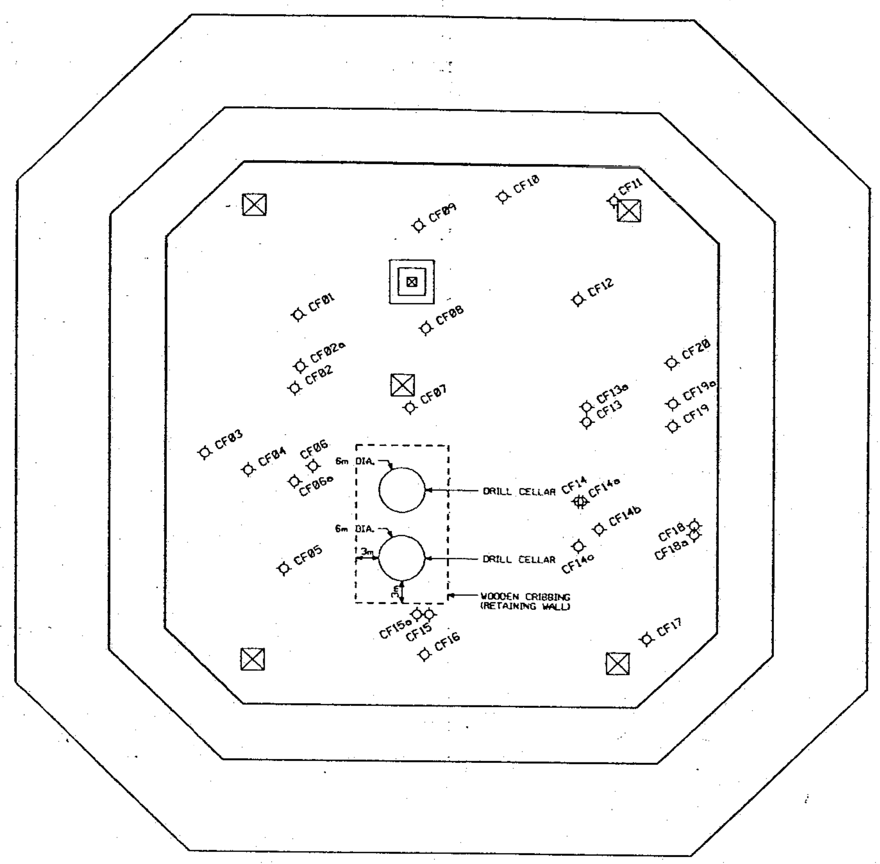
MOLIKPAQ AT AMAULIGAK F-24
AS-BUILT DRAWINGS

CORE BEFORE DENSIFICATION
CORE DURING BLASTING

Figure: 3.16

DATE: 88/03/01 DRAWING No. AF-88-017

PROJECT 84F012

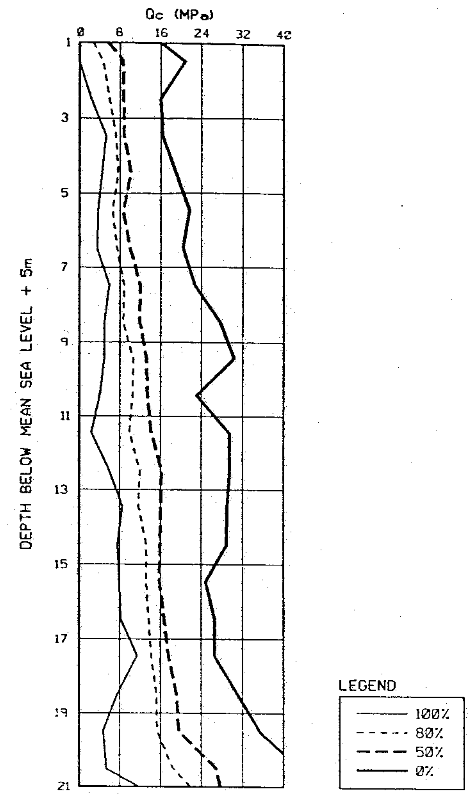


CPT SITES: FINAL CORE VERIFICATION

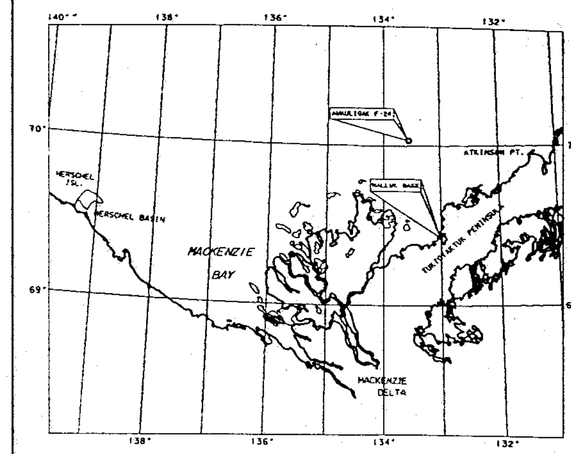
FINAL CORE VERIFICATION TABLE OF COORDINATES

INSTRUMENT LABEL	FROM NORTH (m)	FROM EAST (m)	SURFACE ELEVATION (m)	MAXIMUM DEPTH BELOW MSL
AF87CF01	20.4	54.6	+ 4.4	24.0
AF87CF02	30.1	55.0	+ 4.5	26.0
AF87CF02a	27.2	54.3	+ 4.4	26.5
AF87CF03	38.8	66.7	+ 4.6	37.5
AF87CF04	41.0	61.0	+ 4.5	37.5
AF87CF05	54.1	56.3	+ 4.3	21.0
AF87CF06	40.4	52.6	+ 4.4	15.0
AF87CF06a	42.5	55.0	+ 4.4	15.0
AF87CF07	32.5	40.0	+ 4.6	21.0
AF87CF08	22.0	38.0	+ 4.5	26.0
AF87CF09	8.4	39.0	+ 4.7	26.0
AF87CF10	4.3	28.0	+ 4.8	18.0
AF87CF11	4.6	13.5	+ 4.9	21.0
AF87CF12	17.8	18.2	+ 4.8	15.0
AF87CF13	34.0	17.0	+ 5.0	19.0
AF87CF13a	32.0	17.0	+ 5.0	21.4
AF87CF14	44.7	18.0	+ 5.0	26.0
AF87CF14a	44.7	17.5	+ 4.9	25.0
AF87CF14b	48.4	15.3	+ 4.9	25.0
AF87CF14c	50.7	18.0	+ 4.9	25.0
AF87CF15	60.0	37.4	+ 4.5	25.0
AF87CF15a	60.0	39.0	+ 4.5	27.4
AF87CF16	65.3	38.0	+ 4.5	26.0
AF87CF17	63.0	9.0	+ 5.0	25.0
AF87CF18	47.8	3.0	+ 5.1	26.0
AF87CF18a	49.0	3.0	+ 5.1	25.5
AF87CF19	34.5	5.8	+ 5.1	25.0
AF87CF19a	31.5	5.8	+ 5.1	25.4
AF87CF20	26.0	6.0	+ 5.0	25.0

SUMMARY PLOT OF TIP RESISTANCE FOR FINAL CORE VERIFICATION



NORTH



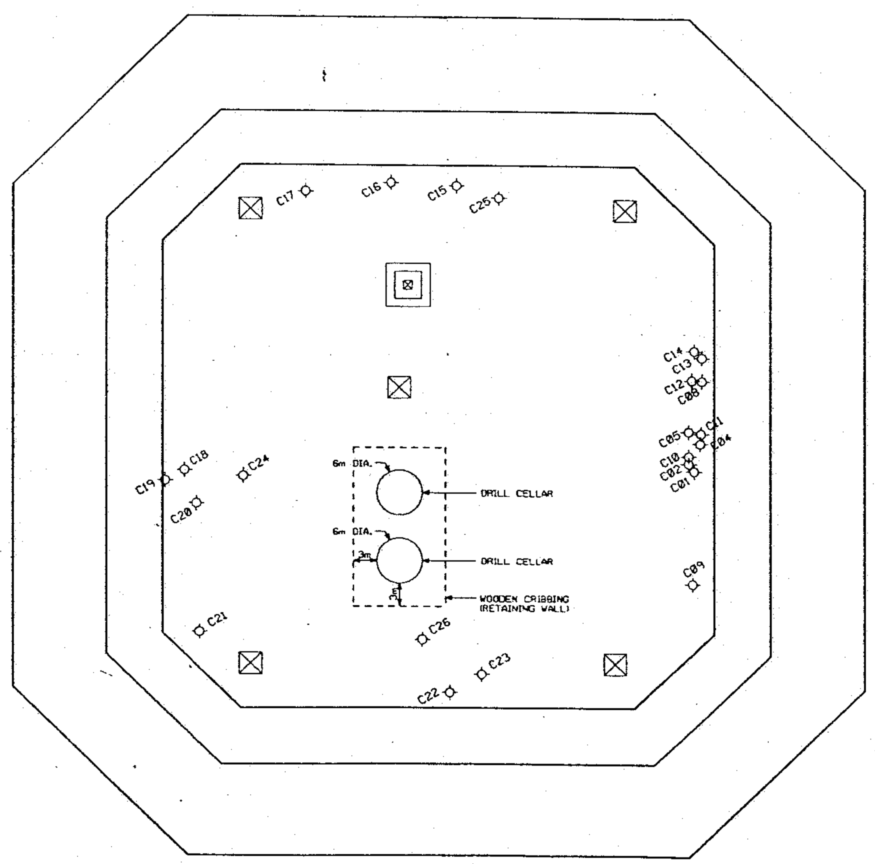
KEY PLAN

NOTES:

1. BERM CENTRE COORDINATES
   GEOGRAPHIC 70°03'17.5" N. 133°37'48.2" W.
   UTM (ZONE 8) 7,772,401 N. 552,154 E.
2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG
3. MOLIKPAQ BEARING
   GRID NORTH - 36°25'21"
   TRUE NORTH - 37°42'36"
4. WATER DENSITY: 10.03 KN/m³

LEGEND

◇ CC INDICATES CONE PENETRATION TEST



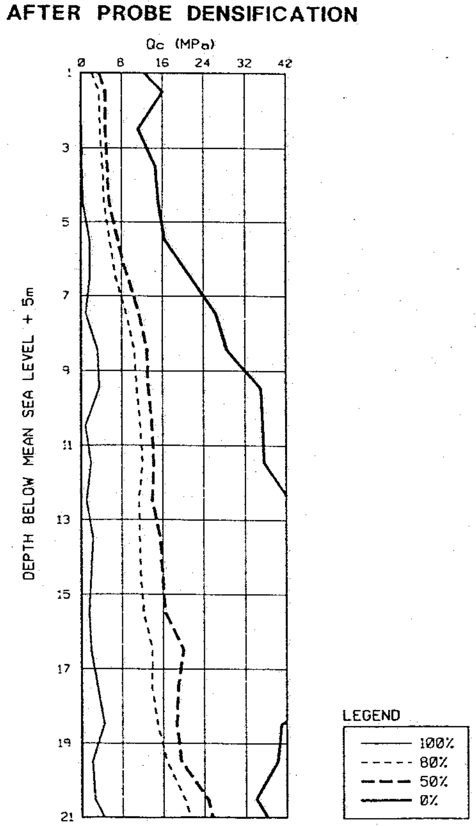
CPT SITES: CORE POST PROBE DENSIFICATION

SCALE (metres) 5 0 5 10 15 20 25

MOLIKPAQ CORE PROBE DENSIFICATION TABLE OF COORDINATES

INSTRUMENT LABEL	FROM NORTH (m)	FROM EAST (m)	SURFACE ELEVATION (m)	MAXIMUM DEPTH BELOW MSL
AF87C01	40.5	2.6	+ 4.8	20.0
AF87C02	39.5	3.3	+ 4.8	17.5
AF87C04	36.8	1.8	+ 5.0	16.5
AF87C05	35.2	3.3	+ 4.8	16.0
AF87C08	28.5	1.6	+ 5.0	17.0
AF87C09	55.5	2.8	+ 4.8	17.0
AF87C10	38.5	3.3	+ 4.8	17.0
AF87C11	35.5	1.7	+ 4.8	18.0
AF87C12	28.4	2.9	+ 4.8	17.5
AF87C13	25.4	1.6	+ 4.8	18.0
AF87C14	24.5	2.6	+ 4.8	8.5
AF88C15	2.7	33.5	+ 4.6	12.5
AF88C16	2.3	42.1	+ 4.5	11.5
AF88C17	3.5	53.2	+ 4.4	18.0
AF88C18	40.6	69.0	+ 4.5	19.4
AF88C19	42.0	71.5	+ 4.4	14.0
AF88C20	45.0	67.4	+ 4.5	16.2
AF88C21	62.0	67.0	+ 4.3	19.5
AF88C22	70.0	34.4	+ 4.4	20.0
AF88C23	67.6	30.2	+ 4.5	12.0
AF88C24	41.3	61.3	+ 4.5	20.0
AF88C25	4.3	28.0	+ 4.3	20.0
AF88C26	63.0	38.0	+ 4.5	19.0

SUMMARY PLOT OF TIP RESISTANCE AFTER PROBE DENSIFICATION



DATE	REV.	ISSUED	CHK'D. BY	APPR. BY
88/03/24	1	AS-BUILT		

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

ACCEPTED FOR CONSTRUCTION — BEAUDRIL PROJECT MANAGER — DATE

	BY	DATE		BY	DATE
DESIGNED	MVH	88/03/01	CHECKED	BTR	88/06/01
DRAWN	VRD	88/03/04	APPROVED	BTR	88/06/01

GULF CANADA RESOURCES
FRONTIER DEVELOPMENT

TITLE

MOLIKPAQ AT AMAULIGAK F-24
AS-BUILT DRAWINGS

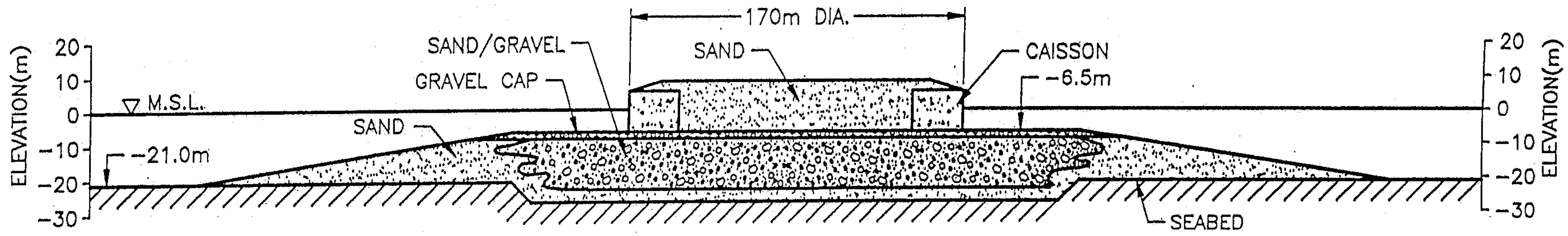
FINAL CORE VERIFICATION
CORE POST PROBE DENSIFICATION

Figure: 3.17

DATE: 88/03/01 DRAWING No. AF-88-018

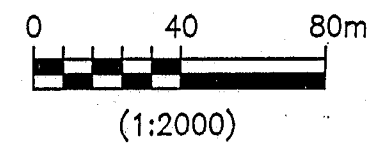
PROJECT 84F012

FILE: ZF1:(163,53)AF88018.;1 88/06/16 WHL



TARSIUT N-44 – CAISSON RETAINED ISLAND

OPERATOR: GULF



KLOHN-CRIPPEN

Figure 3.18: Tarsiut N-44 - Caisson Retained Island

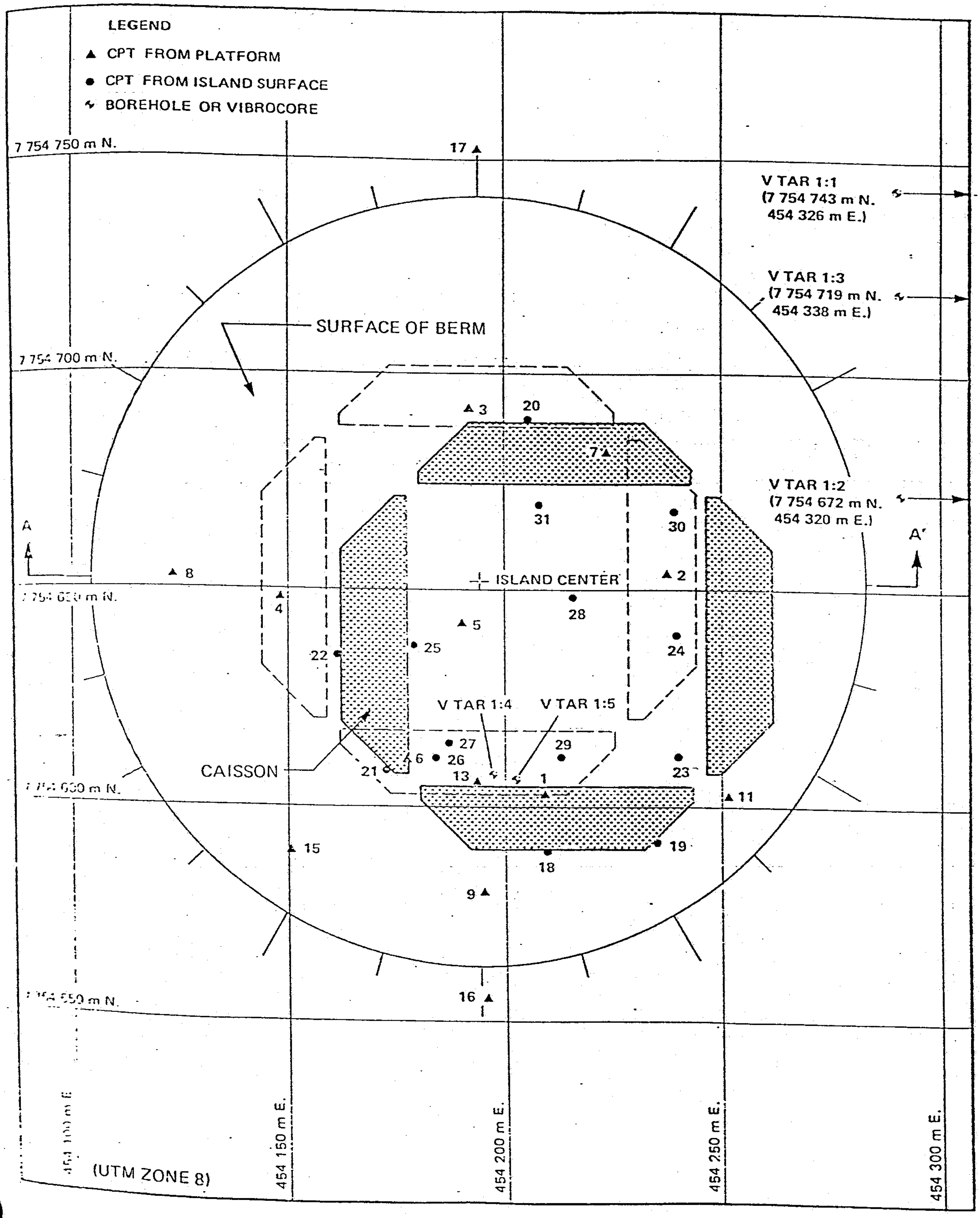
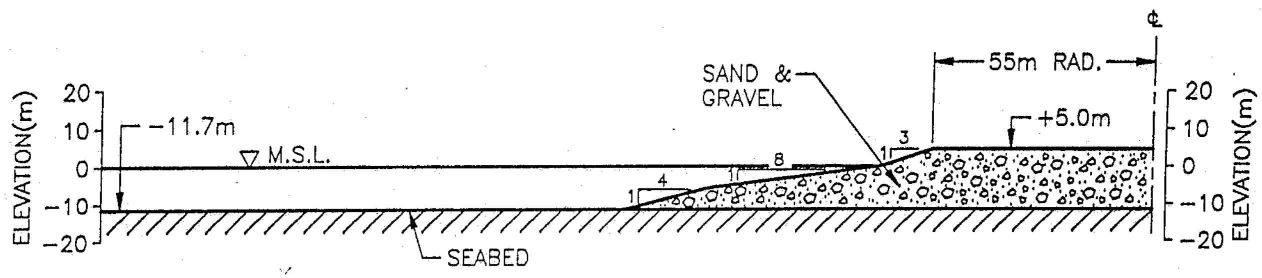


FIGURE 3.19 LOCATION PLAN FOR CONE PENETRATION TESTS AND BOREHOLES



NIPTERK L–19 – SACRIFICIAL BEACH ISLAND

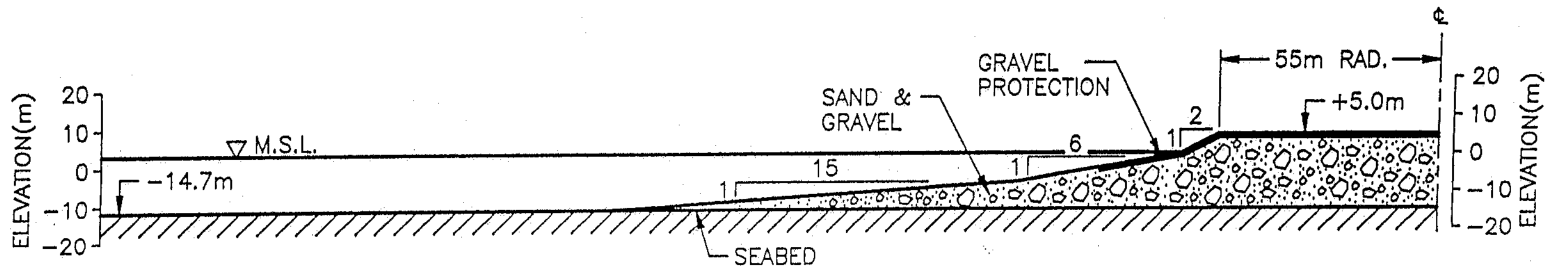
OPERATOR: ESSO

0 40 80m

(1:2000)

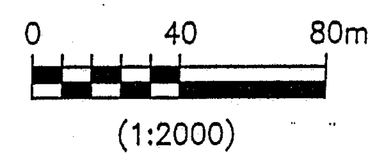
KLOHN-CRIPPEN

Figure 3.20: Nipterk L-19



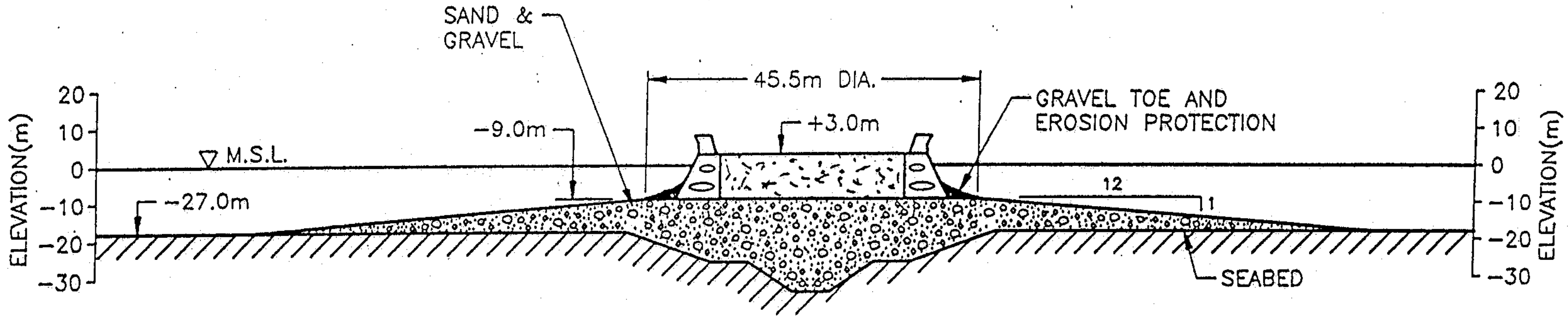
MINUK I–53 – SACRIFICIAL BEACH ISLAND

OPERATOR: ESSO



KLOHN-CRIPPEN

Figure 3.21: Minuk I-53 -Cross-Section

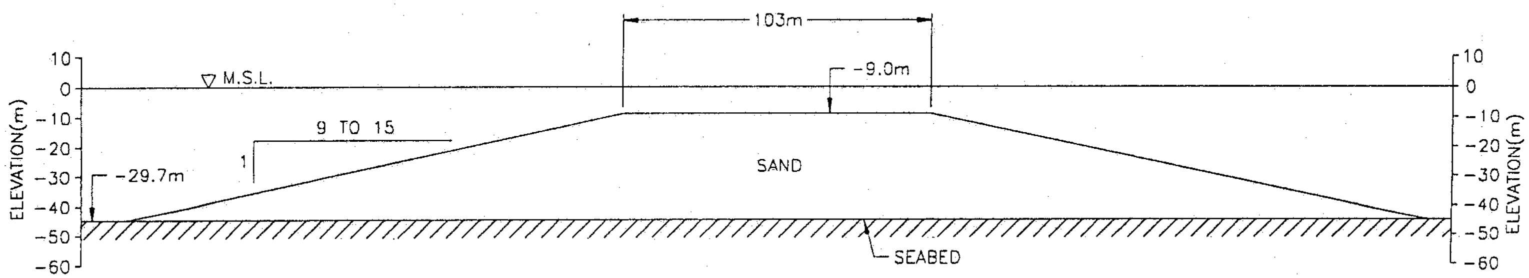


KAUBVIK I-43 – CAISSON RETAINED ISLAND

OPERATOR: ESSO

KLOHN-CRIPPEN

Figure 3.22: Kaubvik I-43 - Cross-Section



NERLERK B-67 - SINGLE STEEL DRILLING CAISSON

OPERATOR: DOME

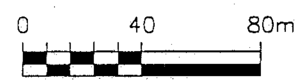
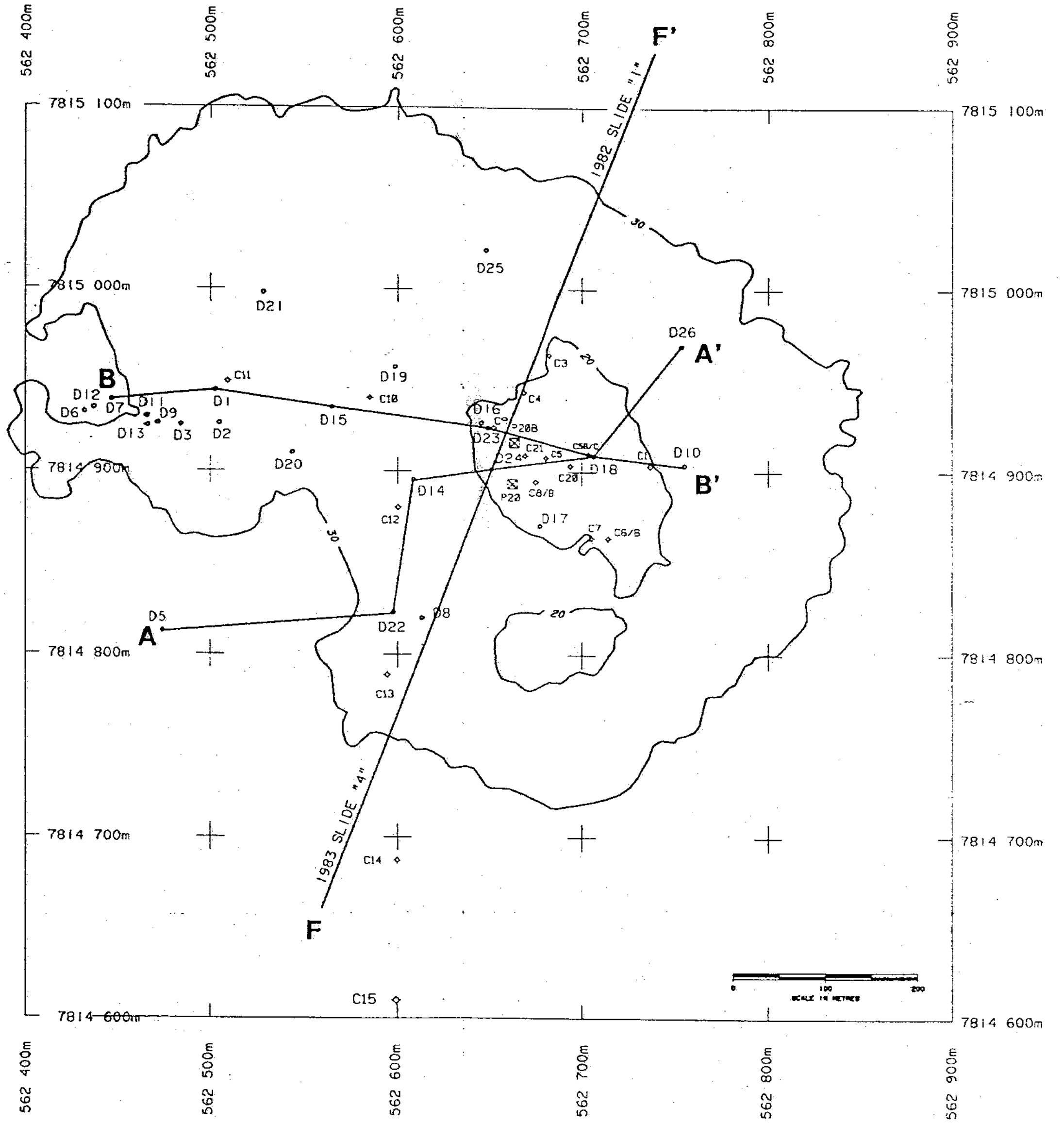


Figure: 3.23

NERLERK B-67 BERM
CPT LOCATIONS AND CROSS SECTIONS

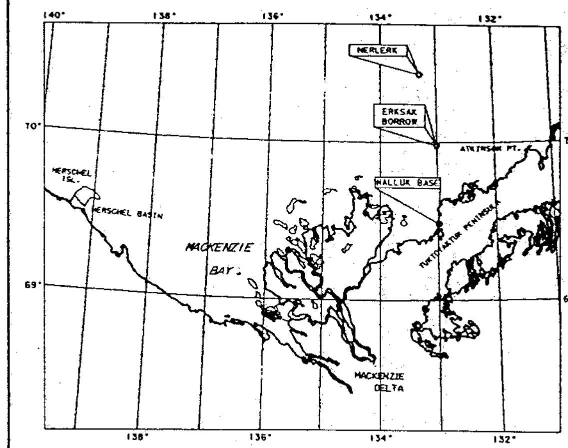


1988 PROGRAM

CPT	NORTHING	EASTING	WATER DEPTH	PENETRATION
NEB6788C1	7814904N	562737E	21.4m	21.5m
NEB6788C3	7814969N	562681E	21.3m	17.6m
NEB6788C4	7814942N	562668E	20.4m	19.5m
NEB6788C5	7814908N	562680E	19.7m	16.0m
NEB6788C5B/C	7814911N	562702E	20.7/43.1	19.3/1.8
NEB6788C6/B	781868N	562717E	21.7/42.8	17.5/27
NEB6788C7	7814864N	562706E	20.7m	18.5m
NEB6788C8/B	7814897N	562673E	20.1/42.5	19.3/2.6
NEB6788C9	7814925N	562653E	20.1M	23.0M
NEB6788C10	7814935N	562590E	22.2m	12.0m
NEB6788C11	7814947N	562509E	27.2m	15.0m
NEB6788C12	7814874N	562585E	26.7m	18.5m
NEB6788C13	7814794N	562587E	27.7m	17.4m
NEB6788C14	7814693N	562600E	34.9m	11.5m
NEB6788C15	7814617N	562598E	38.5m	9.0m
NEB6788C20	7814904N	562691E	20.5/42.7	18.5/1.8
NEB6788C21R	7814910N	562666E	20.4m	18.5m

1983 PROGRAM

TYPE	LOCATION No.	DATE	TOP OF SEABED (m)	TOTAL PENETRATION (m)	UTM COORDINATES NORTHING (mN)	EASTING (mE)
CPT	D1	07/14/83	25.6	14.5	7814945	562503
CPT	D2	07/17/83	21.0	17.1	7814926	562505
CPT	D3	07/17/83	21.0	7.5	7814925	562485
CPT	D4	07/18/83	27.6	9.2	7814823	562460
CPT	D5	07/18/83	27.3	18.7	7814812	562475
CPT	D6	07/19/83	25.7	9.8	7814934	562437
CPT	D7	07/19/83	25.2	10.0	7814939	562447
CPT	D8	07/22/83	21.8	2.7	7814820	562612
CPT	D9	07/25/83	20.5	9.6	7814926	562471
CPT	D10	07/26/83	18.2	10.5	7814903	562755
CPT	D11	07/27/83	19.7	14.8	7814930	562465
CPT	D12	08/06/83	25.6	20.0	7814834	562437
CPT	D13	08/06/83	23.5	15.5	7814925	562465
CPT	D14	08/06/83	25.0	13.4	7814895	562608
CPT	D15	08/06/83	22.4	9.8	7814935	562564
CPT	D16	08/06/83	20.8	12.2	7814926	562646
CPT	D17	08/07/83	19.4	10.2	7814871	562677
CPT	D18	08/07/83	19.2	11.0	7814909	562706
CPT	D19	08/07/83	20.2	9.8	7814957	562598
CPT	D20	08/07/83	23.9	13.9	7814910	562544
CPT	D21	08/07/83	21.9	16.3	7814998	562528
CPT	D22	08/07/83	19.5	17.0	7814822	562597
CPT	D23	08/11/83	18.6	12.0	7814924	562649
CPT	D24	08/11/83	17.8	13.3	7814901	562651
CPT	D25	08/11/83	21.5	14.0	7815022	562650
CPT	D26	08/11/83	23.2	14.8	7814969	562753



