Ice regimes off the west coast of Newfoundland

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Ice Regimes Off the West Coast of Newfoundland

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Report for

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Ottawa, Ont.
Canada

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1 SUMMARY

Sandwell Engineering Inc. and Canatec Consultants Ltd., both of Calgary, Alberta, were contracted to perform a study of Ice Regimes off the West Coast of Newfoundland. The study had two major aspects, the first was to access and document available ice and environmental data for the geographical area of investigation. The second aspect was to use this ice and environmental data to provide recommendations, where appropriate, on offshore oil/gas drilling structure concepts and structures. The study was to concentrate on data applicable to the offshore oil lease areas on the West Coast of Newfoundland.

As part of the project, Canatec accessed and reviewed available environmental data for the West Coast of Newfoundland. Sandwell reviewed offshore oil production structure concepts from ice covered regions of the world. The wave and ice regimes data were used by Sandwell to estimate environmental forces on representative structure geometries. The types of geometries considered were a vertically sided gravity-based structure 100 m by 100 m and a cylindrical structure from 5 m to 20 m diameter. These two candidate geometries were selected because it is impractical to perform load estimate on the many structural concepts that have been proposed for drilling structures in ice covered waters. The candidate geometries were used to investigate, in general terms, which aspects of the environmental forces are dominant as the water depth is varied. The ice force was calculated from either the 1 in 100 year iceberg impact or from a 1 in 100-year first-year pressure ridge. The wave force was calculated using the 1 in 100 year wave for the region. In water depths of less than approximately 30 m, the wave force is governed by the wave breaking action. As the water depth increases, the ice force remains essentially constant whereas the wave force increases. This relationship along with the calculations on the various cylindrical structures indicates that structures, which reduce the wave forces, could be considered. Examples of such geometry are multi-legged structures, monocones or monopods.

Floating production systems may have difficulty in resisting the ridge ice force or the iceberg force and thus bottom-founded structures may be more appropriate for this region. A summer only production system could be investigated given the relatively long open water season.

The ice and wave environment off the West Coast of Newfoundland was compared to other regions of the world. The Canadian and American Beaufort Seas have a more severe ice environment but less severe wave environment. The Grand Banks region have a more severe wave environment but the West Coat of Newfoundland has first-year pressure ridges as the significant ice event rather than iceberg impact. The wave, ice and water depths are similar to that in the Sakhalin region of the Sea of Okhotsk. This suggests that designs and concepts for that region may be applied to the West Coast of Newfoundland.

2 INTRODUCTION

Sandwell Engineering Inc. and Canatec Consultants Ltd., both of Calgary, Alberta, were contracted to perform a study of Ice Regimes off the West Coast of Newfoundland. The study had two major aspects, the first was to access and document available ice and environmental data for the geographical area of investigation. The second aspect was to use this ice and environmental data to provide recommendations, where appropriate, on offshore oil/gas drilling structure concepts and structures. The study was to concentrate on data applicable to the offshore oil lease areas on the West Coast of Newfoundland.

The ice and environmental conditions off the West Coast of Newfoundland are presented in the companion report by Canatec Consultants Ltd. The Canatec report will function as a stand-alone document for available ice, wave, current, wind, temperature, geotechnical and seismic data. As outlined in the Canatec Report, there are deficiencies in the completeness of the publicly available data sources. The data that are available will be used to make an initial selection of the general type of oil or gas production structures that could be investigated for deployment in the West Coast of Newfoundland offshore region.

The oil and gas lease areas off the West Coast of Newfoundland cover a considerable geographical area with ice conditions being generally more severe in the northern part of the region. The ice is mainly first year and is fairly mobile. First-year pressure ridges along with areas of deformed ice are common.

The wave climate is typical for its location in the North Atlantic Ocean. There does not appear to be enough data on the wave climate to distinguish separate region in the north or the south.

