481

GEOTECHNICAL INVESTIGATION

KOPANOAR M–13

BEAUFORT SEA, OFFSHORE CANADA

Report To

DOME PETROLEUM LTD.

Calgary, Alberta

FUGRO GULF, INC.

Consulting Geotechnical Engineers and Geologists

EBA Engineering Consultants Ltd.

GEOTECHNICAL INVESTIGATION
KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA
REPORT NO. 80-058-3

* * *

Report

to

DOME PETROLEUM LTD.

Calgary, Alberta

* * *

By

FUGRO GULF, INC.
Houston, Texas

October 1981

FUGRO

Fugro Gulf, Inc./Consulting Engineers and Geologists

5884 Point West Drive, Houston, Texas 77036
Phone (713) 777-2641 Telex 775494

Report No. 80-058-3
October 30, 1981

Dome Petroleum Ltd.
800-6th Avenue S.W.
P.O. Box 200
Calgary, Alberta
Canada T2P ON1

Attention: Mr. M. Gajtani

GEOTECHNICAL INVESTIGATION
KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA

Gentlemen:

Submitted is the final report for our geotechnical investigation at the above location in the Beaufort Sea. This report presents a description of the field and laboratory testing programs and our engineering analyses for pile design.

Dome Petroleum Ltd. authorized this investigation on April 15, 1980. We conducted the study in general accordance with our contract with Canadian Marine Drilling, Ltd. dated June 9, 1980. Our draft field report was sent to you on October 30, 1980.

We received laboratory data in preliminary form from EBA Engineering Consultants Ltd. on March 10, 1981. Additional test data were received in a telex dated March 31, 1981. Our draft final report was sent to Dome on February 26, 1981 and approved by Mr. M. Gajtani on May 1, 1981.

Final laboratory data were received from EBA on August 12, 1981. These data are included here as Appendix A. Appendix D was received from EBA on October 27, 1981 and was included in this report on request of Mr. M. Gajtani.

We appreciate the opportunity to serve you on this investigation. Please call us for further assistance.

Very truly yours,

Ronald H. Pitts P.E.
Project Engineer

Larry S. Marr P.E. fr
Deputy Manager
Marine Division

RCW/LSM:ab
Copies Submitted: (10)

CONTENTS

ILLUSTRATIONS

SUMMARY

Fugro Gulf, Inc. conducted a geotechnical investigation to determine soil and foundation conditions and pile capacity at Kopanoar M-13 in the Beaufort Sea, offshore Canada. We investigated soil conditions by (1) drilling one borehole (B-1) to a depth of 151 feet (46m), (2) conducting in situ cone penetrometer (CPT) soundings, (3) taking high quality push samples, and (4) performing tests on samples recovered from the borehole. The measured water depth at B-1 was 179 feet (54.6m).

EBA Engineering Consultants Ltd. made laboratory tests on the soil samples; test results are presented in Appendix A. Fugro's engineering analyses for B-1 included determining ultimate pile capacity using the CPT Method for a 30-inch driven pipe pile.

INTRODUCTION

Project Description

Dome Petroleum Ltd. is performing studies in the Beaufort Sea, offshore Canada to determine soil and foundation conditions. Fugro Gulf, Inc. conducted a geotechnical investigation for Dome to determine foundation conditions and pile design parameters at a proposed offshore structure site designated Kopanoar M-13. Dome previously investigated this location during the 1978 drilling season. Results of that study are presented in this report as requested by Dome.

Purposes and Scope of Study

The purposes of the study were to obtain in situ information on soil and foundation conditions at the proposed location and to compute ultimate pile capacity for driven pipe piles. Fugro accomplished these objectives in the following phases:

1. Fugro drilled one borehole to a depth of 151 feet (46m) to determine soil stratigraphy and to obtain soil samples. In situ cone penetrometer soundings and temperature tests were made in the borehole.
2. EBA Engineering Consultants Ltd. performed laboratory tests on soil samples recovered from the borehole to define pertinent enginnering properties.
3. Fugro conducted engineering analyses of the CPT soundings from B-1 to develop the required pile design recommendations.

Subsequent sections of this report contain brief descriptions of the field investigation, laboratory testing program, and general soil conditions at the site. Axial pile design results are presented for a driven pipe pile. Results of the 1978 investigation are presented on Plate 3.

FIELD INVESTIGATION

Fugro investigated soil conditions from the M/V CANMAR SUPPLIER V provided by Canadian Marine Drilling Ltd. (CANMAR). We drilled one borehole, designated B-1, to a depth of 151 feet (46m) below the seafloor. Geographical coordinates provided by Offshore Navigation, Inc. are Latitude 70°22'14" North and Longitude 135°3'24" West. The water depth was 179 feet (54.6m) measured at 0445 hours on August 2, 1980. A map illustrating the borehole location is presented on Plate 1. Plate 2 shows the log of B-1.

The borehole was drilled using CANMAR's Failing 1500 drill rig and conventional rotary drilling techniques. Fugro conducted cone penetrometer (CPT) soundings with our WISON equipment. A temperature sensitive cone was periodically used to measure in situ soil temperatures. In addition, we recovered high quality soil samples with our WIP push sampling tool with thin wall sample tubes. Appropriate drilling subs made up the drill string to enable using the CPT and push sampling tools. The WISON and WIP normally use the weight of the drill string as a reaction force to push the cone or thin wall tube into the soil. A drill string anchor (packer) was used in some cases to increase the reaction force allowing additional penetration into hard or dense soils. Our field report to Dome dated August 10, 1981 presents a detailed discussion of the drilling, sampling, and in situ testing techniques and equipment used.

Descriptions of the soils encountered in B-1 are given on the left portion of the boring log on Plate 2. Plate 4 shows a key to soil classification and symbols used on the boring log. Plate 5 is a brief chronological summary of field activities.

Dome investigated soil conditions at this location in 1978. Tabulated below are coordinates of both this year's and the previous year's boreholes:

Borehole	Coordinates, Meters North	Coordinates, Meters East
1978	7807004	497875
1980	7807016	497877

The boreholes were drilled about 40 feet (12.2m) apart. At the request of Dome, results of the 1978 study are presented here as a log of boring and test results on Plate 3. The primary difference in boreholes is that the frozen silty fine sand was encountered some 5 feet (1.5m) deeper in the 1980 borehole.

FIELD AND LABORATORY TESTS

Field Tests

The field testing program was designed to evaluate the pertinent physical properties of the foundation soils encountered in the boring. Shear strength of cohesive soils was obtained using miniature vane, Torvane, and pocket penetrometer devices. Water content tests were performed for most samples. Selected samples were photographed by EBA; photographs are not included in this report.

Laboratory Tests

Laboratory tests were made for soil identification and classification and to provide detailed information on shear strength and compressibility characteristics of soils. The laboratory testing program was developed jointly by Fugro, EBA, and Dome. EBA conducted the laboratory tests in Edmonton, Alberta.

Most soil test results are presented graphically on the boring log on Plate 2. EBA included test data for this location in their report of test results for the Kopanoar I-44 location. Complete laboratory test results for this location are presented in Appendix A and laboratory test procedures are discussed in Appendix D.

GENERAL SOIL CONDITIONS

Stratigraphy

The following major soil strata were encountered in B-1:

Stratum	Depth, Ft. From - To	Soil Description
I	0 - 16	Very Soft Dark Gray Clay
II	16 - 86	Medium Dense to Dense Silty Fine Sand
III	86 - 151+	Frozen Silty Fine Sand

The boring log shows minor textural and color variations and inclusions of other soil types within each stratum.

Axial pile design analyses presented in this report are based only on the soil stratigraphy and conditions disclosed by B-1. We have not considered possible stratigraphy changes, faulting, or other regional differences that could influence foundation design.

Soil Properties

Soils encountered in B-1 consist of very soft clay overlying silty sand. The silty sand is frozen below 86 feet (26.2m). Above the frozen layer, the silty sand is medium dense to dense in condition and contains occasional clayey sand layers from 81 to 85 feet (24.7 to 25.9m). The WISON temperature sensitive cone measured an in situ temperature of -0.9° Celsius at 55 feet (16.8m).

The cone penetrometer encountered high resistances in the frozen silty sand. Correlations of cone resistance and in situ density were not made in the frozen soil since cone resistances were influenced by the frozen nature of the soil as well as in situ density. In situ temperature measurements were -4.3° and -2.9° Celsius.

Soil Design Parameters

We developed a curve of interpreted shear strength for Stratum I cohesive soil based on an evaluation of the field and laboratory test results. This shear strength profile is shown on the soil properties graph on Plate 6 together with the estimated submerged unit weight.

For the granular materials encountered at this site, design strength parameters were selected on the basis of in situ cone penetrometer data and estimated grain size distribution. The design strength parameters and submerged unit weights for these granular materials are also summarized on Plate 6.

AXIAL PILE DESIGN ANALYSES AND RECOMMENDATIONS

Axial Load Capacity

Fugro used the static method of analysis to predict the ultimate compressive axial load capacities of piles. We computed axial pile capacity using a procedure based on cone penetrometer test results (CPT Method). The CPT pile capacity method evolved from data gathered from many CPT soundings and associated pile load tests in unfrozen soils. For this reason, CPT pile capacity was not computed in the frozen soils found at this site. A detailed description of the method used to calculate unit skin friction and unit end bearing is presented in Appendix B.

Cone penetrometer test (CPT) results are presented in Appendix C in the form of cone resistance and sleeve friction measurements. Since cone soundings are in situ tests, they implicitly reflect in situ properties such as density, strength, grain crushability, and stress conditions. The CPT Method, therefore, uses these in situ soundings directly for pile capacity computations. Curves of unit skin friction and unit end bearing were developed using the CPT data for driven piles; these curves are shown on Plate 6.

Pile Capacity Curves

Ultimate pile capacity curves for a 30-inch driven pipe pile are presented on Plate 7. The CPT Method predicts the following ultimate loads:

Depth Below Seafloor		Ultimate Pile Capacity, Tons (Kilonewtons)	
Feet	Meters	Tension	Compression
86	26.2	111 (988)	292 (2598)

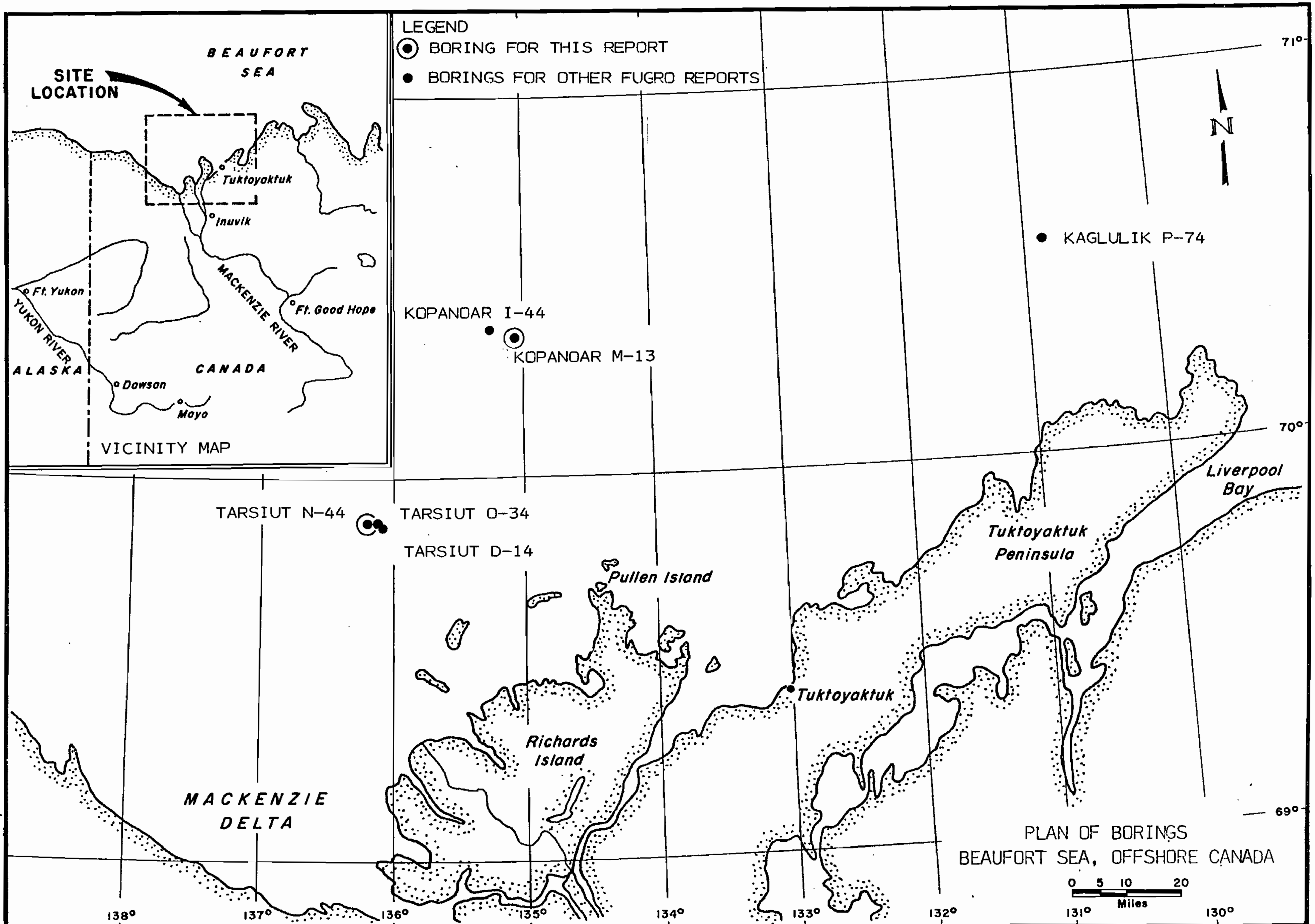
The computed skin frictional capacity of the soil plug formed inside the pile was less than the computed end bearing below Stratum I. As a result, end bearing was limited to the soil plug frictional resistance as indicated on the right graph on Plate 6. Increased end bearing can be achieved by replacing the soil plug by a grout plug.

Factors of Safety

We computed ultimate pile capacity curves as requested by Dome. Factors of safety used to compute design (allowable) pile capacity should be selected considering several factors including (1) storm frequency; (2) wave, current, and ice forces; (3) economic importance of the structure; (4) methods used in determining subsurface conditions and predetermining pile capacities; and (5) sensitivity of the structure to vertical movement. Factors of safety appropriate for pile capacities determined from CPT data are 1.5 for design and storm loads and 2.0 for operating loads.