KEY PLAN

NOTES:

1. WATER DEPTHS NOT CORRECTED FOR EFFECTS OF WIND OR TIDE. POSSIBLE SEA LEVEL FLUCTUATIONS OF ±1 METRE.
2. ALL COORDINATES ARE REFERRED TO NA 1927 DATUM WESTERN AERODIST 1972 ADJUSTMENTS (CHS MAY 1972) COORDINATES SHOWN ARE UTM ZONE 8, CENTRAL MERIDIAN 135°W LONG.
3. NERLERK B-67 BERM CENTRE COORDINATES: 7 814 900mN, 562 620mE

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

ACCEPTED FOR CONSTRUCTION — BEAUFORT PROJECT MANAGER — DATE

	BY	DATE		BY	DATE
DESIGNED	BTR	89/03/30	CHECKED		
DRAWN	PGW	89/03/30	APPROVED		

GULF CANADA RESOURCES LIMITED
FRONTIER DEVELOPMENT

TITLE
FIGURE 4.6
NERLERK CASE HISTORY
NERLERK B-67 CPT LOCATIONS

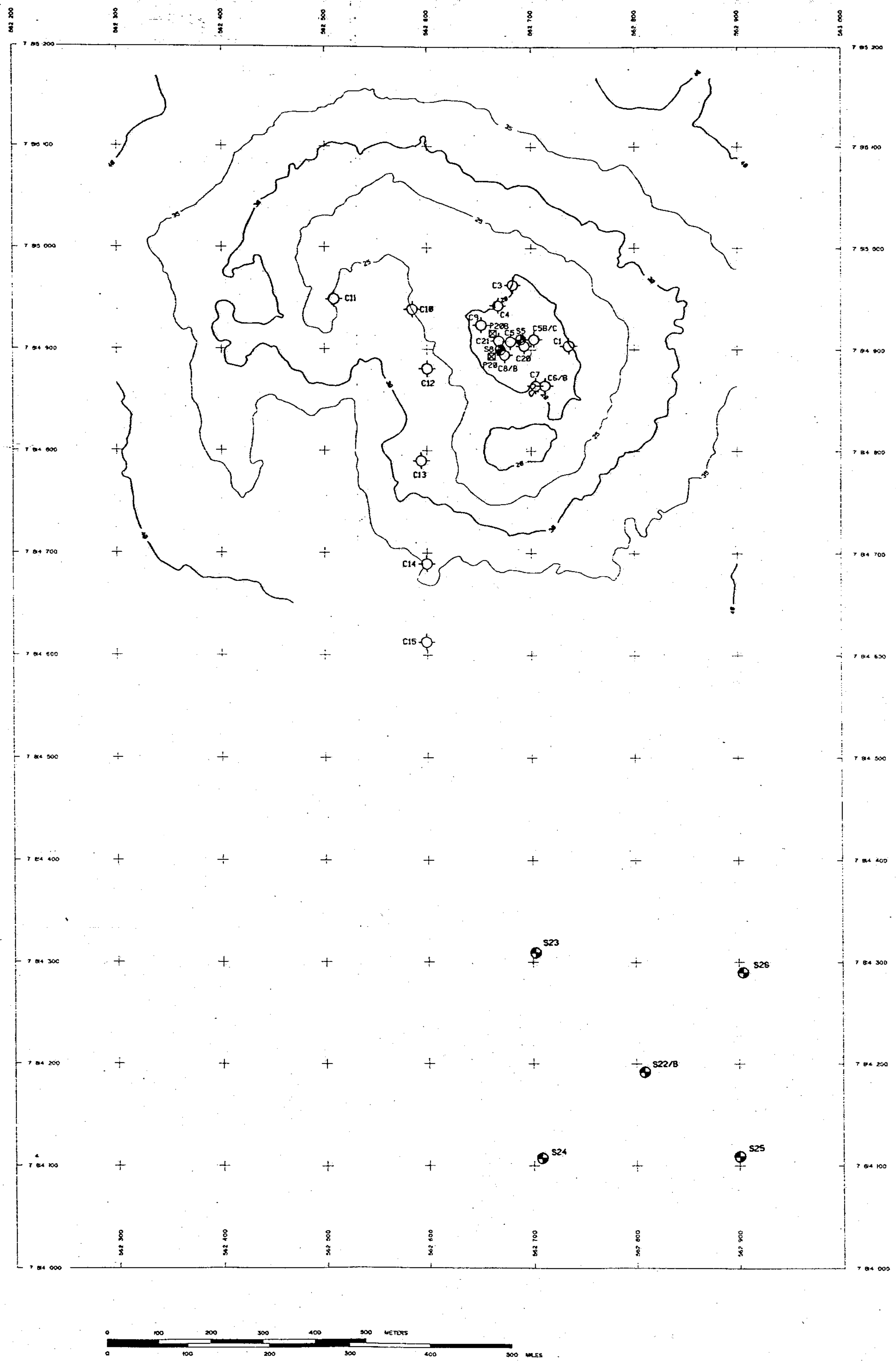
Figure: 3.24

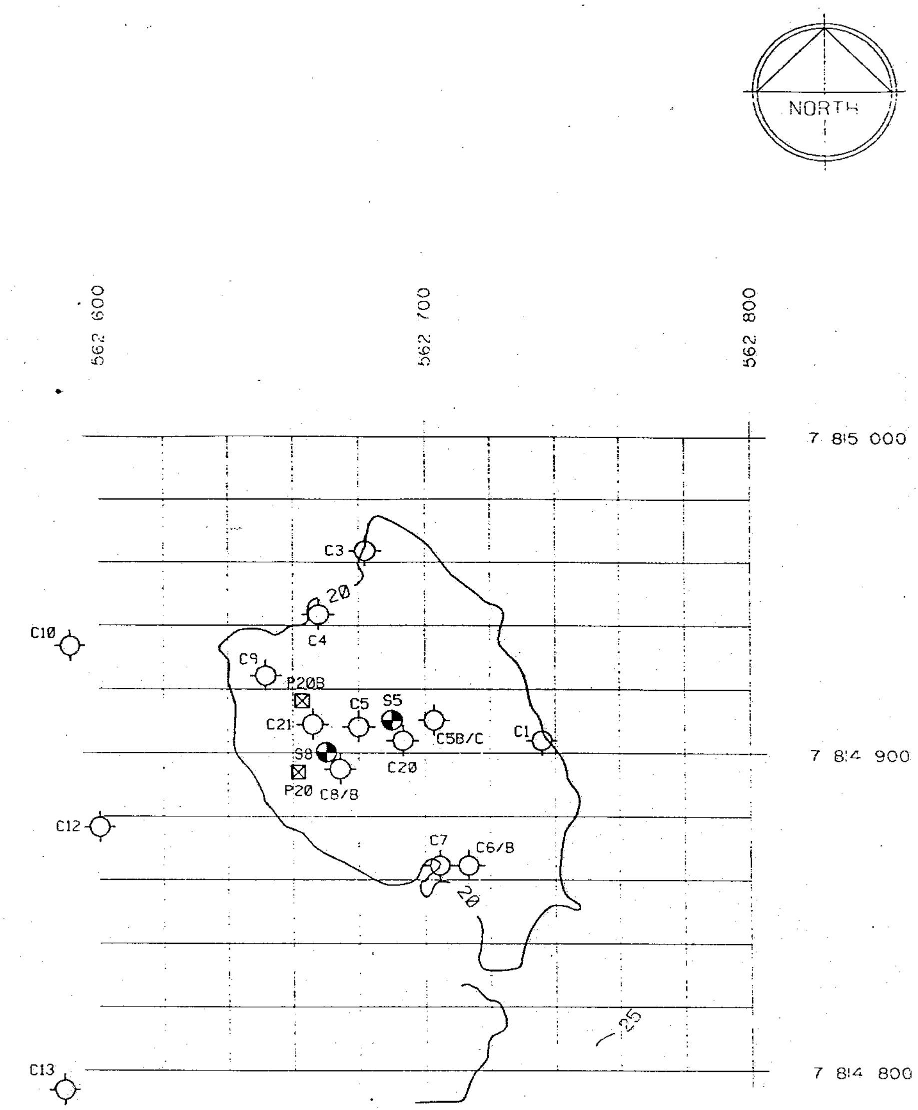
AOE CONSULTANTS LTD.

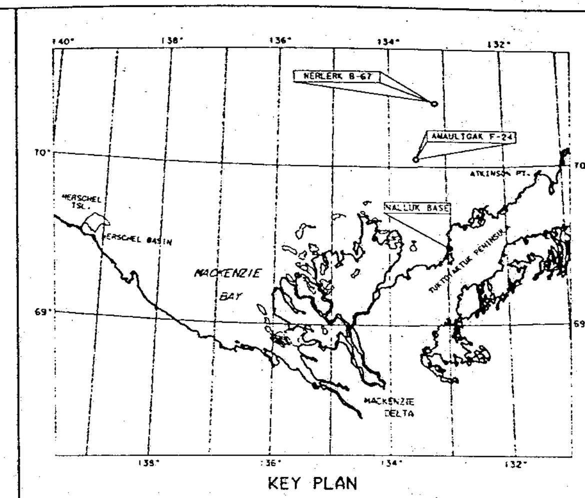
DATE: 89/03/30 | DRAWING No. DIA-154

NAME: ENG:163.053:DIA154.d 89/06/06 WHL

88F004







LEGEND

CPT TESTS

SBP TESTS

BOREHOLES

NOTES :

1. WATER DEPTHS NOT CORRECTED FOR EFFECTS OF WIND OR TIDE. POSSIBLE SEA LEVEL FLUCTUATIONS OF + 1 METER.

2. ALL COORDINATES ARE REFERENCED TO NA 1927 DATA WESTERN AEROSIST 1972 ADJUSTMENTS (CHS MAY 1972). COORDINATES SHOWN ARE UTM ZONE 8. CENTRAL MERIDIAN 135°W LONG.

3. VESSEL FRANK BRODERICK

4. POSITIONING SYSTEM : SERCEL SYLEDIS SR3 IN COMBINED MODE.

BOREHOLE	NORTHING	EASTING	WATER DEPTH	PENETRATION
NEB6788S5	7814910N	562691E	19.9m	28.2m
NEB6788S8	7814899N	562669E	19.7m	27.8m
NEB6788S22/B	7814192N	562813E	45.1m	0.7/1.4
NEB6788S23	7814312N	562703E	46.3m	1.6m
NEB6788S24	7814108N	562705E	46.1m	1.8m
NEB6788S25	7814115N	562900E	45.2m	2.5m
NEB6788S26	7814288N	562907E	46.2m	1.8m

SBP TEST	NORTHING	EASTING	WATER DEPTH	PENETRATION
NEB6788P2			22.4m	3.8m
NEB6788P20	7814894N	562662E	22.6m	3.7m
NEB6788P20B	7814917N	562664E	22.4m	16.0m

CPT	NORTHING	EASTING	WATER DEPTH	PENETRATION
NEB6788C1	7814904N	562737E	21.4m	21.5m
NEB6788C3	7814969N	562681E	21.3m	17.6m
NEB6788C4	7814942N	562668E	20.4m	19.5m
NEB6788C5	7814908N	562680E	19.7m	18.2m
NEB6788C5B/C	7814911N	562702E	20.7/43.1	19.3/1.8
NEB6788C6/B	781858N	562717E	21.7/42.8	17.5/27
NEB6788C7	7814864N	562706E	22.7m	18.5m
NEB6788C8/B	7814897N	562673E	20.1/42.5	19.3/2.6
NEB6788C9	7814925N	562653E	20.1M	23.0M
NEB6788C10	7814935N	562590E	22.2m	12.2m
NEB6788C11	7814947N	562509E	27.2m	15.0m
NEB6788C12	7814874N	562585E	26.7m	18.5m
NEB6788C13	7814794N	562587E	27.7m	17.4m
NEB6788C14	7814693N	562620E	34.9m	11.5m
NEB6788C15	7814617N	562598E	38.5m	9.0m
NEB6788C20	7814924N	562691E	20.5/42.7	18.5/1.8
NEB6788C21R	7814910N	562666E	20.4m	18.5m

DATE	REV.	ISSUED	CHK'D. BY	APPR. BY

DRAWING RECORD

NO.	DATE	REVISIONS	DWN. BY	CHK'D. BY	APPR. BY

	BY	DATE		BY	DATE
DESIGNED	MDH	88/08/10	CHECKED	MDH	88/09/10
DRAWN	WHL	88/08/10	APPROVED	B.R.	88/08/10

GULF CANADA RESOURCES LIMITED

FRONTIER DEVELOPMENT

TITLE

FIG. 5.1
NERLERK CASE HISTORY
1988
NERLERK SITE INVESTIGATION

Figure: 3.25

DATE: 89/03/17 DRAWING No. MAP-011

FILE NO. ZF1(163.53)MAP011: 89/05/31 PGW

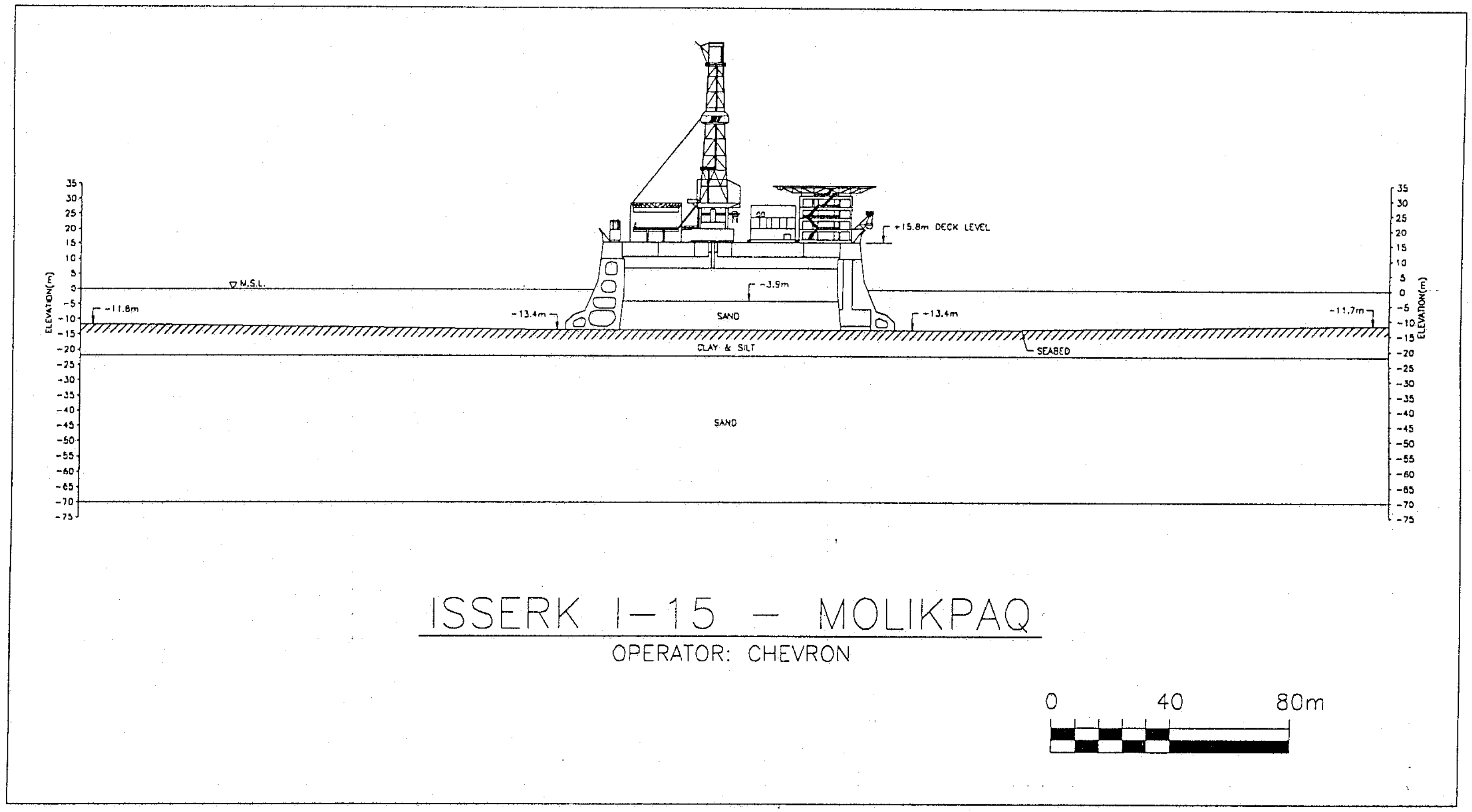
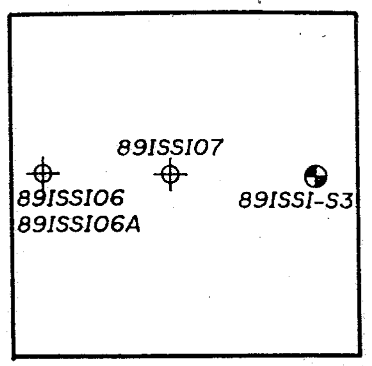

ELEVATION(m)
M.S.L.
-11.8m
-13.4m
-3.9m
SAND
-13.4m
-11.7m
+15.8m DECK LEVEL
CLAY & SILT
SEABED
SAND
ISSERK I-15 – MOLIKPAQ
OPERATOR: CHEVRON
0
40
80m

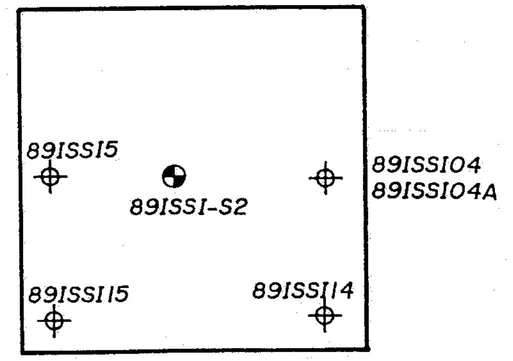
Figure: 3.26

BOREHOLE AND CPT LOCATION PLAN

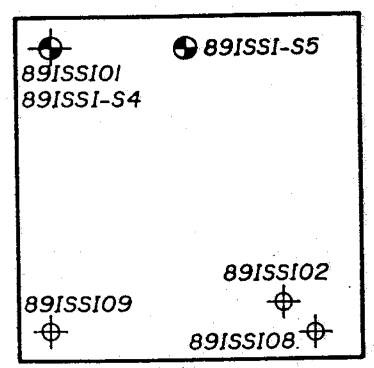
Figure 2.2



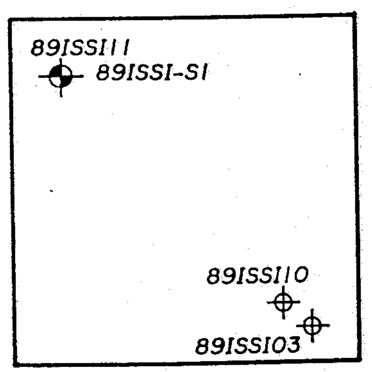
NORTHWEST DREDGE HATCH



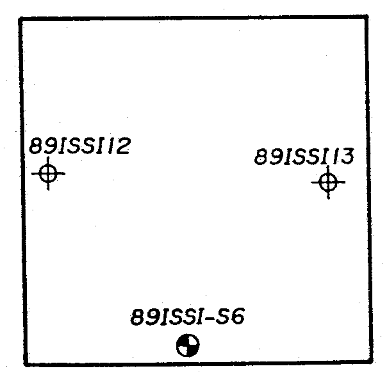
NORTHEAST DREDGE HATCH



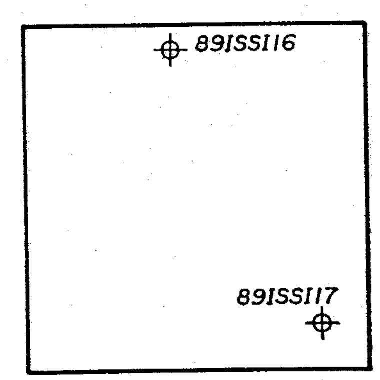
CENTER DREDGE HATCH



SOUTH DRILL CELLAR



SOUTHWEST DREDGE HATCH



SOUTHEAST DREDGE HATCH

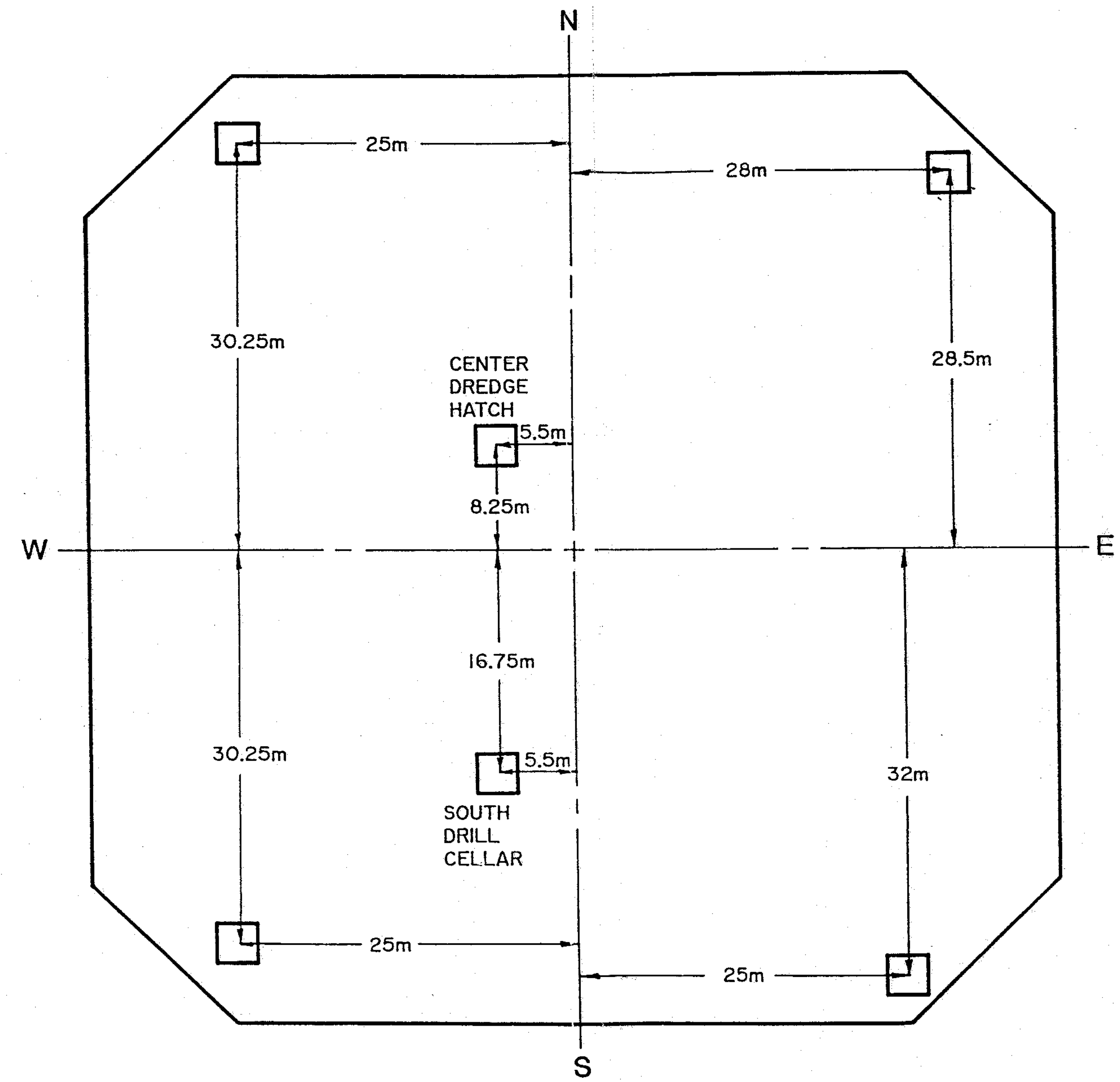
NOTES

1. CONE PENETRATION TESTS 01 TO 04 AND BOREHOLE S1 COMPLETED PRIOR TO START OF CORE FILL
2. CONE PENETRATION TESTS 04A TO 07 AND BOREHOLES S2 & S3 COMPLETED DURING CORE FILLING OPERATIONS
3. CONE PENETRATION TESTS 08 TO 17 AND BOREHOLES S4 TO S6 COMPLETED AFTER CORE FILLING

LEGEND

⌖ CONE PENETRATION TEST

◐ SAMPLED BOREHOLE



5 0 5 10 15meters

SCALE: 1:500

RIG CENTER COORDINATES:

UTM ZONE 8: 7,756,086.89N
526,867.30E

GEOGRAPHIC: 69°54'44.771"N
134°17'56.743"W

ORIENTATION: 359°T

NOTE

EACH DREDGE HATCH OPENING IS 3X3 m

Golder Associates

PROJECT: 892-2069A	DRAWN BY: RK	DATE: OCT 89	REVIEWED: JB

Figure: 3.27

APPENDIX I

INSTRUCTIONS FOR BeauFILL CD ROM

BeauFILL Help File

The BeauFILL program allows access to a database of information on the artificial islands constructed in the Beaufort Sea. The functions of the program are described below, with the information grouped by the menu command, as found at the top of the main program window.

File

Island — Selects the artificial island for which data will be extracted from the database. The main program window will remain blank until an island is selected from this list.

Other — Selects the trial berm or borrow source for which data will be extracted from the database. The main program window will remain blank until a location from the list is selected.

Exit — Exits the BeauFILL program and returns to the Windows operating system.

Construction

Description — Displays a description of the construction process used to construct the selected artificial island.

Surveys — Displays the survey data for the island, including freeboard, area, side slopes, subcut depth and fill quantities.

Borrow

Fill Quantity and Source — Displays the volumes of fill material placed as part of the artificial island, itemized from the borrow sources used.

Fill Quality — Displays the results of grain size tests done on the fill material.

Drawings

Cross Section — Displays a cross section of the artificial island. The cross section may be resized using the commands from the Display Options menu in the image window.

Bathymetry — Displays a bathymetric plan of the artificial island at the specified date. The bathymetric plan may be resized using the commands from the Display Options menu in the image window.

CPT Locations — Displays a plan view of the CPT data locations for the specified data series. The image may be resized using the commands from the Display Options menu in the image window.

Location Plan — Displays a plan view of the specified borrow area. The image may be resized using the commands from the Display Options menu in the image window.

Display Options

Reset Image — Resets the image to the initial size and aspect ratio.

Zoom In — Zooms in (enlarges) the image by a factor of two.

Zoom Out — Zooms out (reduces) the image by a factor of two.

Scale to Fit — Scales the image to fit the size of the window, retaining the aspect ratio. The window may be resized by clicking and dragging the border of the window.

Stretch to Fit — Stretched the image to fill the window, distorting the aspect ratio if necessary. Once distorted, the image may be returned to the correct relative scale by selecting Scale to Fit, above.

It is possible to zoom in to an area of the image by clicking on the image and dragging the mouse to draw a rectangle that outlines that area to be zoomed in to. The image may then be resized to fill the window by selecting Scale to Fit from the Display Options menu.

File

Print Image	Prints the image to the default Windows printer.
Exit	Closes the image window and returns to the main database window.

In-Situ Data

Reference Loads MS Word and displays a scanned reference document, "The Application of Microcomputers to CPT Testing in the Arctic Offshore."

CPT Data Displays a list of the cone penetration test (CPT) data that were performed at the artificial island or trial berm. Selecting one of the tests brings up a window which summarizes the data from that test. Selecting the View Data File button on this window loads the CPT data file into MS Word so that the actual data may be viewed. Selecting the Exit button closes the CPT window and returns to the main database window.

Base Map

Displays a location map of the Canadian Beaufort sea, showing the locations of the artificial islands. See the help description for the Drawings menu for a description of the tools available to resize and zoom the image.

Help

Loads the help file in MS Word.

Beaufort Granular Resources

Installation Instructions for the BeauFILL CD-ROM

Minimum hardware requirements:

- 486-66 CPU (Pentium 120 or faster recommended)
- 16 Mb RAM (32 Mb recommended)
- CD-ROM drive
- 800 x 600 video resolution
- 30 Mb available hard drive space for the Windows swap file. The actual installation requires less than 300 kb, since the data files are stored on the CD-ROM.

Software requirements:

- Windows 95
- Microsoft Word 7 or above

The BeauFILL program and the accompanying database is stored on a CD-ROM. To install the BeauFILL program, run the program SETUP.EXE which is located in the INSTALL directory. This program will install BEAUFIL.EXE into the directory C:\PROGRAM FILES\BEAUFIL.

Before running the program, the following parameter must be set: Using Windows Explorer, open the C:\WINDOWS\START MENU\PROGRAMS directory. Right-click on the BEAUFIL shortcut, and select Properties. Click on the Shortcut tab. The text in the Target: box will be

"C:\Program Files\Beaufil\Beaufil.exe"

Change this to read:

"C:\Program Files\Beaufil\Beaufil.exe" C:\Program Files\Beaufil\Beaufil.ini

The BeauFILL program can now be run by clicking on the Start Menu, Programs, Beaufil.

Note that when running BeauFILL with the data files on CD-ROM, the CD should be left in the CD-ROM drive until the drive stops spinning prior to running BeauFILL. This is because it may take the system some time to initially read the directory structure off the recordable CD, and attempting to access the CD prior to the completion of this process may cause access errors. This is particularly true for slow CD-ROM drives.

11/29/99

To store the data files in a location other than on the CD-ROM drive, such as on a server, copy the directories and all the files from the CD-ROM to the desired location. Edit the Beaufil.ini file in the C:\Program Files\Beaufil directory to point to the new locations for the data files.

APPENDIX II

REFERENCE PAPERS

ISLAND CONSTRUCTION IN THE CANADIAN BEAUFORT SEA

M.G. Jefferies, B.T. Rogers and H.R. Stewart
GULF CANADA RESOURCES LTD.
Calgary, Alberta, CANADA
AND

S.Shinde, D. James & S. Williams-Fitzpatrick
ESSO RESOURCES CANADA LTD.

ABSTRACT

Thirty six artificial islands were constructed in the shallow waters of the Canadian Beaufort Shelf during the period 1972-87. These islands extensively used hydraulically placed sand and a substantial body of experience was accumulated with the engineered construction of hydraulic fill structures. The Paper provides an overview of this experience.

INTRODUCTION

Canadian resource companies (exploration "operators") have used artificial islands as drilling platforms for hydrocarbon exploration in the shallow waters (<50m) of Arctic offshore since 1973. The Beaufort Sea is ice covered for 9 months of the year which constrains exploration by limiting the open water season for drillships. Islands are resistant to ice forces and offer the exploration operator the potential of a substantially longer drilling period than drillships (which may take 3 calendar years to complete a single well).

Islands are mainly constructed with hydraulically placed sandfills. Designs vary from simple surface piercing islands in one metre of water to monolithic steel caisson structures which utilize sand cores to provide ice resisting mass and are founded on zoned sandfill berms (Fig. 1). Thirty six islands have been constructed since the start of offshore exploration in 1973. Key details of these islands are summarized on Table 1 and their locations are shown on Fig. 2.

Islands were initially constructed in the shallow, near-shore areas but have been constructed in increasing water depths with time, (Fig. 3). To date, islands have been successfully constructed and withstood substantial design loadings in as much as 32 m of water but the one island attempted in 45 m of water (Nerlerk) had to be abandoned when it collapsed during construction.