The area is one of low seismic risk and seismic forces on the structures are not likely to be a major limitation on structural designs. While the wave climate is assumed to be largely independent of water depth, the wave forces on bottom founded structures generally increase with increasing water depth. The ice forces on the other hand will be largely independent of the water depth. The major exception being the grounding of icebergs in the shallow water thereby reducing their flux in the shallow water. Iceberg interactions with the structure are rare however. Thus as the installation depth of structures go from shallow water to deeper water the dominant environmental loading could change from ice forces to wave forces. This aspect could then change the overall geometry of the production structures to ones that have lower wave forces. In addition, some structural concepts are applicable to shallow water and some are applicable to deep water. The geotechnical information on the strength of the seabed sediments and materials are sparse. The available information does indicate that the surface layers may have low shear strength. While strength information would be required in the detailed or even conceptual design of structures, in this study we will have to assume that suitable seabed strengths are attainable. The type of structures will be classified by water depth rather than by the geographical region within the lease area.

The geometry of the various concepts covered a wide range of types. However, from an inspection of them, the critical aspects of the environmental forces could be obtained by considering idealized shapes. The shapes selected were a square vertically sided structure 100m wide and a vertical cylinder from 5.0 to 20 meters in diameter. Each of the shapes was constant from the seabed to above the waterline. The available environmental data have been used to estimate the ice and wave forces on these idealized structure shapes as a function of the water depth between 20 and 80 meters. The relative wave and ice forces suggest that the wave climate can provide greater constraints to the structure than the ice loading. A review of drilling and production structure concepts is presented along with general recommendations on the water depth at which these concepts may be applicable. The wave and ice loading information is furthermore used to provide direction on the generalized shape for these offshore structures.

The major outcome of the review of concepts is to suggest what structures are, in principal, candidates. Within the limited scope of the project and given that no information about the oil or gas field characteristics are available, it is not appropriate to recommend particular structures or concepts. What is appropriate is to provide general recommendations on which structures could be included in a conceptual design exercise.

3 STRUCTURAL CONCEPTS

There are many design aspects that enter into the selection of a suitable production structure. These include the water depth of the structure, the wave force, the ice force, the seismic force, the area required for processing and drilling facilities, the product transfer system and the strength characteristics of the seabed. In addition to these structure properties there are properties of the reservoir in terms of expected production rates, production methodologies, lifetime of the field, properties of the oil and the number of wells required. Furthermore there are many financial considerations relating to the costing of structure including outfitting and the cash flow for the whole system

Most of the structural dimensional parameters result from the particulars of the producing field. These are not known, even in general, as exploratory drilling has not been conducted for most of the lease areas. For a conceptual design investigation the general field particulars would be known or would have been estimated by the company requesting the study. Thus the current study could be considered as pre-conceptual and even more non-specific that a conceptual design. In order to proceed with the structural review part of the project, the following was used. A review of structure concepts and structures constructed was performed. The output was then used to select candidate structures for which wave and ice forces could be estimated. Note that given the complete lack of data on the producing field it is not appropriate to make recommendations on particular structures. What is appropriate is to make general recommendations on the types of structures and the generalized characteristics.

A review of structural concepts proposed and/or constructed for ice covered waters was conducted in 1983 (Buslov and Krahl, 1983). The various concepts were classified by principal characteristics. The main classification was for the overall geometry of the structure. The classes were Gravity Permanent, Gravity Movable, Piled Permanent, Mixed Permanent, Moored Permanent, Moored Movable, Dynamic Position Movable, Retained Permanent, Retained Movable, and Non-retained Permanent. Information was also given on the concept originator's recommendations for the water depth for which the concept would be applicable. Also, information on the geographical location and the type of ice loading was given. There was not a consistent method of defining the ice loading; sometimes it was defined in terms of ice thickness and sometimes in terms of the allowable ice pressure. Also there was not a consistent way of defining if the structure were suitable as an exploration structure or a drilling and exploration structure. In practice this distinction is not clear as there are structures, for example the Molikpaq that was designed and used as an exploration structure in the Canadian Beaufort and was then outfitted for use as a production facility in the Sakhalin region of Russia. The main output of this review is contained within Table 3.1.