ILLUSTRATIONS

Job No. 80-058-1

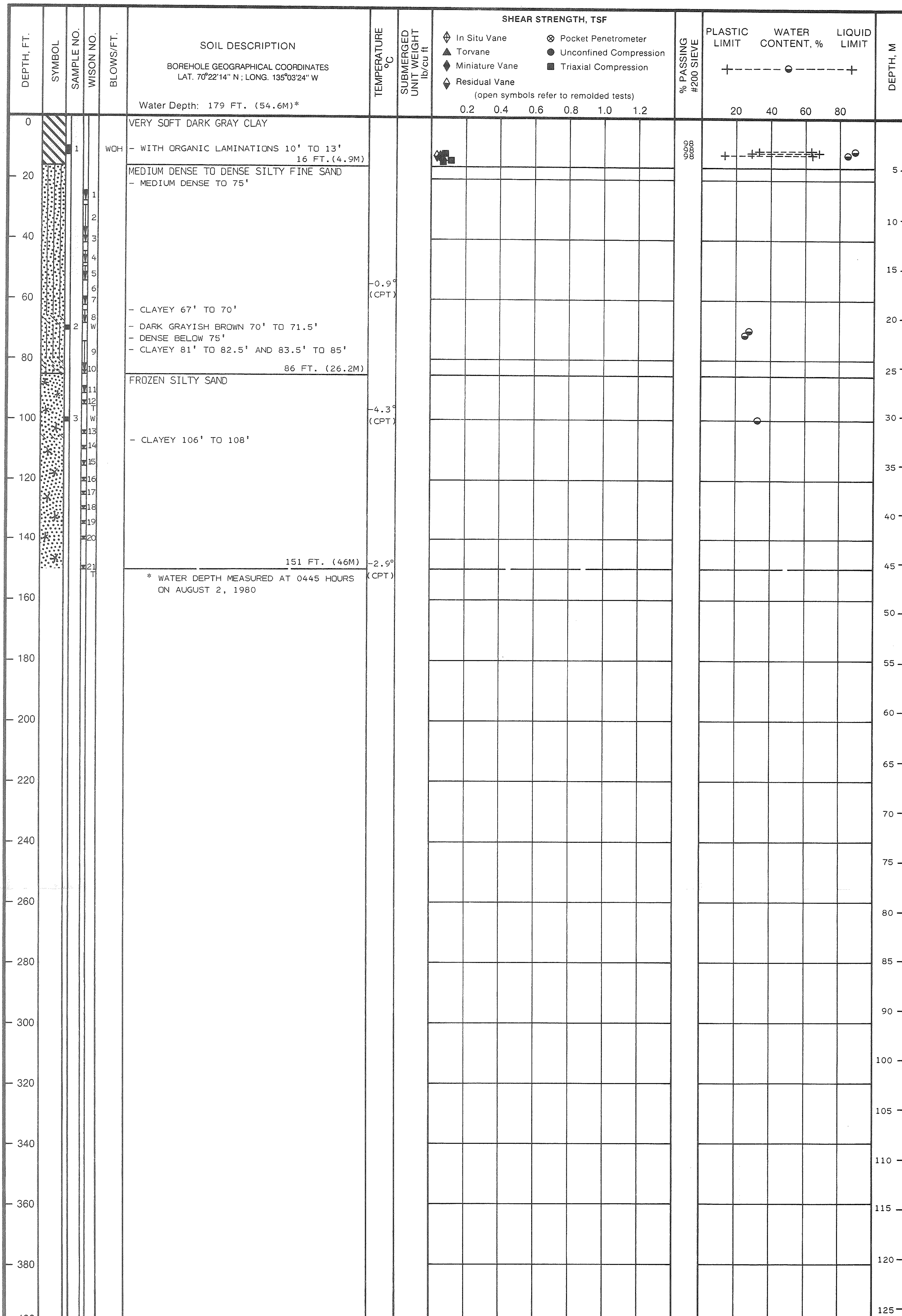


PLAN OF BORINGS
BEAUFORT SEA, OFFSHORE CANADA

FUGRO GULF, INC.
Consulting Engineers and Geologists

PLATE 1

FUGRO GULF, INC.
Consulting Engineers and Geologists



Final Penetration: 151 FT. (46M)
Field Engineer: L. S. MARR

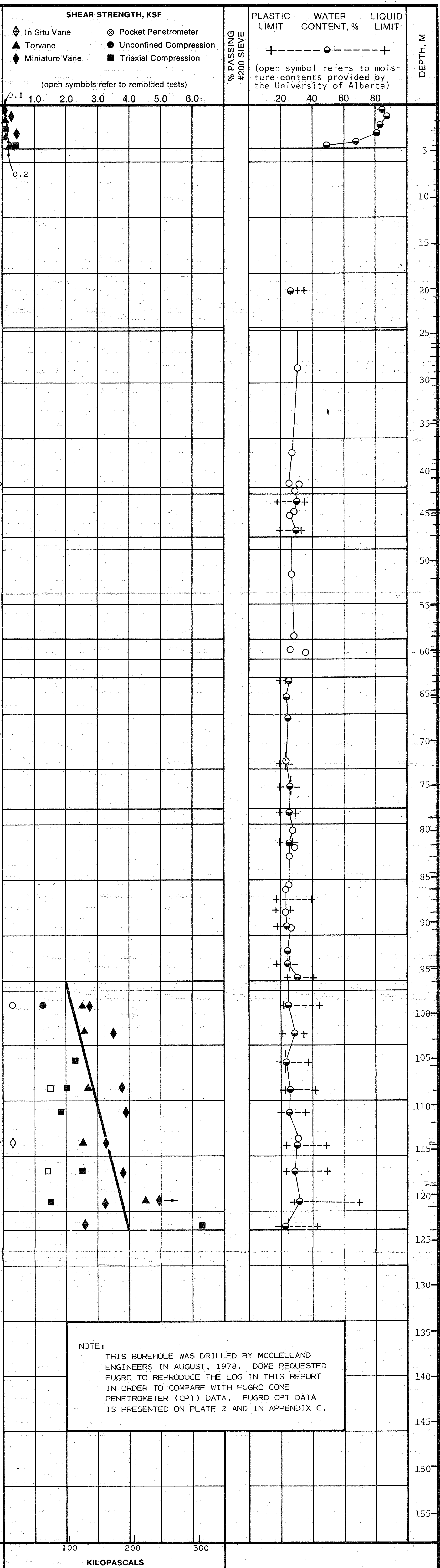
LOG OF BORING AND TEST RESULTS

BORING 1, KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA

PLATE 2

FUGRO GULF, INC.
Consulting Engineers and Geologists

DEPTH, FT.	SAMPLE NO.	BLOWS/FT.**	SOIL DESCRIPTION	SUBMERGED UNIT WEIGHT lb/cu ft
			Water Depth: 186' AT 1925 HRS, AUG 3, 1978	
0	1	PUSH	VERY SOFT OLIVE GRAY CLAY	30
	2	1	- WITH BLACK ORGANIC STREAK TO 18'	30
	3	PUSH	- MEDIUM GRAY, 3' TO 12'	32
	4	PUSH		36
	5	1	15.5'(4.7M)	44
	6	12	LIGHT OLIVE GRAY CALCAREOUS SILTY FINE SAND	
20	7	20	- SLIGHTLY CALCAREOUS AT 16'	
	8	20	- WITH CLAYEY SAND LAYER AND SHELL FRAGMENTS AT 16'	
	9	20		
	10	20		
	11	15		
	12	15	- WITH FINE GRAVEL, 24' TO 26'	
	13	15	- WITH ORGANIC STREAK AT 24.5'	
40	14	13	- OLIVE GRAY BELOW 30'	
	15	13	- WITH BLACK SEAMS, 39' TO 48'	
60	16	13	- WITH CLAY SEAMS AT 56' - WITH BLACK POCKETS, 56' TO 58'	
	17	15	- WITH CLAYEY SILT LAYER, 66' TO 66.5' - SLIGHTLY SILTY, 66.5' TO 67.5'	
	18	17	- WITH FROZEN SEAMS AT 77' 81'(24.7M)	
80			FROZEN OLIVE GRAY FINE SAND	
	19	17	- SILTY TO 116'	
	20	20		
	21	20	- CALCAREOUS TO 120'	49*
	NR			
	22	20	- TRACE OF BLACK ORGANICS BELOW 85'	
100	23		- WITH CLAY AND SILT LAMINATIONS AT 87.5' - WITH VISIBLE ICE AT 87.5'	
	24	40	- WITH CLAY SEAMS AND LAYERS, 91' TO 92'	
	25	60	- WITH SMALL WOOD PIECES, 91' TO 92' AND AT 112'	
120	26	100	- WITH SANDY SILT LAYERS, 96' TO 97'	
	27	80	- WITH CLAY POCKETS AND VISIBLE ICE FRAGMENTS, 97' TO 106'	
	28		- SLIGHTLY CALCAREOUS BELOW 120'	
	NR			
	29	90	- WITH CLAY POCKETS AND SHELL FRAGMENTS BELOW 136' 138'(42.0M)	46*/49*
140	30		FROZEN LAMINATED SILT, SAND AND CLAY	60*
	NR			51*
	31		- WITH VISIBLE 1/8" ICE VEINS TO 147' AND 151' TO 155'	50*
	32			53*
	33		- WITH SANDY SILT LAYER, 142' TO 143' 156'	
	34		- LIGHT OLIVE GRAY, 146' TO 193' (47.6M)	52*
160			FROZEN OLIVE GRAY FINE SAND, SLIGHTLY CALCAREOUS	
	35		- WITH CLAY AND SILT SEAMS, 166' TO 170' - WITH ORGANIC POCKETS AND STREAKS, 166' TO 170' AND 180' TO 180.5' - WITH SANDY SILT LAYER, 167' TO 168'	
180	NR			
	36	60	- SILTY, 180' TO 190'	
	37		- WITH WOOD PIECES BELOW 182'	
	NR			
	38	44	- WITH COARSE SAND SEAM AT 192.5'	
	39	120	193'(58.8M)	
	40	35	FROZEN OLIVE GRAY LAMINATED SAND, SILT & CLAY	58*
200	41			51*
	NR			
	42		- WITH 1/8" ICE VEINS, 197' TO 198' 207'(63.1M)	
	43	30		
	44	50	FROZEN OLIVE GRAY LAMINATED CALCAREOUS SAND AND SILT	
	45	60		
	46		- WITH VISIBLE ICE AT 209' AND 226'	
220	NR	20	- CLAYEY, 205' TO 215' AND 235' TO 245'	
	47	150	- WITH WOOD AND SHELL FRAGMENTS, 224' TO 225'	
	48	48		
240	49	50		
	50			
	51	55	- WITH SAND POCKETS, 251' TO 252'	
	52	50	255'(77.7M)	
	53	50	FROZEN OLIVE GRAY LAMINATED SILTY CLAY AND SILT WITH INCLINED PARALLEL BEDDING	
260	54	50		59*
	55	70	- TRACE OF BLACK ORGANICS AT 267' AND 285' TO 305'	
	56	20		
	57	70	- CLAY LAYER AT 286.5'	
280	58	60	- SANDY, 277' TO 287'	59*
	59	20	- WITH FINE GRAVEL AT 287.5'	
	60	20 35	- WITH WOOD FRAGMENTS AT 289' - WITH 1/16" VISIBLE ICE VEIN AT 291'	
	61	13	- SANDY SILT, 296' TO 297.5'	59*
300			- SANDY CLAYEY SILT, 306' TO 306.5'	
	62	60		
	63	30		
	64	30	317'(96.6M)	
320			VERY STIFF OLIVE GRAY CLAY - CALCAREOUS TO 340'	
	65	30	- SILTY WITH NUMEROUS INCLINED SILT PARTINGS AND SEAMS TO 367'	51
		20		
340	66	20	- EVEN, HORIZONTAL, PARALLEL BEDDING BELOW 336'	
	67	20	- WITH ORGANIC STREAKS, 346' TO 348'	58
360	68	20		59
	69	20		64
380	70	20	- WITH 1/4" VISIBLE ICE WEDGE AT 377' - WITH BLACK ORGANIC STREAKS, 386' TO 398'	54*/59*
	71	15	- GASSY, FLOCCULATED, 396' TO 397.5' - WITH ORGANIC ODOR BELOW 396'	58
400	72	20	- SILTY WITH SAND AND SILT SEAMS AND POCKETS BELOW 406'	54
	73	30	- WITH SHELL FRAGMENTS BELOW 406' 407'(124.0M)	



* SUBMERGED UNIT WEIGHTS PROVIDED BY THE UNIVERSITY OF ALBERTA.

**NUMBER OF BLOWS OF A 175-LB WEIGHT DROPPED APPROXIMATELY 5-FT REQUIRED TO PRODUCE A 12-IN. PENETRATION, EXCEPT AS NOTED; OF A 2.25-IN. OD, 2.125-IN. ID SAMPLER. 2-IN. SPLIT-BARREL USED AT 86', 135' AND 180' PENETRATION. HEIGHT OF DROP OF 175-LB WEIGHT APPROXIMATELY 1' TO 2' BETWEEN 81' AND 300' PENETRATION.

NOTE: THIS BOREHOLE WAS DRILLED BY MCCLELLAND ENGINEERS IN AUGUST, 1978. DOME REQUESTED FUGRO TO REPRODUCE THE LOG IN THIS REPORT IN ORDER TO COMPARE WITH FUGRO CONE PENETROMETER (CPT) DATA. FUGRO CPT DATA IS PRESENTED ON PLATE 2 AND IN APPENDIX C.

Final Penetration: 407' (124M)
Field Engineer: J. A. WOOLEY

LOG OF BORING AND TEST RESULTS

KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA

KEY TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPE
(Shown in Symbol Column)

SAMPLE TYPE
(Shown in Samples Column)

Sand Silt Clay

Sandy Silty Clayey Wison CPT Undisturbed Rock Core Split Spoon No Recovery

Predominant type shown heavy

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (Major Portion Retained on No. 200 Sieve)

Includes (1) clean gravels & sand described as fine, medium or coarse, depending on distribution of grain sizes & (2) silty or clayey gravels & sands (3) fine grained low plasticity soils (PI < 10) such as sandy silts. Condition is rated according to relative density, as determined by lab tests or estimated from resistance to sampler penetration.