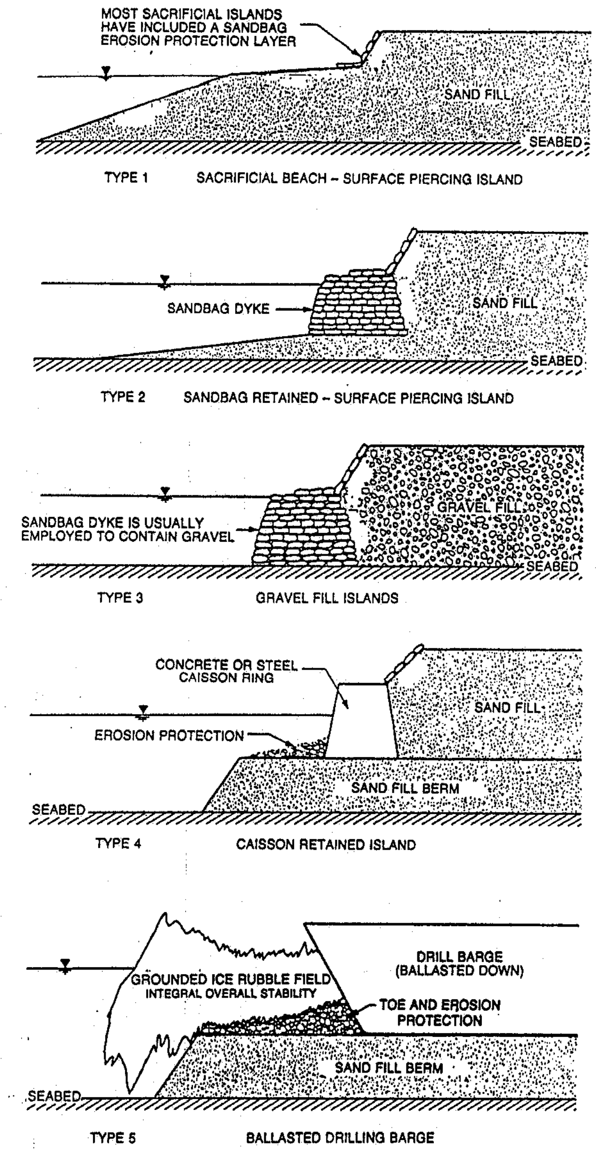


Figure 1
TYPES OF ISLANDS USED FOR EXPLORATION STRUCTURES

TABLE 1
SUMMARY OF ARTIFICIAL ISLAND CONSTRUCTION IN CANADIAN BEAUFORT SEA
1973 - 1987

	ISLAND NAME	EXPLORATION OPERATOR	YEAR COMP-LETED	WATER DEPTH m	FILL QUANTITY: m^3	ISLE TYPE	REF.
1	Immerk B-48	Esso Resources	1973	3.0	180,000	SBI	1,8
2	Adgo F-28	Esso Resources	1973	2.1	36,000	SRI	8
3	Pullen E-17	Esso Resources	1973	1.5	65,000	GHI	
4	Unark L-24	Sun Oil	1974	1.3	44,000	GHI	
5	Pelly B-35	Sun Oil	1974	2.0	35,000	BCI	
6	Netserk B-44	Esso Resources	1974	4.6	306,000	SRI	
7	Adgo P-25	Esso Resources	1974	1.5	27,000	SRI	
8	Adgo C-15	Esso Resources	1975	1.5	70,000	GHI	
9	Netserk F-40	Esso Resources	1975	7.0	291,000	SRI	
10	Sarpik B-35	Esso Resources	1975	4.3	118,000	GHI	
11	Ikkatok J-47	Esso Resources	1975	1.5	38,000	SRI	
12	Kugmallit H-59	Esso Resources	1976	5.3	160,000	SRI	
13	Adgo J-27	Esso Resources	1976	1.8	69,000	SRI	
14	Arnak L-30	Esso Resources	1976	8.5	1,070,000	SBI	
15	Kannerk G-42	Esso Resources	1976	8.5	1,070,000	SBI	
16	Isserk E-27	Esso Resources	1977	13.0	1,561,000	SBI	8
17	Issungnak 0-61	Esso Resources	1979	19.0	4,100,000	SBI	3,38,7
18	Issungnak 2-061	Esso Resources	1980	19.0	1,000,000	SBI	NOTE 1
19	Alerk P-23	Esso Resources	1981	10.5	2,360,000	SBI	61,58
20	N. Protection Island	Dome Petroleum	1981	4.6	2,000,000		NOTE 2
21	W. Atkinson L-23	Esso Resources	1981	7.5	955,000	SBI	
22	Tarsiut N-44	Gulf Canada	1981	21.0	1,800,000	CRI	13,10,11
23	Uviluk P-66	Dome Petroleum	1982	29.7	1,900,000	SSDC	65
24	Itiyok I-27	Esso Resources	1982	15.0	1,940,000	SBI	
25	Nerlerk B-67	Dome Petroleum	1983	45.1	4,000,000	SSDC	64,65, 66,67
26	Kogyuk N-67	Gulf Canada	1983	28.1	1,450,000	SSDC	42,65
27	Kadluk 0-07	Esso Resources	1983	14.0	436,000	CRI	37
28	Amerk P-09	Esso Resources	1984	26.0	1,162,000	CRI	6,5,61
29	Adgo H-29	Esso Resources	1984	3.0	75,000	SBI	
30	Nipterk I-19	Esso Resources	1984	11.7	931,000	SBI	
31	Tarsiut P-45	Gulf Canada	1984	26.0	343,000	CRI	4,12
32	Minuk I-53	Esso Resources	1985	14.7	1,986,000	SBI	
33	Amauligak I-65	Gulf Canada	1985	31.0	1,408,000	CRI	43,19
34	Kaubvik I-43	Esso Resources	1985	17.9	552,000	CRI	45
35	Arnak K-06	Esso Resources	1985	7.2	700,000	SBI	
36	Amauligak F-24	Gulf Canada	1987	32.0	2,000,000	SBI	69

LEGEND: SRI - Sandbag Retained Island
GHI - Island Constructed Using Dumped Gravel
SBI - Sacrificial Beach Island
CRI - Caisson Retained Island

NOTE: 1. Refurbishment of existing island.
2. Protection island was constructed in McKinley Bay as support facility, not an exploration island.

In 1983 Esso Resources Canada (ERCL) and Gulf Canada Resources (GCRL) initiated a co-operative study to synthesize the experience gained with hydraulic fill construction. This Paper, which presents an overview of the Canadian experience, is a summary of the findings of that joint project.

ISLAND TYPES

Sandbag Retained Islands

The sandbag retained island (SRI) was one of two types of islands constructed during the first 5 years of Beaufort exploration. A protective dike around the perimeter of the island consisting of a gravel berm below the water topped with sandbags to about a metre above mean sea level (msl) was used to contain the fill. The method is attractive in areas where fill is scarce and must be hauled some distance. The central core may be filled with either sand (7 cases) or gravel (12 cases). In most instances the fill is placed using clamshell grabs to transfer the fill from the transport barge to the island core. Some aspects of this type of islands are discussed in reference [1] with regard to the Netserk and Immerk projects.

Gravel Hauled Islands

Three gravel hauled islands (GHI) were constructed during the winter seasons from 1973 to 1976. Water depths ranged from 1.5 m to 4.3 m. To construct these islands, gravel was trucked from the near shore Ya-Ya lakes area over ice roads and then placed through slots cut into the offshore ice. Sandbags and filter cloth were placed during the summer season for erosion protection. However, Government discouraged use of the limited onshore gravel reserve in order to preserve the material for other uses. Since alternative gravel sources were too far away to be cost effective, this type of island construction ceased in the Canadian region of the Beaufort Sea.

Sacrificial Beach Islands

Sacrificial beach islands (SBI) consist of a drilling surface surrounded by long gradual beaches. As the name suggests, there is no expectation that the island will have other than a short life. The island geometry provides a working surface in the order of 100 m diameter that is resistant to wave action for the latter part of the open water season and to ice attack for approximately one year. This is achieved by using very flat perimeter beaches (typically 1V to 25H) where the island penetrates the waterline. Depending on the gradation of the beach sand sandbags may be used to protect the steep slope at the inshore end of the beach. Erosion of the beach is expected during the open water season, but the beach is sized for the probable storm scenario so that it is still adequate after erosion. In winter, the beaches cause the ice to fail in bending some distance from the work surfaces creating a protective ice rubble field around the island. This type of island has been discussed in the literature [1, 2] and such islands have been built in as much as 19 m of water (Issungnak) [3, 7].

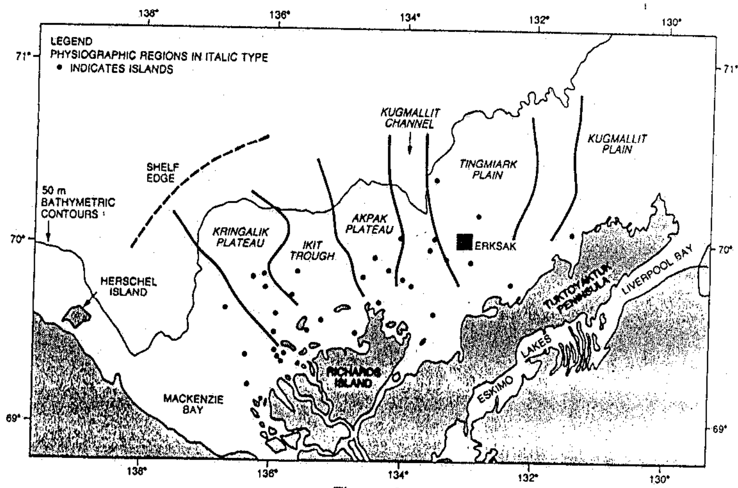


Figure 2
ISLAND LOCATIONS, PHYSIOGRAPHIC REGIONS AND ERKSAK BORROW PIT

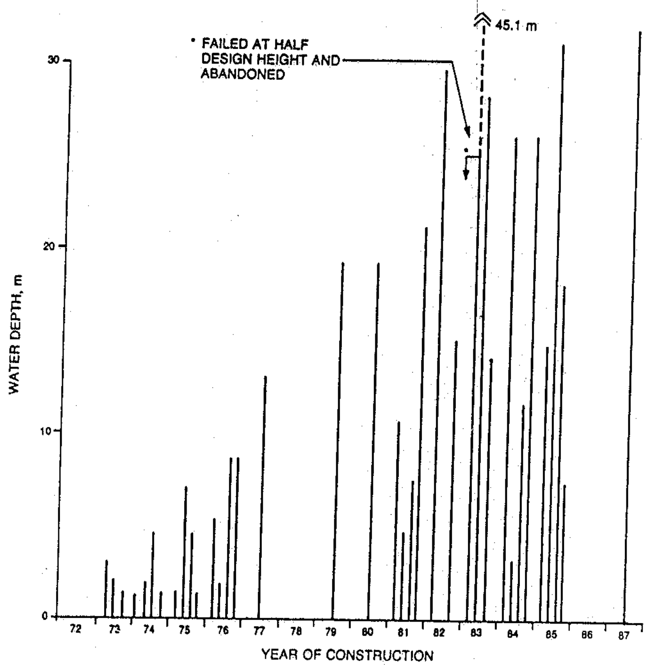


Figure 3
HISTORY OF ISLAND SIZE

Caisson Retained Islands

Caisson retained islands (CRI) use a caisson to substantially reduce the fill volume in the island but still retain sufficient sand mass in the core to resist ice load. As exploration moves to increasing water depths, the savings in sand volume become progressively larger. Moreover, when sand must be hauled over a long distance (eg: > 100 km at Tarsiut P-45,[4]) the advantage increases. The motivation for employing caisson islands was strictly economic - it was argued that halving the fill volume would halve the dredging cost [5]. Although this argument is not strictly correct, CRIs are significantly less expensive than SBIs in deeper (>20 m) water.

Caissons constructed to date are designed for exploration and are intended to be movable and used at several sites with differing water depths. This requirement for use in different water depths is met by placing the caisson on an underwater pad - called a berm - which provides the appropriate set-down depth regardless of water depth at a particular site. The berm is constructed with hydraulically placed granular material which is not compacted. The inner core of the caissons are also infilled with hydraulically placed sand.

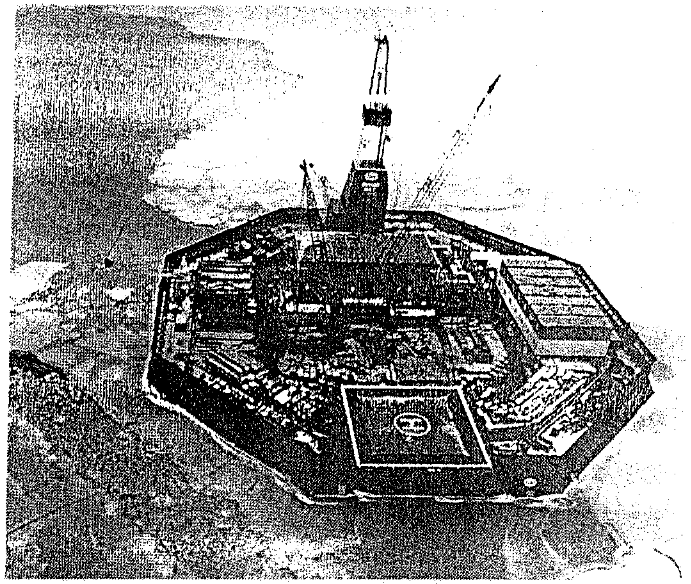
There are three CRI systems that have been used in the Canadian Beaufort Sea to date, each substantially different for the other. The first CRI to be designed was that for Esso Resources Canada [5, 6, 7], Fig. 4. This caisson comprises an octagonal ring with each face of the ring being an independent unit. The units interlock and are held together with post-tensioned cables. The CRI is founded at 9 m and has 3 m of freeboard to island surface with wave or ice deflectors providing another 5 m of overtopping protection. The core is filled with sand or gravel which provided the drilling surface. The unit has been installed at three locations to date (Table 1).

The first CRI to be constructed was Tarsiut N-44 [9, 10, 11]. This CRI comprised four independent concrete caissons that were founded at 6.5 m while the berm extended down to the seabed at 21 m. The core was infilled with sand which provided the drilling surface. The units were only used at Tarsiut N-44, in 1981-3, and have since been abandoned.

The other CRI deployed in the Canadian Beaufort Sea is the Gulf Molikpaq [4, 12], Figure 5. Unlike the other two CRI's, the Molikpaq is a monolithic caisson with an integral deck which supports the drilling modules. To date, the Molikpaq has been deployed at 3 sites (Table 1).

Ballasted Barge Islands

Ballasted drilling barges are comparable in many respects to CRI's. Two such barges have been employed to date, one in 1974 at the Sun Oil Pelly B-35 Island, and one, the Canmar SSDC [13], which has a strengthened hull for extended arctic drilling. This latter unit has been used at two island sites in the Canadian Beaufort (Table 1). The SSDC used sandfill berms as it is founded at 9 m and was deployed in approximately 30 m of water at both Uviluk [14] and Kogyuk.



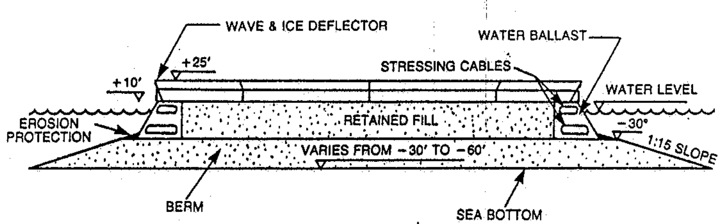
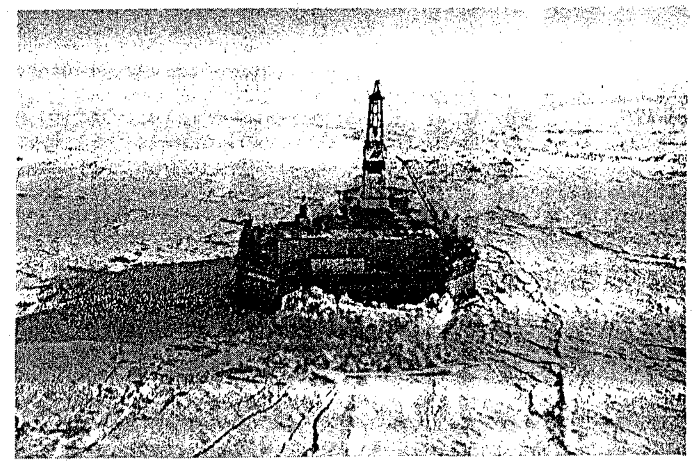


Figure 4
ERCL CAISSON ISLAND



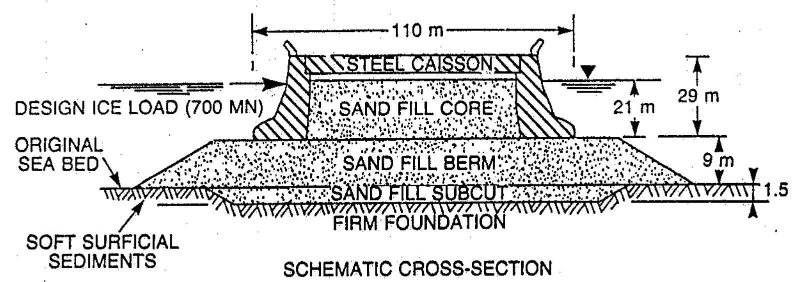


Figure 5
BEAUDRIL MOLIKPAQ

DESIGN

The purpose of design is to consider the loadings on the island and to develop the appropriate combination of island geometry and materials that can withstand the loads within a particular margin of safety at least cost. Conventionally, this is done by considering the factors independently and then making calculations on the overall system. Several iterations are usually required. For the present purpose it is sufficient to discuss each item separately without describing the optimization stage which is well understood.

Design Standards

The design and construction of offshore structures is usually carried out according to a code of practice (or its equivalent) that varies with the jurisdictional authority. The situation in Canada is rather different as no national code of practice has been published nor has any external code been approved for general use. The work reported in this Paper has been approved by the Canadian Oil and Gas Lands Administration (COGLA) on a case by case basis.

Exploration for hydrocarbons is a site specific activity of limited duration. In the Beaufort Sea most locations have only a single well, although a few sites have been considered for two or more wells. The required service life of the island is short, generally, less than two years. In this context, the advice of the British offshore code [15] is particularly pertinent:

> "...... If the intended service life of the structure is significantly less than thirty years, it may be appropriate to use an extreme design environmental condition having a recurrence period of less than 50 years."

This greater risk approach has become the norm for exploration structures in the Canadian Beaufort and is combined with monitoring programs to provide advance warning of possible overload or deviation between projected and actual performance [16].

Ice Loads

As noted in the introduction, ice cover is a major environmental feature of the Beaufort Sea. This ice may produce considerable horizontal loads on stationary structures. Ice loads used for design are influenced by two major factors: risk of exposure to thick ice; and, nature of the island.

In the shallow, near shore areas where the ice is landfast for much of the winter and extreme thicknesses of ice do not occur, design might be based on an ice sheet 2.0 m thick moving against the island/structure. The landfast ice extends (in an average winter) out from the shore to approximately the 20 m isobath. Beyond the 20 m isobath and extending as far as the shelf-edge a band of intermittently moving ice (called the shear zone) separates the near-stationary landfast ice from the constantly gyrating polar pack. Ice in the shear zone will normally be 'first-year' ice typically less than 2.0 m thick but there is a finite probability of incursions of older ice, called 'multi-year' ice in the form of floes that may be tens of miles in diameter with thicknesses typically between 3 to 6 m. Floes may also contain line features, known as 'pressure ridges' with substantially greater thickness of ice than the average ice floe thickness. Thus, the expected or design ice thickness to be resisted by the island depends very much on its location.

Ice loads may be computed from the design ice/structure interaction scenario using several different methods as there is not, as yet, an agreed practice. An excellent survey of the commonly used approaches is given in [17]. The important point to appreciate is that the expected load for any given thickness of ice depends on the structure itself with regard to both scale and failure mode.

Scale effects are significant in ice. The crushing pressure of thin ice in a laboratory tank against a model structure of a few centimetres width may exceed 5 MPa. Yet, the same type of ice may fail against a full size structure whose width is 100 m with a crushing pressure of less than 1 MPa even where geometric similitude has been preserved. Thus a two order of magnitude change of scale produces a five fold reduction in unit loading. Further data and discussion on this most important phenomenon may be found in [18].

The geometry of the island structure also has considerable influence on the ice load. If the ice/island interface is near vertical (eg; a CRI) then ice failure will tend to pure crushing whereby intact ice is fractured to a granulated material which is then extruded from the failure zone (see [19] for photographs of this process). On the other hand, if the ice/island interface slopes as in a SBI, the ice will tend to fail in bending with blocks of ice being pushed up the beach. The load to cause bending failure in ice is generally less than that required to cause crushing except in the case of very thick (> 12 m) sheet ice. Geometry also affects the ice/island interaction in the horizontal plane as, for example, in promoting splitting of ice floes (which lowers ice forces) or in the reduction in the importance of edge effects (which raises ice forces).

Overall, the evaluation of design ice load is a complicated process with scale effects, geometric effects, rate effects superimposed on uncertainty with regard to ice features. Although these factors might be expected to lead to considerable variation in opinion as to appropriate design loads, in fact, a consistent pattern emerges as may be seen on Table 2 which summarizes the design ice loading for four island projects. It illustrates that design load increases with distance offshore (water depth), and reflects the island type. Design ice loads are also comparable between operators. Probalistic studies suggests the load levels are associated with about a 4 per cent annual probability of being exceeded.

A final aspect of ice loads is the issue of load cycling. Several types of ice failure mode produce cycling of ice load and pure crushing of ice in particular produces systematic load cycling with an amplitude in the order of 50 per cent of the peak load [19].

TABLE 2

Project	Water Depth	Island Type	Design Ice Load
NIPTERK I-19	12 m	SBI	170 MN
AMERK P-09	26 m	CRI	400 MN
TARSIUT P-45	25 m	CRI	500 MN
AMAULIGAK F-24	32 m	CRI	700 MN

Frequencies of cycling are commonly in the order of 1 Hz while durations of such ice failure events may be of the order of 30 minutes. As will be readily appreciated, this type of loading has particular significance for hydraulically placed sandfills.

Sandfill Strength (Density)

The issue of sandfill density is one that attracts much attention in the context of hydraulic fills because sand strength is a strong function of sand density and hydraulically placed sandfills have a reputation of not being dense [20] unless compacted. The reality of underwater hydraulic fills is that they can never be made very dense even after extensive compaction. Because of this reality, comprehensive laboratory sand testing programs (approaching a total cost of $1M) were sponsored by the exploration operators to evaluate the drained, undrained and cyclic strength of Beaufort sands as functions of sand density and gradation.

A fundamental aspect of sand behaviour is dilatant response to stress. Dilatancy is the increase in volume with shear strain and is the mechanism by which sands may exhibit high friction angles [21]. Dilatancy is very desirable in engineered sandfills and is usually obtained by ensuring the sand is dense. However, it became apparent at an early stage of testing that loose clean Beaufort Sands (less than 5% silt) were dilatant, commonly at relative densities loose as low as 20%. But, there were many inconsistencies in sand behaviour as a function of relative density [22].

An alternative to relative density, the state parameter, was introduced [22] as a characterizing parameter for sandfill. The state parameter concept is an empirical adaption of critical state soil mechanics [23] and uses the distance from the steady (critical) state in e-p' space as a normalizing parameter for sand. It was found that both peak dilation rate and peak friction angle were strongly a function of the state parameter (ψ), regardless of sand gradation, mineralogy, silt content, etc. [22, 24]. Figure 6 shows both the definition of state parameter and its relation to peak friction angle and dilatancy. Although the state parameter approach is founded in

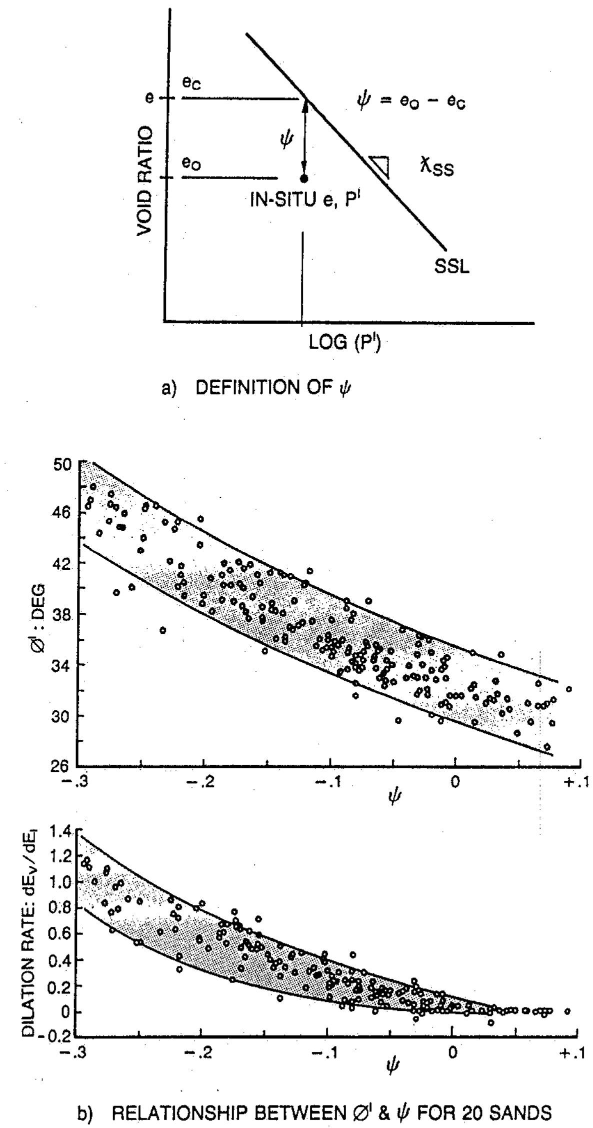


Figure 6
DEFINITION AND USE OF STATE PARAMETER

critical state soil mechanics it has the implicit operational form of stress-dilatancy theory [25]. These theories together provide theoretical assurance that the state parameter is a proper approach to the engineering of sandfills.

State is not the only parameter which influences the constitutive behaviour of sand. A second parameter, usually referred to as fabric, is equally important [26, 27]. The fabric parameter describes the relative orientation of sand particles and a whole range of fabrics can exist in sand at any particular density [28]. It has been shown, [26, 29] that fabric can change the shear stress - shear strain relationship by more than a factor of two and the peak friction angle by several degrees. However, the routine measurement of fabric has not been achieved and the avoidance of fabric in sand engineering necessarily introduces significant constraints on the precision of calculations.

Stability calculations of hydraulic fills placed considerable reliance on the 'steady state' concept [30]. This concept indicates that undrained strength of sand at large strain is a simple (logarithmic) function of initial void ratio regardless of whether the sand is monotonically or cyclically loaded. The steady state strength may be treated as an assured minimum value [31]. Stable behaviour of a sandfill may be obtained, according to this steady state concept, by ensuring the sand is dilatant, since a dilatant sand has a residual undrained strength greater than the peak drained (or partially drained strength).

Ice Resistance

As ice is a principal environmental load, providing adequate resistance to this loading is a major objective of island design. To date, design has employed upper-bound methods whereby a kinematically admissible failure mechanism is postulated and the work (force) required to drive this mechanism is compared to the ability of the mechanism to dissipate energy plasticly. Upper-bound approaches require consideration of a variety of mechanisms to find that with the least ratio of external work/plastic dissipation. Although an infinity of admissible mechanisms may exist, they can broadly be classed into two groups: those only involving the sandfill; and, those also involving foundation soils. The first group are often referred to as 'decapitation' failures by analogy to the top of the island moving as a block, shown as 'Mode 1' on Fig. 7. The second group are correspondingly referred to as 'basal' failures because of the importance to the foundation to this mechanism, shown as 'Mode 2' on Fig. 7.

'Mode 1' type failures usually display a logarithmic spiral velocity discontinuity with a factor of safety typically 15% less than that calculated by a planar sliding analysis. A factor of safety of 1.5 defined on the frictional peak stress ratio is usually sought for

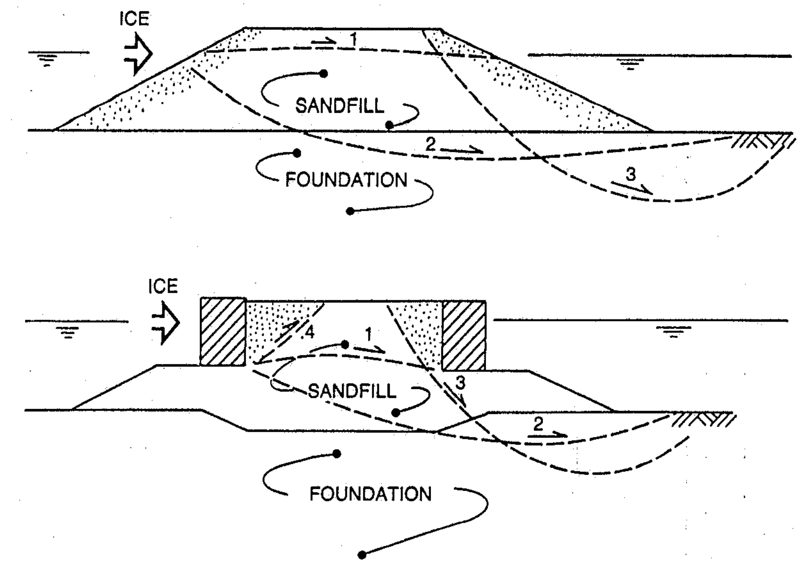


Figure 7
EXAMPLES OF KINEMATICALLY ADMISSABLE MECHANISMS FOR ISLAND DESIGN

'Mode 1' type failure. In the case of SBI platforms and other islands where the earthworks pierce the water surface, sufficient effective mass above the failure surface usually exists because of other design considerations. Experience has also shown that cyclic loading is not an issue with SBI type platforms.

Analysis of 'Mode 1' failures is quite complex in the case of CRI platforms because of the two issues of soil-structure interaction and cyclic loading. Although a factor of safety of 1.5 is still sought it is necessary to consider whether the caisson can enforce the unified action of 'Mode 1' or whether local failures ('Mode 3' on Fig. 7) might dominate. Both drained and steady-state analyses are usually carried out and may be supplemented with finite element analysis and physical model (centrifuge) tests [4].

Design action against 'Mode 1' failures of CRIs fall into two categories: i) structural and ii) site specific. Structural design of the caisson involves selecting the appropriate caisson geometry, set-down depth, structural system, etc. to meet the design load criteria. Discussion of these issues may be found in [32] and [34]. Once the caisson exists the question of its adequacy at a particular site arises. If the original design criteria are more stringent than the site specific ones then no further action is required. However, if the site specific criteria are more stringent than those assumed in the original design, then upgrading is required. There are three possible remedial measures:

- improve the sand properties;
- increase the effective stress regime;
- modify the sand.

Improvement of sand properties can be achieved either by using more permeable sand, which improves the time factor of the sand response to loading or by compacting the sand. More pervious sand was used at Kadluk O-07, Tarsiut P-45, Amerk P-09, Amauligak I-65, Kaubvik I-43 and Amauligak F-24. The core sand was compacted at Amauligak F-24.

Increasing the effective stress regime can be achieved by raising the set-down depth and/or installing a core dewatering system. Both actions reduce the internal phreatic surface with respect to the top of the core. Both actions were employed at Amauligak F-24 which used a 5 m raised set-down elevation and ten 200 mm dia. dewatering wells with a total of 225 hp installed pumping capacity.

Sand modification can be achieved either by structural grouting or freezing. To date neither technique has been used in any Canadian island project.

Basal failure, 'Mode 2', has not been found as critical as gravitational failure because the sliding plane is longer and flatter. However, concerns about basal sliding arose in the case of Tarsiut N-44, and Kadluk O-07 when pore pressures in the foundation clays continued to rise after construction had been completed [35, 36, 37, 38]. It was subsequently determined that the phenomenon was one of delayed peak pore pressure, not progressive failure.

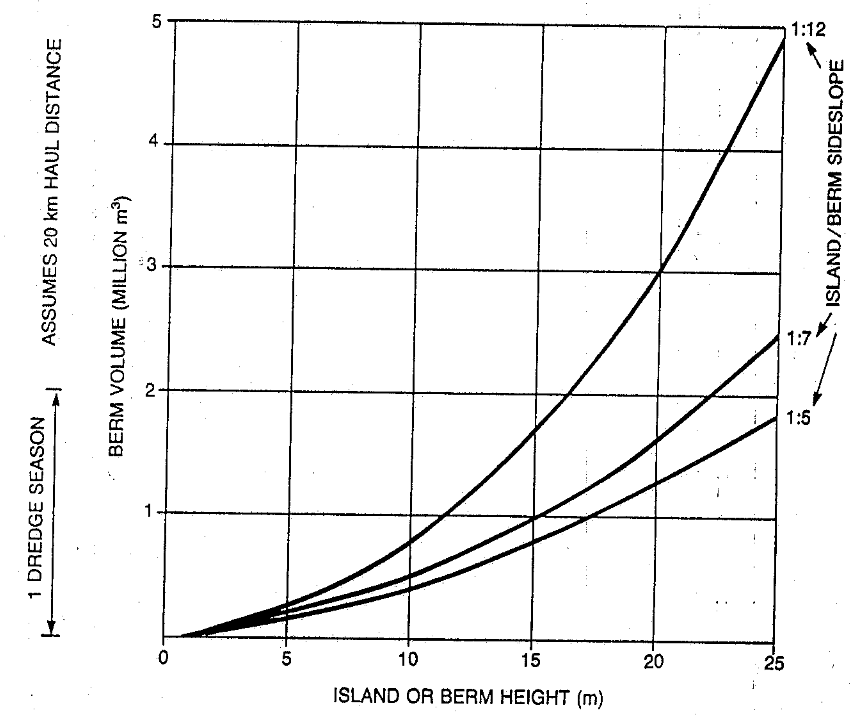
Because of this phenomenon, design action for 'Mode 2' failures has continued to use short term (undrained) strengths. No allowance is made for consolidation induced strength gain. A factor of safety of 1.5 is usually taken as appropriate for 'Mode 2'.

The previous considerations relate to the overall action of the CRI. In addition local passive failure, 'Mode 4', is considered with the requirement that the factor of safety be at least equal to 'Mode 1'.

Gravitational Stability

Offshore islands impose substantial loads on the foundations because of self-weight. Although ice loading attracts attention because of the large horizontal forces involved, the fact remains that gravitational loads are at least three times greater than ice loads. Consideration of self-weight failure, 'Mode 3' on Fig. 7, is an important aspect of island design.

The approaches adopted by ERCL and GCRL have tended to diverge with regard to foundation treatment and its effect on design sideslope. The required construction effort and cost of the island/berm is strongly a function of sandfill volume, as halving fill volume may allow substantial reduction in the dredging fleet. But, the process is not one of proportionality because dredges tend to be charged on a per season basis. Typical trailer dredge production capabilities in the Canadian Beaufort Sea over an average haul of, say, 15 km, are 2.0 million m^3 sand/dredge/season. With such a production rate and a minimum commitment of one dredge it is quite clear from Figure 8 that it is irrelevant whether one builds a 1:5 or a 1:12 sloped island for island heights in the order of 10 m. The situation is much changed when island heights exceed 20 m. Again reference to Figure 8 shows



NOTE: Assumes 6000 m^3 hopper dredge @ 20 hrs/day For 100 day open water season.