For further information about these concepts refer to Appendix A.

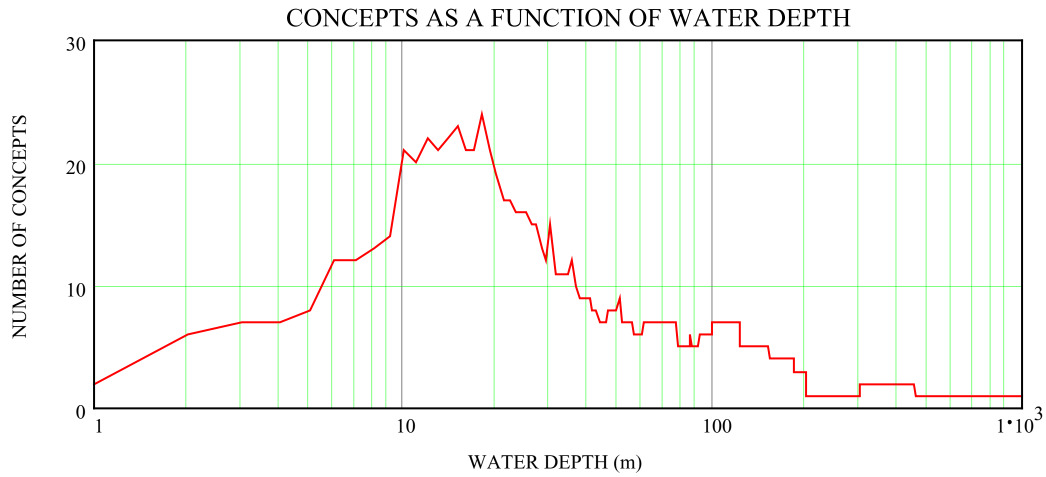
Table 3-1: Structural Concepts as of 1983

	Description	Location	Water Depth Min (m)	Water Depth Max (m)	Ice Conditions
Gravity Permanent					
BGP 1	Gravity Structure on a submerged mound				
BGP 2	North Sea type three legged tower				
BGP 3	Monotower				
BGP 4	Monotower with ice-breaking cone				
BGP 5	Concrete conical production & storage structure				
BGP 6	Controlled stiffness steel arctic platform	Alaskan Beaufort	14	14	Sheet 2.1 m M.Y. Ridge 15 m
BGP 7	NAC friction ring fender platform	Canadian East Coast	85	85	Icebergs 10-50 mm tonne @1.5 cwt.
BGP 8	Concrete production island	Norton Sound	10	30	First-year sheet
BGP 9	Technomare Steel Gravity	Labrador	100	200	
BGP 10	Bottom Mounted Gravity/detachable	Beaufort	30	30	15m floe
BGP 11	Bottom Mounted Gravity/Permanent	Beaufort, Labrador	50	200	M.Y. ridges, icebergs
Gravity Moveable					
BGM 1	Arctic Drill Barge (ADB)	Near shore Beaufort	2	10	Sheet 2.1m, Ridge 5m
BGM 2	Monopod Exploration Drilling Rig	Canadian Beaufort	2	12	Sheet 3.0m, Ridge 15m
BGM 3	Mobile Gravity platform (Monocone)	Southern Beaufort	10	42	8.4 Mpa on 5m^2
BGM 4	Arctic mobile drilling structure (AMDS)	Alaskan Beaufort	6	19	First-year with M.Y. fragments
BGM 5	Mobile Arctic gravity platform	Beaufort	20	50	1.6m sheet, M.Y. ridges
BGM 6	Single steel drilling caisson (SSDC)	Canadian Beaufort	10	25`	1.8m sheet and M.Y. floes
BGM 7	BWA Caisson (BWACS)	Beaufort, Chukchi, Bering	10	20	1.8m sheet, 5.2m consolidated rubble
BGM 8	Arctic Cone Exploration Structure (ACES)	Arctic	15	35	
BGM 9	Mobile Arctic Drilling Structure (MADS)	Beaufort	1.8	6.1	2.5 sheet
BGM 10	Monopod Jackup drilling rig	Beaufort	4.5	27	
BGM 11	Concrete Island Drilling System (CIDS)	Beaufort	5.5	14.5	
BGM 12	Portable Arctic Drilling Structure (PADS)	Beaufort	6	15	
BGM 13	Bottom Mounted Ice-cutting Platform	Arctic	15	36	
BGM 14	Mobile Arctic Island (MAI)	Beaufort	6	23	3.4 MN/m to 9.0 MN/m Load
BGM 15	Sonat Hybrid Arctic Drilling Structure (SHADS)	Alaskan Beaufort	9.1	20	450 MN load
BGM 16	Modular Concrete Platform	Alaskan Beaufort	9.1	24	
Piled Permanent					
BPP 1	Cook Inlet Multi-legged Structure	Cook Inlet	21	30	1.1m sheet
BPP 2	Cook Inlet Monopod Structure	Cook Inlet	19	19	1.8m sheet