Descriptive Term	Relative Density
Loose	0 to 40 %
Medium Dense	40 to 70 %
Dense	70 to 90 %
Very Dense	90 to 100 %

FINE GRAINED SOILS (Major Portion Passing No. 200 Sieve)

Includes (1) inorganic & organic silts & clays, (2) sandy, gravelly or silty clays, & (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests for soils with PI ≥ 10

Descriptive Term	Cohesive Shear Strength Tons/Sq. Ft.
Very Soft	Less Than 0.125
Soft	0.125 to 0.25
Firm	0.25 to 0.50
Stiff	0.50 to 1.00
Very Stiff	1.00 to 2.00
Hard	2.00 and Higher

NOTE: SLICKENSIDED AND FISSURED CLAY MAY HAVE LOWER UNCONFINED COMPRESSIVE STRENGTHS THAN SHOWN ABOVE, BECAUSE OF PLANES OF WEAKNESS OR SHRINKAGE CRACKS; CONSISTENCY RATINGS OF SUCH SOILS ARE BASED ON HAND PENETROMETER READINGS

TERMS CHARACTERIZING SOIL STRUCTURE

Parting:	paper thin in size
Seam:	1/8"–3" thick
Layer:	greater than 3"
Fissured:	containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical
Sensitive:	pertaining to cohesive soils that are subject to appreciable loss of strength when remolded
Interbedded:	composed of alternate layers of different soil types
Laminated:	composed of thin layers of varying color and texture
Calcareous:	containing appreciable quanities of calcium carbonate
Well Graded:	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Poorly Graded:	predominately of one grain size, or having a range of sizes with some intermediate size missing
Flocculated:	pertaining to cohesive soils that exhibit a loose knit or flakey structure
Slickensided:	having inclined planes of weakness that are slick and glossy in appearance

DEGREE OF SLICKENSIDED DEVELOPMENT

Slightly Slickensided:	slickensides present at intervals of 1'-2'; soil does not easily break along these planes
Moderately Slickensided:	slickensides spaced at intervals of 1'-2'; soil breaks easily along these planes
Extremely Slickensided:	continuous and interconnected slickensides spaced at intervals of 4"–12"; soil breaks along the slickensides into pieces 3"–6" in size
Intensely Slickensided:	slickensides spaced at intervals of less than 4", continuous in all directions; soil breaks down along planes into nodules 1/4"–2" in size

FUGRO GULF, INC.
Consulting Engineers and Geologists

PLATE 4

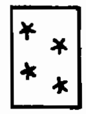
Job No. ______

FORM D 101

Job No. 80-058

KEY TO SOIL CLASSIFICATION AND SYMBOLS FOR FROZEN SOILS

SYMBOL TYPE
(Shown in Symbol Column)



FROZEN SOIL
(Overlays Soil Symbol)

ICE DESCRIPTION (After NRC TM No. 79) *

GROUP	ABBREVIATION	DESCRIPTION
NON VISIBLE ICE	Nf	Poorly Bonded
	Nbn	Well Bonded
	Nbe	Excess Ice
VISIBLE ICE LESS THAN 1 INCH THICK	Vx	Individual Ice Crystals or Inclusions
	Vc	Ice Coatings or Particles
	Vr	Random or Irregularly Oriented Ice Formations
	Vs	Stratified or Distinctly Oriented Ice Formations
VISIBLE ICE GREATER THAN 1 INCH THICK	ICE+	(Soil Type) Ice with Soil Inclusions
	ICE	Ice without Soil Inclusions

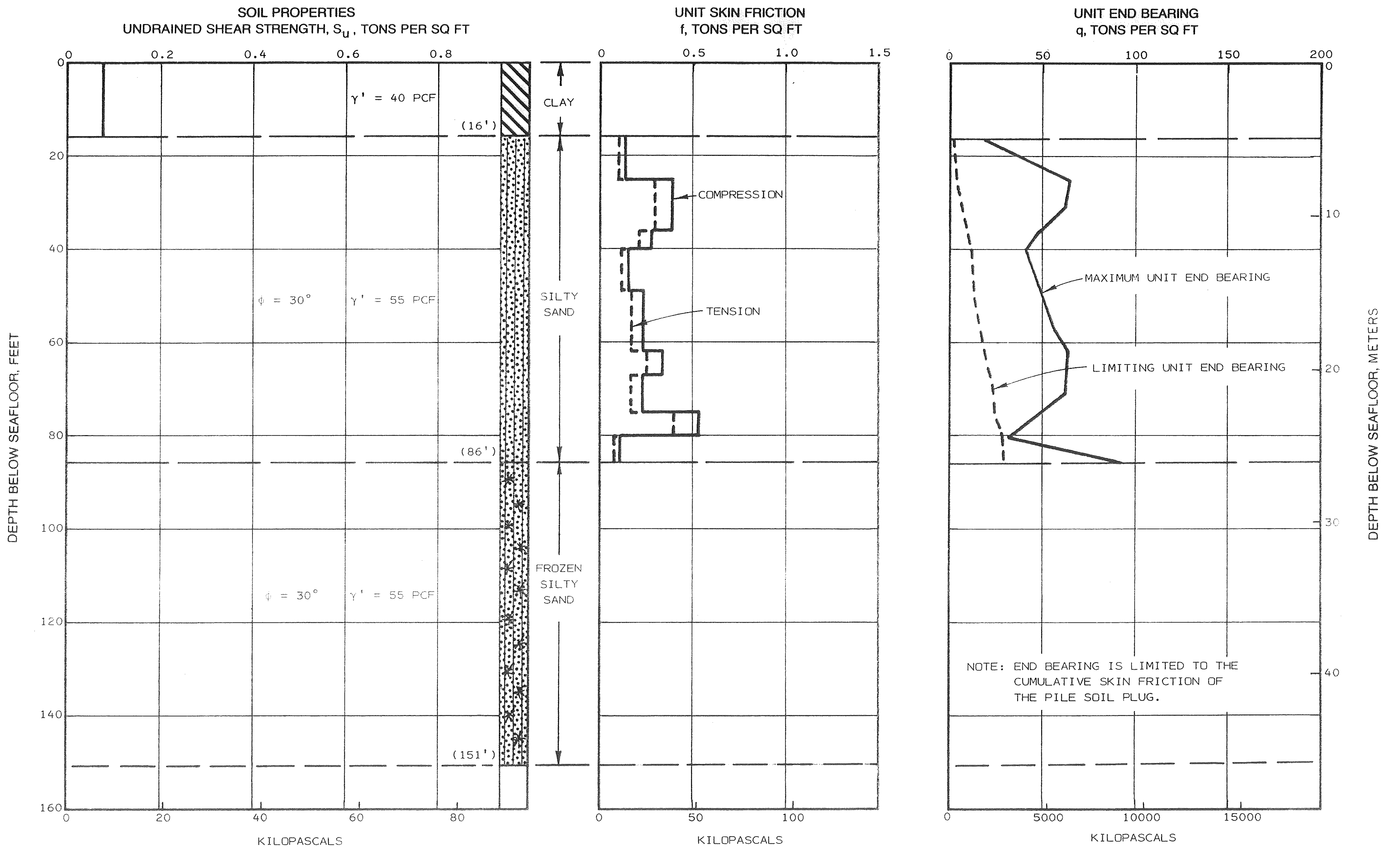
* NATIONAL RESEARCH COUNCIL (CANADA) TECHNICAL MANUAL

FUGRO GULF, INC.
Consulting Engineers and Geologists

Job No. 80-058-3

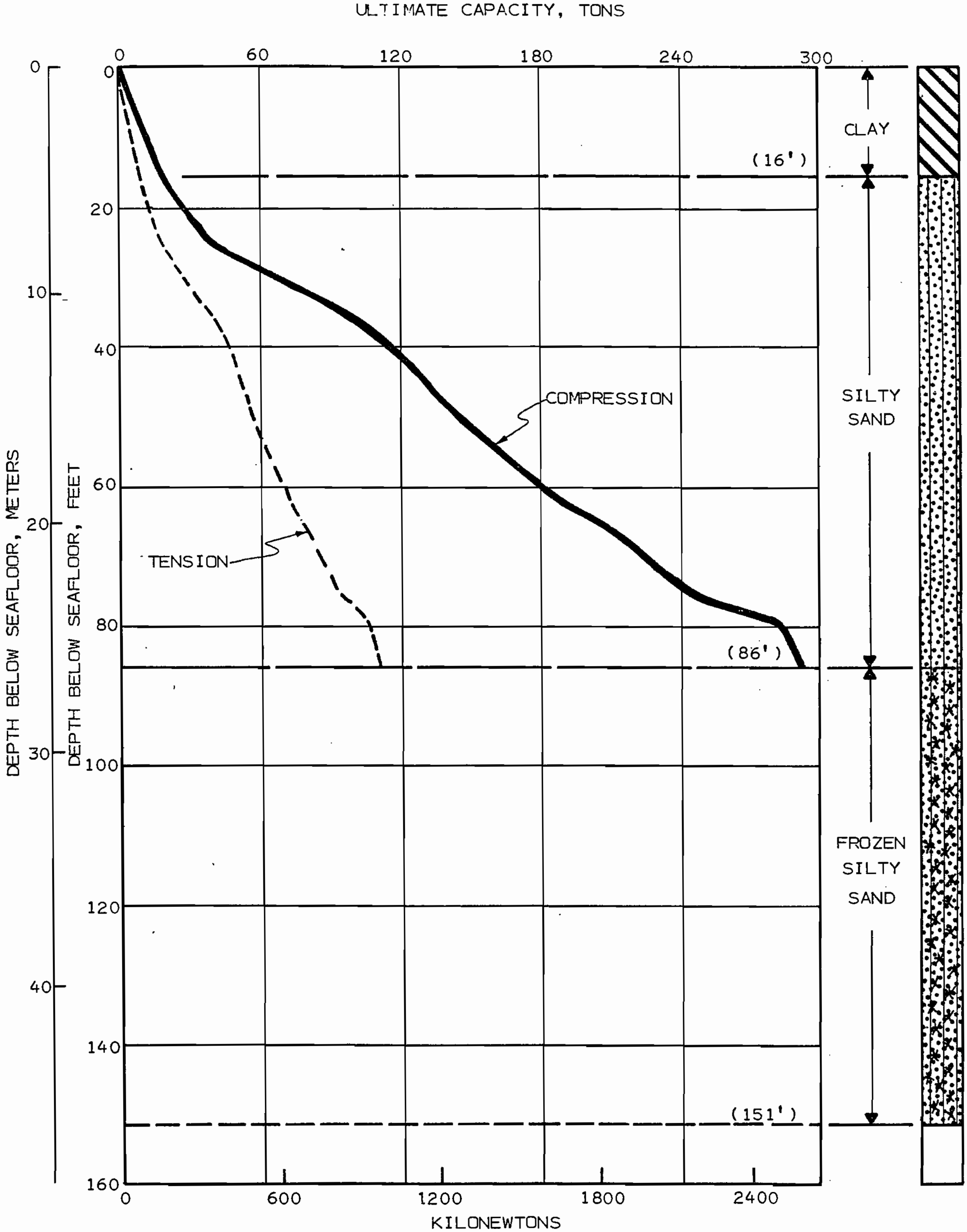
SUMMARY OF FIELD ACTIVITIES
KOPANOAR, M-13
BEAUFORT SEA, OFFSHORE CANADA

DATE	TIME FROM	TIME TO	DESCRIPTION OF ACTIVITY
8/01/80	13:50	23:00	Travel to Kopanoar Area M-13.
	23:00	24:00	Calibrate ARGO system.
8/02/80	00:00	03:50	Set anchors; reset one anchor.
	03:50	06:00	Prepare for drilling. Water depth is 179 ft. at 0445 hours.
	06:00	10:30	Lower drill pipe. Location of previous Kopanoar M-13 boring: Lat. 70° 22' 13" N Long. 135° 03' 24" W. Location of this boring: Lat. 70° 22' 14" N Long. 135° 03' 24" W.
	10:30	20:00	Drill and Wison test 0 to 25 ft.
	20:00	22:00	Reverse combicable.
	22:00	24:00	Drill and Wison test 25 to 40 ft.
8/03/80	00:00	24:00	Drill and Wison test 40 to 120 ft.
8/04/80	00:00	10:30	Drill and Wison test 120 to 150 ft.
	10:30	-	Lost drill string while trying to retrieve 1.5m Wison.
	10:30	14:30	Wait at location for divers. Lost drill bit, packer, top sub, spacer sub, signaling sub, locking sub, 1.5m Wison, temp. cone, RMC/Wison connector, sixteen 20-ft joints, two 10-ft. joints, one 5-ft. joint.
	14:30	21:00	Wait at location for divers; seas building and too rough to drill.
	21:00	24:00	Wait at location. Divers on board: seas too rough to drill. Divers unable to find drill pipe.
8/05/80	00:00	01:25	Pull anchors.