Figure 8

EFFECT OF SLOPE & HEIGHT ON ISLAND (BERM) VOLUME

that for these island heights a change from 1:8 to 1:12 slopes produces an incremental volume of approximately 1 million m^3. This requires the chartering of an additional dredge, or the lengthening of the construction period to two summer seasons. This marked difference in consequence of sideslope angle for project economics has dominated the issue of foundation treatment. Foundation treatment has been carried out at Amerk (ERCL), Kogyuk, Tarsiut N-44, Tarsiut P-45, Amauligak I-65 and Amauligak F-24 (all GCRL). Sideslope flattening was undertaken at Kaubvik I-43 (ERCL).

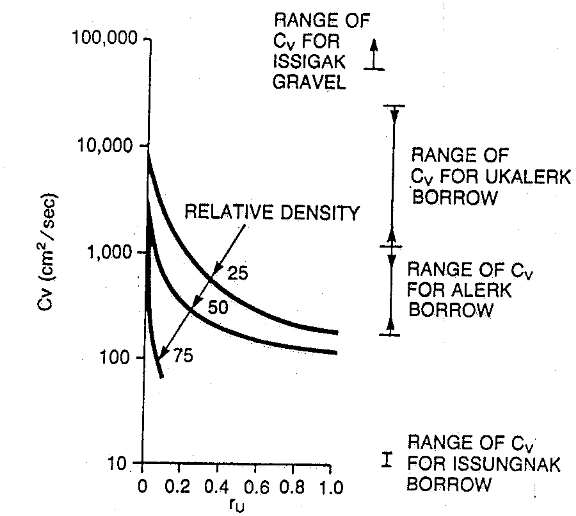
Design factors of safety have increased substantially with the increasing water depth where consequences of failure are very much more serious. In shallow water, where 'failure' amounts to slope slumping of no practical consequence to the function of the island, factors of safety less than 1.3 have been used. On the other hand, the stress concentration and monolithic nature of the Molikpaq caused factors of safety of 2.0 to be adopted against foundation failure by self-weight. Extensive monitoring of foundation performance has also been undertaken at CRI sites (Amerk P-09, Kaubvik I-43, Tarsiut N-44, Tarsiut P-45, Amauligak I-65, Amauligak F-24) to provide confirmation of predicted foundation performance.

Sand Permeability

The cyclic loading action of storm waves on island sandfills may fluidize the sand, which can then "flow", slump or become rapidly eroded. Cyclic inducing of excess pore water pressure has been analysed using one proposed method [48] for a typical 8:1 side slope berm (actually Amerk) for a typical design storm. The results are summarized on Fig. 9 in terms of the coefficient of consolidation, C_v, plotted against the predicted normalized residual porewater pressure ratio associated with the design storm.

As shown on Fig. 9, material type expressed in terms of C_v is the major factor in determining susceptibility to wave induced liquefaction. Wave induced excess pore pressures may be expected to be minimal provided C_v > 1000 cm^2/sec for the design Beaufort storm, regardless of sand density. This may be compared with a previous analysis [39] that concluded K>10 $^{-4}$m/sec for the same reasons.

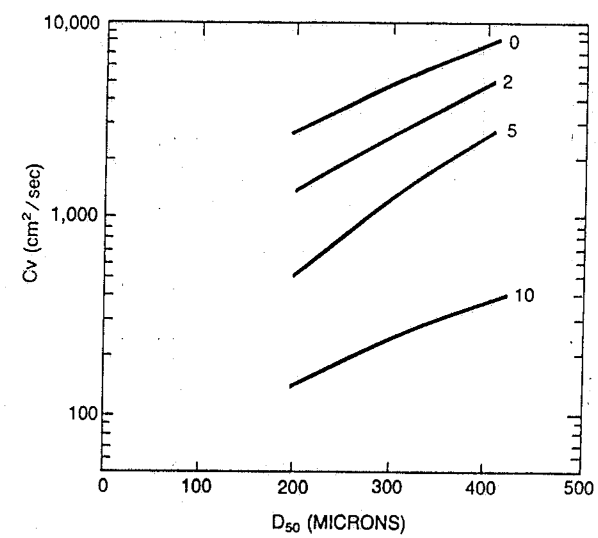
The coefficient of consolidation for sands is difficult to measure directly using standard laboratory equipment. The approach adopted was to measure the individual components, permeability and compressibility. Permeability, was measured either in triaxial test equipment or in a permeameter. Compressibility was measured in an oedometer. The range of C_v values for several Beaufort Sea sands are indicated on Fig. 9, while Fig. 10 shows the effect of grain size and silt content on C_v for Erksak sand.



NOTE: Predicted by method in ref [48]

Figure 9

WAVE INDUCED EXCESS PORE PRESSURE



COEFFICIENT OF CONSOLIDATION

Figure 10

HYDRAULIC PROPERTIES OF ERKSAK SANDS

Specifications

The specification for sand quality used on the project must result in both an adequately dilatant and permeable sand in-situ. In fact, as will be subsequently shown, both objectives are complementary as the in-situ state of sand (which controls dilatancy) is linked with the permeability of the as-placed sand for a given construction method.

A specification for use during construction must be given in terms of variables easily observed and measured during construction. This conventionally implies sand gradation, which was adopted for the island fills reported here. It became recognized that a coarse sand could tolerate more silt content for any particular mechanical properties. Specification of sand quality therefore developed into a smooth range of median grain size/silt content combinations which were subdivided into material classes, Fig. 11. This figure shows a typical specification used for a zoned fill-quality berm.

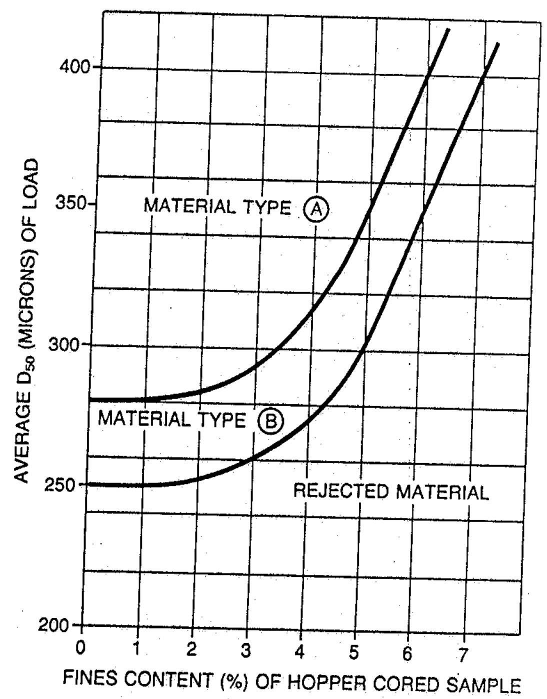


Figure 11
GRANULAR FILL SPECIFICATION

CONSTRUCTION

Once the desired berm/island geometry and sand properties have been defined, other engineering issues become important. These may be conveniently grouped into four topics:

i) sand availability and location with respect to project site
ii) construction technique including vessel selection and scheduling
iii) achieved in-situ sand density, island slopes and erosion protection
iv) potential for remedial works, if required.

The issues are all interrelated as, for example, both sand type and placement technique affect the achieved fill density. However, it is convenient to discuss each topic in turn before considering how they interelate.

Sand Availability and Location

Although sand is quite widespread in the Eastern portion of the Canadian Beaufort Sea, it is virtually absent in the Western part; as a rule of thumb, the 136°W longitude provides a divider as to those regions which may be expected to contain sand and those that do not (greater definition is contained in the physiographic studies [40]). Islands, however, have been constructed throughout the Beaufort Shelf.

Although many earlier projects used anchored dredges and local borrow, the trend since 1980 has been to use one particular borrow area and trailing suction hopper dredges. This borrow area is variously known as the Ukalerk/Erksak pit and its location is shown on Fig. 2.

The Erksak pit came into use with all three oil companies because it has shown an average grain size some 50% coarser than is typical for Beaufort Shelf sands. The coarsest, cleanest possible sand is desirable to achieve the maximum as placed density, steeper side slopes and, maximum resistance to wave induced liquefaction.

The in-situ gradation of the Ukalerk/Erksak pit is summarized on Fig. 12 in terms of median grain size, D_{50}, and silt content. Also shown for comparison are data from other, typical sand sources in the Beaufort Shelf. It can be seen from this figure that the Ukalerk/Erksak sand exhibits a median grain size of 300 - 400 microns and a silt content of <8% in the pit. Commonly at least half the silt content is lost during construction because of the washing action of the dredge.

Vessel Type and Selection

Dredges used to date may be classed into one of three types: anchored suction dredges, anchored cutter suction dredges and trailing suction hopper dredges. Figs. 13 and 14 illustrate a suction dredge and a hopper dredge, respectively.

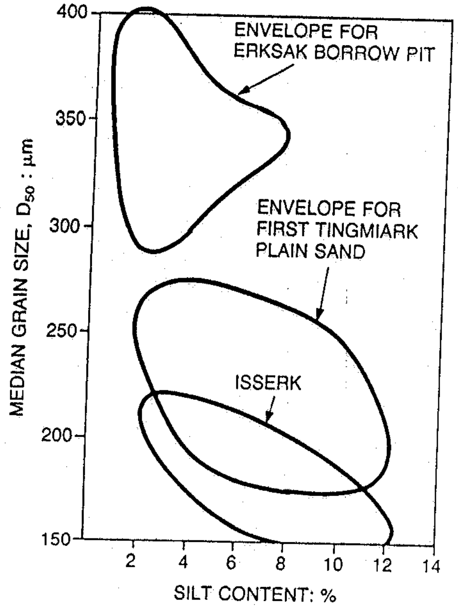


Figure 12
SUMMARY OF IN-SITU GRADATION OF BEAUFORT SEA CONSTRUCTION SANDS

An anchored suction or cutter suction dredge is moored during dredging operations and utilizes a large pump mounted on a barge or a vessel. The two types of dredge differ in that a mechanical cutter is provided on a cutter suction dredge to assist in fluidizing sand in the borrow pit; the suction dredge relies on the eroding action of the suction pipe sometimes supplemented with water-jetting. Sand, once fluidized is sucked up from the pit through the pumps and transported as a slurry in a pipeline to the project site. The pipeline may be either anchored itself at the project site providing a central spigot arrangement or alternatively, the pipeline may be connected to a construction barge having a means of directing the flow of sand. Anchored dredges require sand deposits at least 5 m thick to be effective, and these deposits should be within about 1 km of the project site. But, the delivered sand quality is not unduly sensitive to thin layers of overlying clay.

A trailing suction/hopper dredge is a vessel with the prime function of recovering material from the seabed and transporting it. A typical layout of such a vessel is shown on Fig. 14. Sand is recovered by the dredge pulling the drag arm across the seabed and pumping the

Figure 13
ILLUSTRATION OF SUCTION DREDGE

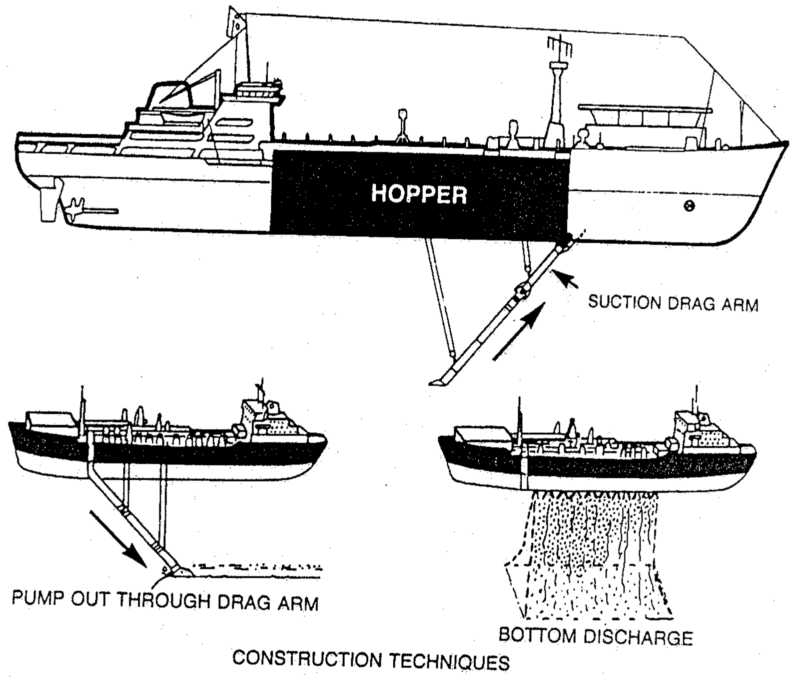


Figure 14
DETAILS OF TRAILER-HOPPER DREDGE

slurried sand into the hopper. Once the sand has been moved to the construction site it may be deposited in three ways. First, the dredge may unload the hopper using valves or doors in the base of the hopper - referred to as 'bottom discharging'. Second, the sand in the hopper may be slurried using the dredging pumps with supplementary water and discharged back down a drag arm. Thirdly, the sand may be slurried and bow discharged into a floating pipeline or through a nozzle/diffuser.

Trailer dredges operate most efficiently if sand recovery is continuous. The areal extent of the borrow should therefore typically be greater than 0.3 by 2 kms. Such sand deposits need not be thick and deposits as thin as 2 m have been worked in the Canadian Beaufort Sea. The sand deposits must be exposed at the seabed as even 0.1 m layer of overlying clay significantly degrades delivered sand quality. The clay can be stripped from potential borrow pits, but such stripping may severely affect project economics. Alternatively, abandoned islands have been used as borrow sources with the dredges either sailing around the island or anchored against the island slopes.

Sacrificial Beach Construction Method

Originally, SBI's were constructed using a stationary dredge using sand from the seabed close to the island. This approach was used in the construction of Issungnak [3]. A new construction method pioneered by ERCL at the Nipterk SBI used a trailing suction hopper dredge to haul fill to the site. Bottom dumping techniques are utilized to an elevation of approximately 8 m below mean sea level (bmsl) at which time draft restrictions prevent any further deposition. Bow and side discharging technique are then used to build up the center of the island. The drilling surface is constructed by bow discharging through a nozzle and then using land equipment to level the surface to the correct elevation. The beaches are extended by depositing material directly onto the desired area through a diffuser attached to the bow of the dredge. The development of the diffuser allows for the build up of the surface without the use of a pipeline, which is preferrable since pipeline use is weather sensitive because of hook-up requirements.

CRI Berm Construction - ERCL

ERCL carried out a comprehensive model study on the influence of sand placement technique on berm geometry. This work and the field trials that followed it are reported in detail in [41].

The ERCL berm design for their CRI comprises a berm rising from seabed to an elevation of 9 m bmsl. Sideslopes of 1:8 are desired if the foundation is sufficiently strong; this has not been the case at either Amerk or Kaubvik and berm sideslopes have been flattened to 1:15 in the lower zones so as not to overstress the clay. Excavation of soft clay was also carried out at Amerk.

The ERCL berm construction technique is based entirely on the bottom discharge technique. This technique - which provides excellent densities as subsequently documented - has the tendancy to produce rather flat sideslopes, less than 1V:20H if used indiscriminantly. The sequence developed by ERCL overcomes this difficulty by following steps:

i) the dredge is orientated radially to the island centre during discharge.

ii) a sequential ordering of discharge locations is followed with two discharges (approximately 12000 m^3) per location.

iii) the radius of the desired dump zones is a function of both water depth and berm crest diameter. In the case of the Amerk and Kaubvik islands, a dumping radius of 105 m was found effective in achieving the desired slopes.

iv) topping-off the berm is conducted with a much reduced dumping radius.

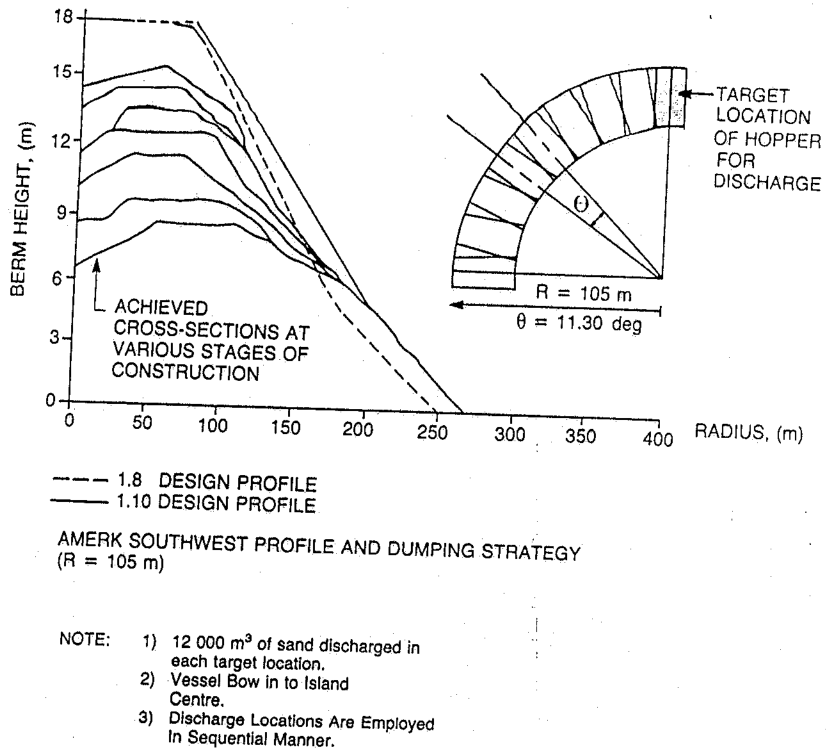


Figure 15
ERCL BERM CONSTRUCTION TECHNIQUE

The sequence of operations is shown on Figure 15 together with the achieved berm profile.

CRI Berm Construction - GCRL

The technique used by Gulf for four CRI berms (Kogyuk N-67, Tarsiut P-45, Amauligak I-65 and Amauligak F-24) amounts to an extension of the technique developed by Volker-Stevin [11] for the first caisson project, Tarsiut N-44. The original Gulf technique has been previously presented in some detail [42], but has subsequently received significant improvements [43]. The Gulf technique, in both original and upgraded form, involves sand discharge by both pump-out and bottom valve methods. Only trailing hopper dredges are employed.

The technique is based on constructing the berm in several "lifts". Each lift starts with the construction for a perimeter bund by pump-out discharge, the drag arm being positioned about 2.5 m above the sand surface and within a narrow horizontal zone dictated by the desired berm geometry. Drag arm positioning is detailed on Fig. 16. Perimeter bunds provide a means of obtaining steep slopes. Once a bund has been constructed, the centre zone is infilled by bottom valve discharge.

Subsequent modifications to the basic technique have arisen for two reasons. First, on both the Tarsiut P-45 and Amauligak I-65 projects simultaneous use of two and four dredges respectively was required and formal scheduling was necessary. Second, it was observed that attempting to fill in a low spot by bottom discharge increased rather than decreased the hole; presumably because of density-flow induced sour currents. The solution to both situations is a formal dump-zone scheduling for bottom valve discharge in a similar exactly similar manner to that used by ERCL. The scheduling sequence used at Amauligak F-24 is shown on Fig. 16.

Positioning and Surveying

The sand placement methods described above all require a precise (standard deviation better than 1.5 m) navigational/surveying system. The navigational network used has been predominantly a Syledis one, which is a shore based UHF radio ranging system. Positional accuracies for this equipment are +-4 m to a distance of 100 km offshore [44], using a minimum of 3, carefully located shore stations. On each vessel a syledis receiver is mounted whose output is transferred to an engineering workstation for enhancement and display to construction operatives (such as the vessels helmsman).

The display technology used was coloured graphic 'cartoons' which display both the current vessel or spigot position (in one colour) and that desired at the particular stage of construction operations (in a different colour). The barge master or helmsman then adjusts the vessel position to maintain an image match between what is desired and what is achieved.

In addition to the real time display of the position information, measurement of progress and verification of compliance with design geometry is necessary. Bathymetric surveys performed by independent survey vessels were to check that material was deposited in the correct location. Dual high frequency sounders (33 and 210 KH_z), adequately calibrated with velocity profile measurements, and bar checks combined with real time offshore tide gauge measurements enabled accurate and consistent surveys to be performed. Radio modems enable the corrected and reduced data to be transmitted directly from the survey vessel to the post-processing system on the dredge. This data was then used to produce the survey drawings which provide the basis of subsequent construction directives.

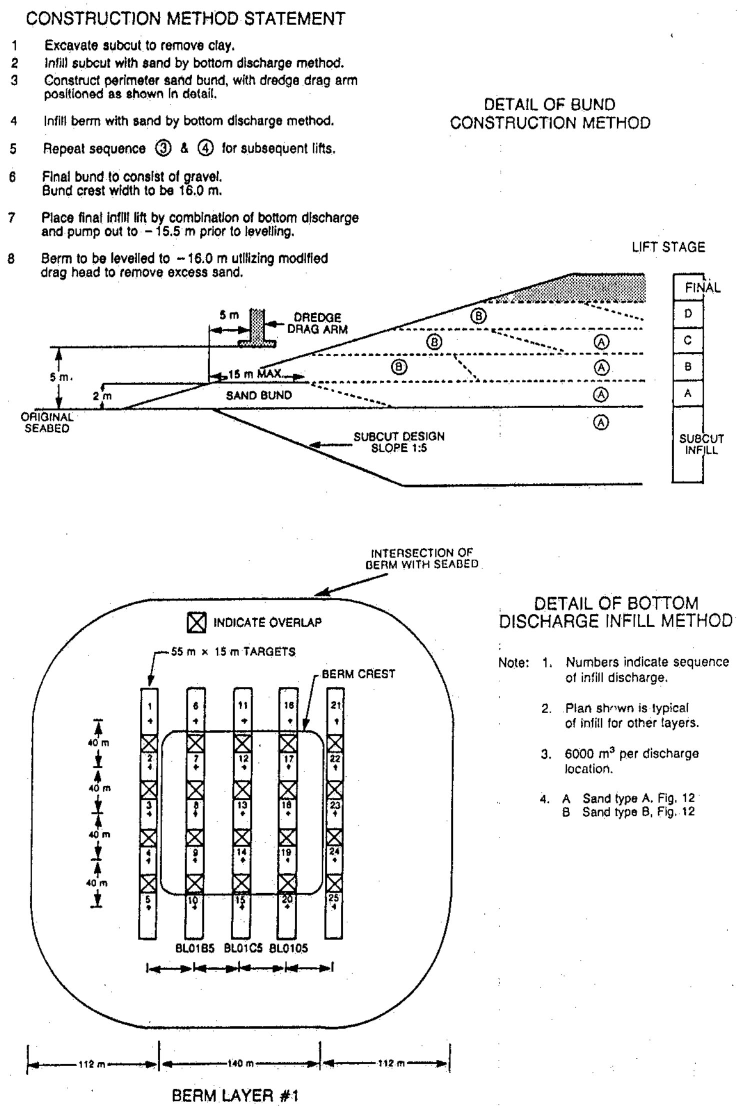


Figure 16

GCRL BERM CONSTRUCTION TECHNIQUE

Quality Control Methods

Early experience with in-situ densities achieved by purely hydraulic means led to the conclusion that better densities were achieved with coarser sand. In addition, it became apparent from both experience as well as theoretical studies [11] that steeper side slopes could be achieved with coarser sand. Considerable motivation therefore developed to construct with as coarse a sand as possible. Two sources of information on material gradation exist during construction:

- by drilling and sampling the intended borrow pit;
- by monitoring sand gradation on board the dredge.

It is usual to carry out both activities, regardless of whether suction or trailer hopper dredges are involved. Detailed borrow investigations have been made in previous years (eg: that reported in [45]). Recently, most attention has been given, to assessment of what grading of sand has been dredged, which is carried out in real-time on board the vessel. If out-of-specification material is encountered then that sand is wasted rather than placed in the island/berm.

There are a number of sampling procedures and protocols that have been used on the various dredges to date. Samples for gradation analysis have been obtained from a small spigot pipe off the main slurry intake or from sounding tubes in the hopper itself. The first sampling method gives reasonable estimates (when subsequently compared to borehole samples in the placed fill) of median grain size but markedly underestimates the silt content.

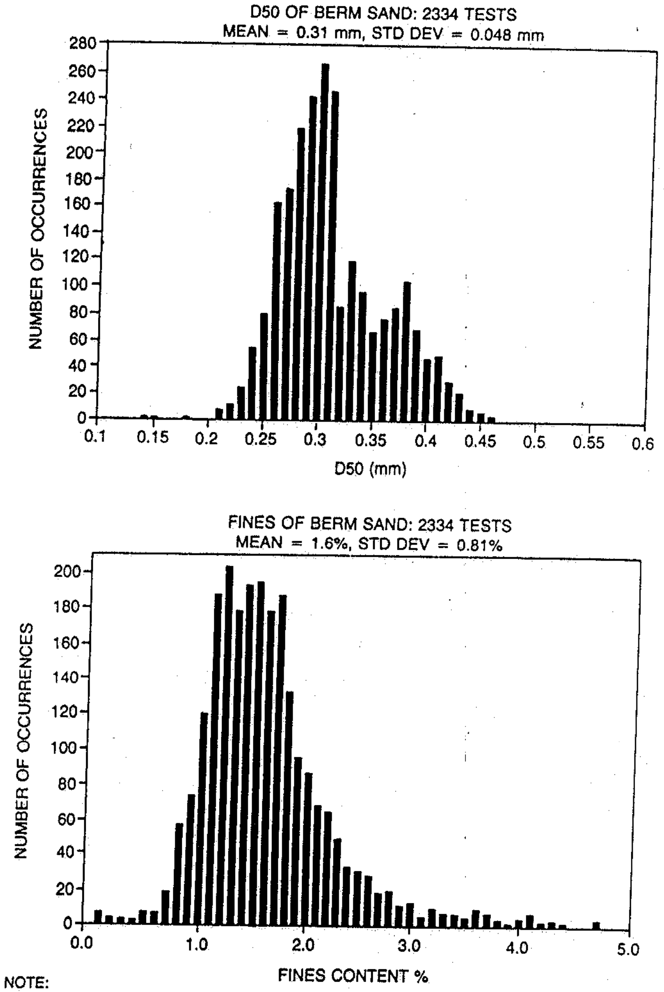
Concern over whether sampling methods were indeed representative caused GCRL to mount a drillrig to core the sand in the hopper of the dredge. Gradation analysis of samples was also advanced by using particle counting technology [46]. Together, these methods allow confidence in the material in the hopper; subsequent placement of the fill may result in minor reduction in silt content, which is desirable.

Results of these sand quality monitoring programs show a distribution in both median grain size and silt content; typical examples of these distributions are shown on Fig. 17.

Costs

Cost is an important issue that has received little attention in the published record of offshore island construction.

Some recent experience with the cost of constructing berms for CRI projects is summarized on Table 3, subdivided into accounting items. This table excludes any amortization cost of the caisson system itself and also excludes all costs of operating the drill rig. The cost picture is best understood by a simple picture, Fig. 18.



NOTE:
1 21 fines contents were in excess of 5% and are not shown on histogram (maximum was 13.5%).
2 2331 tests on Erksak, 3 tests on Amauligak I-65 sand.

HISTOGRAMS OF SAND QUALITY IN SUBCUT AND BERM

Figure 17

EXAMPLE OF PARTICLE SIZE DISTRIBUTION FROM QUALITY CONTROL PROGRAMS (AMAULIGAK F-24 BERM)

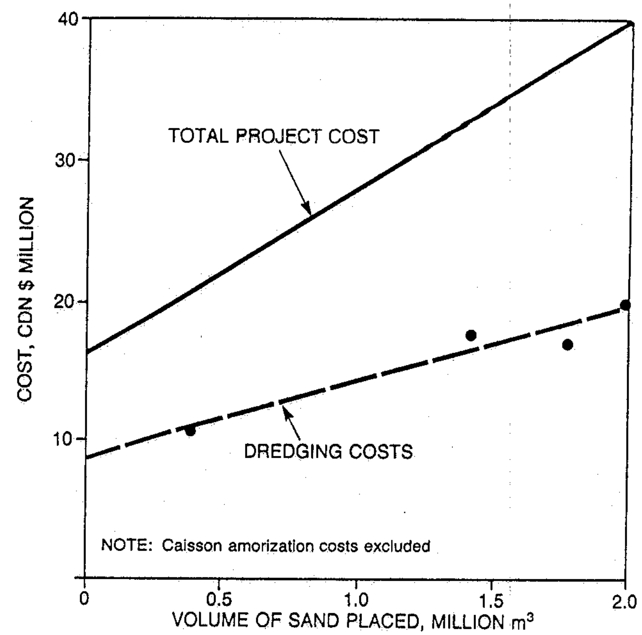


Figure 18

SUMMARY OF RECENT BERM COSTS

TABLE 3

Activity / Project	Tarsiut P-45	Amauligak I-65	Amauligak F-24
1. Berm Design	88	72	195
2. Site Investigation	427	821	805
3. Dredge Charter	10,617	17,631	19,756
4. Quality Control & Surveying	2,608	2,514	1,318
5. Densification & Dewatering	N/A	N/A	2,881
6. Support Costs	7,585	10,029	13,643
7. Post-Construction Verification	103	118	177
8. Miscellaneous Costs	466	795	1,091
TOTAL:	21,892	31,979	39,866

NOTE: 1. Costs include caisson core infill but exclude caisson amortization charges.
2. All costs are CDN$K.

It is clear from Fig. 18 that costs are not proportional to sand volume as assumed during the genesis of caisson retained islands [5]. Specifically, the cost experience shows that the project cost of earthworks in the Arctic offshore is approximated by:

$$\$ = 16 \text{ Million} + 12/m^3$$

That is, for three recent berms, the fixed cost have been in excess of 40% of total project cost without any capital amortization of the caisson system.

An identical pattern emerges for the dredging costs alone. The direct expenditures on dredge charter/operation are also plotted on Fig. 18 for four berm projects (3 GCRL, 1 ERCL) and it may be seen that the dredge costs approximate:

$$\$ = 9 \text{ million} + 5.5/m^3$$

Caution should be used if any extrapolation is made from Fig. 18 as it contains several hidden factors, the most important of which are:

- One site involved a long (>100 km) haul of sand.
- Another site involved substantial foundation treatment and costs were further increased by bad ice conditions in the open water season which forced the chartering of additional dredges.
- One project included densification and dewatering work of the hydraulically placed sand but offset this cost by particularly efficient use of dredge equipment.
- Dredge charter rates softened substantially (by nearly half) in the 1985-7 period compared to 1980-3. This reflected a worldwide surplus of dredging capacity.

In short, while Fig. 18 may provide a guide, there is no substitute for cost estimation from first principles using the concept of 'method related charges' [47] and allowing for the potential effect of external factors.

SANDFILL DENSITY ACHIEVED BY HYDRAULIC PLACEMENT

Need for Experience

The missing knowledge in the design and construction of hydraulic fill structures was how may in-situ density be controlled during hydraulic placement. No theory exists on the subject. The only available approach is to formalize existing experience and introduce this into design.

It should be appreciated that the onshore approach of specifying a design density (i.e. a 'performance specification') is unrealistic in the offshore. The dredging contractor does not have the option of making another pass with the equivalent of a compaction roller. Either acceptable results are to be achieved by the dredge on its own or a densification project is to be specified which has the effect of substantially increasing the capital cost and duration of the works. If owners attempt to lay this cost risk on contractors, then excessive bid prices are encountered. It is beholden on owners to recognize this fact, to understand the limitations and opportunities of hydraulic fill placement, and to structure construction contracts accordingly. Owners should use 'method specifications' and concept of method-related charges appears to provide the appropriate contract basis.

Measurement of Sandfill Density

Experience of sandfill density implies measurement, which is often done in terms of penetration resistance. Although the SPT is the commonest test for sandfill assessment, it has seen little use in the Beaufort Sea where the CPT has been preferred for several compelling reasons:

- Long rod lengths are required for the SPT in the offshore and the effects of these lengths on the SPT are uncertain. No such difficulty exists with the CPT which is a simple and precise test even in the offshore.
- Whereas only a few SPT's can be accomplished per hour, the same time is sufficient for as much as 20 m of CPT sounding. Further each metre of CPT contains approximately 70 individual readings. The CPT thus allows a wealth of data to be gathered in the limited time window available during offshore construction.
- Hydraulic fills are quite variable with regard to penetration resistance or density. For this variance to be understood and characterized it is necessary to have a test method that has reproducible results in standard conditions. Experience with the CPT around the world show a repeatability of end resistance, q_c, better than +-2% with occasionally +-0.5% being reported. Well controlled SPT work is at least an order of magnitude worse than this.
- Empirical correlations exist between CPT and SPT [49]. While this may appear to introduce additional uncertainty, this is not the case as using the CPT and a correlation to SPT equivalents has a lesser uncertainty that the scatter commonly encountered in SPT measurements.

These reasons were judged sufficiently compelling by all three exploration companies in the Canadian Beaufort Sea for the universal adoption of CPT soundings as the basic method of sandfill characterization. All testing since 1981 (which includes 96% of the data base) has been carried out with right-cylindrical, 10 cm^2 cones with 60 deg. tips.