Table 3.1 : Structural Concepts as of 1983 (continued)

	Description	Location	Water Depth Min (m)	Water Depth Max (m)	Ice Conditions
Mixed Permanent					
BMP 1	Arctic Production Monocone	Canadian Beaufort	46	76	M.Y. 3m sheet 35m ridge
BMP 2	Ice-resistant integrated deck platform (Hicone)	Beaufort	9	18	4.8m sheet, 15m ridge
BMP 3	Production platform for Sakhalin fields	Sakhalin	30	30	0.9 sheet
BMP 4	Deepwater Actively Frozen Seabed (DAFS)	Beaufort	25	50	25MN/m global
BMM 1	Soviet Ice Platform	Sakhalin	12	20	1.1m sheet
BMM 2	Sohio Arctic mobile structure (SAMS)	Alaskan Beaufort	12	18	
Moored Permanent					
FMP 1	Floating drilling production and storage caisson	Low Arctic	300	1000	M.Y. 3m, ,15m consolidated ridge
Moored Moveable					
FMM 1	Swivel Drillship	Beaufort	18	122	Extended open water season
FMM 2	Round Drillship	Beaufort	18	122	1.8m sheet
FMM 3	Egg-shaped Ice-resistant barge				
FMM 4	Floating rig inside ice-free zone	Beaufort			
FMM 5	Ice-class Semi-submersible	Arctic			
FMM 6	Conical mobile drilling unit (KULLUK)	Beaufort	24	55	1.2m sheet
FMM 7	Ice-resistant Semi-submersible drilling unit	Beaufort, Chukchi, Bering			2.1m rafted, 32 ridge
FMM 8	Arctic drill hull	Beaufort	18	183	1.5m sheet
Dynamic Positioned Moveable					
FDM 1	Ice-cutting Semi-submersible drilling vessel	Labrador, Canadian Arctic	91	450	17m ridge
Retained Permanent					
IRP 1	Sandtube retained Island	Beaufort	3	3	Landfast ice
IRP 2	Arctic production (drilling) sand isle	Beaufort	60	60	2.1 sheet, 15m ridge
IRP 3	Man-made rock island (NORPEX)	Hibernia	30	152	Icebergs
IRP4	Arctic Production Loading Atoll (APLA)	Beaufort	61	76	Ice Islands
IRP 5	Arctic mooring storage caisson	Beaufort	15	28	Grounded Rubble
IRP 6	Gulf/Dome Tarsuit Caisson	Beaufort	6.5	22	4.2 Mpa on 4.6 m^2
IRP 7	Arctic single-point mooring	Alaskan Beaufort	35	35	
IRP 8	Cellular Island	Alaskan Beaufort	2	27	7 Mpa local pressure
Retained Moveable					
IRM 1	Necklace	Beaufort	7.5	18	2.1m sheet
IRM 2	Caisson Berm-protected Island	Beaufort	3.3	3.3	
IRM 3	Caisson-retained Island	Beaufort	10	18	2.1m sheet
IRM 4	Mobile Arctic Caisson (Molikpaq)	Beaufort	21	40	7.5m floe, 21m ridge
IRM 5	Stacked steel caisson system	Arctic	4	20	
Non-Retained Permanent					
INP 1	Gravel Islands	Beaufort	1	15	Landfast to 2.1m
INP 2	Sacrificial Islands	Canadian Beaufort	1	19	Landfast & transition zone