SOIL PROPERTIES AND PILE CAPACITY DATA
CPT METHOD
BORING 1, KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA

Job No. 80-058-3



ULTIMATE PILE CAPACITY CURVES
CPT METHOD
30-INCH DRIVEN PIPE PILE
BORING 1, KOPANOAR M-13
BEAUFORT SEA, OFFSHORE CANADA

FUGRO GULF, INC.
Consulting Engineers and Geologists

PLATE 7

APPENDIX A

LABORATORY TEST RESULTS (EBA)

TABLE B-1 SUMMARY OF ATTERBERG LIMIT TESTING, KOPANOAR I-44

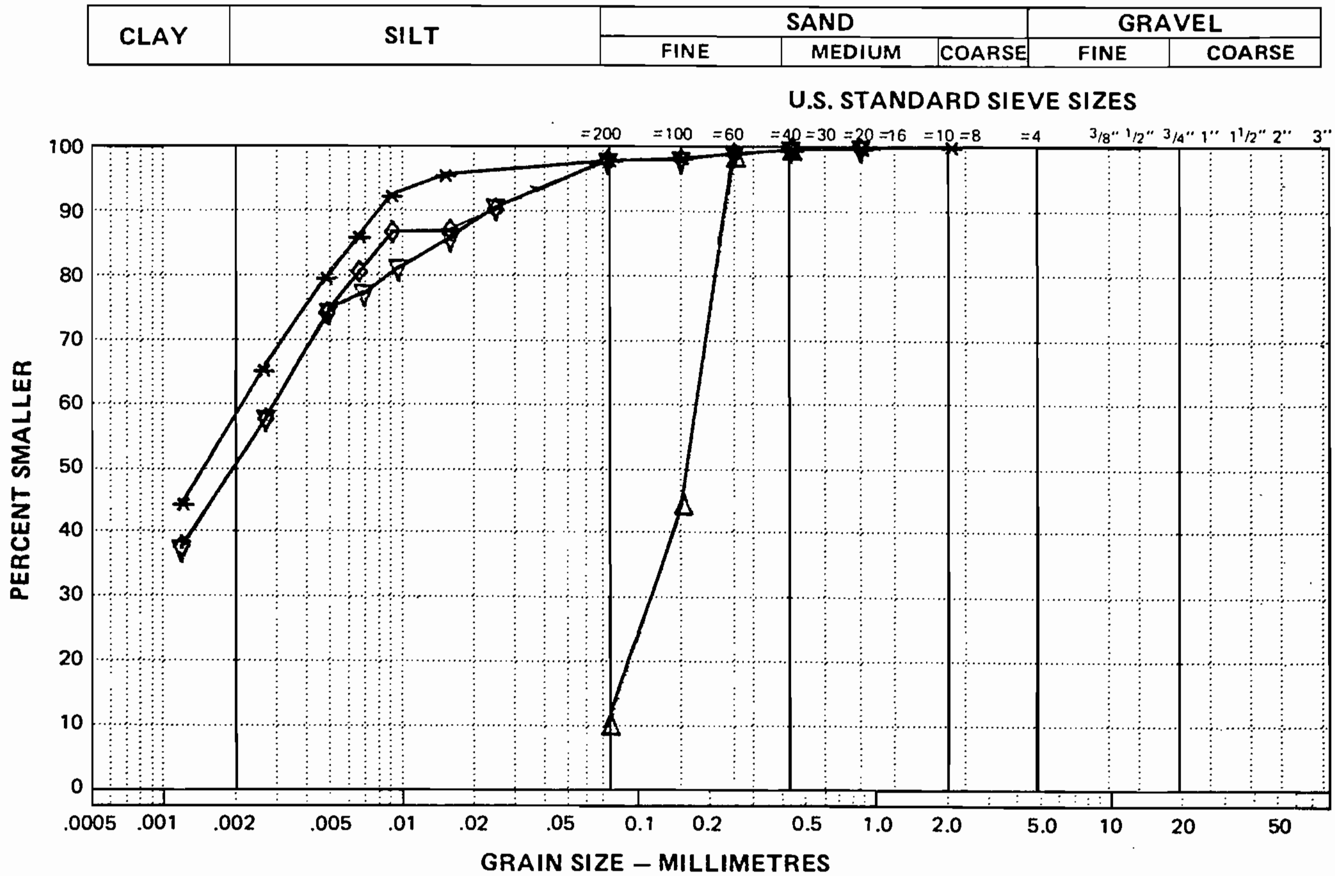
SAMPLE NUMBER	DEPTH INTERVAL (m)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	USC	LIQUIDITY INDEX
1C	0.25 - 0.30	73	55	27	28	CH	1.6
2B	0.70 - 0.85	--	64	32	32	CH	---
3A	1.59 - 1.63	87	66	30	36	CH	1.6
4A	2.15 - 2.28	78	56	31	25	MH	1.9
5D	2.87 - 3.00	85	64	32	32	MH	1.7
6D	3.48 - 3.60	87	64	31	33	CH	1.7
7D	4.07 - 4.20	63	53	23	30	CH	1.3
8A	4.65 - 4.80	78	63	28	35	CH	1.4
9A	5.25 - 5.32	81	53	31	28	CH	1.8
10C	5.80 - 5.93	48	51	27	24	CH	0.9
12A	6.90 - 7.00	50	45	19	26	CL	1.2
13B	7.57 - 7.70	37	32	17	15	CL	1.3
15A	8.60 - 8.65	38	27	13	14	CL	1.8
22A	17.70 - 17.85	22	29	18	11	CL	0.4
23B	18.05 - 18.15	26	25	15	10	CL	1.1
24A	20.95 - 21.15	29	nonplastic			ML	
24B	21.15 - 21.25	27	32	20	12	CL	0.6
25B	24.15 - 24.27	25	29	16	13	CL	0.7
26C	27.15 - 27.30	24	30	15	15	CL	0.6
27A	30.10 - 30.25	25	30	16	14	CL	0.6
28A	33.25 - 33.45	22	23	19	4	CL-ML	0.8
29C	36.25 - 36.40	23	26	15	11	CL	0.7
32A	45.20 - 45.35	24	24	21	3	CL-ML	1.0
35C	54.53 - 54.66	--	22	14	8	CL	---
36A	57.55 - 57.70	--	25	21	4	CL-ML	---
37B	60.60 - 60.75	--	31	18	13	CL	---
38A	63.68 - 63.80	23	28	17	11	CL	0.5
39C	66.65 - 66.80	21	23	16	7	CL-ML	0.7
43B	79.15 - 79.25	22	25	17	8	CL	0.6
45C	84.55 - 84.70	28	nonplastic			ML	
51A	92.30 - 92.35	26	21	18	3	ML	2.7
58B	104.20 - 104.35	28	nonplastic			ML	
59B	106.10 - 106.30	21	24	15	9	CL	0.7
63A	112.65 - 112.80	22	23	17	6	CL-ML	0.8
67C	121.55 - 121.75	27	nonplastic			ML	
1A(CPT)	3.15 - 3.30	92	64	33	31	MH	1.9
1B(CPT)	3.30 - 3.45	86	68	29	39	CH	1.5
1C(CPT)	3.45 - 3.60	81	64	14	50	CH	1.3
1F(CPT)	3.90 - 4.00	85	62	28	34	CH	1.7

SUMMARY OF LABORATORY TESTING RESULTS – PERMAFROST

SAMPLE NO.	SAMPLE TYPE	DEPTH INTERVAL (metres)	MOIST. CONT. (%)	BULK DENS. (Mg/m³)	GROUND ICE DESCRIPTION (%)	ATTERBERG LIMITS LL (%)	ATTERBERG LIMITS PL (%)	ATTERBERG LIMITS PI (%)	GRAIN SIZE DISTRIBUTION CLAY (%)	GRAIN SIZE DISTRIBUTION SILT (%)	GRAIN SIZE DISTRIBUTION SAND (%)	GRAIN SIZE DISTRIBUTION GRAV. (%)	ORGANIC CONTENT (%)	SOIL DESCRIPTION
1A	L	3.15-3.30	92	1.52	NOT FROZEN	64	33	31	59	39	2	0		(MH) SILT & CLAY
1B	L	3.30-3.45	86	1.53	NOT FROZEN	68	29	39	51	47	2	0		(CH) CLAY & SILT
1C	L	3.45-3.60	81	1.52	NOT FROZEN	64	14	50	51	47	7	0		(CH) CLAY & SILT
1D	V	3.60-3.75			NOT FROZEN									
1E	V	3.75-3.90	89		NOT FROZEN									
1F	V	3.90-4.00	85		NOT FROZEN	62	28	34						(CH) CLAY
2A	B	21.30-21.35	28		NOT FROZEN				-	10	90	0		SAND
2B	C	21.55-21.70			NOT FROZEN	Nonplastic			1	11	88	0		(SP-SM) SAND
2C	B	21.70-21.75	26		NOT FROZEN				-	20	80	0		(SM/SC) SAND
3A	PF	30.50-30.65			FROZEN	Nonplastic			0	13	87	0		(SM) SAND
3B	B	30.65-30.70	33		FROZEN				4	20	76			(SM/SC) SAND

PROJECT NUMBER 101-2941 SITE NUMBER KOPANOAR CPT HOLE

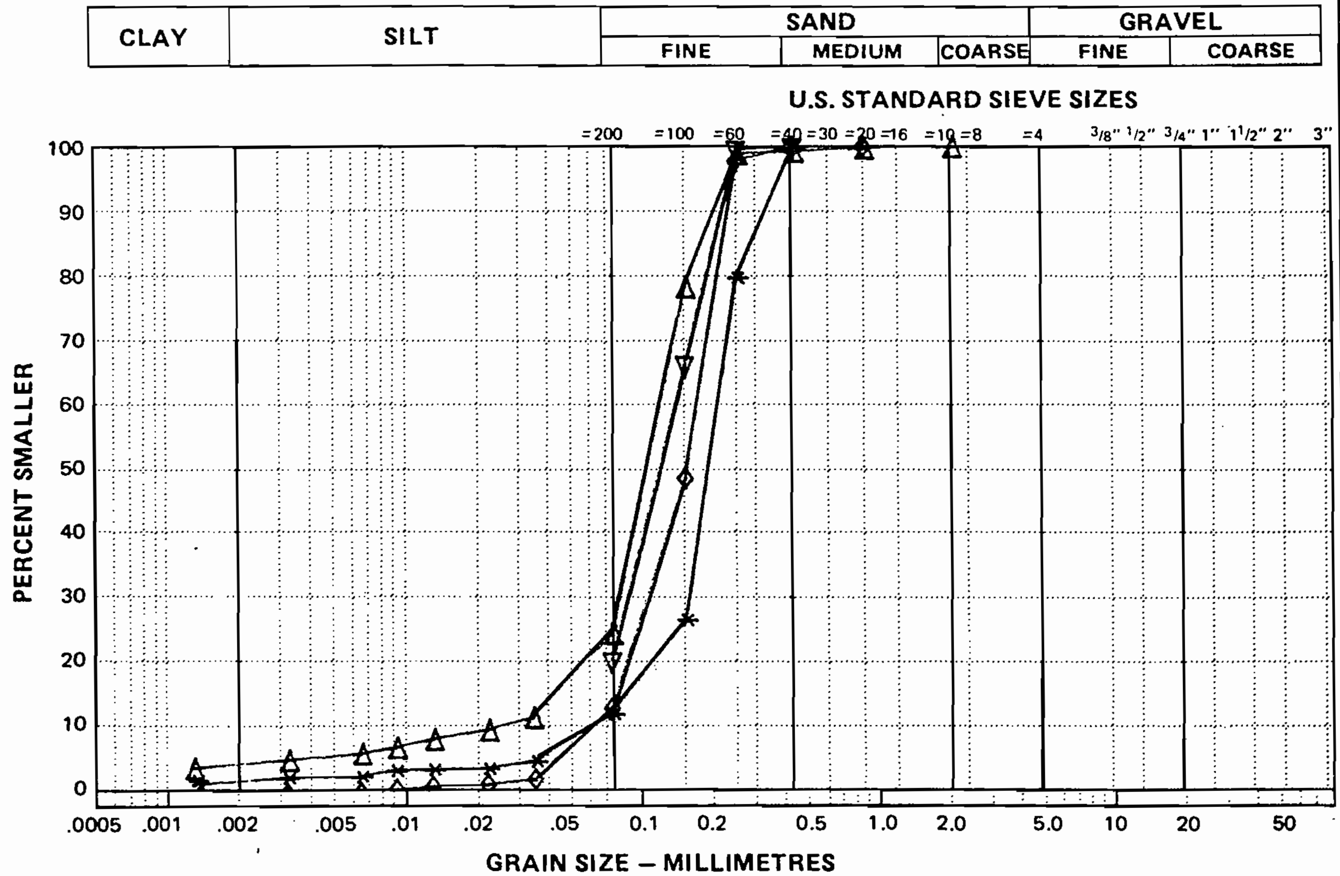
PARTICLE - SIZE ANALYSIS OF SOILS



SYMBOL	BOREHOLE NUMBER	DEPTH (m)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)	Cu	Cc	U.S.C.
—✳—	KOP CPT	3.15– 3.30	58	40	2	Ø	–	–	MH
—▽—	KOP CPT	3.30– 3.45	50	47	2	Ø	–	–	CH
—◇—	KOP CPT	3.45– 3.60	51	48	2	Ø	–	–	CH
—△—	KOP CPT	21.30– 21.35	–	10	90	Ø	–	–	

JOB NO. 1Ø1–2941 **DATE** 81– 4–15

PARTICLE - SIZE ANALYSIS OF SOILS



SYMBOL	BOREHOLE NUMBER	DEPTH (m)	DESCRIPTION CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)	Cu	Cc	U.S.C.
—✳—	KOP CPT	21.55– 21.70	1	10	88	0	3.4	1.9	SP–SM
—▽—	KOP CPT	21.70– 21.75	–	20	80	0	–	–	SM/SC
—◇—	KOP CPT	30.50– 30.65	0	13	87	0	2.7	1.0	SM
—△—	KOP CPT	30.65– 30.70	4	20	76	0	4.6	2.1	SM/SC

JOB NO. 101–2941 **DATE** 81– 4–15

TABLE D-1 SUMMARY OF LABORATORY VANE SHEAR TEST RESULTS, KOPANOAR I-44

SAMPLE NUMBER	DEPTH INTERVAL (m)	WATER CONTENT (%)	MINIATURE VANE PEAK (kPa)	MINIATURE VANE RESIDUAL (kPa)	SENSITIVITY
1C	0.25 - 0.30	73	5	2	2.5
2D	1.00 - 1.15	-	4	1	4.0
3A	1.50 - 1.63	87	6	1	6.0
4A	2.15 - 2.28	78	6	2	3.0
5D	2.87 - 3.00	85	9	3	3.0
6D	3.48 - 3.60	87	5	2	2.5
7D	4.07 - 4.20	63	4	2	2.0
8A	4.65 - 4.80	78	5	2	2.5
10C	5.80 - 5.93	48	9	3	3.0
12C	7.15 - 7.30	36	12	5	2.4
1D(CPT)	3.60 - 3.75	-	7	3	2.3
1E(CPT)	3.75 - 3.90	89	2	1	2.0