CPT soundings have varied in number on individual projects from a low of 3 soundings per island to a maximum of 106. Generally, there

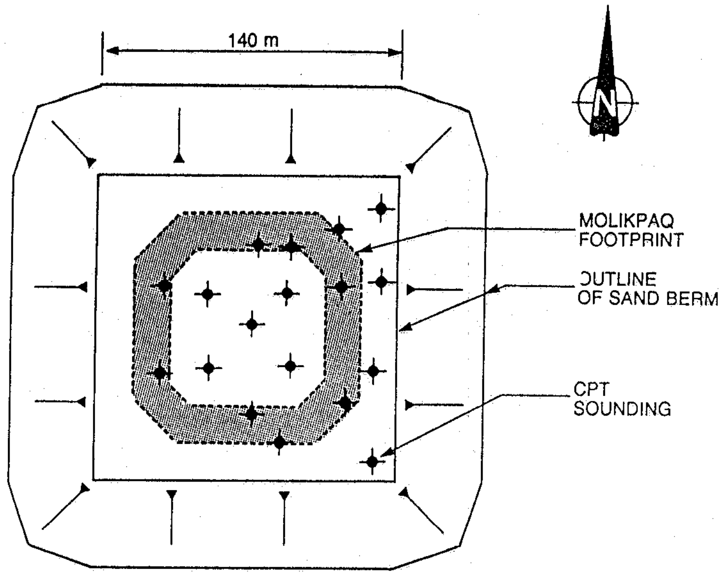


Figure 19

PLAN OF TYPICAL CPT PROGRAM FOR BERM DENSITY VERIFICATION

have been about 8 soundings at any particular stage of a project, a number which appears sufficient to provide a stable statistical estimate of sand state [50]. The layout of soundings is not necessarily regular, but rather is a regular pattern modified for convenience as required at the time. Figure 19 is typical and shows the arrangement of CPT soundings carried out to verify the density of the Tarsiut P-45 berm before placement and ballasting of the caisson onto it.

Individual CPT soundings in a 'uniform' sandfill show substantial variations in the various parameters measured, particularly end bearing resistance, q_c. A typical example of CPT data is shown on Fig. 20, which is taken from the Amauligak I-65 project. This figure shows a general increase in q_c with depth and display a local variation $\pm30\%$ of the mean value at any particular depth.

The variation in tip resistance during a sounding led to a questioning as to whether the low q_c values - which were associated with 'loose pockets' - had any significant lateral continuity. The issue was first examined in detail for the Kogyuk N-67 berm constructed in 1982 [42]. A total of 33 soundings were systematically arranged on a berm surface 140 x 140 m; it was found that no apparent lateral continuity of 'loose zones' existed at this scale. A further examination of the issue was carried out at Tarsiut P-45 [4] where a total of 32 CPT's were carried in an area 75 x 75 m. A proportion of

these tests were at 9 m centres and Fig. 21 shows the data from this close-spaced portion of the testing program. It can be observed from Fig. 21 that the continuity of 'loose pockets' in hydraulic fills is apparently quite limited spatially.

With the understanding that 'loose pockets' were a random rather than discrete phenomenon, then it is reasonable to seek a statistical characterization of fills. The abundance of data from CPT soundings allowed the state probability density function (PDF) to be calculated directly from the CPT data without assumption about the nature of the PDF [50]. It turns out that it was fortunate that such an approach was adopted because a normal distribution - which experience with other materials would suggest as a first approximation to the PDF - is, in fact, a quite erroneous representation of sand property distribution.

The characterization approach used to date involves engineering workstation based processing of CPT data [50, 51]. Individual CPT soundings are read and each scan is classified into a two-dimensional domain: q_c value and depth. The q_c discretization is at intervals of 1 MPa (ie: from 0 to 50 MPa in 1 MPa steps); the depth discretization is from top of the sounding to the bottom in (usually) 1 m steps. A simple running total of each class then provides frequency histograms as a function of depth (or vertical effective stress). The results of this procedure are illustrated on Fig. 22 for the depth interval 10 to 11 m below berm surface at Amauligak I-65 based on 8 CPT soundings. It can be seen that - as previously stated - the distribution is not normal but is in fact well skewed. Skewness changes with placement technique and stress level (see, for example [42]).

A question then arises as to how the calculated frequency distributions may be summarized. Although the tendency in offshore soil mechanics is to use a 'conservatively assessed mean' [51] as the characteristic value for a particular soil property, it is more usual in general engineering practice to use the 90 or 95 per centile exceedence criteria. Ideally, one should explore the interplay of property distribution and overall behaviour with an advanced constitutive model properly implemented in a finite element code to define the characteristic value. This has not been done yet. It was proposed [42] as a compromise that the value exceeded by 80% of the berm volume at a given stress level be taken as characteristic; as this was the lower limit at which consistent trends existed between inferred density and log vertical effective stress. The location of the 80% exceedance value with regard to the PDF is shown on Fig. 22, the more traditional median (50 per centile) also being illustrated.

CPT Data on Underwater Sandfills

Cone resistance, q_c, is strongly affected by confining stress. Strictly, it is the mean effective stress that should be used to normalize cone resistance [53, 54]. However, mean stress involves use of horizontal geostatic stress which has often not been measured and where it has been measured the accuracy has been disputed.

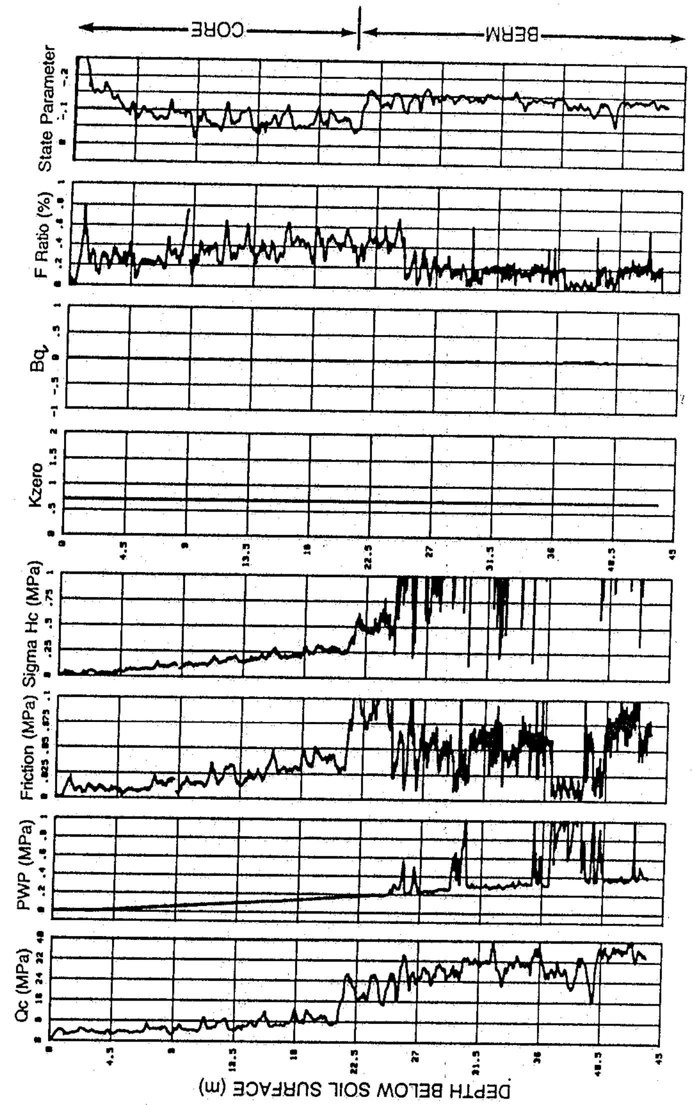


Figure 20
TYPICAL CPT DATA

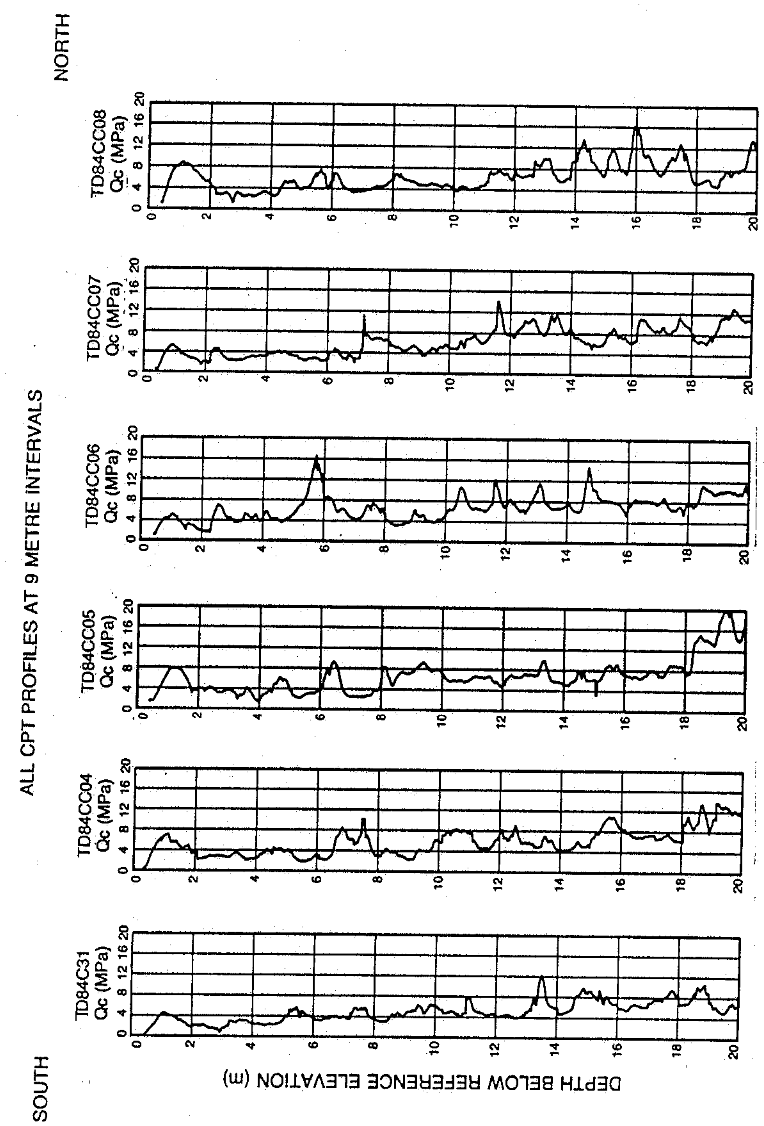


Figure 21
LATERAL CONTINUITY OF "LOOSE POCKETS" MOLIKPAQ CORE

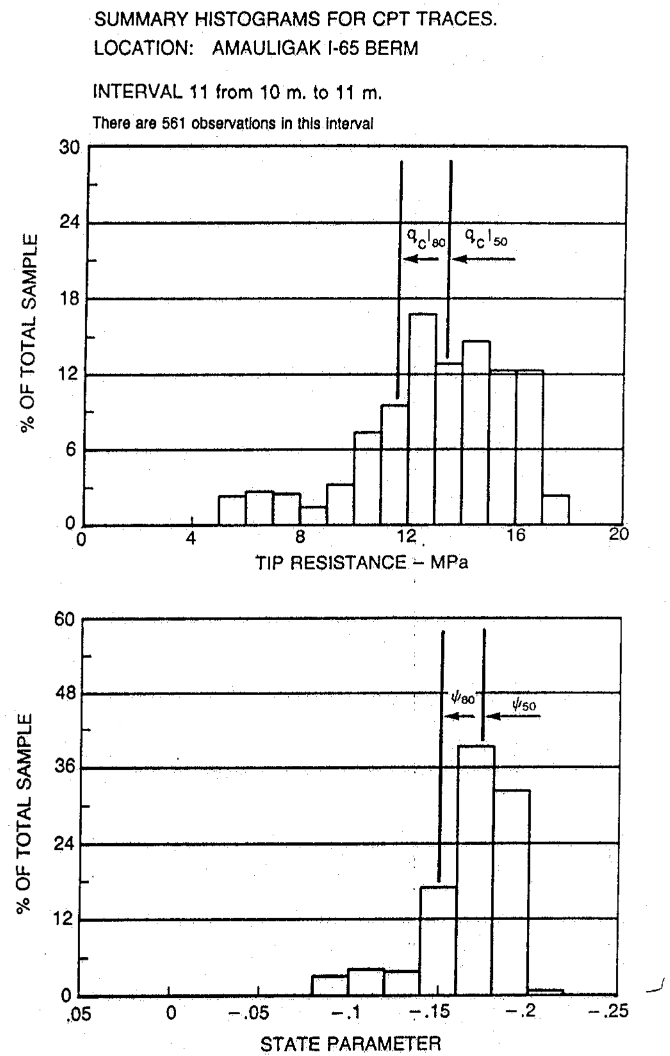


Figure 22
TYPICAL DISTRIBUTION FUNCTIONS OF q_c AND ψ

Accordingly, the q_c case history data will be presented against the vertical effective stress since that is a simple transform of piezometric conditions and the depth below sand surface subject to errors of only +-5%.

Nine of the islands constructed in the Canadian Beaufort Sea had sandfill density evaluated by CPT. In addition to these projects three trial fills have been constructed purely for evaluation of placement technique or sand quality on achieved density. The total amount of CPT data available is some 6000 m of sounding. Within this available data base, several different gradations of fill have been used as well as three distinct placement techniques. Each sand type or placement technique has been differentiated as a separate case record. The data base is summarized on Table 4. The measured CPT data that corresponds with Table 4 is presented on Fig. 23 for the 80% q_c trends.

Interpretation of CPT Data

The CPT q_c data presented above is not a direct measure of any particular sand property; rather the bearing resistance is the product of the sands initial stress state and constitutive behaviour. Extraction of a characterizing parameter for sand requires interpretation of the CPT data.

Earlier in the Paper the state parameter was introduced as an appropriate characterizing measure for sand. Considerable work was carried out on how state might be measured in-situ following introduction of the state parameter for normalization of laboratory data [53, 54, 55]. It was found that the state parameter may be inferred from CPT data regardless of sand type with a precision approximately +-10% of the range of state encountered with most sands [54,55].

The determination of state from CPT data uses the mean effective stress, p' as the stress measure and this requires knowledge of horizontal geostatic stress (or ko). Conventionally, one might calculate p' using an estimated ko from the relation ko = (1- sin Ø'). But, in-situ data obtained with the self-bored pressuremeter (SBP), has shown ko is rarely close to the conventionally expected value [56].

There is a tendency to dismiss SBP data as prone to the effects of imperfect self-boring (ie: disturbance). While it is true that perfect self-boring is rarely achieved, it is also hard to ignore the fact that similar results were obtained in some 200 tests carried out over five years by various testing companies and site investigation contractors. The average value from many SBP tests is likely to be an approximation of 'ground truth'; this average value is ko = 0.7 in hydraulically placed, undensified sandfill, about double what might be conventionally expected. Use of a conventionally expected value (ko = 0.45) is not conservative and indeed may lead to quite dangerous errors when dealing with loose fills. The interpretation of CPT data in this Paper uses a constant k_o = 0.7.

TABLE 4

PROJECT	YEAR	SAND GRAD- ATION	FILL PLACEMENT METHOD	NO. OF CPT SOUN- DINGS	DIS- TANCE SOUNDED (m)	KEY # FOR FIG. 23,24	REMARKS
Alerk P-23	1981	210/6	Pipeline Spigotted	3	52	a	undercut Delft cone
Tarsiut N-44	1981	350/4	Berm: Mainly Bottom Dumped	18	190	b	
			Infil: Spig- otted	8	96	c	
Uviluk P-66	1982	350/2	Berm: Bottom Dumped	18	368	d	wave densi fication
Ukalerk #1 Trial Fill	1982	350/2	Bottom Dumped	5	28	e	
Ukalerk #2 Trial Fill	1982	350/2	Pump-Out	10	62	f	
Kogyuk N-67	1982	350/2	Bottom Dumped Pump-Out	33 33	165 132	g h	
	1983	350/2	Bottom Dumped Pump-Out	3 3	15 15	j k	prior to 1983 const
Kogyuk N067	1983	350/2	1982 BD fill 1982 PO fill 1983 BD fill	9 9 9	36 45 54	l m n	midway through 1983 const
	1983	350/2	1982 BD fill 1982 PO fill 1983 BD fill	6 6 6	24 30 60	o p q	Carried out end of 1983 const
Kogyuk N-67	1983	350/2	1982 BD fill 1982 PO fill 1983 BD fill	6 6 6	24 30 60	r s t	With SSDC in place.
	1984	350/2	1982 PO fill 1983 BD fill	1 6	3 50	u v	After SSDC removal
Nerlerk	1983	320/2 260/2	Bottom Dump Spigotted	26 26	Total 280m	nb ns	Ukalerk s. Nerlerk sand
Isserk Trial Berm	1984	210/1	Bottom Dump	8	57	w	Silty fine sand
Amerk P-09	1984	290/1	Berm: Bottom Dump	3	58	x	Caisson in place
			Core: S	3	36	y	
Tarsiut P-45	1984	320/1	Berm: BD	18	180	aa	
	1984		Berm: Bottom Dump	7	63	ab	Caisson in place
Tarsiut P-45	1984 1985	320/1 320/1	Core: Cent. S Core: After 11 Mon. aging	32 5	784 140	ac ad	

TABLE 4

PROJECT	YEAR	SAND GRAD- ATION	FILL PLACEMENT METHOD	NO. OF CPT SOUN- DINGS	DIS- TANCE SOUNDED (m)	KEY # FOR FIG. 23,24	REMARKS
Amauligak I-65	1985	320/1	Berm: Bottom Dumped	9	182	ae	
	1985	320/1	Berm, Caisson in place	13	220	af	
	1985	320/1	Core: S	21	450	ag	
	1986	320/1	Core: After April 12 1986 event	9	190	ah	
Amauligak F-24	1987	320/2	Berm: Bottom- Dump	11	240	fa	
		320/2	Berm: Caisson in place	17	(see next row)	fb	
		350/1	Core: S	17	440	fc	
		350/1	Core: During Densification	8	210	fd	Series 'A' blasts
				18	530	fe	Series 'B' blasts
		350/1	Core: After Densification	29	845	ff	
	1988	350/1	Core: After Densification	20	430	fg	after vibro probing
	1988	350/1	Core: Check Aging	3	75	fh	
TOTAL					6949m		

S = Spigot
BD = Bottom Dump

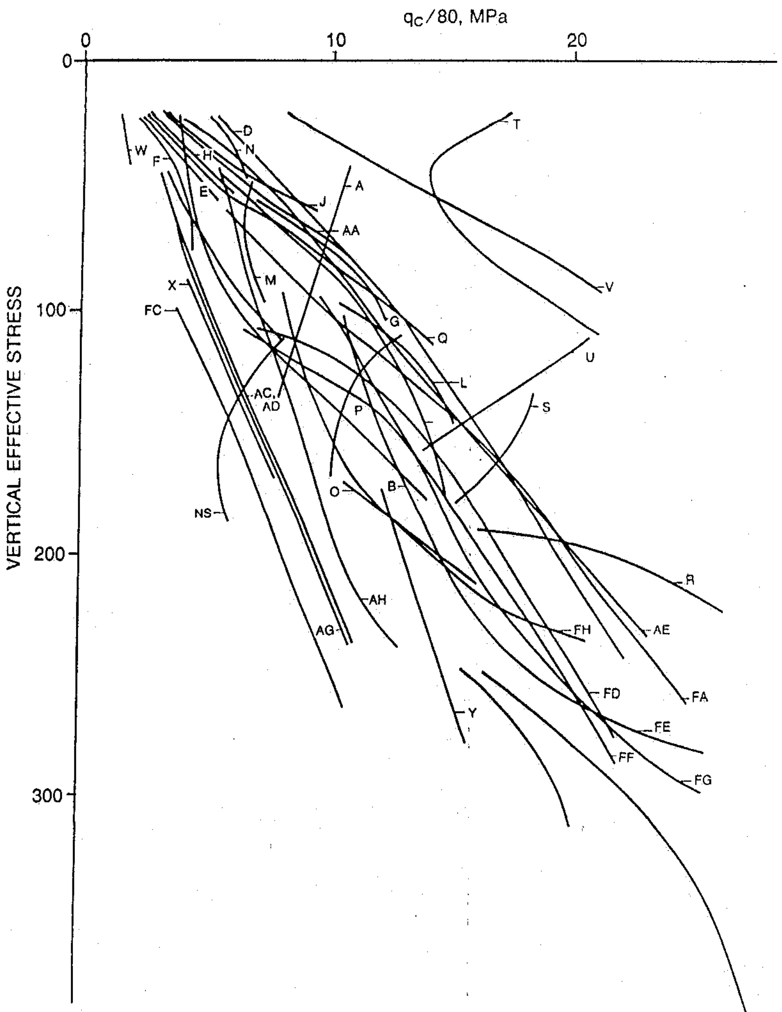


Figure 23

PLOT OF 80% q_c DATA vs VERTICAL EFFECTIVE STRESS

The method of determining state from CPT data [54] uses a scaling parameter which is the slope of the steady state line in e - log p' space, λ_{ss}. Although this parameter displays a wide range of numerical value with different sands [22, 31] considerable data on its value has been obtained for many gradations and several borrow sources for Beaufort Sea sands. Further, the general approach to CPT interpretation has been calibrated for one specific gradation of Erksak sand by chamber referrence testing [57].

The results of applying the procedure for determining from CPT data are shown on Fig. 24 which shows the same data as Fig. 23 but now transformed to state. It can be seen from Fig. 24 that a whole range of characteristic states have been obtained by purely hydraulic placement of sand. The densest fill has $\psi \approx -.18$ which corresponds to a substantially dilatant sand ($\phi' > 38°$ deg, Fig. 6). Equally, there are several cases where $\psi \approx 0.0$ which corresponds to a complete absence of dilatancy ($\phi' \approx 31°$ deg, Fig. 6). The picture is therefore a complicated one and the challenge is to identify those factors that produced substantially dilatant fills by purely hydraulic placement.

EFFECT OF CONSTRUCTION VARIABLES ON ACHIEVED SAND STATE

Factors Considered

Hydraulically placed sandfill makes the transition from a water supported condition without grain-grain contact to a grain-grain skeleton so it is natural to postulate that the process is related to consolidation. Consolidation parameters of sand vary with gradation, silt content in particular having a marked influence. However, the process of hydraulic transport also suggests that placement technique, which is a mixture of slurry density, energy, depth and erosion should be considered. To examine the influence of these various factors, the achieved state of underwater sandfills will be examined in the following steps:

i) Variation in state with water depth at constant stress level, placement technique and gradation.

ii) Variation in state with placement technique for constant water depth, constant sand type and constant stress level.

iii) Variation in state with stress level for constant placement method, constant sand type.

iv) Variation in state with sand type with constant placement technique, constant stress level, constant water depth.

Effect of Water Depth on Achieved State

Bottom dumping involves a free fall through the water column, which is a possible variable. However, several cases exist using nominally the same sand with different water depths. Specifically, the

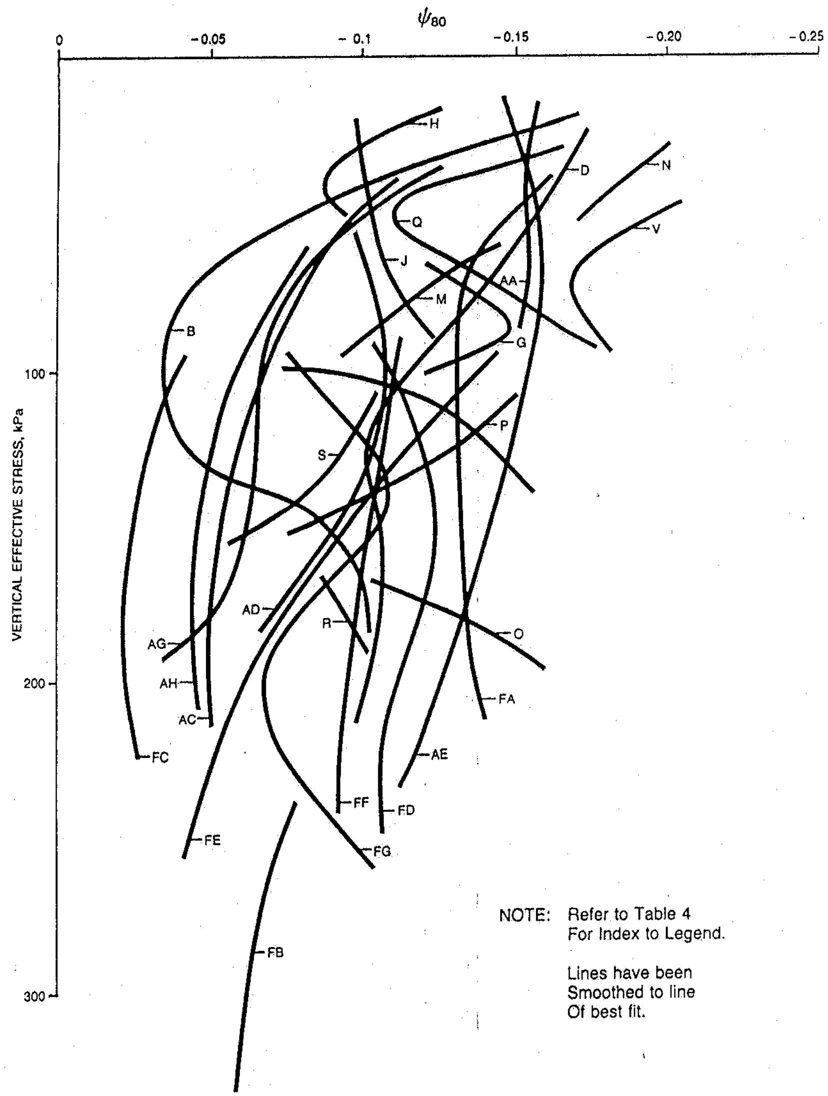


Figure 24
PLOT OF ψ_{80} vs VERTICAL EFFECTIVE STRESS

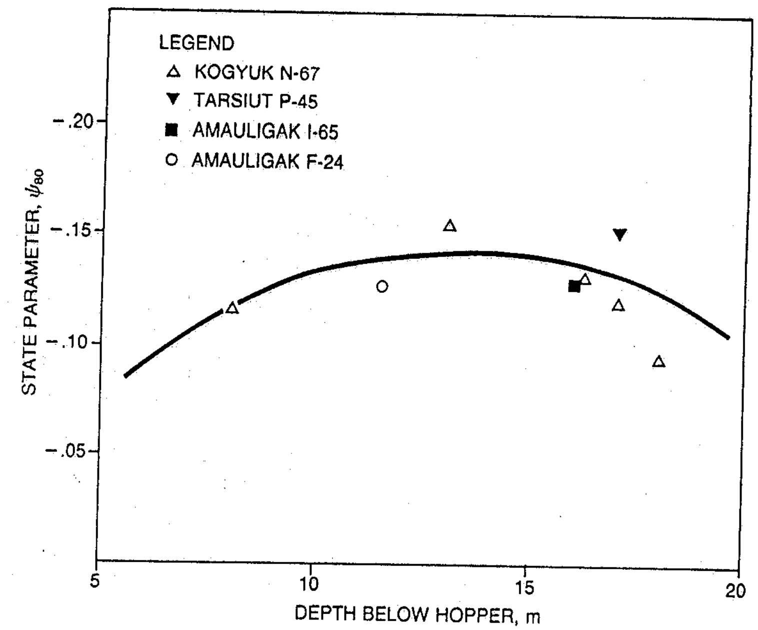


Figure 25
EFFECT OF DROP HEIGHT ON SAND STATE

1983 Kogyuk fill (10 m minimum depth) can be compared with 2 Molikpaq berms (20 m minimum depth) and the 1987 Molikpaq berm at Amauligak F-24 (15 m minimum depth). This is done on Fig. 25 in terms of the 80 per centile state. As can be seen, substantially the same state is achieved; water depth appears not to be a significant variable with the range 0 - 25 m, although there is the suggestion of possible shortfalls in greater water depths.

Effect of Placement Method on Achieved State

Three distinct placement methods have been used to date:

- Discharge directly from the base of a hopper using valves or gates; the sand descends to its final location as a dense plume. This technique is called bottom dumping.

- Discharge as a slurry from a pipe located only a few metres above the accretion surface at relatively low velocities. This may be achieved either from a floating dredge using its drag arm or by an anchored barge equipped with a tremie pipe; it is referred to as the 'pump-out' method.

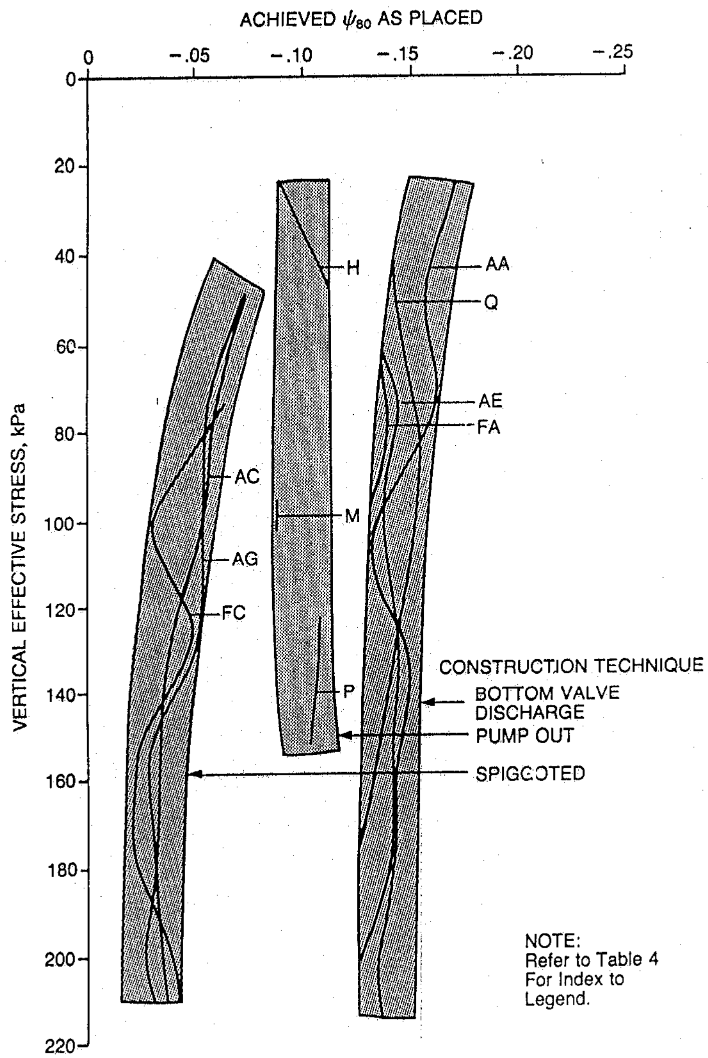


Figure 26
SUMMARY OF ACHIEVED STATES FOR ERKSAK SAND

- Discharge as a slurry at relatively high velocity (several m/s) some distance from the sand surface. Discharge may not even be underwater. This method is referred to as spigotting.

Substantial data exists on spigotted sands from four caisson infills all using to Erksak sand. These cases include Tarsiut P-45; Amauligak I-65 and Amauligak F-24 which specifically used the same gradation of sand and Amerk P-09 which used sightly finer sand from the same borrow source. The state achieved with these various case histories is remarkably consistent, as shown on Fig. 26. This figure also shows the corresponding data for 'bottom dumped' placement of the same sand. It may be observed that the difference in achieved state for the same sand is substantial; spigotting produces $\psi \simeq -.05$ at $\sigma_v' =$ 100 kpa where as bottom dumping produces $\psi \simeq -.14$ at the same stress level. Both procedures are repeatable over several case histories.

Although pump-out fill placement has seen substantial use in the three Molikpaq berms, little CPT data has been obtained because this method of fill placement is only used in the construction of retaining bunds which have no function in supporting the structure. The data base that allows comparison between pump-out and other placement procedures is restricted to the two 1982 trial fills and the upper layer of the 1982 Kogyuk berm, which was placed entirely by pump-out methods. This latter case was tested at several times as the 1982 berm was raised in 1983 to accommodate the SSDC, which provides information on pump-out fill at several stress levels. Both the trial fills and the 1982 Kogyuk berm, Erksak sand. The comparison is presented on Fig. 26 and shows that bottom dumped procedures produce a somewhat denser fill than pump-out but that pump-out is preferable to spigotting.

State as a Function of Stress Level

Soils consolidate with increases in confining stress and, if they are stressed in excess of any previous value, then consolidation will exhibit substantial irrecoverable (plastic) strains. The manner in which sands are deposited by hydraulic methods means that they are, by definition, normally consolidated and plastic strain may be expected.

All of the case histories for which CPT data exists have a significant range of stress. Unfortunately, the data was obtained at a single time and since construction in the case of bottom valve or spigot fills is carried out from near a constant elevation, the data contain the results of both a variable fall height as well as variable stress level.

Data on unarguable stress level effects does, nevertheless, exist. The Kogyuk N-67 case history had CPT testing carried out at three stages in construction, while the three Molikpaq berms were both tested on completion of construction and with the Molikpaq in place. The information on stress level effects is presented in Fig. 27, which shows that state becomes progressively more positive as stress level increases. This implies that normally consolidated sand exhibits a consolidation index, somewhat less than λ_{ss} . This conclusion corroberates the similar finding reported in [58].