Figure 3-1: Number of Concepts as a function of water depth



The data contained in Table 3.1 was processed to create a histogram of the number of concepts as a function of the water depth. From figure 3.1 it can be seen that between 10 m and 20 m water depth there are between 20 and 24 concepts at any particular depth and in water depths greater than 20 m the number of concepts decreases. At a water depth of approximately 100 meters there are approximately 6 to 8 concepts at a particular depth. Between a water depth of 20 and 100 meters there are 34 different concepts in total. The large number of concepts between 10 and 20 meters water depth represents the area of interest prior to 1983. There were a number of exploration wells that had been drilled in the near-shore region of the Canadian and American Beaufort Seas and there was interest in extending the exploratory drilling into deeper water.

A review of structures in the Beaufort Sea was conducted and reported by Masterson et al (1991). A total of 141 offshore wells were drilled in the Canadian and the American Beaufort Sea. The types of drilling structures included gravel islands caissons, drill ships and specifically designed floating drilling vessels (e.g. Kulluk). The overall water depths ranged from approximately 1.0 m to 67 m, a range that includes a significant portion of the water depth in the West Coast of Newfoundland lease areas. For more information refer to Appendix A.

Many of the offshore drilling concepts and structures were designed by Sandwell for a variety of clients. Further information on these designs is contained within Appendix B.

A number of developments have occurred since the review of 1983. For example the SSDC and the Molikpaq have had spacers constructed so that the water depth can be increased without the requirement of constructing a gravel berm. Developments off the east coat of Canada have occurred principally the oil development at Hibernia on the Grand Banks and the gas development at Sable Island off the coast of Nova Scotia. Hibernia is a concrete bottom founded structure and the development of Sable a jacket type structure.

The list of concepts presented represent a large variety of shapes and configurations. For this project it in impractical to calculate or estimate the wave and ice forces on each of them. Based on other design projects such calculations for even one structure requires a level of effort greater than for the whole of this project. What can be done, however is to estimate the wave and the ice forces for idealized structure shapes. By doing these calculations the general features and trends can be demonstrated. The idealized shapes and dimensions selected were based on structures that have been constructed and/or proposed. The idealized shapes were a square bottom founded structure 100 meter on a side and vertical columns with a diameter between 5 and 20 meter. Using the vertical cylindrical columns a multi-legged structure can be represented. This selection captures some of the essential elements of the structures.

The water depth range considered was from 20 m to 80 m. At less than a water depth of 20 m, it has been assumed that directional drilling would be able to reach the field. The upper limit corresponds to the water depth at the perimeters of the lease areas. As will be demonstrated in the following sections, the general trends are not that sensitive to water depth if it is about 100 m.

4 WAVE CLIMATE AND FORCES

4.1 WAVE CLIMATE

The wave climate for the region was monitored in a T.D.C. sponsored study. (See Canatec Report)

Data on wave characteristics are usually given in terms of wave height and wave period. Our analysis of the data from a T.D.C. sponsored wave climate study in the region, indicated that a linear relationship between the significant wave height and wave period was a reasonable representation to the wave data:

$$T = 4.007 + 1.364 H_{sig} \qquad 4.1$$

The significant wave height H_{sig} is the average height of the largest third of the waves. The extreme or largest wave height for engineering purpose can be expressed in terms of the significant wave height as:

$$H_{exe} = 1.87 H_{sig} \qquad 4.2$$

The wave height and wave period data were collected in deep water. There is a maximum wave height that can occur in shallow water due to the breaking of the wave. Thus the wave height data may need to be modified to allow for the relatively shallow water depth range considered. The approximate relationship for the maximum wave height in shallow water is given by (Morris, 1963):

$$H_{\max} = \sqrt{\frac{8D^3}{\lambda}} \quad (m) \qquad 4.3$$

Using a third order Stokian representation, the wavelength is calculated in terms of the wave period, the wave height and the water depth and is given in equation 4.4 (Dorf, 1995). This expression has as limiting cases, the shallow water approximation and the deep-water approximation for wavelength. Note that the expression also takes into account the effect of finite wave height on wavelength. Because equation 4.4 explicitly includes the wavelength on the right hand side it has to be numerically solved.