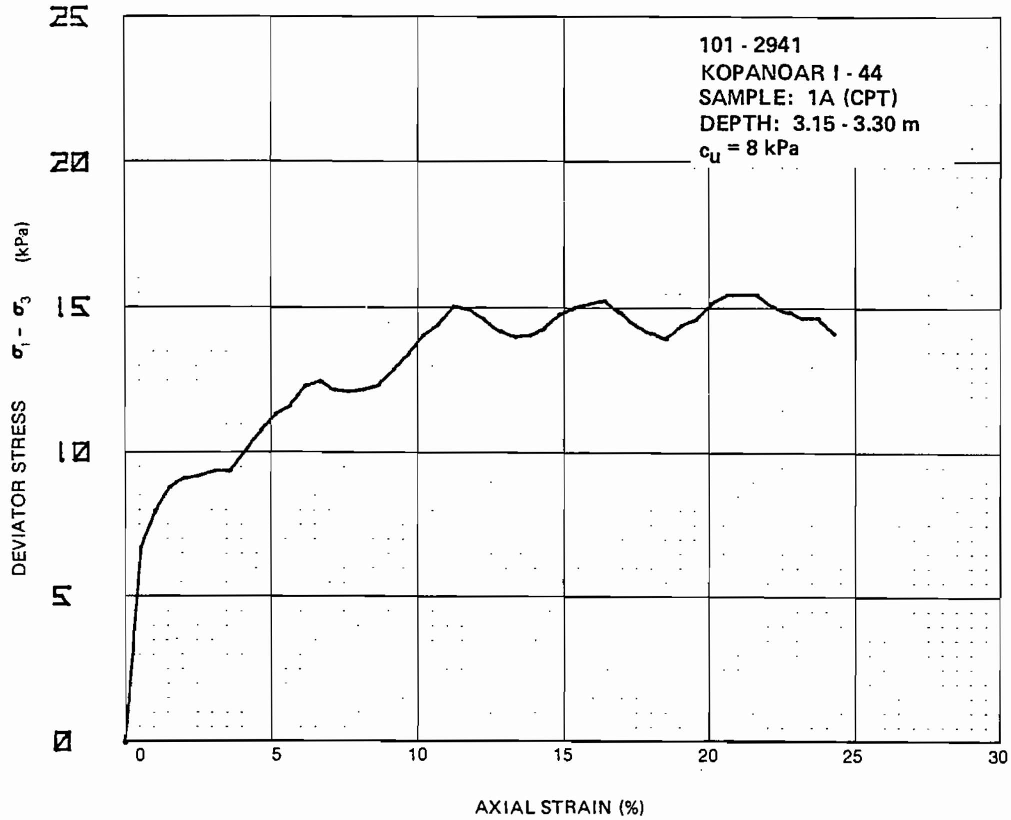
NOTE: 1. Tests performed in field laboratory.

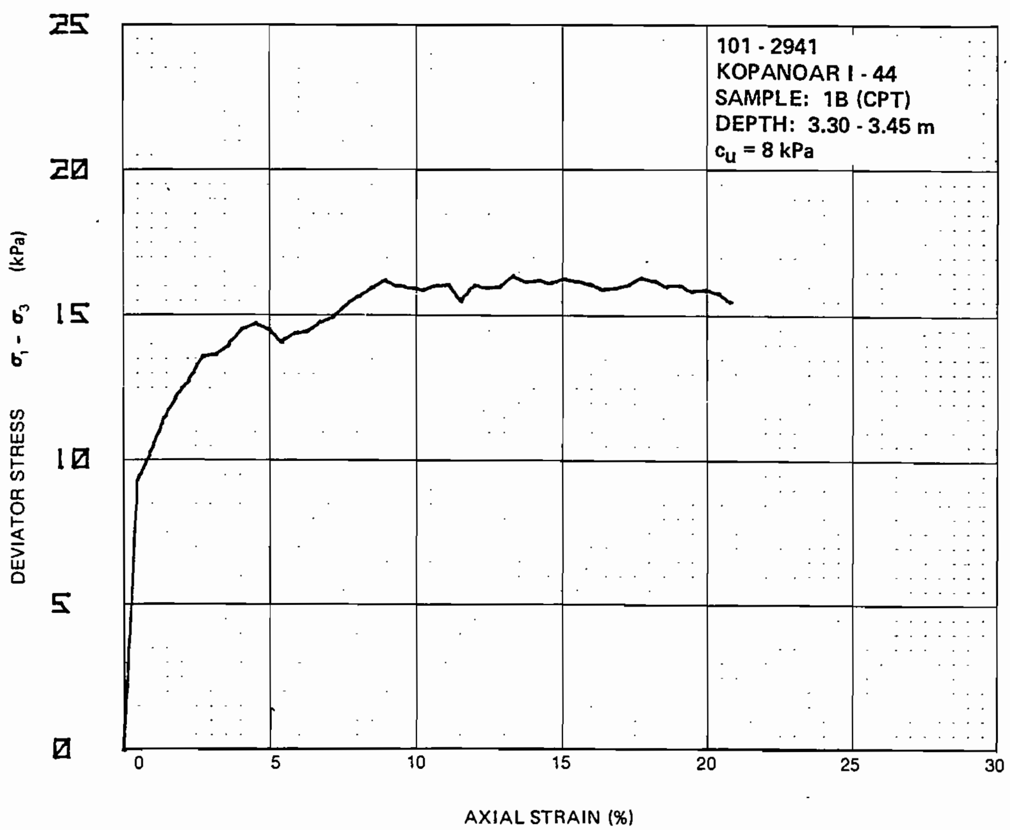
TABLE D-3 SUMMARY OF RESULTS FROM UNCONSOLIDATED-UNDRAINED TRIAXIAL TESTS, KOPANOAR I-44

SAMPLE NUMBER	DEPTH INTERVAL (m)	USC	MOISTURE CONTENT (%)	INITIAL BULK DENSITY (Mg/m^3)	CONFINING PRESSURE (kPa)	AXIAL STRAIN AT FAILURE (%)	UNDRAINED SHEAR STRENGTH (JACKETED TRIAXIAL) (kPa)	REMARKS
2B	0.70-0.85	CH						Disturbed
5C	2.75-2.87	CH	79	1.50	360	15	6	
9B	5.32-5.45	CH	40	1.88	385	12	17	
12B	7.30-7.45	CL						Disturbed
23A	17.45-18.05	CL	24	2.05	405	15	50	
26C	27.15-27.30	ML	24	2.08	625	15	58	
27B	30.37-30.40	CL	25	2.01	660	15	65	
29D	36.40-36.55	CL	22	2.10	730	15	67	
34B	51.60-51.75	ML	21	2.10	910	15	81	
35C	54.53-54.66	ML	21	2.09	940	15	64	
38A	63.68-63.80	ML	23	2.03	1045	15	63	
41B*	72.90-73.10	ML	20	2.05	1155	15	149	
43A*	79.05-79.15	ML	21	2.05	1225	15	45	
46B*	85.10-85.25	ML	24	1.97	1295	14	393	
52A*	93.65-93.90	ML	28	1.89	1395	15	104	
53A*	95.20-95.40	ML	27	1.90	1415	15	600	
+56B*	98.85-99.05	ML	22	2.02	1407	15	106	Low Value
59A*	106.30-106.70	ML	28	1.89	1540	15	150	
64B*	113.90-114.10	ML	24	1.94	1630	15	165	
65A*	115.60-115.80	ML	18	2.10	1650	15	180	
1A(CPT)	3.15- 3.30	MH	92	1.52	250	15	8	
1B(CPT)	3.30- 3.45	CH	86	1.53	350	15	8	
1C(CPT)	3.45- 3.60	CH	81	1.52	450	15	11	

Note:
1. 15% axial strain assumed as failure unless peak stress obtained earlier.
2. Asterisk indicates sample in frozen state prior to testing. Initial bulk density reported is therefore frozen bulk density.
3. Test with pore pressure measurement (+) were done at slower rates.

UNCONSOLIDATED — UNDRAINED TRIAXIAL TESTS





UNCONSOLIDATED — UNDRAINED TRIAXIAL TESTS

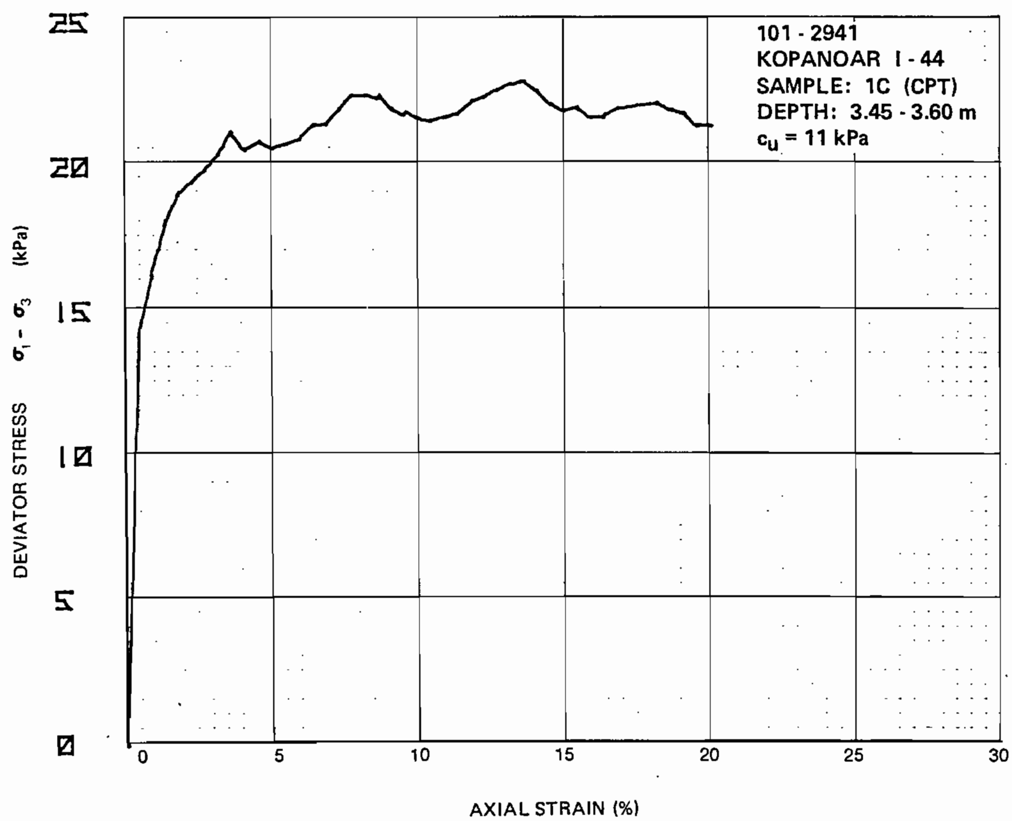


TABLE D-7 TRIAXIAL TEST RESULTS - CURVE HOPPING TECHNIQUE

KOPANOAR CPT BOREHOLE

SAMPLE NUMBER	DEPTH INTERVAL	USC	SAMPLE HEIGHT	SAMPLE DIAMETER	BACK PRESSURE	STRAIN RATE	MOISTURE CONTENT INITIAL	MOISTURE CONTENT FINAL	INITIAL DRY DENSITY	EFFECTIVE CONFINING PRESSURE	CONDITIONS AT FAILURE $\frac{\sigma_1' + \sigma_3'}{2}$	CONDITIONS AT FAILURE $\frac{\sigma_1' - \sigma_3'}{2}$	CONDITIONS AT FAILURE AXIAL STRAIN	EFFECTIVE COHESION INTERCEPT	EFFECTIVE FRICTION ANGLE
	(m)		(mm)	(mm)	(kPa)	(%/min)	(%)	(%)	(Mg/m^3)	(kPa)	(kPa)	(kPa)	(%)	(kPa)	(degrees)
CONSOLIDATED-DRAINED (RECONSTITUTED)															
2B	21.55-21.7	SP/SM	70	38	552	0.1	9	24	1.46	100	238	138	12		
										200	428	228	11	28	28
CONSOLIDATED-UNDRAINED (PERMAFROST)															
3A	30.5 -30.65	SM	123	61	276	0.1	30	31	1.42	124	413	239	7		
										248	500	286	8	20	33

NOTES:

1. The reconstituted sand sample formed in a split mould and the sand placed at a moisture content of 9%. A relative density of 56% was obtained for Sample 2B. The permafrost sample 3A was at a relative density of 47% when thawed.
2. Minimum porewater pressure response of 0.95 for both samples obtained by back pressure saturation.
3. Samples consolidated isotropically.
4. Cell pressure alternated and equilibrated at intervals of 1% axial strain.
5. Peak stress ratio failure was considered in CD triaxial test on Sample 2B. Stress conditions at peak obliquity agreed with shear strength envelopes defined by failure at maximum deviatoric stress.
6. Samples 2B and 3A were texturally similar. Considering all four failure points on the same envelope results in effective shear strength parameters as follows: Friction Angle: 33 degrees Cohesion Intercept: 6 kPa.