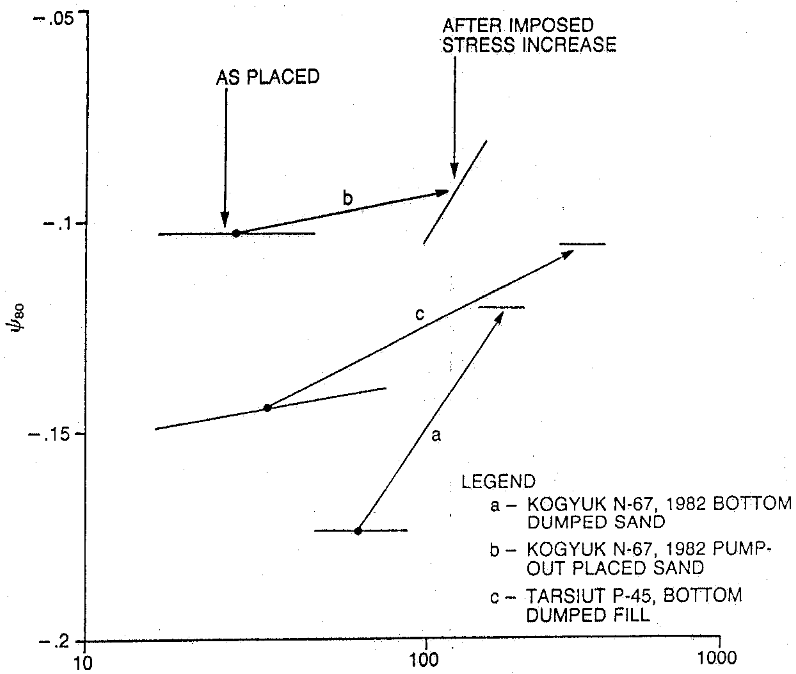


Figure 27

EFFECT OF IMPOSED STRESS INCREASE ON STATE OF HYDRAULICALLY PLACED SAND

A note of caution should be added regarding this conclusion. The analysis has been conducted with a constant k_o. There is insufficient precision in the existing data base to establish whether this is a reasonable assumption. If k_o becomes less with increasing stress level - as might be inferred from laboratory tests - then the influence of stress has been overstated.

State as a Function of Material Type

The final variable to be considered is the effect of material type. Material type is in itself not an appropriate property for comparison with achieved state; rather, material type influences permeability, compressibility and other mechanical attributes of the sand. As noted earlier, the process of changing from a slurry to sediment would be most naturally regarded as one related to consolidation; accordingly, the relationship of achieved state to a coefficient of consolidation will be explored.

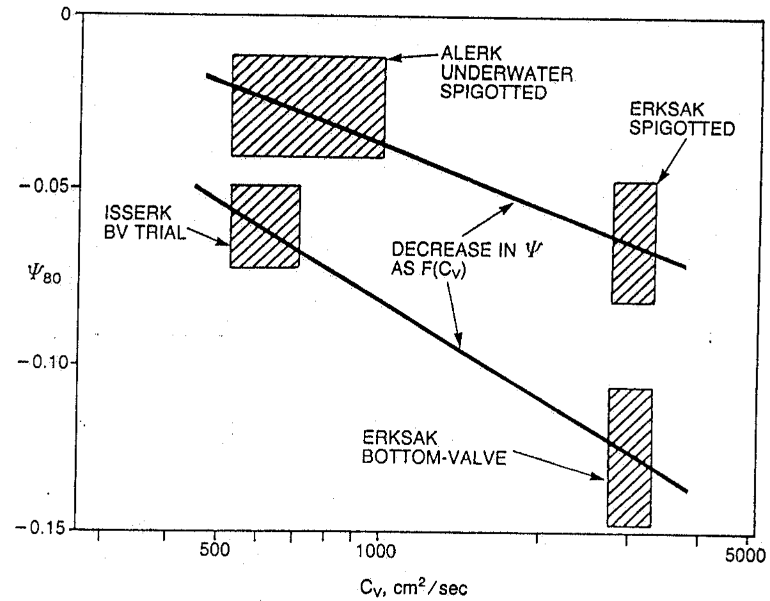


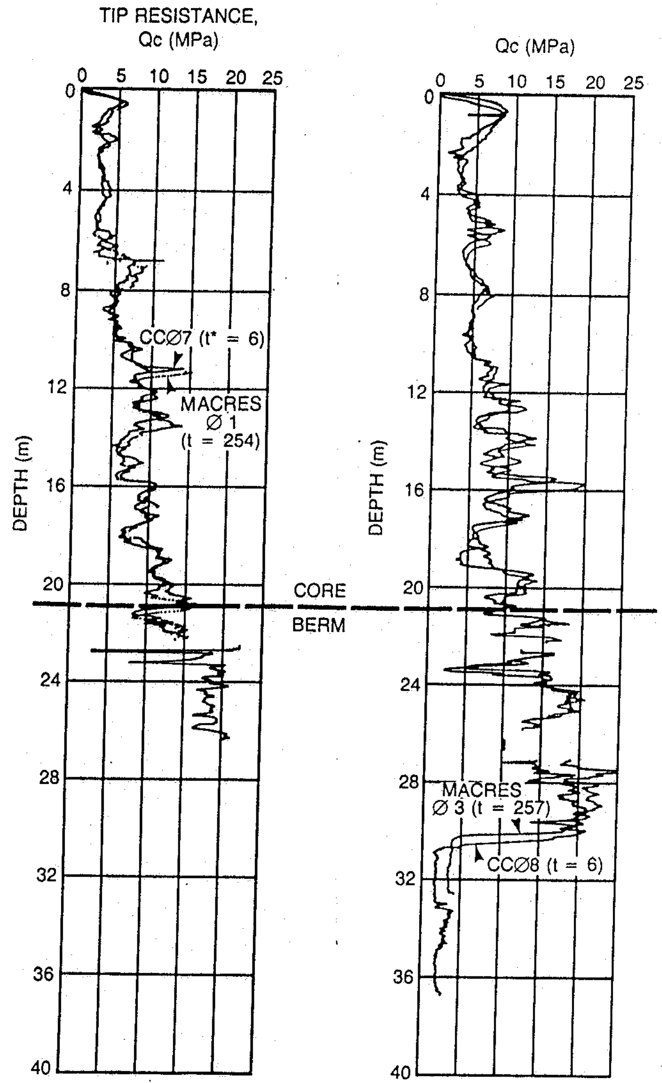
Figure 28

ACHIEVED Ψ AS A FUNCTION OF C_V

There are two comparisons we can make to gain insight on the influence of soil type. First, we can contrast bottom valve placement of Ukalerk and Isserk sands. Second we can contrast Erksak sand placed in CRI cores (spigoted) with the Alerk fill, which was spigotted from a pipeline. The results are shown of Fig. 28 and there appears to be a reasonable relation of state to coefficient of consolidation for a particular placement method. The change in state with coefficient of consolidation is similar for both placement methods.

AGING EFFECTS

There is evidence in the literature [59] to suggest that cone penetration resistance of sand increases significantly with time even when no settlement (densification) has occurred. This is a particularly interesting observation from both practical and theoretical points of view. It is interesting in the purely practical sense because doing nothing but waiting would appear to improve a fill. It is interesting from the theoretical point of view since it would imply the cone shows a substantial response to a variable other than state - which contradicts direct laboratory testing [54]. Careful tests were therefore carried out as part of the Tarsiut P-45 project to see whether any such aging effects existed with a clean Erksak sandfill over an 11 month period.



LEGEND:
– t * = time in days since completion of construction in Oct. 84.

Figure 29
EFFECT OF AGING ON SPIGOTTED ERKSAK SAND

The tests for aging were careful in the sense that CPT soundings were carried out as pairs with each member of a pair being less than 1.5 m horizontally from the other member. Five such sounding pairs were carried out. An example of the results obtained is shown on Figure 29 in terms of the raw q_c value; the data has not been subject to any interpretation. It can be seen that the q_c profile at 260 days differs from that at 6 days by less than 2%, which is essentially the repeatability between two different CPT transducers. Aging effects are not present on the timescale investigated for spigotted sandfills.

SUMMARY OF "UNTOWARD" INCIDENTS

Although 36 islands have been built, their construction has not been entirely a success story as eight islands have exhibited unexpected or untoward behaviour as summarized on Table 5. It should be remembered when reviewing these incidents that an "incident" is not necessarily a "failure". A "failure" means an occurrence which causes substantial loss of either money or life; only two such incidents might be regarded as such; Nerlerk B-67 and Minuk I-53. Nevertheless, "incidents" are a caution since a mechanism that only constitutes an incident in 5 m of water where remedial action may be trivial, can equally be a catastrophic failure in 50 m of water. Accordingly, much pertinent experience can be gained by reviewing the "untoward" incident record.

Excess Pore Pressures Induced by Construction

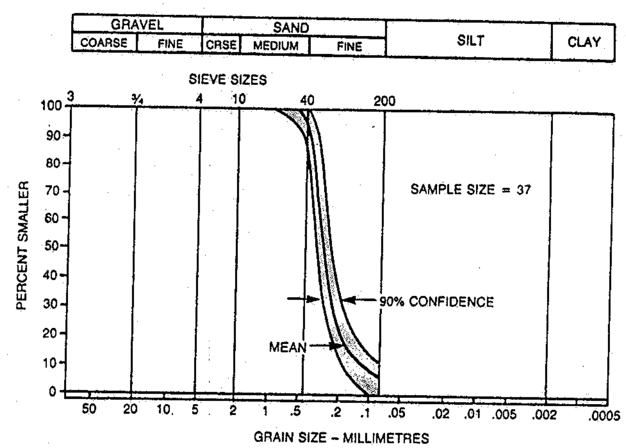
The first indication that hydraulic fill construction might be difficult using the sand and procedures available in the Beaufort Sea came from the Arnak L-30 project in 1976. This island was constructed using a typical Beaufort Sea sand which was a little siltier than those used more recently; the envelope of gradations for this island is shown on Fig. 30. The island construction was typical of the early years. The suction dredge "Beaver MacKenzie" was used to excavate local sand which was discharged via a floating pipeline at the island site. The above water portion of the island required the use of bulldozers and a backhoe to form a circular dyke which was then infilled with sand slurry. Trafficability of the fill was poor during dyke construction and the beach subsequently slumped downslope several tens of metres. This slumping was caused by transient excess pore pressures in the sand, as evinced by the sand boils observed on the beach [Fig. 30]. Since there were no outside disturbances it is reasonable to infer that these large excess pore pressures were entirely due to the sandfill and construction procedures used. The field estimated C_v based on the measured decay of excess pore pressure in a piezometer was 0.2 cm^2/sec [71]. It is difficult to explain the excess pore pressure even with this value of C_v using accreting layer theory [60]. Considerable caution should be used when assessing the transient stability of hydraulic sandfills.

TABLE 5
SUMMARY OF "UNTOWARD" INCIDENTS

PROJECT	YEAR	INCIDENT	CAUSE	ACTION
Arnak L-30	1976	Slumping of recently constructed beach	excess pore water pressure caused by consolidation of spigotted sand	Additional sand placed; island became stable as pwp dissipated
Issugnak O-61	1979	Substantial erosion of the island fill when close to waterline	Storm waves on two separate occasions; wave induced liquefaction possible factor	Additional sandfill placed and island brought into service
Tarsiut N-44	1980	Caissons misplaced by 20 m (25% of island surface)	Surveying error	Toe berm constructed to avoid foundation failure and island brought into service
Alerk P-23	1981	Nearly half of island disappeared	Postulated as wave induced liquefaction [61] triggering slumping of loose sand	Additional sandfill placed and island brought into service
Nerlerk B-67	1983	General slope failure during construction when at half height	see Text	Project abandoned with loss > $100 M
Adgo H-29	1984	Cracking and downward movement of sandfill in NE corner	Foundation failure in soft clay	Stabilizing toe berm placed and island brought into service
Minuk I-53	1985	Island erosion leading to loss of rig and supplies	Storm substantially in excess of design conditions	Rebuilt and brought into service
Amauligak I-65	1986	Partial fluidization of sand	Extended cyclic loading of platform by ice crushing	Densification specified for subsequent Molikpaq deployments

PHOTOGRAPH OF SAND BOILS – ARNAK ISLAND BEACH

(a) EVIDENCE



(b) GRADATION OF SAND INVOLVED

Figure 30

EVIDENCE OF LARGE EXCESS PWP INDUCED BY HYDRAULIC FILL CONSTRUCTION METHODS

Wave Induced Cyclic Mobility

A simple examination of Table 5 indicates that wave action was a common factor in several instances. Wave action can lead to the generation of excess pore pressure because of cyclic total stress reversals: Material type, expressed in terms of C_v, is the major factor in determining susceptibility to wave induced liquefaction [48]. Shown on Fig. 9 are the ranges of C_v for various borrow materials, and it is clear that Erksak sand is sufficiently clean and coarse (i.e. incompressible and with high permeability) to ensure that excess porewater pressures will not be developed under the "design" storm even if it is in a loose state.

Alerk and Issungnak islands were both constructed in the early 1980s by pipeline placement of local sand borrow which was relatively fine and dirty. As indicated on Fig. 9, the Issungnak sand had $C_v <$ 100 cm^2/sec and this material would have been expected to liquefy under a less severe storm than the design storm. During the construction of Issungnak, observations by field staff indicated that on at least two occasions the above water portion rapidly disappeared during storm action. Although the C_v for the Alerk material is greater than Issugnak sand, its performance in the design storm would be expected to be marginal; during 1981 half the island disappeared during a storm at the end of the construction stage. Analysis of failure survey data has led to the conclusion that Alerk experienced instability because of its near liquefiable state coupled with cyclic induced excess pore pressures [61].

Erosion

A late season storm in September 1985 resulted in major damage to the Minuk I-53 island. The storm eroded near half the island, toppled the drilling rig and swept part of the camp out to sea. However, the island had been evacuated and there was no loss of life [62]. Although the experience at Alerk and Issugnak might suggest that wave induced liquifaction was also the cause of the Minuk incident, review of available data suggests it was direct erosion by wave action.

Minuk I-53 was a sacrificial beach island. Such islands are designed by balancing the expected erosion in a design storm with the size of the beach, a very non-linear calculation that depends not only on significant wave height but also storm duration, sand size, modifications to the beach itself by the erosion process, etc. The complexity of design is further compounded by the limited history of environmental conditions in the Beaufort Sea (there are few records before the start of oil exploration in 1973). The design approach adopted for SBIs was to use physical model tests at 1:25 scale combined with numerical modelling to develop an adequate island for an estimated storm but also to recognize that the issue was one of economic risk for the operator provided drilling was not started prior to freeze-up and provided a means of evacuating personnel existed.

Post-failure analysis of the Minuk I-53 performance during the storm [63] indicated that the uncertainties in the extrapolation of model tests, the grain size of fill materials and the wave climate experienced were not critical to the incident. Excessive erosion was caused by a relatively rare storm with a duration approximating four times that used in the original design, although with a comparable wave heights.

The Minuk storm event is the first case where the design philosophy of accepting economic risk while preventing environmental damage/loss of life was tested. The investigation carried out after the event by the regulatory agency concluded that this design philosophy was still acceptable [63]. If anything, the lesson to be learned from Minuk is to be very cautious of probabilistic predictions of environmental conditions from short duration data bases.

Ice Induced Cyclic Mobility

Although ice loading is often considered as a quasi-static situation, much of the ice loading of the Molikpaq at significant load levels (ice Loads > 100 MN) has been associated with cyclic, dynamic loading [19]. These cyclic loads have induced vibration of structure at frequencies commonly in the order of 0.5 to 3 Hz and with durations as long as about 30 minutes. Although the ice induced vibrations were comparable to a moderate earthquake of extended duration, the view developed that the platform was behaving rather better than predicted in design.

Unlike the previous several years experience in the Beaufort Sea, ice conditions at Amauligak I-65 in winter 1985/86 included substantial amounts of multi-year ice whose thickness was commonly in the range 2 to 4 m. Ice loads on the structure were correspondingly greater. At 0800 hrs on Saturday April 12, 1986, a moderate size (2 km x 1 km) multi-year floe was continuously crushed against the side of the Molikpaq. This crushing caused peak loads on the structure marginally in excess of the design load (500MN) with large concurrent vibrations. The vibrations were sufficiently severe to induce fatigue of the sand core, the principal element by which the Molikpaq resists ice load, to the extent that the ability of the platform to withstand further ice loading was degraded.

The nature by which cyclic ice loads degraded platform stability can be observed on Fig. 31(a). This figure shows 10 cycles of ice loading on the structure and the corresponding response at a typical piezometer in the sand core. For any single load increase, a positive pore pressure is induced but the situation is stable in the sense that there is no 'run-away' generation of pore pressure. However, the induced pore pressure is not completely recovered during the unloading part of the cycle so that the effect of continuous load cycling is to ramp the average pore pressure upwards, as may be observed on Fig. 31(a). The overall pattern of behaviour at one piezometer is shown on Fig. 31(b). In this case, ramping continued in a linear manner at an average rate of accumulation of excess pwp of 0.8 kPa/cycle. The process terminated when the condition $p' = 0$ was reached

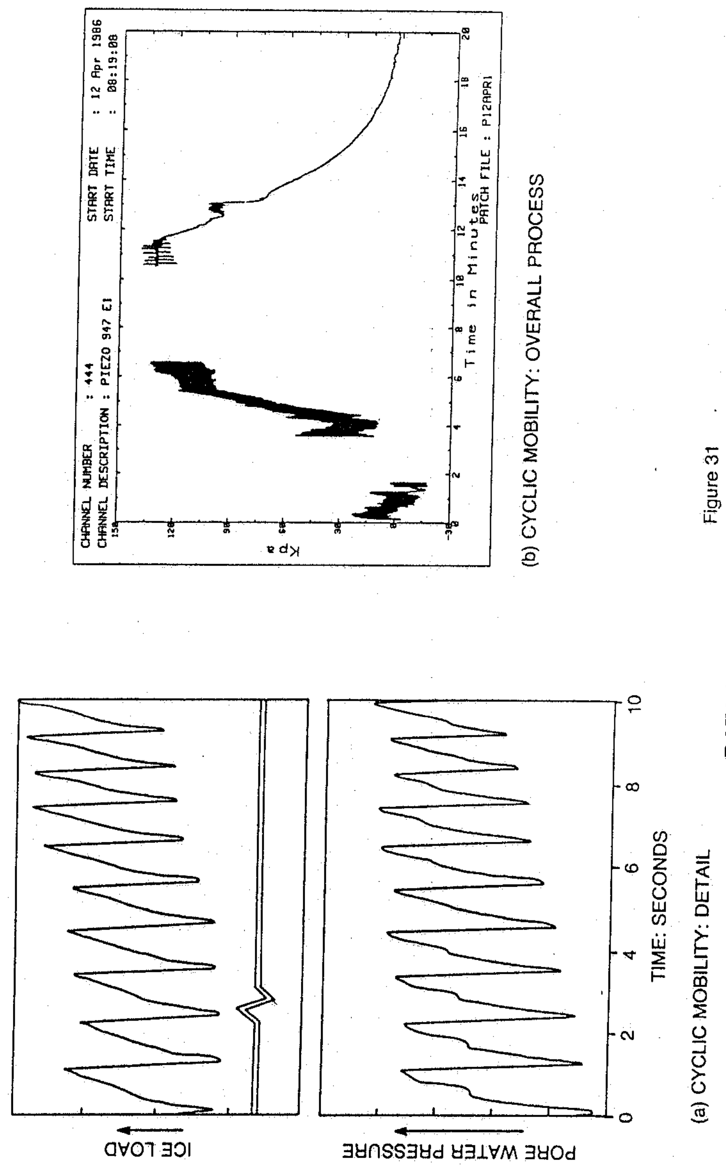


Figure 31

DATA FROM AMAULIGAK I-65 SHOWING NATURE OF CYCLIC PORE PRESSURE BUILDUP

(i.e. liquefaction as defined by Seed [33]), as may be observed on Fig. 31(b). The increase in pore pressure that subsequently took place was entirely caused by an increase in hydrostatic pressure as the sand and the piezometer settled 1.5 m.

Fluidization of the sand core was a progressive phenomenon that initiated at mid-height of the loaded side and propogated rearwards and downwards, as documented by other piezometers. Fortunately, the ice action that caused fluidization ceased before pore pressures had risen sufficiently to cause overall platform instability.

The data obtained at Amauligak on the nature of excess pore pressure generation by cyclic loasing is presently being studied under a joint National Science Foundation/National Research Council project. The results of this study will be eventually published, but for the present it should be noted that the Amauligak data shows that:

i) Lightly dilatant sands ($\psi \approx -.05$) have substantial reserves of cyclic strength.

ii) Excess pore pressures continue to be generated as long as cyclic loading occurs.

iii) Redistribution of pore pressure occurs on the same timescale as cyclic loading; the assumptions inherent in the 'steady state' concept of minimum assured strength [31] are false.

Large Strain Strength of Sand

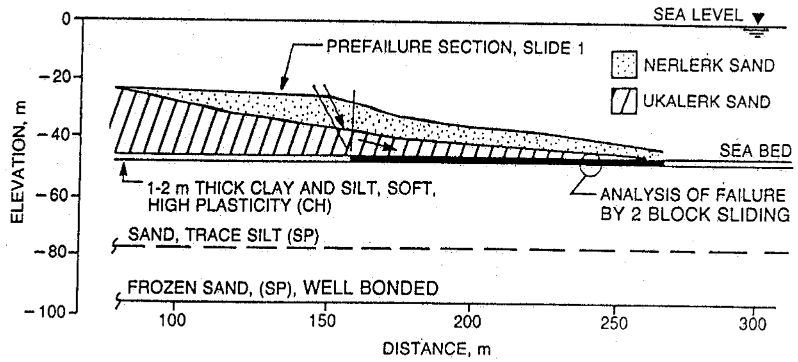
Nerlerk is the major failure in the Arctic offshore. Nerlerk was proposed as a 35 m high berm for the SSDC; unlike the previous successful berms it was to be built using the powerful suction dredge, Aquarius, and a local sand that was both finer and siltier than the precedent experience. It was also placed by a previously unused nozzle technique [65]. The Nerlerk sand was somewhat loose although examination of Figure 23 illustrates it was comparable to that at Alerk.

Construction at Nerlerk had to be abandoned because of large scale slope failures at a berm height well short of design with a direct loss exceeding $100 M. Specifically five slides occurred in the berm when it was at 25 M above seabed [63]. Fortunately, failure occurred before the SSDC was ballasted onto the berm.

A cross-section of the Nerlerk berm immediately prior to the first slide is shown on Fig. 32 together with a typical failure mechanism used in back analysis. Analysis is complicated by the difficulty in distinguishing between three potential failure modes:

° extrusion of the underlying soft clay which initiated sand slumping

° static liquefaction of the sand under self-weight



SECTION THROUGH NERLERK BERM BEFORE FIRST SLIDE

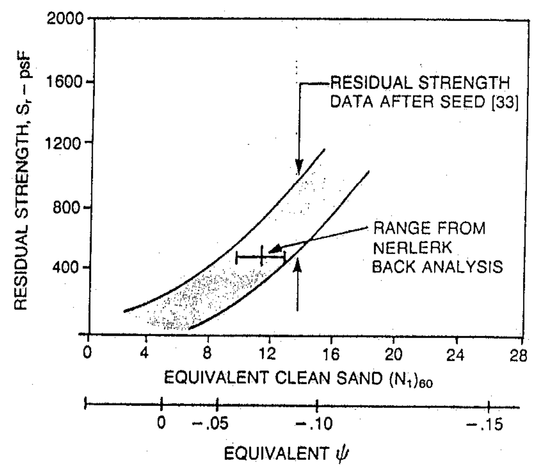


Figure 32
SUMMARY OF NERLERK BACK-ANALYSIS

- transient instability because of large excess pore water pressures induced by the construction method.

Considerable discussion has occurred in the literature on the Nerlerk failure [66, 67]. On one hand [66] it has been asserted that Nerlerk failed by static liquefaction of the sandfill in the sense defined by Castro & Poulos [30]. This assertion depends on the morphology of the failure as diagnostic of static liquefaction. The validity of morphology for this purpose has been disputed [67] and a recent review of the data base on fill failures [68] concludes that static liquefaction has actually never occurred to date for an underwater sand fill.

Caution should be used when considering the "results" of Nerlerk back analysis. As pointed out [67] there is no data on the undrained behaviour of Nerlerk sand at the lightly dilatant densities, there is no data on the drained behaviour of Nerlerk sand, there is no data on the clay strength in the soft foundation layer.

The clays and sands still exist at the Nerlerk site. There appears to be considerable merit in obtaining the relevant samples and conducting a thorough study of the failure. The Nerlerk case history is at least as important as the Amauligak ice event, as Nerlerk probably can define the limiting strength of sand with reasonable precision once all the soil data is known.

Until Nerlerk is thoroughly analysed, the fact that apparently lightly dilatant sand slumped in near-undrained loading suggests that prudent engineering should consider $\psi < -.1$ as the limiting condition for design of saturated sandfills. This is in accordance with a recent summary study of large scale sand strengths [33].

DENSIFICATION

Concern over the adequacy of undensified sand when exposed to cyclic ice loads existed from the early days of CRI type platforms. The approach adopted was to construct an undensified fill and then test it for adequacy, contingency plans being drawn up prior to construction so that densification could be undertaken if required. Examples of contingency plans are:

- 4 vibroprobes were mobilized to the Tarsiut N-44 site in 1981;
- an explosive densification method statement was engineered for the Molikpaq core at Tarsiut P-45;
- a high-powered vibroneedle was mobilized to Hay River and maintained on standby during construction of the Amauligak I-65 berm.

In all the above circumstances density verification testing showed the design target density had been achieved by purely hydraulic placement and no contingency plan was ever executed.

The view on what comprised an adequate density in the core of the Molikpaq changed substantially after the April/86 ice loading experience at Amauligak I-65. The manner in which pore pressures were generated during the April/86 ice load event strongly suggested that sliding of the platform would have occurred had cyclic ice loading continued. This conclusion about the adequacy of the fill under cyclic loading was at marked variance with the original reliance on the steady state concept supported by centrifuge models [4]. However, the original design had recognized uncertainty in behaviour of sand which led to the installation of fast response piezometers and a high speed data acquisition system. It was the data obtained by this system that forced the change in views, in itself an interesting example of the role of instrumentation in civil engineering works.

The Molikpaq was deployed at Amauligak F-24 in September, 1987 and densification of the core by blasting was adopted. Blasting was carried out in two 'passes', each pass consisting of holes drilled at 6 m c/c on a rectangular grid. The two passes were offset diagonally by 4.2 m . The plastic-cased holes were charged with 3 x 3.6 kg charges of 'Nitropel' explosive separated by gravel stemming. Holes were shot in groups of typically 14 using delay detonators to both minimize shock transmission to the steel structure and to prolong the duration of vibration. Extensive details of the densification methodology may be found in [69].

The costs for the explosive densification approximated $25/m^3 and some 65,000 m^3 of sand were so treated. The work was substantially completed in only sixteen days on site, using two rigs operating 24 hours/day. Although these costs might seem quite significant, it should be noted that offshore Arctic construction usually shows cost factors between two to three times greater than expected costs in onshore, southern locations.

CPT soundings were carried out before, during and after blasting operations to provide data that blasting had produced the required densification. Some of this data is reproduced on Fig. 33 which shows the q_c and state profiles for the as-placed core and after the first pass of blasting (which produced most of the densification) was completed. As can be seen, the blasting has near-doubled the q_c values. The computed state profiles are also shown and it can be seen that state has typically changed by -0.07, a value which correlates well with the measured surface settlement of 0.6 m at the end of the first pass and a densification zone of about 16 m. Further details on settlements and achieved CPT resistance may be found in [69]. Perhaps the most important feature of the densification is that even after considerable effort the characteristic state of the undensified, hydraulically placed berm is more dilatant than the densified core as may be seen by comparing Fig. 33 with Figs. 26, 24 and 23.

Densification of the Molikpaq core was not limited by considerations of temporary instability. The fill densified was entirely contained by the caisson so that the sand fluidization caused by the densification process could not lead to short term instability.

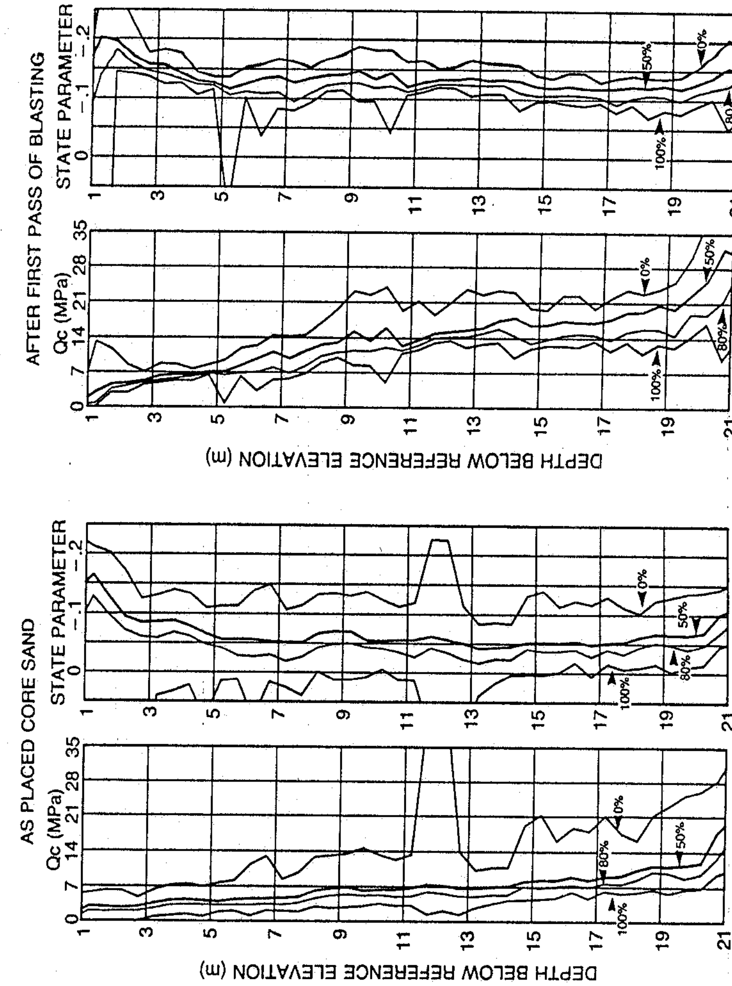


Figure 33
EFFECT OF EXPLOSIVE DENSIFICATION AT AMAULIGAK F-24
% BY VOLUME GREATER THAN VALUE SHOWN

Obviously, this would not be the case during densification of a berm or island where densification might well provoke a failure. Thus, although explosive densification was successfully applied to a clean hydraulically placed sand and gave the expected results, it should not be assumed to be an appropriate treatment for all hydraulically placed sands without proper consideration of temporary, transient conditions.

CONCLUSION

Experience gained in the construction of some 36 artificial islands in the Canadian Beaufort Sea has been reviewed and summarized. This experience shows that it is possible to repeatably construct quite dense fills purely by hydraulic placement of sand: specifically, this may be achieved by 'bottom dumping' of a medium sand with no more than five per cent silt from a trailer hopper dredge. Other placement techniques produce looser fills, also in an apparently systematic manner.

A central issue of hydraulic fills is that of adequate density, a particular concern when dealing with saturated hydraulic fills that may be subject to shock or cyclic loadings. The Amauligak I-65 and Nerlerk B-67 case histories were reviewed and force the conclusion that while dilatancy is a necessary criterion for stable behaviour of loose sands, it is not sufficient. The question then arises as to what is sufficient.

The state parameter approach was reviewed and its applicability to sandfill engineering summarized. This approach has particular relevance to steady state strength because the state parameter is defined with respect to the steady state locus in e - p' space. On the basis of the existing data and for vertical effective stress $\sigma_v' < 300$ KPa the condition that the steady state strength be at least as great as the drained strength will be met provided the characteristic state parameter is $\psi = -0.1$ or more negative (more dilatant). This corresponds to the condition that the total volumetric strain be dilatant at peak stress ratio, rather than just the dilatancy condition (which is the rate of volumetric strain change with shear strain change).

Both the existing data of full-scale steady state strengths as well as a simple critical state framework show that the strength may be expected to change exponentially with state or density. For typical quartz sands with only a few per cent silt content, the reduction in strength will be an order of magnitude for every 0.05 change to less dilatant states. This extreme sensitivity of strength to state, when combined with the state measurement precision of +-0.03, means that prudent engineering with saturated sandfills should involve characteristic states that are at least as dilatant as $\psi = -0.1$ regardless of any consideration of required steady state strength.

Selection of sand type to be used in construction is obviously an important decision. The Paper has illustrated the case for sand selection on the basis of the coefficient of consolidation, c_v. This presents a particular uniformity because, whether one assesses Beaufort Sea island construction in terms of construction induced pore pressure, or wave induced pore pressure, or achieved undensified state, the same value emerges as the criterion of a good construction sand: $c_v > 2000$ cm^2/sec. Fundamentally, this value should be project specific to the extent that the existence of excess pore pressure must also scale with fill size (drainage path), fill placement rate and slurry density. Nevertheless, these latter factors have a much smaller range of values in full scale construction than c_v for sand, so that use of a criterion that $c_v > 2000$ cm^2/sec is a reasonable starting point for other projects.