$$\lambda = \frac{g T^2}{2\pi} \cdot \tanh\left(\frac{2\pi D}{\lambda}\right) \cdot \left[1 + \left(\frac{\pi H}{\lambda}\right) \cdot \left(\frac{5 + 2\cosh\left(\frac{4\pi D}{\lambda}\right) + 2\cosh\left(\frac{2\pi D}{\lambda}\right)^2}{8\sinh\left(\frac{2\pi D}{\lambda}\right)^4} \right) \right] \qquad 4.4$$

where:

$g = acceleration\ due\ to\ gravity\ (m/s^2)$

$T = period\ (s)$

What is useful from a design point of view is the load for waves of various return periods. From the TDC study on the Gulf of St. Lawrence the significant wave height for three different return periods are given in Table 4.1. Also given are the extreme wave height calculated using equation 4.2. Also provided in Table 4.1 are the 90% confidence limits for the significant and the extreme wave height (numbers given in the brackets). Note that these data are derived from only a few years of wave data. Continuous time series data for the region over long periods do not appear to be available. There may thus be additional uncertainty in the 10 year and 100 year return period values.

Table 4-1: Wave Heights and Return Periods

RETURN PERIOD (Year)	Significant Wave Height (m)	Extreme Wave Height (m)
2	6.6 (6.2 ; 7.0)	12.3 (11.6 ; 13.1)
10	8.5 (7.6 ; 9.4)	15.3 (14.2 ; 17.6)
100	10.6 (8.9 ; 12.5)	19.8 (16.6 ; 23.4)

The data in Table 4.1 are appropriate for deep water. The wave breaking relationship in equation 4.3 was used to truncate where appropriate, the wave height data given in Table 4.1. The limiting occurred mainly for the extreme wave height in shallow water of about 20 meter.

4.2 WAVE FORCES

Wave forces also depend upon the detailed geometry of the structure including both the characteristics at the water line and the characteristics below the waterline. In order to provide directions on the type of structure that may be suitable for the area, the simplified geometry is assumed. The two cases considered are a long vertical wall and a relatively narrow cylindrical structure.

In some cases (small members), where the structure does not influence the waves very much, the loads can be defined using Morison type equations and non-linear wave theory. In some cases, Morison's equation must be adopted for breaking waves. For large structure, where the wave-structure interaction is important, (wall – GBS's) the loads must be defined using an appropriate diffraction theory, giving due account to the impulsive loading (inertial terms) due to the non-linear and breaking nature of the waves.

The wave force per unit length of a long wall extending from the seabed to well above the waterline is given by (Morris, 1963). This expression does not take into account the effect of the breaking wave.

$$F_{wall} = \frac{\gamma H}{6}\left[4\pi H \frac{D}{\lambda} + 3 \cdot \left(D + \frac{H}{4}\right)\right] \cdot \sin^2(\alpha) \quad (N/m) \qquad 4.5$$

where:

$$\gamma = specific\ weight\ of\ sea\ water\ \ (N/m^3)$$
$$D = water\,depth\,(m)$$
$$H = wave\ height\,(m)$$
$$\lambda = wave\ length\,(m)$$
$$\alpha = angle\,of\ \ wall\,to\,horizontal\ (\text{deg})$$

A methodology for estimating wave loads on square caissons has been presented (Mogridge and Jamieson, 1976). Similarly, to the expressions given in Morris, 1963, it does not take into account the effect of the breaking wave which may occur in the shallow water depths. It does, however, include the two-dimensional character of the structure rather than just being a wall. In general, the method of Mogridge and Jamieson gives higher loads than does Morris. For the 100-year extreme wave height of 19.8 m, the Mogridge and Jamieson may not provide reliable estimates at a water depth of less than 30 m. Because of the wave breaking and slamming effect, the wave interaction force has not been reduced at water depths of less than 30 m but has been held at the 30 m value. Note that this extrapolation is not the result of detailed engineering calculation but is based on engineering judgement coupled with the results of model scale tests of similar shaped structures.