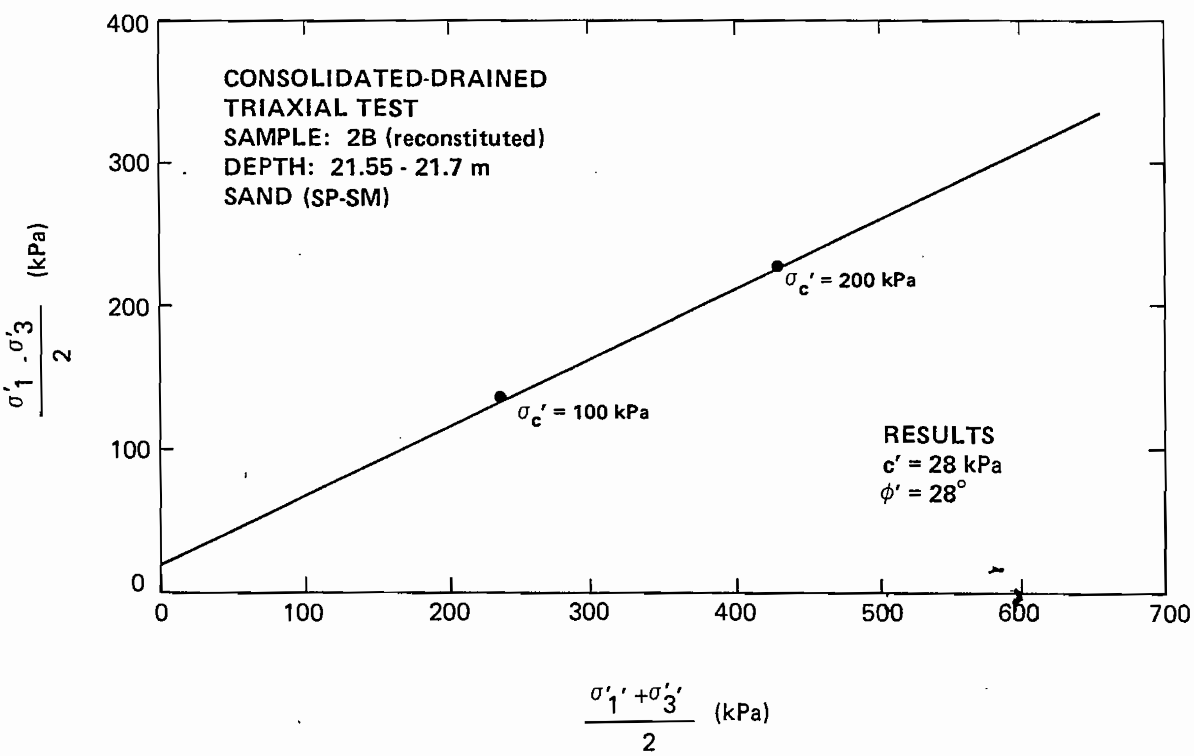
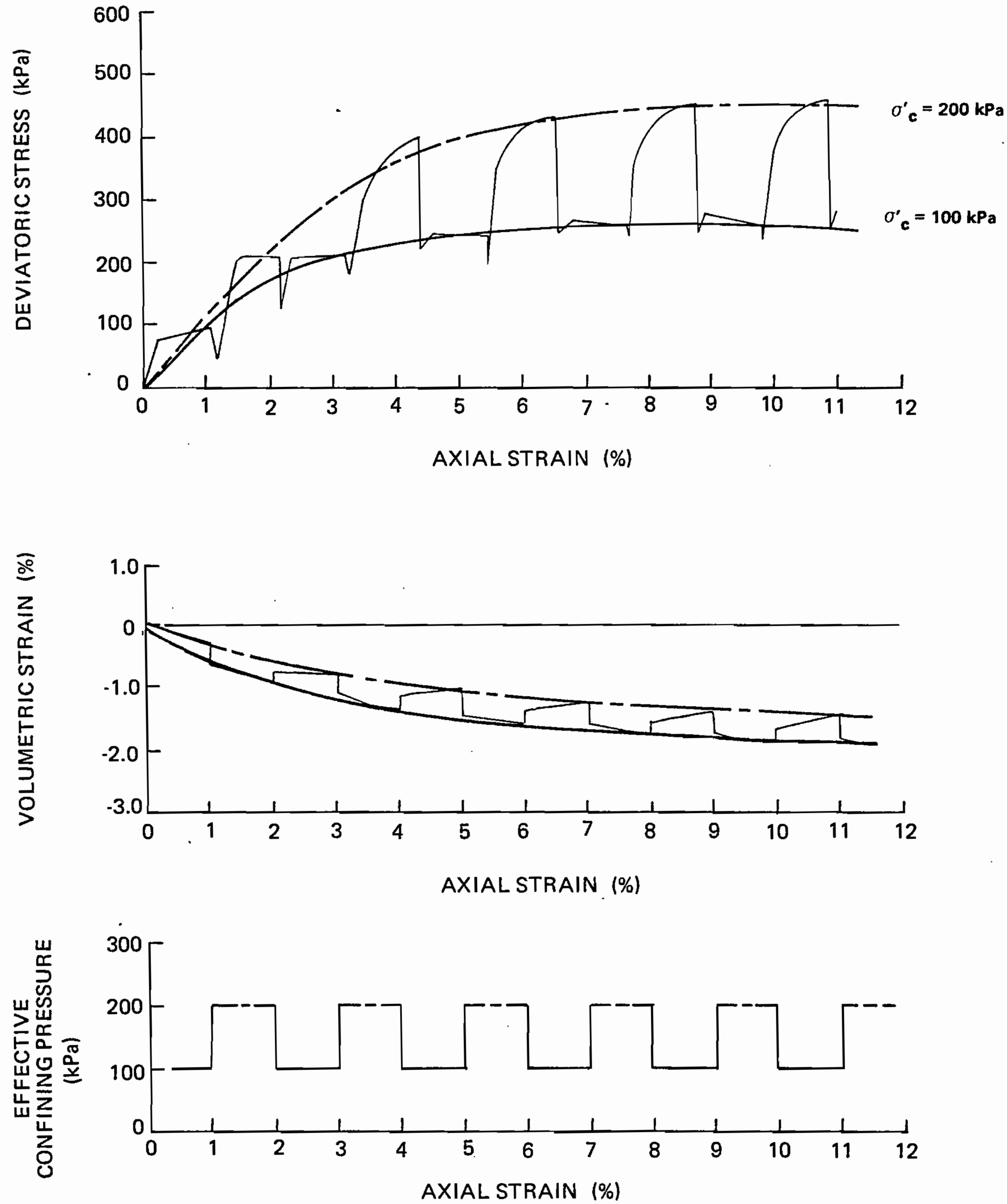


FIGURE D - 4 SHEAR STRENGTH ENVELOPE FOR TRIAXIAL TEST RESULTS, KOPANOAR CPT BOREHOLE

TRIAXIAL TEST RESULTS
(CURVE HOPPING TECHNIQUE)



KOPANOAR CPT BOREHOLE
SAMPLE: 2B (reconstituted)
DEPTH: 21.55 - 21.7 m
SAND (SP-SM)
CONSOLIDATED-DRAINED
TRIAXIAL TEST

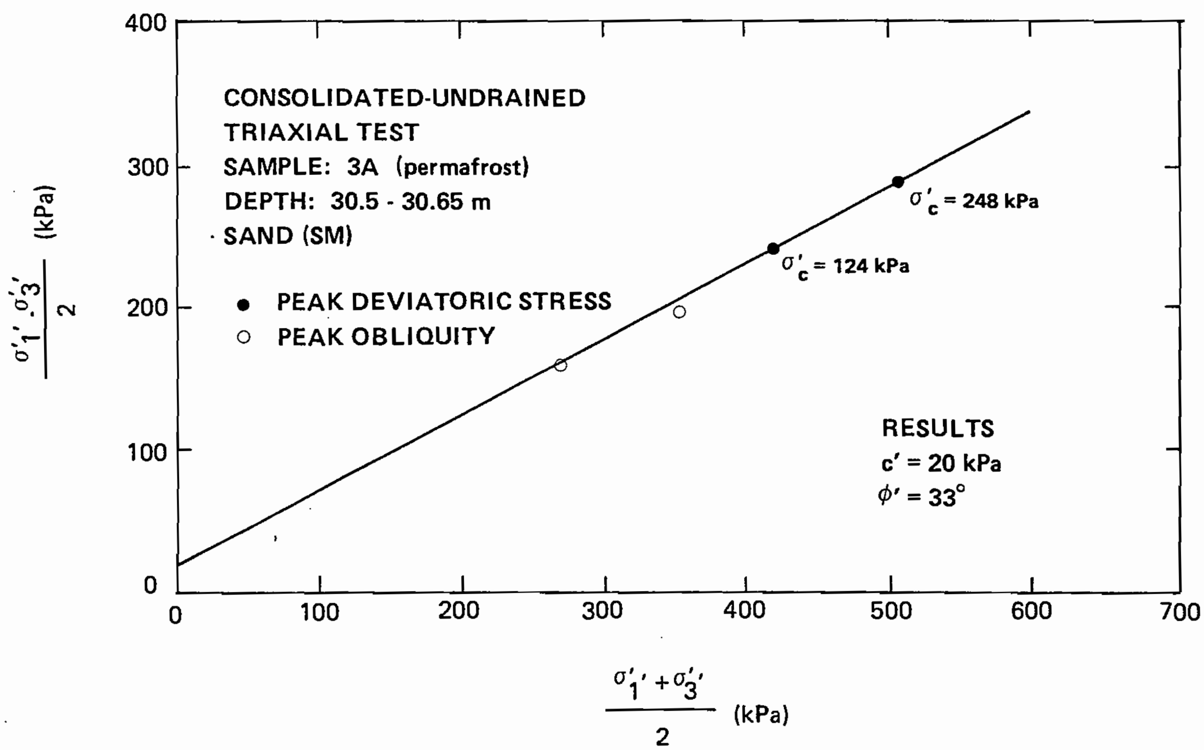
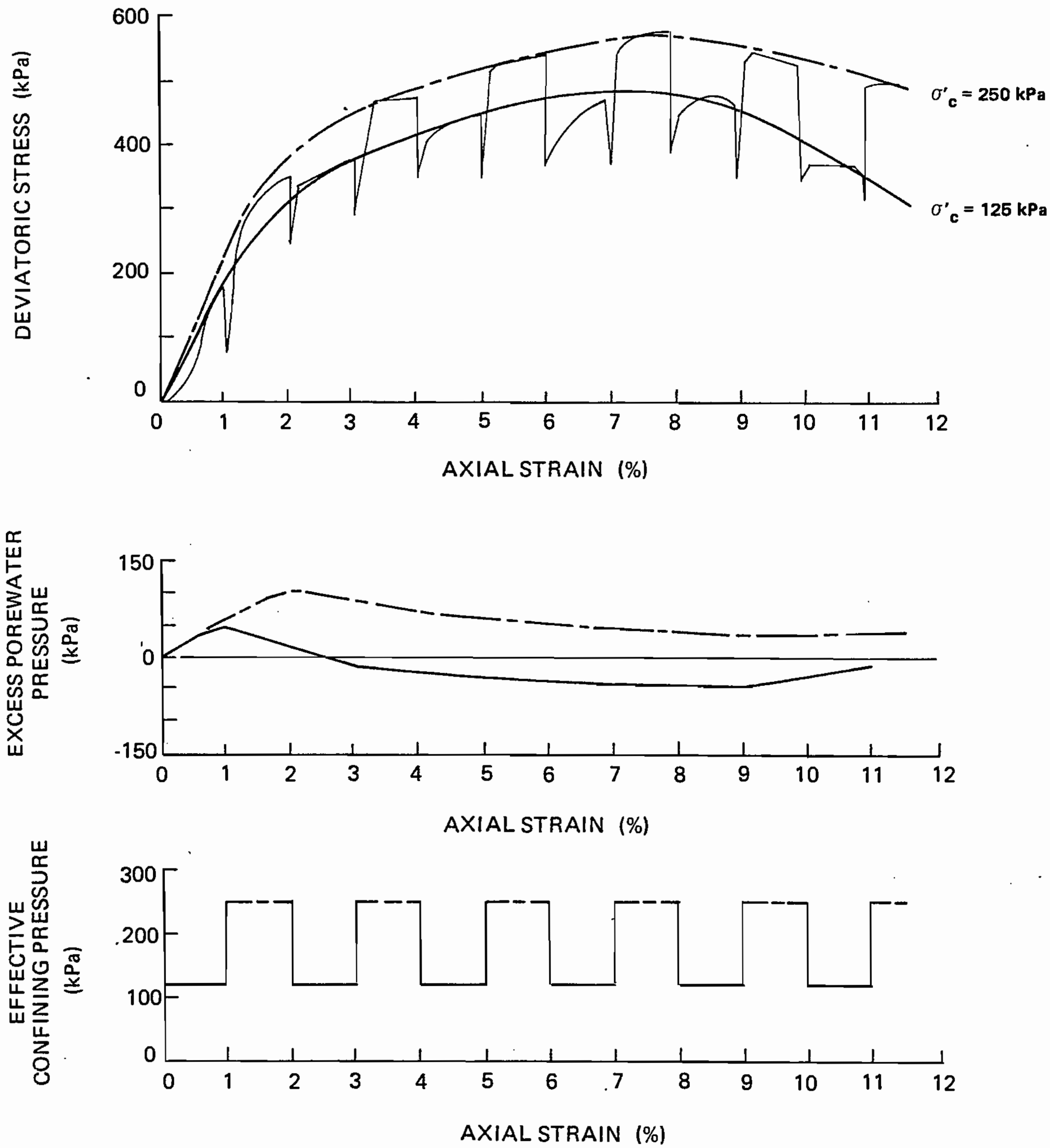


FIGURE D-5 SHEAR STRENGTH ENVELOPE FOR TRIAXIAL TEST RESULTS, KOPANOAR CPT BOREHOLE

TRIAXIAL TEST RESULTS
(CURVE HOPPING TECHNIQUE)



KOPANOAR CPT BOREHOLE
SAMPLE: 3A (permafrost)
DEPTH: 30.5 - 30.65 mm
SAND (SM)
CONSOLIDATED-UNDRAINED
TRIAXIAL TEST

TABLE G-1 SUMMARY OF PHOTOGRAPHS FROM CORES AND RADIOGRAPHS, KOPANOAR I-44 (Cont'd)

SAMPLE NO.	SAMPLE TYPE	DEPTH (m)	USC	CORE PHOTOS (No. of Prints)	RADIOGRAPHS (No. of Prints)	SHEAR TESTING
CANMAR		3.10 - 3.15	MH	1		
1A (CPT)	L	3.15 - 3.30	MH	2		
1B (CPT)	L	3.30 - 3.45	CH	2		
1C (CPT)	L	3.45 - 3.60	CH	2		
2B (CPT)	G	21.5 - 21.7	CH	1		
3A (CPT)	PF	30.50 - 30.69	SM		2	CU

Note:

1. UU - Unconsolidated-Undrained Triaxial Test
   CU - Consolidated-Undrained Triaxial Test
2. All photos have been included in Volume 7D

APPENDIX B

METHOD FOR PREDICTING PILE CAPACITIES (FUGRO)

CONTENTS

METHOD FOR PREDICTING PILE CAPACITIES

Predetermination of the ultimate axial capacity of piles is defined using the static method of analysis. In this method, the ultimate compressive capacity, Q, for a given penetration is taken as the sum of the skin frictional capacity, Q_s, and the end bearing capacity, Q_p, so that

$$Q = Q_s + Q_p = fA_s + qA_p$$

where A_s and A_p represent, respectively, the embedded pile surface area and the pile tip area; f and q represent, respectively, the unit skin friction and the unit end bearing. When computing ultimate tensile capacity, the second term of this equation is neglected.