The uncertainty in extrapolating the Beaufort experience to other situations indicates the importance of developing a theoretical basis for the consolidation behaviour of sand. The experience in the Beaufort shows that a whole range of densities can be repeatably constructed, all of which comprise "normally consolidated" sands. While some of the important factors have been identified the fact remains that there is no theory for predicting the achieved density in terms of fundamental variables such as rate of accretion, drainage path, c_v, slurry density, etc. Until such a theory exists and has been calibrated against full scale data, a significant restriction will exist on the confident engineering of hydraulic sandfills.

The restriction on extrapolating experience was overcome in Beaufort island construction by a three step approach:

- build using best estimated materials/procedures;
- assess what state has been achieved on completion of construction by testing;
- have contingency plans in place for remedial activities in case they are required;

which in effect is an implementation of the much quoted "observational method".

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Influence of placement method on the *in situ* density of hydraulic sand fills

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The range of densities achievable by hydraulic placement of sand straddles the boundary between values giving acceptable potential performance and those giving unacceptable potential performance. This has led to concerns over the safety of structures using hydraulic fills, such as the artificial drilling islands in the Canadian Beaufort Sea. Liquefaction failures of hydraulically placed sand have occurred at four or more of these islands. Until recently, the factors affecting *in situ* density were little understood. Data obtained from several artificial islands are presented and these are used to demonstrate the overwhelming influence of method of placement on *in situ* density. The possible reasons for this influence and the implications for design are discussed. Recommendations are made for research that, together with conclusions drawn in the paper, should allow hydraulic fills to be used with more confidence in the future.

Key words: sand, hydraulic fill, liquefaction, cone penetration test.

Le domaine de variation des densités qui peuvent être atteintes par mise en place hydraulique du sable chevauche la limite entre une performance potentielle acceptable et une inacceptable. Ceci a causé certaines inquiétudes quant à la sécurité des structures utilisant des remblais hydrauliques, telles que les îles artificielles dans la mer canadienne de Beaufort. Des ruptures par liquéfaction du sable mis en place par la méthode hydraulique se sont produites dans au moins quatre de ces îles. Jusqu'à récemment, les facteurs affectant la densité *in situ* étaient peu compris. Les données obtenues sur plusieurs de ces îles artificielles sont présentées et elles sont utilisées pour démontrer l'influence prépondérante de la méthode de mise en place sur la densité en place. Les causes possibles de cette influence et les implications pour la conception sont discutées. L'on y fait des recommandations de recherches qui, combinées avec les conclusions tirées de cet article, devraient permettre dans le futur l'utilisation des remblais hydrauliques avec plus de confiance.

Mots clés : sable, remblai hydraulique, liquéfaction, essai de pénétration au cône.

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Introduction

It has been recognized for some time that *in situ* density is important to the behaviour of hydraulic fills and that the range of densities achievable by hydraulic placement is large. The factors affecting the *in situ* density achieved have been little understood, however. It is intuitively reasonable that variations in density are related to placement method. This paper examines the effect of placement method on density. Specifically, data are presented concerning the *in situ* density of hydraulically placed sands used in the construction of platforms for hydrocarbon exploration in the Canadian Beaufort Sea. This database is of particular interest, as the performance of these platforms, placed using various techniques, has not always been satisfactory. Clark and Jordaan (1987) reported that 25% of arctic offshore islands have experienced unexpected geotechnical problems. It is shown that placement technique has an overwhelming influence on *in situ* density. On the basis of this review, conclusions are drawn that have implications for future practice, and shortfalls in the present state-of-the-art are identified. Recommendations are presented for future research that should lead to increased confidence in the usage of hydraulic fills. Before the presentation of the Beaufort Sea database, background information concerning the density of hydraulic fills is reviewed.

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In situ density of hydraulic fill—general

It is of interest to compare the placement of hydraulic fills beneath water, which involves settling from a slurry, with the basic test methods used to establish maximum and minimum void ratios for sands (i.e., gentle pluviation versus compaction on a vibrating table (ASTM 1982)). Since there is a large difference in energy input in the basic test methods, it would seem implausible that any form of hydraulic placement could result in an *in situ* void ratio that is significantly closer to the minimum than to the maximum. Indeed, it is intuitive to expect that material pumped as a slurry from a pipeline and allowed to settle gently through water would have a void ratio close to the maximum. It would be reasonable, however, to expect that if a free-draining sand were placed using a technique that imparted some compactive effort, a denser condition might be achieved.

Various workers have presented data concerning the *in situ* density of subaqueous hydraulic fills, including Hanzawa and Matsuda (1977), Hall (1962), Basore and Boitano (1969), Osterberg and Varaksin (1973), Steurerman and Murphy (1957), Whitman (1970), Johnson *et al.* (1972), and de Groot *et al.* (1988) (Beaufort Sea experience excluded). For a general review of the use of hydraulic fills, the reader is referred to Morgenstern and Kupper (1988).

It is not possible within the context of the present paper to present a detailed review of literature concerning the *in situ* density of hydraulically placed sands. The following

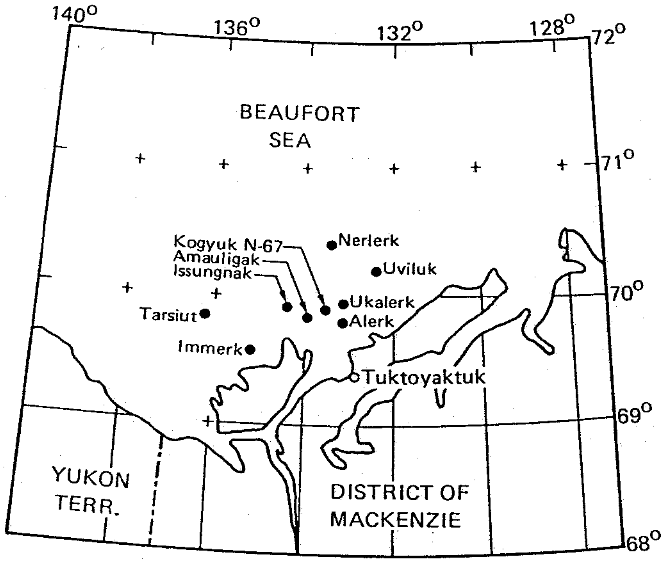

FIG. 1. Location of some island sites and the Ukalerk borrow source in the Canadian Beaufort Sea.

points, however, can be considered representative of worldwide experience in the use of subaqueous hydraulically placed sand fills:

(1) Dense sand cannot be obtained by simple hydraulic placement. (*In situ* densification has been a commonly accepted practice in land reclamation projects.)

(2) At best, average relative densities of up to 60% can be achieved by hydraulic placement but generally they will be less than 50%.

(3) Relative density within a given fill can vary from about 10 to 70%.

(4) The factors affecting *in situ* density are little understood.

It appears that the range of densities achievable by subaqueous hydraulic placement of sand straddles the boundary between values giving acceptable and unacceptable potential performance. As rightly stressed by Morgenstern and Kupper (1988), the majority of hydraulic fill structures have performed satisfactorily and the whole concept of hydraulically placed structures has often been unfairly tainted by the failure of a few.

Failures, however, have occurred. The most dramatic failures have involved liquefaction. The term "liquefaction" in this paper is used in the sense defined by Castro *et al.* (1982). The factors leading to major failures are often contentious and the subject of liquefaction of hydraulically placed sand is no exception. Most liquefaction failures have been triggered by some form of shock loading; several case histories have been reviewed by Castro *et al.* (1982), including the lower San Fernando dam, the Sheffield dam, several tailings dams in Chile, and the Fort Peck dam. Repetitive loading resulted in the liquefaction of part of the core sand of the Molikpaq, an offshore platform in the Canadian Beaufort Sea (Stewart and Hodge 1988). There are, in addition, several well-documented cases of liquefaction induced simply by gravity loading (i.e., Koppejan *et al.* 1948; Andresen and Bjerrum 1967; Cornforth *et al.* 1973; Karlsrud and Edgers 1981; Sladen *et al.* 1985*a*). The challenge that must be met—if undensified hydraulic fills are to be used with confidence in the future—is to explain these failures in view of the satisfactory performance of other fills.

On a cautious note, it should be pointed out that satisfactory performance judged by absence of catastrophic failure does not necessarily mean that a certain hydraulic fill existed in a nonliquefiable state. By its nature, liquefaction involves loss of strength. It follows that the peak strength must be greater than the residual, or steady state, value, and for failure the peak strength must be exceeded. Certain fill geometries and loading conditions may not lead to mobilization of peak strength even if the sand is liquefiable. Little work has been done to define the peak strength of very loose sands, although the subject has recently received some attention (Sladen *et al.* 1985*b*; Kramer and Seed 1988; Sladen and Oswell 1989).

Use of hydraulically placed sands in the Beaufort Sea

The first Beaufort Sea Island, Immerk, was constructed in 1972 in 3 m of water using a 60 cm diameter suction dredge. Figure 1 shows the locations of the major case histories discussed in this paper. Subsequent islands were constructed using a variety of techniques including the use of gravel trucked to the site on winter ice roads and the clam shelling of local silt. A refinement of the silt island technique involved transporting sand fill from a remote borrow pit by bottom-dump scows and stockpiling it on the seabed to be subsequently cast into the island interior by barge-mounted clamshells and draglines (Hayley 1979). The requirement to construct islands in deeper water resulted in the use, from 1976, of a 90 cm diameter stationary suction dredge. Seabed materials were dredged from an adjacent borrow pit and pumped to the site through a floating pipeline with a single discharge point. The largest island constructed using this method was Issungnak in 19 m of water. Islands of this type were known as sacrificial beach islands, since the working surface was surrounded by beaches of sufficient width such that erosion could occur during the open-water season without encroaching on drilling operations. Because of the generally poor quality of the near-site borrow materials (i.e., fine silty sands) these islands had very flat sideslopes, which resulted in the placement of large volumes of fill and, hence, precluded the economical use of this approach at deeper water sites.

The geotechnical performance of these fills is difficult to quantify owing to a number of factors. First, the acceptance criteria for borrow material were set only to ensure the dissipation of pore pressures built up during construction. The quality of the placed fill was generally not verified. Second, no attempt was made to achieve steep slopes (slopes of 1:15 – 1:20 were normal). Flat slopes were desirable to dissipate wave energy. A geotechnical "failure" of a locally steep slope during construction was not considered a failure but, rather, a part of the construction process. Further, such a "failure" would be difficult to distinguish from slope flattening due to wave erosion. Third, because these islands were situated in the landfast ice region, and were generally surrounded by large rubble fields, it is likely that they did not experience significant horizontal shear loads.

However, Shinde *et al.* (1986) reported on some interesting observations regarding failures at Issungnak (1979): "Issungnak material...would have been expected to liquefy under a less severe storm than the design storm. In fact, during the construction of Issungnak, observations by field staff indicated that on at least two occasions the above water portion rapidly disappeared during storm action," and Alerk

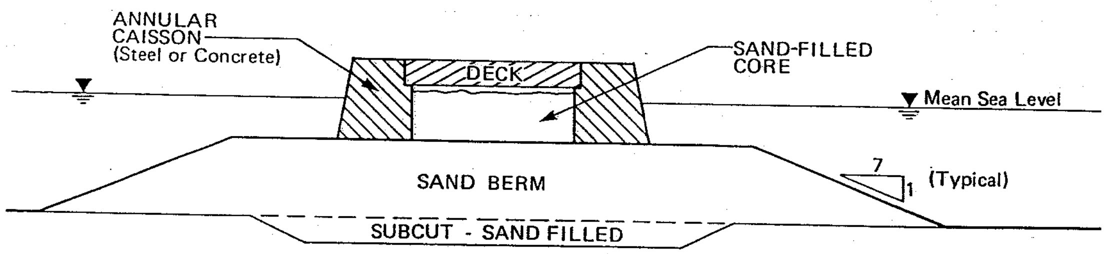

FIG. 2. Schematic illustration (not to scale) of waterline penetration structures.

(1980): "Based on CPT testing and *in situ* porosity measurements, the density of the sand fill at Alerk was determined to be in a relatively loose condition. This fact, as well as an analysis of failure survey data, has led to the conclusion that Alerk experienced instability during a late season storm due to its potentially liquefiable state coupled with cyclic induced excess pore pressures." In fact, at Alerk about half the entire island suddenly disappeared. Given the very flat slope angles prior to failure, liquefaction is indeed the only viable explanation for these failures.

The 1980's saw the introduction of a number of hybrid exploration islands designed primarily to reduce the fill volume requirements at deeper water locations. Four waterline penetration systems were developed: Canmar's concrete caisson system, the "Tarsiut caissons" (1981) (Fitzpatrick and Stenning 1983); Canmar's single steel drilling caisson, the "SSDC" (1982) (Fitzpatrick 1983); Esso's segmented steel caisson, the "CRI" (1983) (de Jong and Bruce 1978); and Gulf Canada Resources Ltd.'s monolithic caisson the "Molikpaq" (1984) (Bruce and Harrington 1982; McCreath *et al.* 1982). The Molikpaq is illustrated schematically in Fig. 2.

Although each system is unique, deployment of all systems commenced with the building of a steep-sided (1:6 – 1:8) subsea sand berm on which the caisson was to be placed. The requirement to place a heavy structure on a berm capable of resisting the large horizontal ice loads that were anticipated in the moving ice or "shear" zone required that the sand meet rigid specifications on gradation and *in situ* density in order to provide assurance against liquefaction. The consequences of a liquefaction failure of such islands are potentially catastrophic. As most proposed sites did not possess suitable local borrow material, trailing suction hopper dredges were introduced to transport sand from remote locations. Trailing suction hopper dredges pick up material from a submarine borrow source by dragging an arm along the seabed. They are capable of carrying up to 8000 m^3 of sand per load.

The construction of these berms also introduced new sand-placement methods. Three methods of sand placement are illustrated schematically in Fig. 3. Hopper dredges can bottom dump sand by discharging it through large valves or doors located on the underside of the vessel. Such dumping involves discharging up to 8000 m^3 of sand in a period of several minutes (Fig. 3*a*). By contrast, the method employed for the construction of the sacrificial beach islands is shown in Fig. 3*b*. Further, "steep" slopes were formed by constructing perimeter containment bunds (i.e., low dykes) and infilling the central area (Fig. 3*c*). The bunds were placed using controlled pipeline discharge techniques. This could be accomplished by pumping material from either a hopper or cutter suction dredge through a floating pipeline to an anchored barge. The sand was then tremied through a modified discharge nozzle during controlled winching of the barge. The drag arm pump-out method was also used to construct bunds. With these procedures it has been possible to construct berms with sideslopes between 5H:1V and 7H:1V. The average Tarsiut and Uviluk berm sideslopes were about 5.6H:1V (Mitchell 1984).

On the basis of data from the first stage of construction of the Kogyuk berm in 1982, it was reported that densities achieved by the bottom-discharge and pipeline-placement techniques were similar (Stewart *et al.* 1983). As shown later in this paper, this conclusion is misleading.

Apart from the SSDC, which was ballasted onto the berm with water, all the systems required backfilling of a central core with sand. This sand was of necessity placed by a pipeline.

In summary, with respect to placement technique, the Beaufort Sea experience has involved two fundamentally different methods, namely bottom dumping and pipeline placement. Pipeline-placement techniques can be further subdivided into discharge from the sea surface, direct single-point discharge near the berm surface, and modified discharge near the berm surface to enhance the creation of steep slopes. Although densification equipment was on standby for the first hybrid island, Canmar's Tarsiut N-44, until recently mechanical densification of the placed sand in islands has not been carried out.

The distinction between the two primary placement methods is important because data obtained from several artificial islands constructed in the Canadian Beaufort Sea have provided evidence that material placed by bottom dumping is significantly denser than pipeline-placed material, other factors being equal. This evidence is presented in this paper. It shows that the relative density of hopper-placed sand is typically 20–30 percentage points higher than pipeline-placed material. This evidence is consistent with that reported by the Dutch as a result of density measurements in the Brouwersdam structure, which is part of the Delta project (Klohn Leonoff Ltd. *et al.* 1984). While the reasons for this difference must presumably result from differences in the hydraulics of placement, they are not clearly understood at this time. The consequences of this consistent difference, however, are more easily assessed.

As mentioned previously, it has been reported that 25% of arctic offshore islands have experienced unexpected geotechnical problems. Specifically, four cases of sand liquefaction have been reported. Unfortunately, full details of only one case history have been published (the Nerlerk berm, Mitchell 1984; Sladen *et al.* 1985*a*), although some details of a second (the Molikpaq at Amauligak I-65) have also been published (Jefferies and Wright 1988). Both these case histories involved liquefaction of hydraulically placed sands,

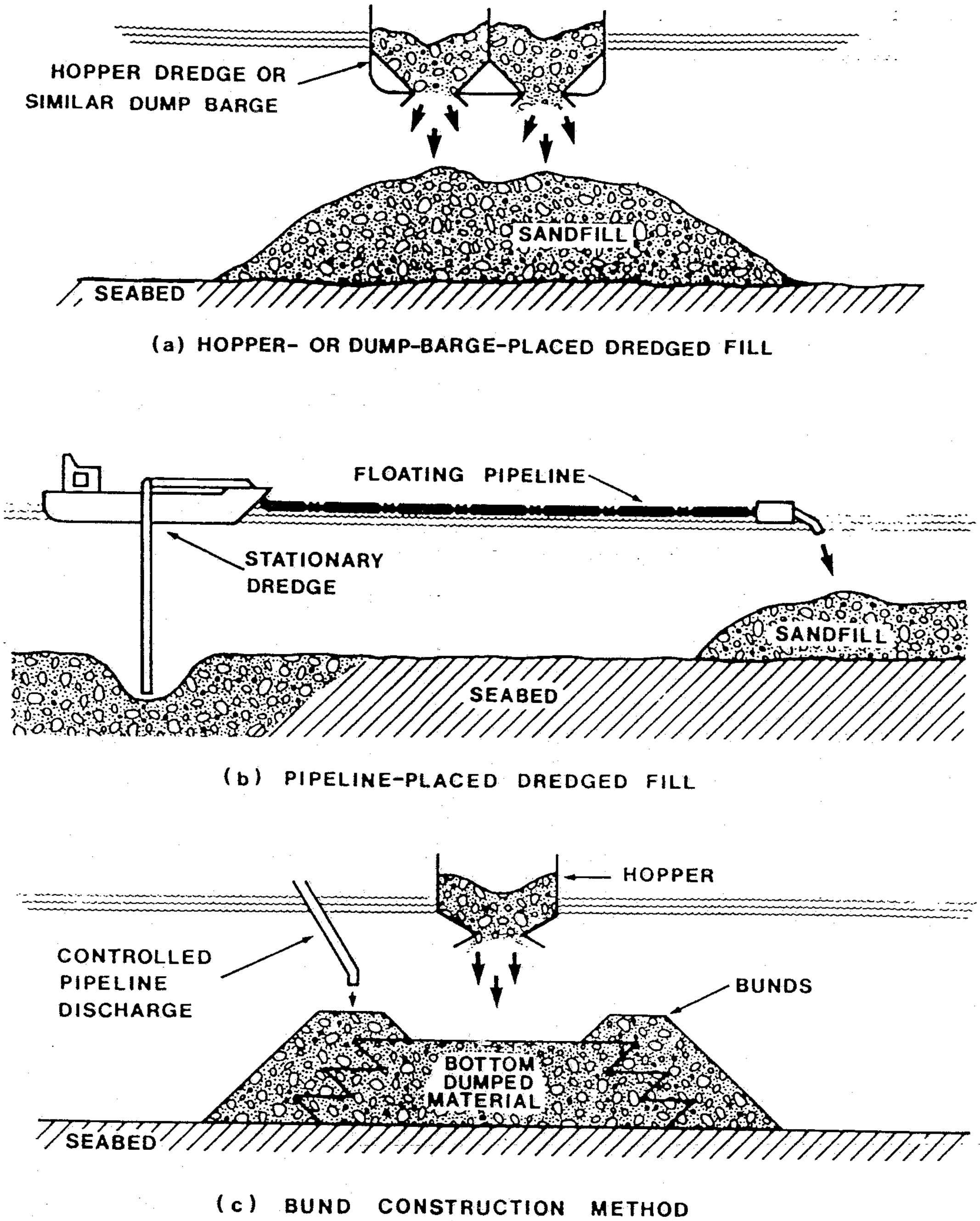

FIG. 3. Schematic illustration of hydraulic placement methods for sand: (*a*) bottom dumping, (*b*) pipeline placement, (*c*) bund construction.

although in different circumstances. Liquefaction slides at Nerlerk in 1983 led to the abandonment of the project. Liquefaction of part of the sand "core" of the Molikpaq, during ice loading in 1986, led to the recognition of the need for mechanical densification of the core for the next deployment (Stewart and Hodge 1988).

Characteristics of Beaufort Sea sands

Sands available for use in hydraulic fill construction are typically fine grained and uniform. Fines (silt and clay size) content is variable. Generally, material has been obtained from the Ukalerk, sometimes called Erksak, borrow area. Figure 1 shows the location of this borrow source. Sand from this source is predominantly quartz and is relatively clean. Typical mineralogical composition of Ukalerk sand is 75% quartz, 12% chalcedony, 10% amphiboles, and 3% feldspar. Some berms have been constructed using lower quality (finer and siltier) material because of the economic advantages of being able to use a local borrow source. Typical index properties of Ukalerk sand are given in Table 1. Figure 4 shows typical grain size distribution. Index properties for the Ticino sand, a standard sand used for *in situ* test calibration, are included in Table 1 for comparison.

TABLE 1. Typical index properties of Ukalerk (Erksak) and Ticino* sands

Property	Value	
	Ukalerk	Ticino
Uniformity coefficient ($C_u = D_{60}/D_{10}$)	1.5–1.8	1.60
Sphericity	0.7–0.8†	0.79
Roundness	0.35–0.5†	0.38
Maximum void ratio	0.80	0.89
Minimum void ratio	0.50	0.60
Specific gravity	2.66	2.67

*Data for Ticino sand from Been *et al.* (1987).
†Test method, Krumbein and Sloss (1951).

Methods of assessing *in situ* sand density

The behaviour of sand is sensitive to small differences in void ratio. The accuracy with which sand void ratio can be measured directly in the field can be low in relation to this sensitivity. The problem is that it is difficult to recover a sample of sand with any certainty that its void ratio has not changed during the sampling process. This is not a new problem and it has led to many attempts to measure *in situ* density indirectly by correlation to the results of *in situ* tests. To date, the cone penetration test (CPT) has been the primary tool used to assess *in situ* density of sand fills in the Beaufort Sea.

Various workers have used large-scale chamber tests to determine relationships between sand density (or void ratio), effective stress level, and CPT tip resistance (Schmertmann 1977; Villet and Mitchell 1981; Baldi *et al.* 1982; Parkin *et al.* 1980; Parkin and Lunne 1982).

For a given sand there is found to be a reasonably unique relationship between sand void ratio, e, mean effective stress, p', and CPT tip resistance, q_c. Baldi *et al.* (1986) have presented the relationship for Ticino sand between relative density (D_r), q_c, and p' shown in Fig. 5. In this figure contours of relative density are shown in a plot of q_c versus p'.

Actually, there is some evidence that tip resistance is more strongly dependent on horizontal effective stress than on mean stress (Baldi *et al.* 1986; Houlsby and Hitchman 1988). Nevertheless, for Ticino sand, the curves in Fig. 5, which were derived through a regression analysis of experimental data, correlate reasonably well with the chamber test results. Values of relative density, expressed in percent, predicted by Fig. 5, are within ± 14 of the actual value, 9 times out of 10. That is within ± 0.05 in terms of void ratio. This error band could be considered to be the limit of accurary of any method of interpreting CPT chamber data that only considers mean effective stress and tip resistance. For sands with less experimental data than for the Ticino sand or where the horizontal effective stress is unknown, greater potential error could be anticipated.

There are some potential problems that arise when applying relationships such as the curves shown in Fig. 5 to field conditions. First, there is no direct evidence to prove that data obtained in a large chamber are directly relevant to field conditions, even for the sand studied in the chamber. Such factors as depositional mode, fabric, nonuniformity, and ageing could affect CPT field performance. Second, it is not usually possible to measure the horizontal effective stress. Third, it is not known if a relationship developed for

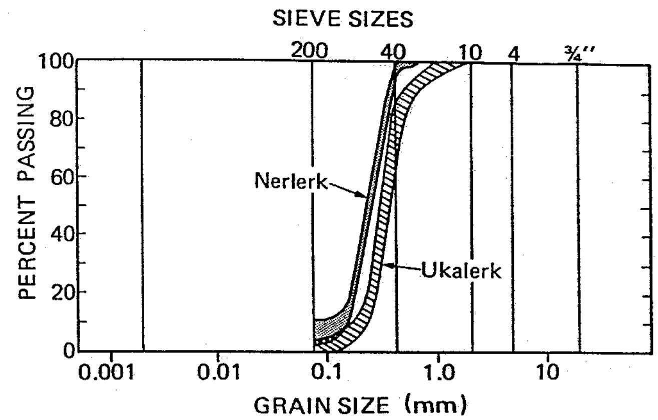

FIG. 4. Typical grain size distributions for Ukalerk (Erksak) and Nerlerk sands.

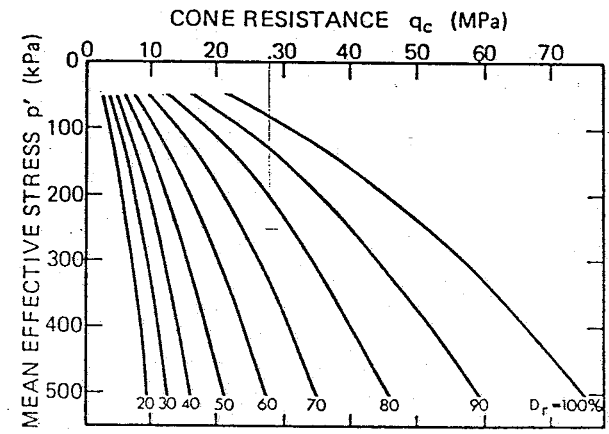

FIG. 5. The relationship between relative density and mean effective stress measured in large-scale chamber tests for Ticino sand (after Baldi *et al.* 1986).

one sand is applicable to any other sand, particularly for a variable deposit. The net effect of these problems, qualitatively, must be a reduced confidence in estimated density. While it is difficult to quantify the level of confidence, a range of ± 25 percentage points relative density may be reasonable for a sand for which little or no reliable chamber test data are available.

Because of these uncertainties, values of relative density obtained by relationships such as Fig. 5 should be treated with caution until there is field evidence to support or otherwise calibrate them. They are more useful as relative indices of density than as absolute measures. Some results of chamber calibration tests on dense and very dense Erksak sand have recently been published (Been *et al.* 1987) but these are not directly relevant to undensified hydraulic fills, which generally exist in a loose to medium dense state.

Beaufort Sea data

As noted above, in the Canadian Beaufort Sea, the primary tool used to date to estimate *in situ* density of sand fills has been the CPT. To avoid issues concerning the interpretation of the CPT at this stage, data will be discussed primarily in terms of CPT tip resistance. While this implies that a given tip resistance at a given vertical stress level can be related to a given density, this is considered to be reasonable given the similarity of the sands used for fill in

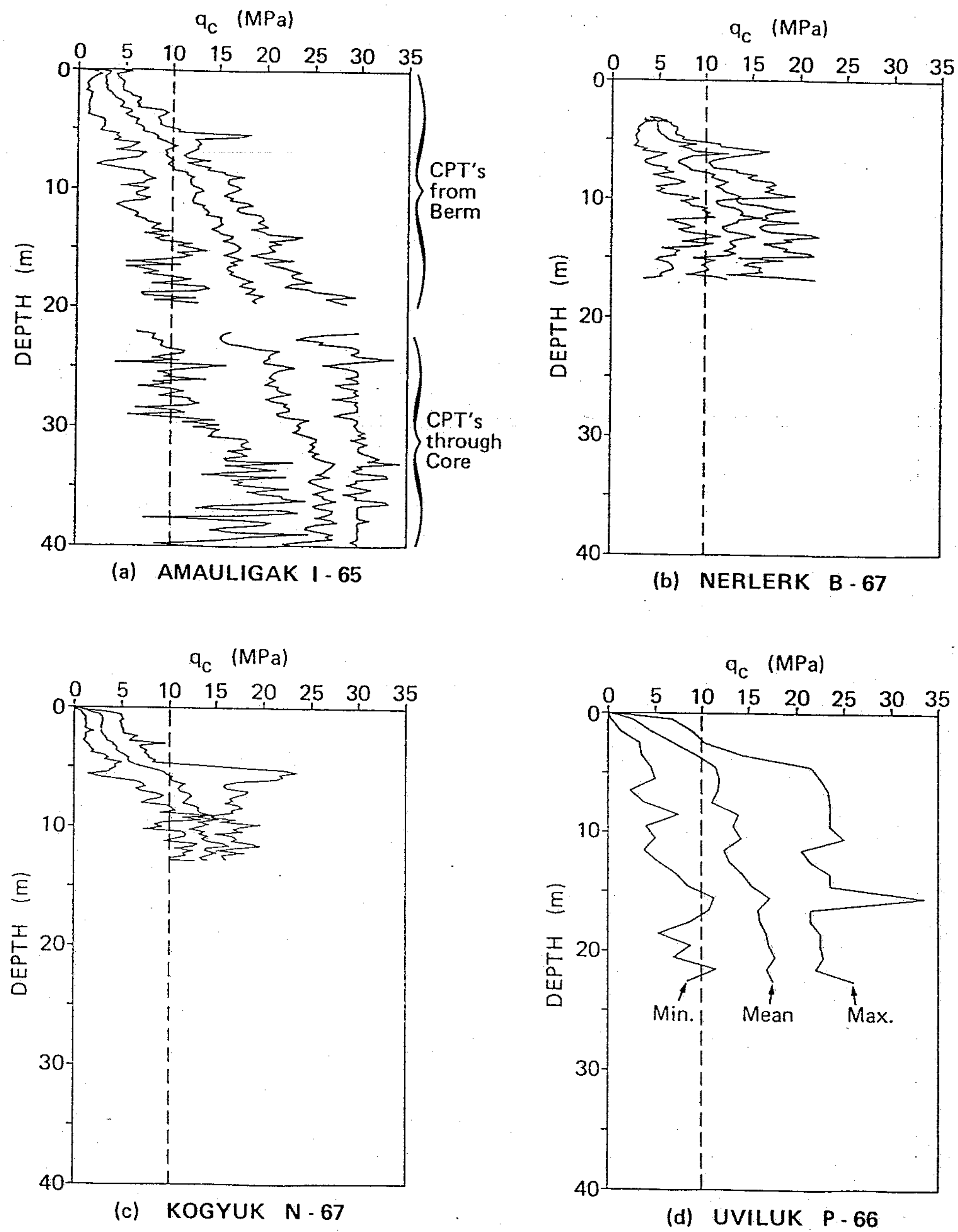

FIG. 6. CPT tip resistance profiles in hopper-placed sand.

the Beaufort Sea. Certainly, for example, a fill whose CPT tip resistance is twice that of a second fill at the same depth must be significantly denser than the second. Implications in terms of relative density, which of necessity make use of some correlation, are given only to illustrate the possible consequences of a given observation.

Experience to date in the Canadian Beaufort Sea has shown that placement technique has an important influence on the *in situ* density of hydraulically placed sands. This is illustrated by reference to CPT data from five subsea berms or structures: Uviluk, Nerlerk, and Kogyuk, each intended for deployment of Canmar's SSDC structure; the Molikpaq deployment at Amauligak I-65; and the Alerk sacrificial beach island. In each of these cases it is possible to separate CPT data according to placement method. Three of the five cases used very similar sands from the Ukalerk borrow source. Since placement is the only significant variable, its effect is therefore easy to isolate. Alerk sand is also similar with a mean grain size of 235 μm and an average percent fines of 3.7. The pipeline-placed sand at Nerlerk was slightly finer and more silty, as shown in Fig. 4. It is shown, however, that CPT response in the Nerlerk and Alerk pipeline-placed sands is in fact very similar to pipeline-placed Ukalerk (Erksak) sand, suggesting that, for all cases, placement technique is the dominant influence on CPT response.

Figures 6 and 7 show the ranges of CPT tip resistance measured at four of the sites. Data for Alerk are not presented in these figures as there are insufficient CPT's to define ranges of tip resistance clearly. The data available at Alerk are presented later. At the other sites, a large number of CPT soundings are available: 19 at Uviluk, 46 at Kogyuk, 26 at Nerlerk, and 29 at Amauligak. In Fig. 6 data for hopper-placed material alone are shown. The mean and the range are illustrated. Similar data are shown in Fig. 7 for

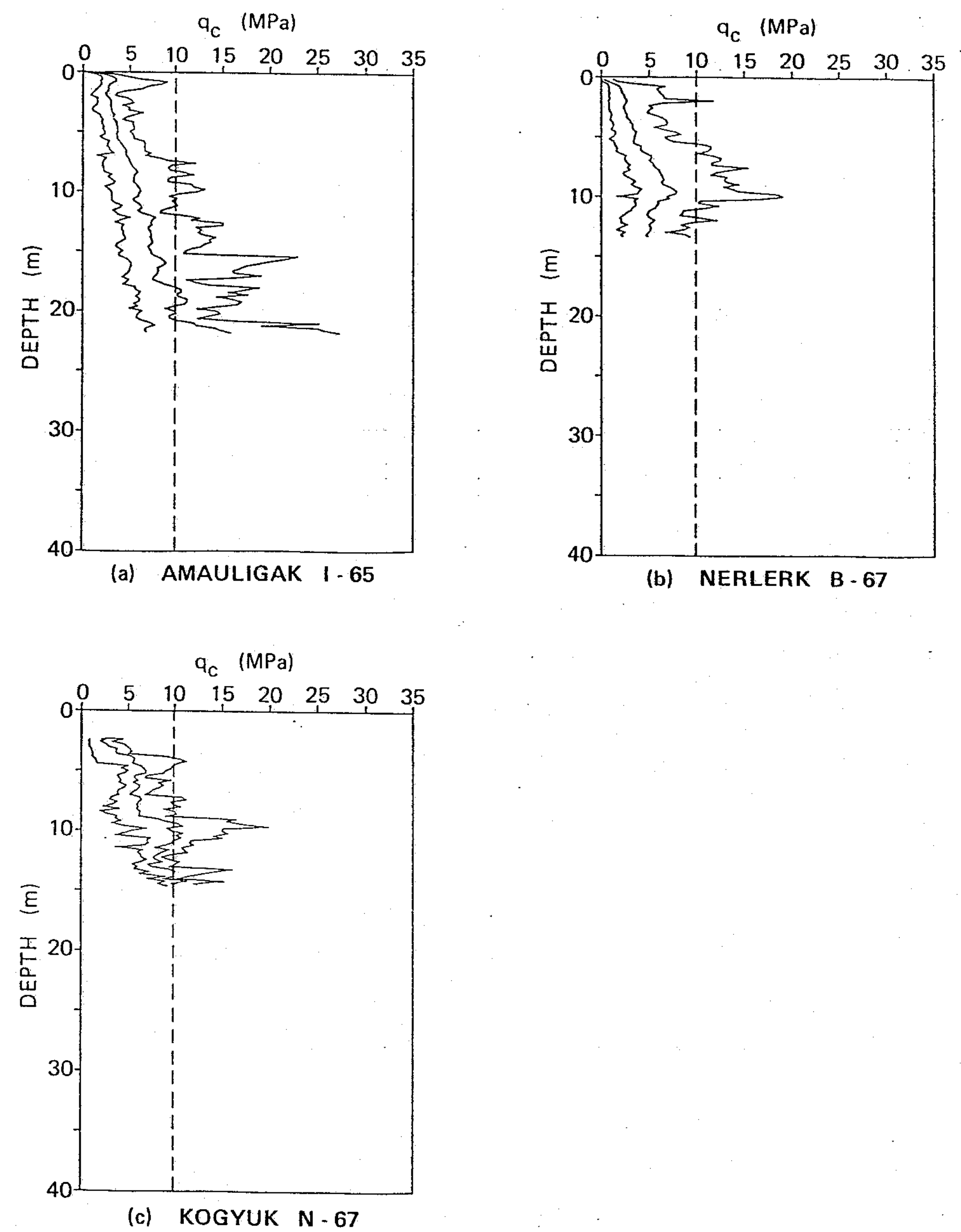

FIG. 7. CPT tip resistance profiles in pipeline-placed sand.

pipeline-placed sand. Note that at Uviluk, no significant quantity of pipeline-placed material was used. At Amauligak I-65, where the berm was predominantly hopper placed, there are two sets of data for this material: that derived from testing prior to caisson setdown and that from CPT's pushed through the completed core into the berm.

It can be observed that mean CPT tip resistance in the hopper-placed material is always greater than that in the pipeline-placed material. At all sites, there is a large range of tip resistance recorded at a given depth. For a given placement method, maximum tip resistances are typically two to four times the minimum values. Given that vertical stress will likely be reasonably uniquely related to depth, at a given site these variations can be attributed to some combination of variations of *in situ* density, horizontal effective stress, and composition. Given the uniform nature of the sand, it can be assumed that *in situ* density is likely the dominant factor, but it should be recognized that there is some uncertainty as to the cause of the spatial variability.

Whatever the cause, this variability raises the question as to what value of tip resistance within the range measured can be considered representative of the fill. A review of individual CPT traces shows that at a given CPT location, alternate layers of higher and lower CPT tip resistance are typical, suggesting some form of horizontal stratification. The fact that it is not usually possible to correlate zones laterally is suggestive of a lenticular nature. Jefferies *et al.* (1988) have recently presented some data illustrating the limited lateral extent of loose zones in hydraulic fills. In this situation, it is conceivable that the zones of lowest CPT resistance, presumably the loosest, could dominate behaviour, so that use of mean CPT resistance could be unconservative in design. Jefferies *et al.* (1988), for example, recommend the use of the 80 percentile value of tip resistance.

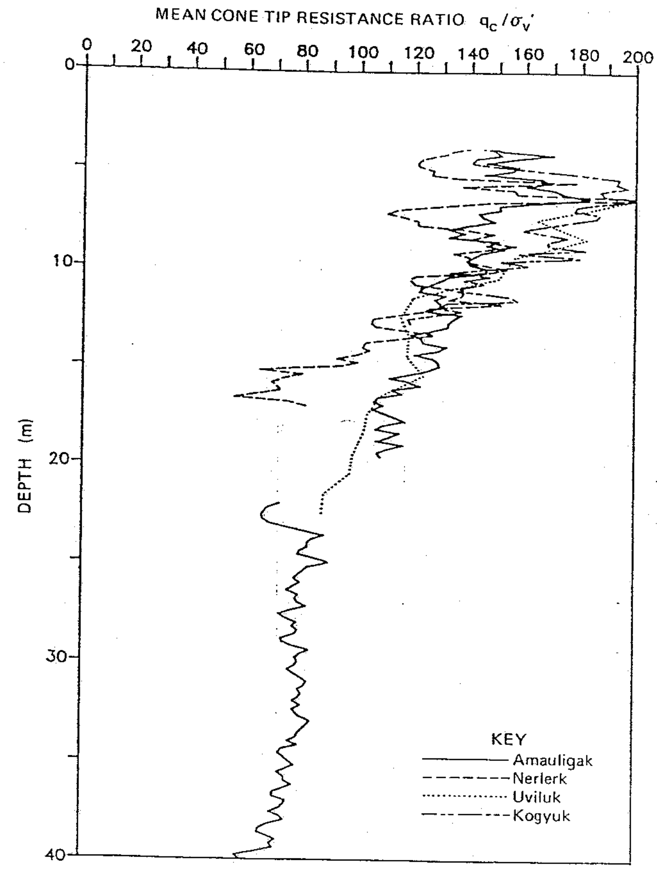

FIG. 8. Mean cone tip resistance normalized by effective overburden pressure versus depth for hopper-placed sand.

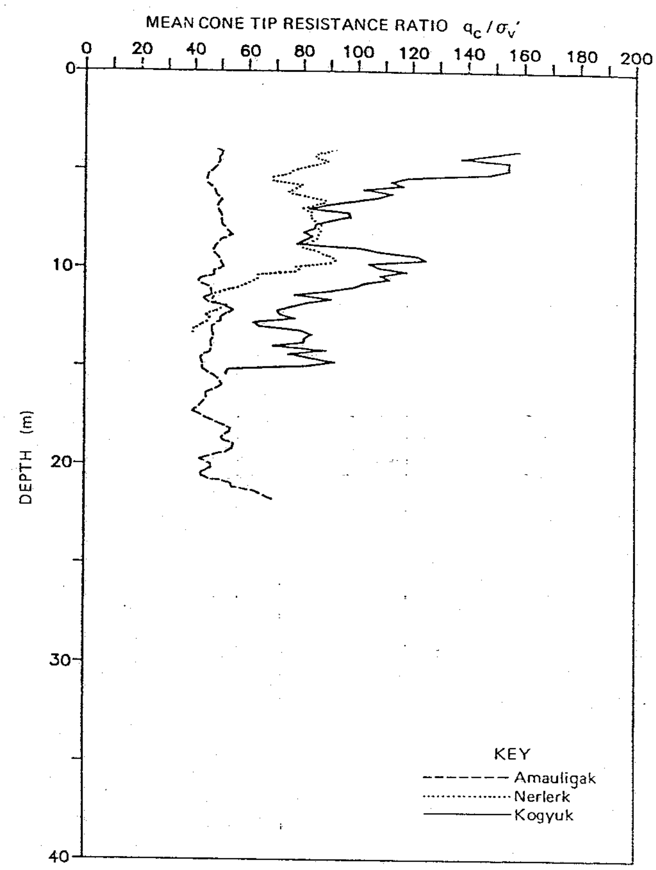

FIG. 9. Mean cone tip resistance normalized by effective overburden pressure versus depth for pipeline-placed sand.

Mean tip resistance can, however, be used for relative comparisons between different sites and placement methods. Figures 8 and 9 show profiles of CPT mean tip resistance for the four sites superimposed for hopper-placed and pipeline-placed material respectively. For these plots, values of tip resistance have been divided by the estimated mean vertical effective stress. This quotient is termed "tip resistance ratio." This was done because for one set of data, that through the core at Amauligak I-65, a portion of the sand tested was above water level, so that at any given depth, vertical effective stresses would be higher than for the other data sets. It should be noted that division by vertical effective stress was undertaken only to allow a relative comparison between profiles to be made, and does not imply that sand with similar values of tip resistance ratio, but at different depths, could be expected to behave in a similar manner.

These figures show that mean values of tip resistance for hopper-dumped and pipeline-placed sand fall within two different ranges. Further, for pipeline-placed material, the data at Amauligak are generally lower than those obtained at Nerlerk, while the Kogyuk data are somewhat higher. Values at Kogyuk are typically twice the values at Amauligak, at the same depth. This suggests that for pipeline-placed sand subtle differences in placement technique, discussed previously, can influence *in situ* density. The range of mean tip resistance ratios for the hopper-placed sand from the four sites is surprisingly narrow. In Fig. 10 the ranges for hopper-placed sands are compared with the pipeline-placed data. It is clear that at a given depth below the top few metres, values of CPT tip resistance within hopper-placed sand are typically about twice values within pipeline-placed material.

As noted above, all sands used were similar and were as close in grading and composition as is ever likely to be obtained in civil engineering construction of this type. Accordingly, variability in composition can be practically eliminated as a reason for this variability in CPT response. It can therefore be concluded that pipeline placement results in lower *in situ* density than does hopper placement. The effect of *in situ* horizontal stress is a remaining uncertainty but it can be noted that generally the looser the sand the higher the value of K_0. This is illustrated in Fig. 11, in which values of K_0 measured for normally consolidated Ticino sand are plotted against void ratio. To the author's knowledge, no reliable measurements of *in situ* horizontal stress in hydraulic sand fills have been made, but if the trends from the Ticino tests were also true for hydraulic fills then they would tend to negate some of the effects of increased void ratio on CPT response. That is, the difference in density between two sands may be even greater than a simple comparison of CPT tip resistances might imply.

To illustrate the implications of this difference, ranges of mean relative density for the two placement methods have been estimated using the correlation provided by Baldi *et al.* (1986). These are shown in Fig. 12. It is not known if this correlation is directly applicable to Beaufort Sea sand. While it is not possible to assume that values of inferred density correspond directly to relative density, it at least allows a relative comparison. As can be seen, mean values of inferred relative density are generally in the range 30–60% for pipeline-placed sand and 50–80% for hopper-dumped sand. It should be noted that these are mean values for each site. At a given site the range is typically ±20 percentage points about the mean. The values of relative density inferred from the Baldi *et al.* correlation for the hopper-placed sand are somewhat higher than expected from precedent, which would suggest a sensible upper limit for hydraulically placed sands of about 65–70%. Therefore, it seems likely that at least for medium dense states this correlation, when applied to Beaufort Sea sands, may overestimate true relative density by up to about 20 percentage points.

Considering that (1) minimum tip resistance is often only half the mean value, at a given depth, (2) sand behaviour is sensitive to small differences in void ratio, (3) the likely range of relative density attainable by hydraulic placement is large, and (4) the likelihood that the boundary between "acceptable" and "unacceptable" behaviour lies somewhere within this range, then it can be stated that a marked difference in behaviour between hopper- and pipeline-placed materials can be reasonably expected.

Discussion

It has been shown that material placed by the bottom-dumping technique is significantly denser than pipeline-placed material, all other factors being equal. Further, subtle differences in the details of pipeline placement can influence *in situ* density. The exact mechanics that lead to these differences are little understood. On review of the data, however, reasonable speculations can be made on the factors that may be causing these differences.

The sand in a hopper dredge, although likely loose and near saturated, probably has a bulk density of around 1.8–1.9 Mg/m^3. Conversely, typical densities of sand slurries in pipelines are around 1.3–1.4 Mg/m^3. The net negative buoyancy of the discharge from a hopper dredge is therefore at least twice that of the discharge from a pipeline. As a result, the sand from the hopper falls as a slug rather than as individual particles. Further, the simultaneous opening of all the valves or doors of the hopper dredge inhibits the entrainment of "fresh" water into the slug that would reduce its fall velocity and expand its size. The fall energy of this discharge is likely dissipated in compaction of the berm through impact and shearing. In effect, the bottom-dumping method is in itself a form of soil compaction.

It is intuitive that pipeline placement results in less compactive effort. If the discharge were a single point held close to the berm surface, there would be little opportunity for entrainment of "fresh" water into the flow. Virtually all the kinetic energy due to the discharge velocity would be absorbed by the berm. As such, one would anticipate some vibratory packing induced by the fluid turbulence in the jet impingement region and some horizontal shearing of the settled sand away from the nozzle. This discharge procedure would be representative of drag arm placement, as was used for part of the berm at Koguyk.

If the single-point discharge were replaced with an inverted "umbrella" that directed the discharge up away from the berm surface, most of the previous benefits would be lost and an even looser *in situ* state would result. This was the procedure used at Nerlerk to promote the creation of steeper slopes (Mitchell 1984).

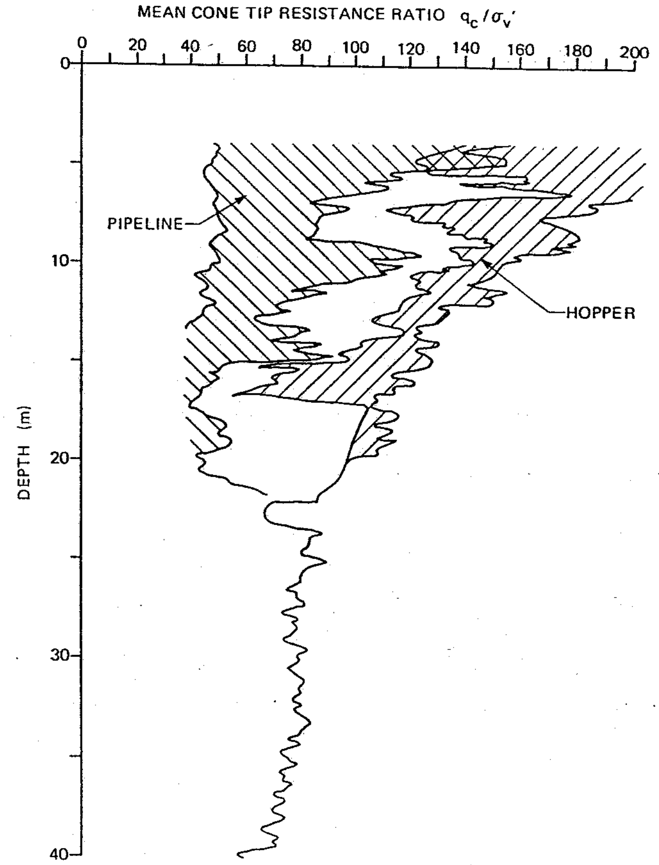

FIG. 10. Range of mean tip resistance normalized by effective overburden pressure for hopper-placed sands compared with profiles in pipeline-placed sand.

The loosest possible state would likely be achieved if the single-point discharge were placed near the sea surface such that maximum water entrainment took place and the discharge flow became a cloud with a fall velocity close to the fall velocity of the individual particles. Each particle would essentially come to rest in the position where it makes contact with the berm. Subsequent impacts of other particles may result in some jostling but the impact velocities and forces would be small. This discharge method was used at Alerk, Issungnak, and for the Molikpaq core.

Interestingly, where variations in density have been noticed between the hopper- and pipeline-placed sands, these have often been attributed to other factors. For example, Stewart *et al.* (1983) attributed an increase in density with depth at Kogyuk to densification due to increased effective stress, whereas in fact it can now be seen to be due to pipeline-placement above hopper-dumped sand.

Berzins and Hewitt (1984) reported a trend of increased values of CPT tip resistance with increased median grain size and reduced percent fines. Such a trend is possible, as it can be shown that even very small additions of clay to a slurry markedly increase the viscosity of the fluid (Western Canada Hydraulic Laboratories Ltd. 1983). Hence, particle fall velocities are reduced and the permeability of the at-rest material is considerably less, which would limit the effect of any subsequent compactive effort. The data were not normalized for placement technique, however, and a review of the data in the light of subsequent findings suggests that

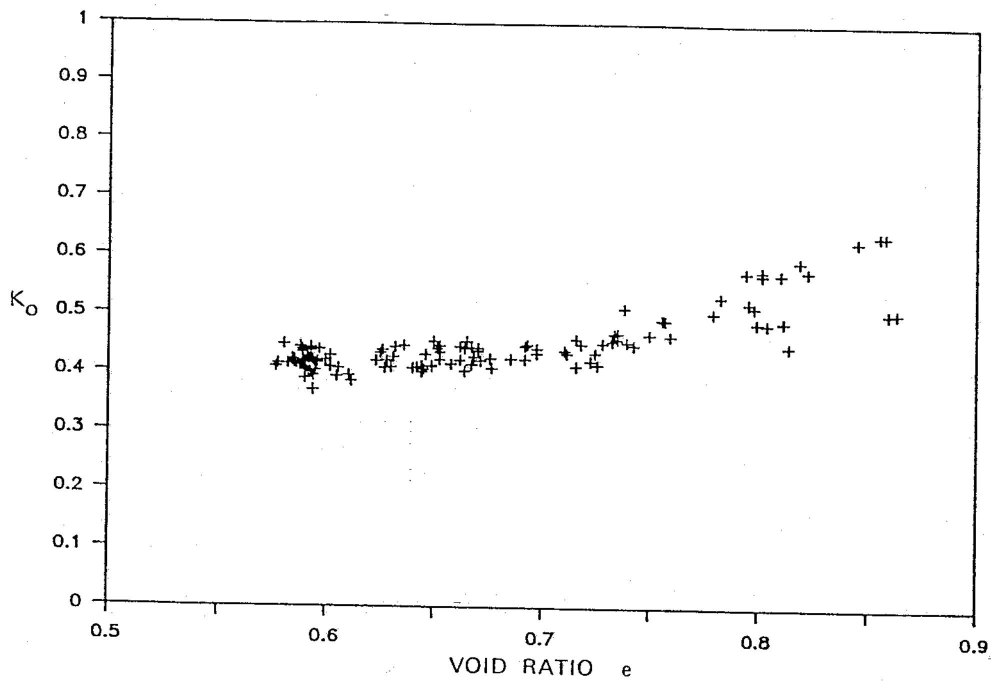

FIG. 11. K_0 versus void ratio for normally consolidated Ticino sand—from chamber tests (data from Baldi *et al.* 1986).

placement technique, and not grain size or percent fines, likely accounted for most of the differences.

Since preparation of initial drafts of the present paper, Jefferies *et al.* (1988) have presented data concerning the effect of placement method on *in situ* density. Their data support the general conclusions of this paper with respect to the differences between hopper- and pipeline-placed sands. Unfortunately, data were presented in terms of inferred sand state using a correlation between CPT tip resistance and state that has subsequently been shown to be in error (Sladen 1989).

Implications for performance

As noted above, there have been four reported cases of liquefaction of hydraulically placed sands in the Canadian Beaufort Sea: Nerlerk, the Molikpaq at Amauligak I-65, and Esso's Issungnak and Alerk islands. The Nerlerk case history has been reported in detail (Sladen *et al.* 1985*a*). Some details of performance at Amauligak I-65 have been provided by Jefferies and Wright (1988). Additional details of preliquefaction state at Amauligak, including the CPT data discussed above, have been made available by Jefferies *et al.* (1988) and as part of a recent research study. This study was undertaken by the National Research Council of Canada in conjunction with Gulf Canada Resources Limited and was managed by the Atlantic Geoscience Centre (AGC). Other details of performance data at Amauligak I-65 remain proprietary. Reports on this research together with available performance data will be put on open file at AGC in 1990. In both cases for which details are available liquefaction was restricted to pipeline-placed sand.

At Nerlerk, the mechanism triggering liquefaction was simple static loading, which occurred when slope angles approached 3H:1V. The liquefaction involved large-scale retrogressive flow slides, which exhibited the classical features of such failures. Volumes of failed material were as much as 200 000 m^3 per slide and postfailure slope angles were as flat as 50H:1V.

At Amauligak, liquefaction was induced by pulsating loading. The liquefied sand was restricted to the core, which is confined by the annular steel structure. Accordingly, large-scale flow slides were prevented. The net results of the liquefaction were an increase in core pore pressure and a large core settlement.

This is not the place to enter into a detailed discussion of the factors affecting liquefaction. Reviews have been provided by various authors (i.e., Castro and Poulos 1977; Sladen *et al.* 1985*b*). For liquefaction flow slides to occur, the initial state of the sand must lie above the steady state line for the sand. Problems with the accurate measurement of *in situ* void ratio and the determination of steady state lines for silty sands (Sladen and Handford 1987) lead to difficulties in the assessment of liquefaction potential with the present state-of-the-art. Both these problems are rooted in laboratory test procedures and the uncertainties concerning the relevance of laboratory data to field conditions. In the case of the determination of *in situ* void ratio, the problem arises because correlations between *in situ* tests and void ratio generally require a laboratory calibration. This issue is discussed by Sladen (1989).

The availability of preliquefaction *in situ* test results at Nerlerk, Amauligak, and Alerk allow these laboratory testing difficulties to be partially circumvented. Inferences of state that can be made from performance can be compared directly with *in situ* test results.

The Nerlerk case history provides CPT tip resistance values in sand fill that later suffered flow sliding. It can be inferred that these CPT traces correspond to a sand whose state lies above the steady state line. It can be shown that the undrained brittleness index (i.e., loss of strength divided by the peak strength) of very loose sand is a function of sand state in relation to the steady state line (Sladen *et al.* 1985*b*).

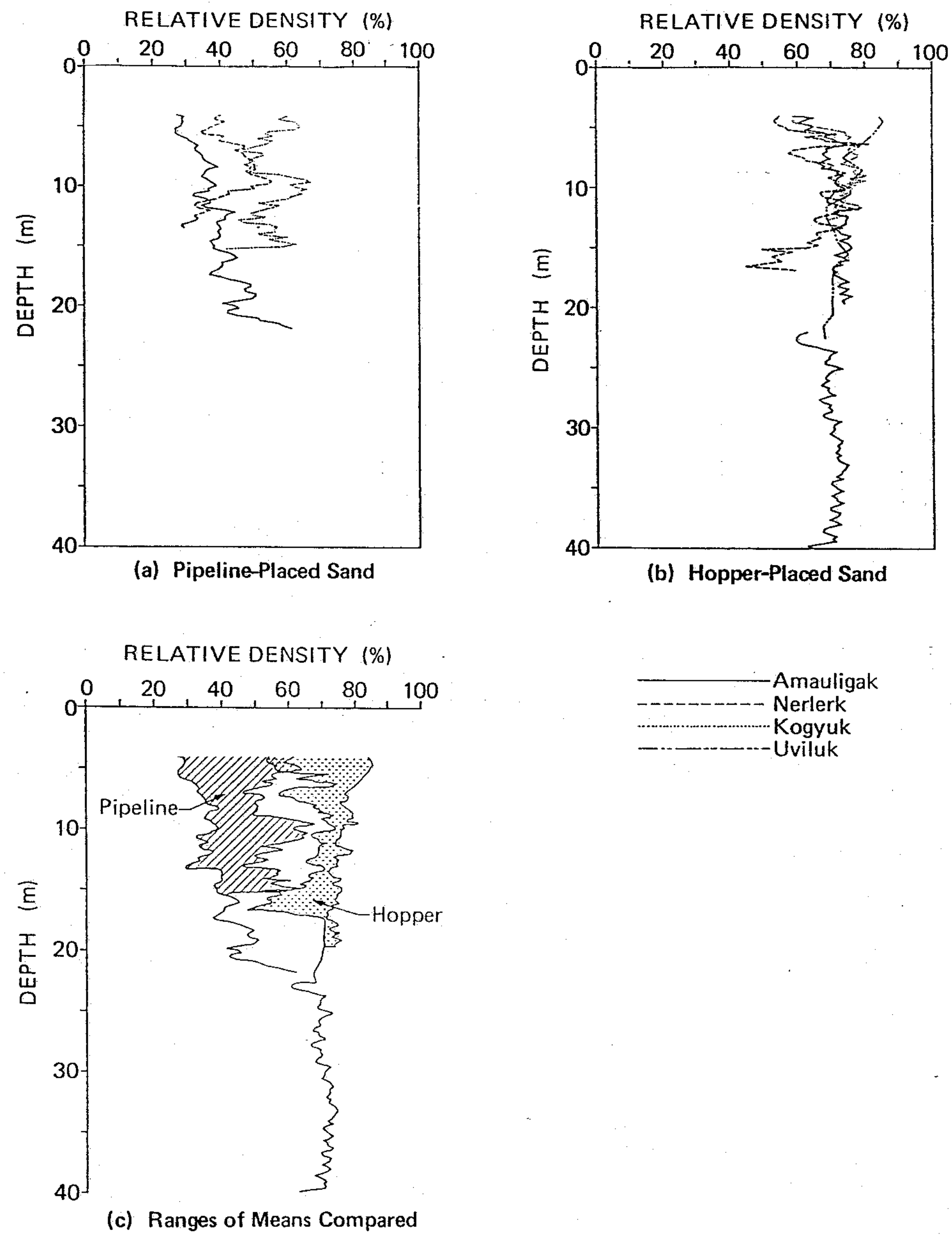

FIG. 12. Profiles of inferred mean relative density for hopper- and pipeline-placed sands compared.

From this relationship, it can be inferred that the sand at Nerlerk was, in terms of void ratio, about 0.05–0.15 above the steady state line. That is about 15–45% above the steady state line in terms of relative density. The higher values apply to greater depths (higher stress levels).

No such direct correlation is available from the Molikpaq case history because the direct evidence for flow sliding is unclear. In that the tip resistance ratios in the Molikpaq core (Fig. 9) were generally less than at Nerlerk, it is likely that *in situ* density was also less, and as liquefaction developed, it can be inferred that the sand in the core was also in a state above the steady state line.

There has, to date, been no evidence of flow sliding in hopper-placed material. As discussed above, absence of liquefaction does not necessarily imply the lack of potential for liquefaction. It can be pointed out, however, that if the above estimated state for the pipeline-placed Nerlerk sand is correct and if the general effect of void ratio on CPT tip resistance is reasonably well represented by the Baldi *et al.* correlation, then this would imply that hopper-placed material exists some 0.05–0.15 below the steady state line in terms of void ratio, that is, it is dilatant. The pipeline-placed material at Kogyuk is interesting. It can be observed that no liquefaction occurred and that the berm supported the SSDC structure for a drilling season. In addition, CPT response is significantly greater than at Amauligak I-65, and slightly higher than at Nerlerk. A reasonable interpretation would be that the material state was close to and possibly slightly below the steady state line in a plot of void ratio versus mean effective stress.

Figure 13 shows mean values of tip resistance versus estimated mean average effective stress, for the case histories discussed above and for the Alerk Island. Accepting, for a given sand within a certain tolerance (say ± 0.05 in terms

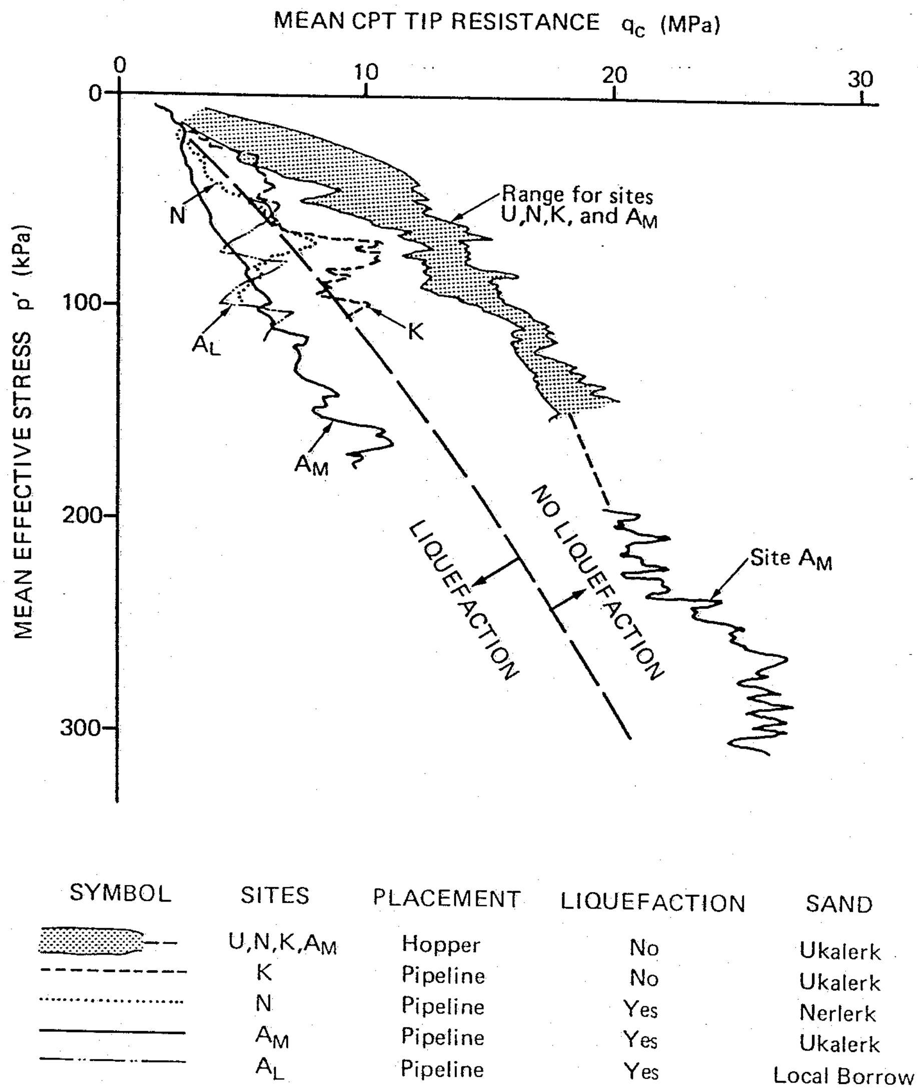

FIG. 13. Mean tip resistance versus mean effective stress for the cases studied. A line separating sands that have exhibited liquefaction from those that have not is indicatea. Note that although liquefaction has been reported for these sites, loading conditions were different (see text for details). Key to site names: U, Uviluk P-66; N, Nerlerk B-67; K, Kogyuk N-67; A_M, Amauligak I-65; A_L, Alerk.

of void ratio within normal stress levels), that CPT tip resistance is uniquely related to mean effective stress and void ratio, then it should be possible to put a line on this graph representing states on the steady state line, that is, to separate liquefiable from nonliquefiable sands. Based on the above discussion, this has been done, and a "best estimate" line is shown. It should be noted that although liquefaction has been reported to have occurred at the three sites, the loading conditions leading to liquefaction were different for each, as discussed earlier in this paper.

With respect to design there are further uncertainties that must be recognized. These uncertainties include the effect of minor changes in sand composition, the appropriateness of the mean tip resistance value, the validity of the assumption that, even for a given sand, tip resistance can be uniquely related to void ratio and mean stress and the value of *in situ* horizontal stress. Accordingly, designers should be cautious before assuming that tip resistance values that plot above this line necessarily indicate nonliquefiable states.

Conclusions

The conclusions drawn in this paper are summarized below in point form for convenience:

(1) A dense sand state cannot be obtained by simple hydraulic placement.

(2) At best, mean relative densities of about 60% might be anticipated, but they may be lower than 20%.

(3) The range of relative density about the mean can be large within a given fill.

(4) The mean relative density may not be appropriate for design.

(5) Some hydraulic fills are loose enough to be susceptible to liquefaction flow slides.

(6) Hopper-placed Beaufort Sea sand is much denser than pipeline-placed sand.

(7) Some pipeline-placed Beaufort Sea sands have suffered liquefaction flow sliding.

(8) The confidence with which void ratio can be inferred from *in situ* tests is low, particularly with regard to the sensitivity of performance to void ratio.
(9) There are problems with laboratory-based methods of interpreting state.
(10) A direct relationship between CPT tip resistance in Beaufort Sea sand and mean effective stress that separates liquefiable from nonliquefiable states can be inferred from the performance of case histories.

Recommendations for future work

The main conclusion that can be drawn from this study is that if any hydraulically placed sands in the Beaufort Sea are to be used with confidence in the future, there is a pressing need to develop means of increasing the confidence with which *in situ* state can be assessed. In the past, the primary tool has been the CPT. In the author's opinion, this is an ideal tool for providing relative profiles that can be used to infer states between known points. It is, in effect, an ideal secondary tool. Because of uncertainties in interpretation, however, it should not be relied upon as a primary means of measuring density. Future research therefore should be directed towards other tools more capable of making direct measurements of density, such as advanced samplers or nuclear methods. Other areas in need of research include a study of the factors that affect *in situ* density achieved from various placement techniques, and quantification of laboratory test errors in the triaxial testing of very loose sands.

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