Cohesive Soils

In cohesive soils, the frictional capacity, Q_s, of a pile at a particular penetration is related to both effective vertical stress and undrained shear strength, c, by a factor λ as follows:

$$Q_s = \lambda(\bar{\sigma}_m + 2c_m)\ A_s$$

where:

λ = dimensionless coefficient (function of pile penetration)

$\bar{\sigma}_m$ = mean effective vertical stress between the ground surface and the pile tip

c_m = mean undrained cohesive shear strength along the pile length

A_s = surface area of the pile.

The values of c_m and $\bar{\sigma}_m$ for various penetrations in the clay strata are computed from the undrained strength and the submerged unit weight values; values of λ are obtained from Fig. 1 of the paper presenting this procedure.[1]

Unit end bearing in clay is estimated using the expression

$$q = cN_c$$

where: c = undrained cohesive shear strength

N_c = a dimensionless bearing capacity factor (N_c = 9 for deep footings).

Granular Soils

The frictional capacity developed in granular soils is determined using the following equation:

$$f = K\bar{\sigma}_v \tan \delta$$

where: K = coefficient of lateral earth pressure

$\bar{\sigma}_v$ = effective vertical stress

δ = angle of friction between foundation soil and steel pile.

The value of K is taken as 0.7 for compressive loads and 0.5 for tensile loads. Effective vertical stress is computed from the submerged unit weight values.

(1) Vijayvergiya, V.N. and Focht, J.A., Jr., "A New Way to Predict the Capacity of Piles in Clays," Proceedings, Fourth Annual Offshore Technology Conference, 1972, Vol. 2, pp. 865-874.

Unit end bearing, q, for piles installed in granular soils is computed using the following equation

$$q = \bar{\sigma}_v N_q$$

where: $\bar{\sigma}_v$ = effective vertical stress

N_q = a dimensionless bearing capacity factor which is a function of $\bar{\phi}$, the angle of internal friction of the soil.

The computed values of f and q are not allowed to exceed certain values[2] given in the table below:

Soil Type	$\bar{\phi}$	δ	f_{max} ksf	N_q	q_{max} ksf
Clean Sand	35°	30°	2.0	40	200
Silty Sand	30°	25°	1.7	20	100
Sandy Silt	25°	20°	1.4	12	60
Silt	20°	15°	1.0	8	40

(2) "Planning, Designing and Constructing Fixed Offshore Platforms," A Recommended Practice by American Petroleum Institute, API RP 2A, October, 1969.

APPENDIX C

IN SITU TEST INTERPRETATION AND RESULTS (FUGRO)

CONTENTS

ILLUSTRATIONS

IN SITU TEST INTERPRETATION AND RESULTS

WISON Cone Penetrometer Test Interpretation

WISON cone penetrometer test (CPT) results are presented on Plates C-1 and C-2 as curves of cone resistance, q_c, and sleeve friction, f_s, versus depth below seafloor. Interpretation of this data provides estimates of soil type and cohesive soil shear strength or granular soil density. Shear strength and density data from CPT results are combined with field and laboratory soil test results to obtain the best estimate of soil parameters needed to compute unit skin friction, f, and unit end bearing, q, for pile design analyses.

The ratio of sleeve friction, f_s, to cone resistance, q_c, is termed the friction ratio, F_R, and has generally been correlated with unfrozen soil types [(1)], as shown in Plate C-3. Friction ratio can be used to identify unfrozen soil type at depths where no samples have been taken.

In cohesive soils, in situ undrained shear strength can be estimated from the equation:

$$s_u = \frac{q_c - \bar{\sigma}}{N_k}.$$

where s_u = undrained cohesive shear strength,

q_c = cone resistance,

$\bar{\sigma}$ = effective vertical stress, and

N_k = cone factor.

Values of N_k are normally based on undrained shear strengths determined

---

(1) te Kamp, W.G.B., Fugro In House Seminar, February 13, 1975.

from triaxial compression test results on soils recovered from the borehole. CPT soundings were not made in Stratum I cohesive soils.

We estimated in situ relative density of unfrozen granular soils using the following:

Density	Cone Resistance, q_c, kgf/cm2	Relative Density, Percent
Loose	less than 40	0 to 40
Medium Dense	40 to 120	40 to 70
Dense	120 to 200	70 to 90
Very Dense	greater than 200	90 to 100

Schmertman [(2)] presented a correlation of q_c, relative density, and vertical effective stress for specific granular soil conditions. Schmertman's correlation is illustrated on Plate C-4 for comparison purposes only.

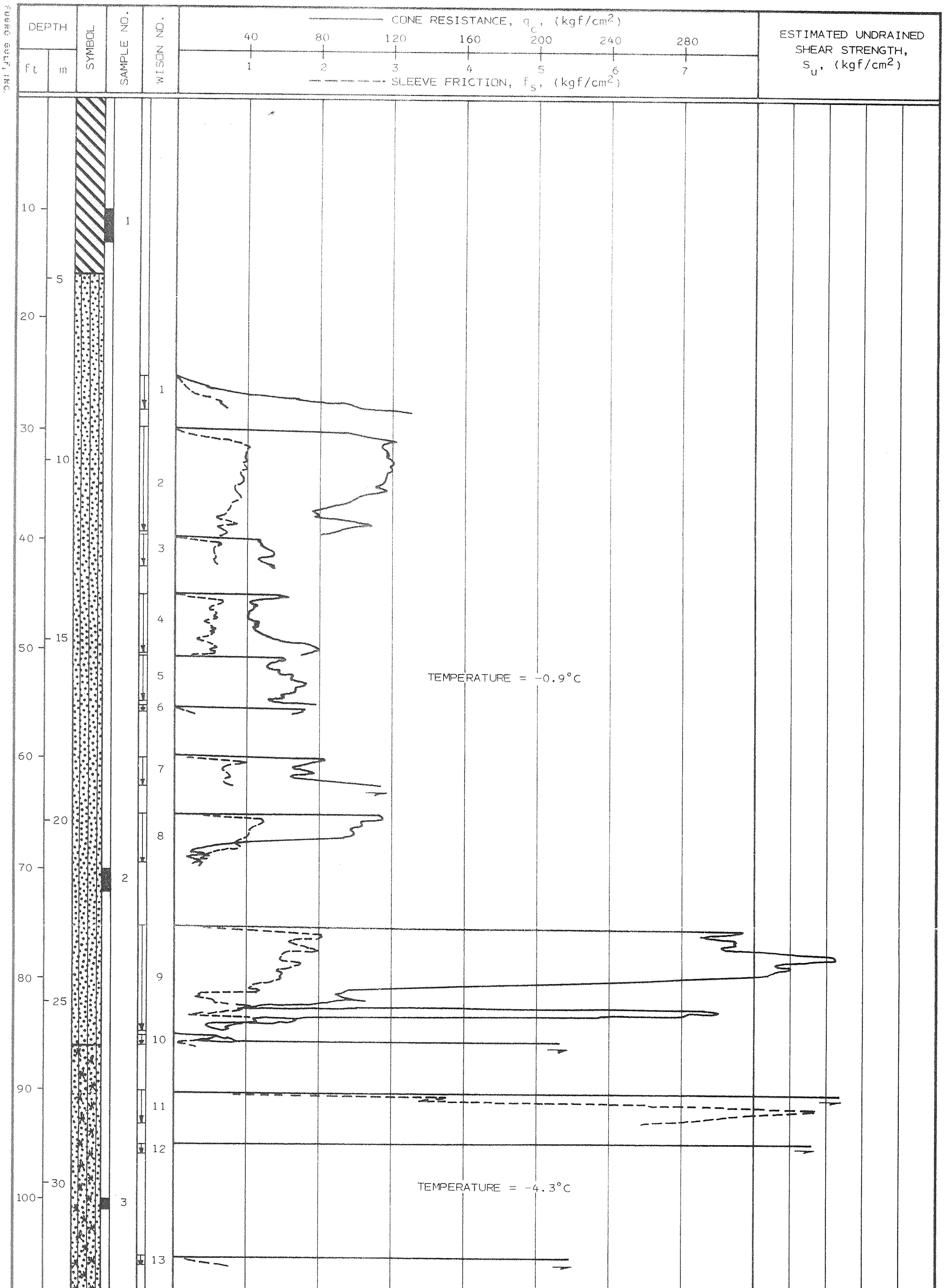
WISON Temperature Probe

In situ temperature measurements made in the borehole are presented on Plates C-1 and C-2 and on the boring log. We measured temperatures using our specially designed temperature sensitive cone. This cone replaces the normal cone and measures cone tip resistance, q_c, and in situ temperature along the sleeve. Sleeve friction is not measured while using the temperature cone. Measured temperatures normally stabilized about 5 to 15 minutes after completion of the cone sounding.

We checked the temperature sensitive cone against a mercury thermometer before and after each test conducted in the field. In addition, the mercury thermometer was checked by a calibration company at the completion of the field work; calibration results are presented on Plate C-5. Temperature measurements presented in this report have been corrected.

(2) Schmertmann, John H., (1977) "Guidelines for Cone Penetration Test Performance and Design", U. S. Department of Transportation, FHWA-75-78-209.

FUGRO GULF, INC.
Consulting Engineers and Geologists

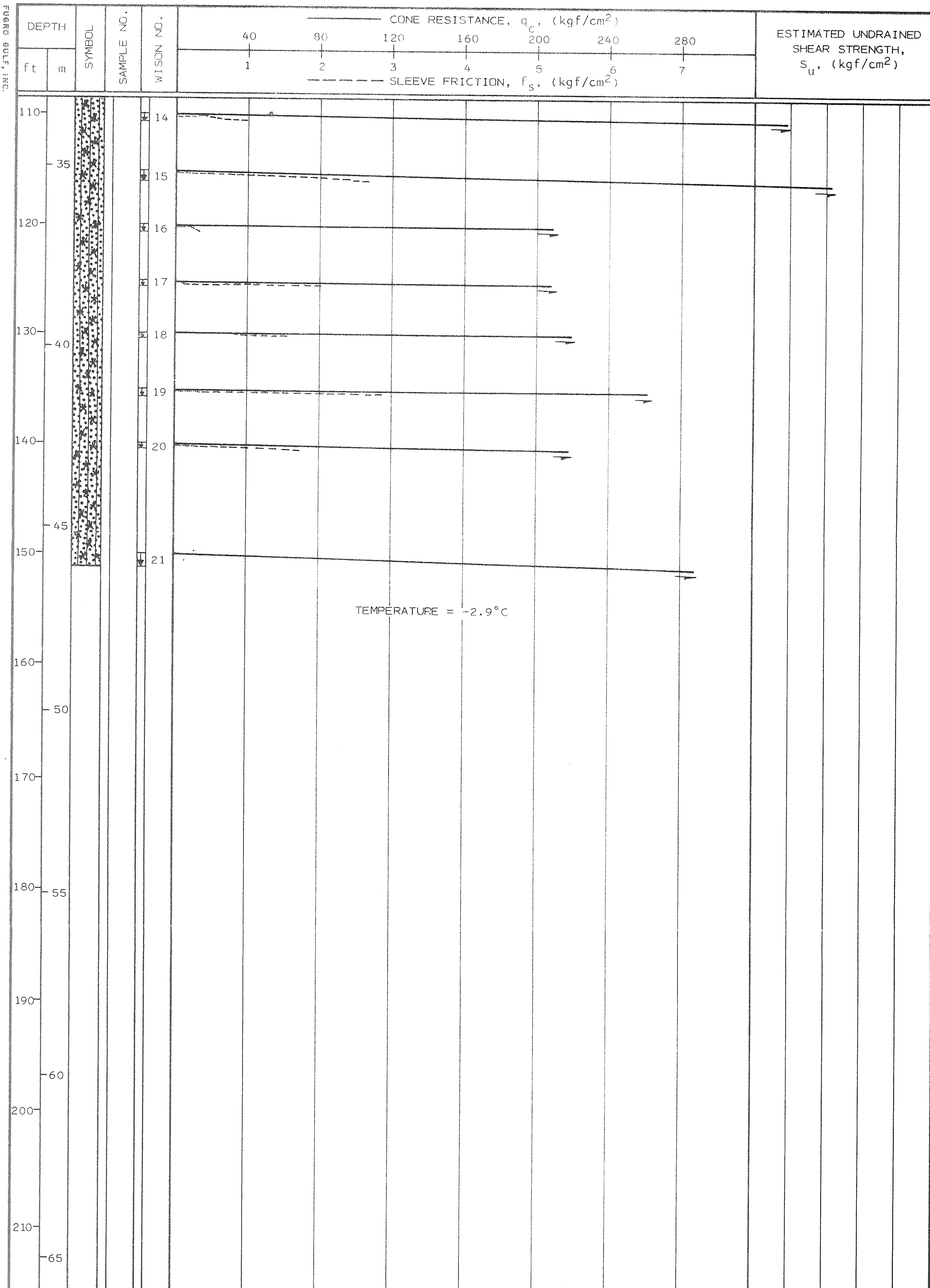


TEST(S) 1-13 | DEPTH 0-109 FT (0-33M) | BORING 1 | LOCATION KOPANOAR M-13

WISON CONE PENETROMETER TESTS
BEAUFORT SEA, OFFSHORE CANADA

PLATE C-1

FUGRO GULF, INC.
Consulting Engineers and Geologists

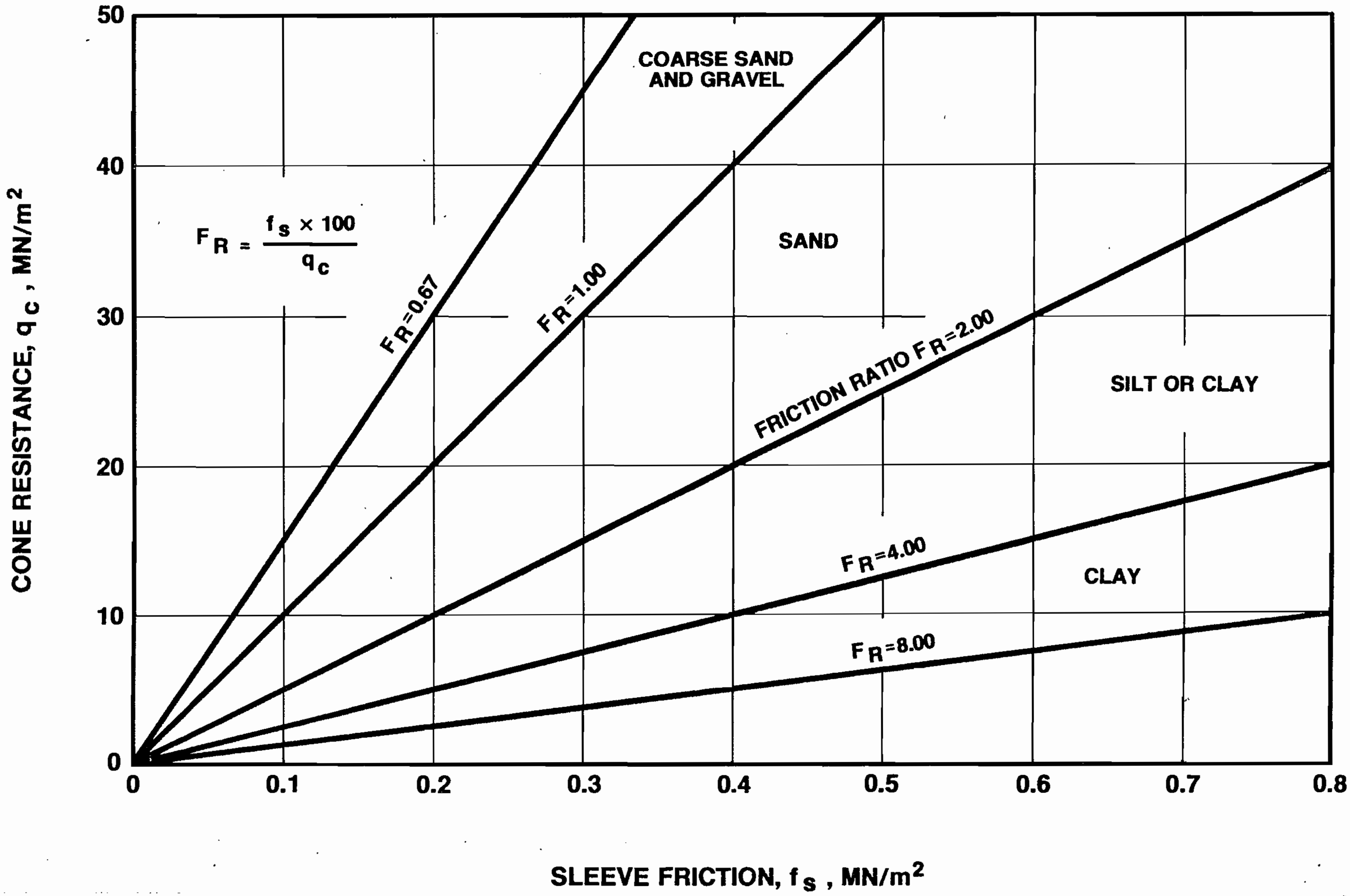


TEST(S) 14-21 DEPTH 109-151 FT. (33-46M) BORING 1 LOCATION KOPANOAR M-13

WISON CONE PENETROMETER TESTS
BEAUFORT SEA, OFFSHORE CANADA

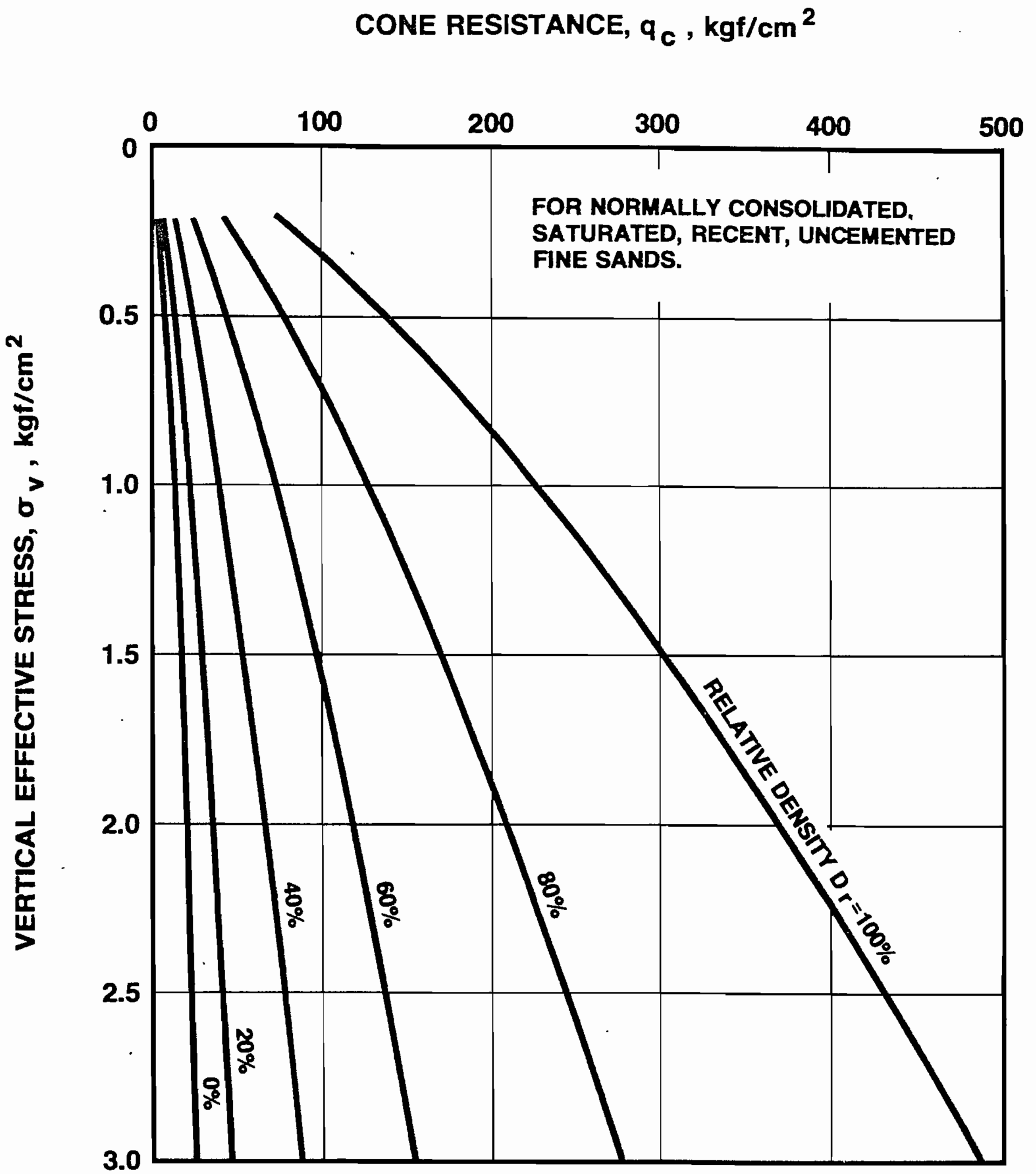
PLATE C-2

FUGRO GULF, INC.
Consulting Engineers and Geologists



UNFROZEN SOIL CLASSIFICATION BY FRICTION RATIO
FOR WISON CONE PENETROMETER
AFTER SANGLERAT AND FUGRO

Job No. 80-058



CORRELATION OF CONE RESISTANCE
AND RELATIVE DENSITY
AFTER SCHMERTMANN (1977)

Job No. ______

MEASURED TEMPERATURE Degrees Celsius	TEMPERATURE CORRECTION Degrees Celsius
+4.0	0
0.0	+0.1
-4.0	+0.1

Notes: 1. Temperature correction uncertainty is less than 0.2° Celcius.

2. Temperatures provided in this report have been corrected.

3. Temperatures certified by Dienst van Het Ijkwezen.

CALIBRATION OF MERCURY THERMOMETER

1980 CANMAR PROJECT

BEAUFORT SEA, OFFSHORE CANADA

A P P E N D I X D

LABORATORY TEST PROCEDURES (EBA)

LABORATORY TEST PROCEDURES

Classification and Index Tests

These tests are quite routine and the standard ASTM procedures employed are listed below:

TEST	ASTM DESIGNATION
Moisture Content	D 2216
Liquid Limit*	D 423
Plastic Limit and Plasticity Index	D 424
Grain Size	D 421 & 422
Specific Gravity	D 854
Relative Density	D 2049
Unified Soil Classification	D 2487
Bulk Unit Weight**	N/A

In addition to the specific procedures described in the following sections, samples programmed for strength and consolidation tests had several other basic tests performed. These were:

1. Moisture content
2. Bulk density
3. Core photography and radiograph (where practical)
4. Detailed description of sedimentological features, and
5. Identification and preservation of any discrete organic material encountered.

NOTES: *All liquid limit determinations were performed with a 3 point procedure.

**Bulk unit weights were determined directly from external dimensions and sample weight in air.

LABORATORY TEST PROCEDURES

Triaxial Shear Tests

Unconsolidated-Undrained Triaxial Tests

Procedure 1* - Standard unconfined compression test procedure. Stress-strain curve produced.

Procedure 2* - Sample mounted in triaxial cell and jacketed. Cell pressure equivalent to estimated <u>total</u> horizontal stress applied without sample drainage. Sample sheared by increasing axial stress at controlled rate of strain. Frozen samples permitted to thaw (undrained) before commencing shear. Stress-strain curve produced.

Consolidated-Undrained Triaxial Tests

Procedure 1 - Sample mounted in triaxial cell and jacketed. Cell pressure equivalent to estimated <u>total</u> horizontal stress applied with drainage permitted. With consolidation complete, drainage is shut off. Frozen samples were placed in a pre-chilled triaxial cell and permitted to thaw before commencing consolidation. A pore pressure response test was carried out prior to shearing. If a B value of less than 0.95 was obtained, back pressure was applied to obtain saturation. Samples were sheared by increasing axial stress at a controlled rate of strain. Stress-strain curves and other diagnostic plots produced. Generally performed minimum of 2 tests on each material type to define strength parameters in terms of effective stress.

LABORATORY TEST PROCEDURES

Triaxial Shear Tests

Consolidated-Drained Triaxial Tests

Procedure 1 - Frozen samples required that the triaxial apparatus be pre-chilled. Sample mounred in triaxial cell and jacketed, thawed under nominal pressure. Consolidated to cell pressure equivalent to estimate horizontal in situ effective stress. With drainage open, sample was sheared by increasing the axial stress at a controlled rate of strain. Stress-strain curve and other diagnostic plots produced. Generally performed minimum of 2 tests on each material type to define strength parameters in terms of effective stress.

Procedure 2 - Lack of undisturbed samples of sand from certain strata required reconstituting disturbed samples for strength testing. Relative density test conducted on the sand and reconstituted samples are prepared to approximately 70% relative density. Sample was consolidated to cell pressure equivalent to the estimated in situ horizontal effective stress. With the drainage permitted, the sample was sheared by increasing the axial stress at a controlled rate of strain. Stress-strain curve and other diagnostic plots produced. Generally performed minimum of 2 tests on each material type to define strength parameters in terms of effective stress.

NOTES:

1. Unconsolidated-undrained triaxial procedure according to ASTM D2850.

2. Consolidated-undrained and-drained triaxial procedures follow those recommended Bishop & Henkel (1969).

3. Samples reconstituted following procedures outlined in Bjerrum, Kringstad, and Kummeneje (1961).

4. Tests denoted by asterix were occasionally performed with back pressure and porewater pressure measurement during shear.

LABORATORY TEST PROCEDURES

Direct Shear Tests

Procedure 1 - Standard direct shear procedure. Frozen samples permitted to thaw and stage-consolidate under applied normal pressure before commencing shear. Resheared strength measured on plane cut after peak strength had been determined. Stress-strain curve and other diagnostic plots produced. Generally performed minimum of 3 tests on each material type to define strength parameters in terms of effective stress.

Procedure 2 - If available sample consisted of disturbed material, test specimen was reconstituted and sheared following the same general procedure indicated above.

NOTES:

1. Standard direct shear procedure according to ASTM D 3080.
2. Samples reconstituted following procedures outlined in Bjerrum, Kringstad, and Kummeneje (1961).

LABORATORY TEST PROCEDURES

Consolidation Tests

Procedure 1 - Sample set up in 64 mm (2.5 in) oedometer with dry stones. Load increments increased by 50% were applied to obtain a specified vertical effective stress exceeding the estimated in situ effective overburden pressure. The oedometer was then flooded, unloaded and permitted to rebound. After rebound, the specimen was reloaded in 50% increments until a vertical effective stress of approximately 400 kPa was obtained. Thereafter, the standard doubling of pressures was resumed to test completion. An e-log p' curve, c_v, k, m_v, and estimate of preconsolidation pressure produced.

Procedure 2 - Sample set up frozen in oedometer, then moved from cold room to standard apparatus. Moderate stress was applied to seat load cap, and sample was then thawed under nominal pressure. Procedure then continued as outlined above. An e-log p' curve, c_v, k, m_v, and estimate of preconsolidation pressure produced. Thaw strain data also obtained.

NOTE: Modifications made to standard procedure (ASTM D 2435) taken from Andresen et al. (1979) and Broms (1980) as recommended for overconsolidated soils.

LABORATORY TEST PROCEDURES

Pore Water Salinity Tests

Procedure 1 - Samples trimmed to remove disturbed material. Porewater extruded from thawed sample. Titration was then performed to establish equivalent salinity (NaCl).

Procedure 2 - Salinities originating from Carbon Systems, Inc. were determined using a modified Mohr titration and verified by conductivity.

NOTES: 1. A silver nitrate titration was performed to determine the chloride ion content (chlorinity) following ASTM D 512, Method B.

2. Chloride ion content was converted to an equivalent salinity using the empirical relationship that follows.

Salinity (o/oo) = 0.03 + (1.805 x Chlorinity (o/oo))

REFERENCES

Andresen, A.T. Berre, A. Kleven, and T. Lunne. 1979. Procedures Used to Obtain Soil Parameters for Foundation Engineering in the North Sea. Marine Geotechnology, Vol. 3, No. 3, pp. 201-265.

Bishop, A.W. and J.D. Henkel. 1969. The Measurement of Soil Properties in the Triaxial Test. 2nd Edition, London, Edward Arnold, 228 pp.

Bjerrum, L., S. Kringstad, and O. Kummeneje. 1961, The Shear Strength of a Fine Sand. Proceeding 5th International Conference of Soil Mechanics and Foundation Engineering, Paris, 1, pp. 29-37.

Broms, Bengt B. 1980. Soil Sampling in Europe: State-of-the-Art. ASCE Journal of the Geotechnical Engineering Division, Vol. 106, No. GT1, pp. 65-97.