For cylindrical piles extending from the seabed to well above the waterline there are drag forces and inertia forces which are given by (Morris, 1963). These expressions do not take into account the effect of the breaking wave.

$$F_{drag} = \frac{\gamma C_D\, d\, H^2}{16} \cdot \frac{\sinh\left(\frac{4\pi}{\lambda} \cdot \left(D + \frac{H}{2}\right)\right) + \frac{4\pi}{\lambda} \cdot \left(D + \frac{H}{2}\right)}{\sinh\left(\frac{4\pi D}{\lambda}\right)} \quad (N) \qquad 4.6$$

$$F_{inertia} = \frac{\gamma\, \pi\, C_m\, d^2 H}{8} \cdot \tanh\left(\frac{2\pi D}{\lambda}\right) \quad (N) \qquad 4.7$$

where:

$$C_D = 1.2\ ,1.5\ ,2.4\ \ (for\,circular;square;H;Piles)$$
$$C_m = 2.5\ ,3.1\ ,5.0\ \ (for\,circular;square\!:H;Piles)$$
$$d = pile\,diameter\ (m)$$

The pile force is calculated as the maximum of the drag force or the inertia force plus 40% of the drag force as a result of the drag and inertia forces being approximately 90 degree out of phase with each other.

$$F_{pile} = MAXIMUM\left[\left(F_{drag}\right) or \left(F_{inertia} + 0.4 \cdot F_{drag}\right)\right] \quad 4.8$$

Calculations were performed to estimate the wave force on the 100 meter wide vertical faced structure along with the force on the vertical cylindrical piles of various diameters. The calculations were performed for the three return periods and for water depths between 20 and 80 meter. The water depth range covers the water depth range in the lease areas. For less than a water depth of less than 20 meter, it has been assumed that directional drilling would be able to reach the field and that an offshore production structure may not be required

The various calculations were performed using the Mathcad program language and the results are illustrated in Figures 4.1 to 4.6 and also given numerically in Tables 4.2 to 4.5.

Table 4-2: Unfactored Wave Forces for structural elements in 20 meter water depth

	2 Year Return Period		10 Year Return Period		100 Year Return Period	
Case	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)
Wall						529
5m leg	1.1	1.4	1.3	1.4	1.4	1.4
10m leg	4.2	4.4	4.6	4.1	4.8	4.1
20m leg	16	15	18	13	17	14

Table 4-3: Unfactored Wave Forces for structural elements in 40 meter water depth

	2 Year Return Period		10 Year Return Period		100 Year Return Period	
Case	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)
Wall						715
5m leg	1.4	2.1	1.7	2.5	1.9	2.9
10m leg	5.5	7.6	6.4	8.4	7.1	8.8
20m leg	22	28	25	30	30	29

Table 4-4: Unfactored Wave Forces for structural elements in 60 meter water depth

	2 Year Return Period		10 Year Return Period		100 Year Return Period	
Case	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)
Wall						990
5m leg	1.6	2.5	1.9	2.9	2.2	3.5
10m leg	6.1	8.9	7.2	10.2	8.2	11.3
20m leg	24	34	28	37	32	40

Table 4-5: Unfactored Wave Forces for structural elements in 80 meter water depth

	2 YEAR RETURN PERIOD		**10 YEAR RETURN PERIOD**		**100 YEAR RETURN PERIOD**	
Case	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)	Sig. Wave Force (MN)	Ext. Wave Force (MN)
Wall						1225
5m leg	1.6	2.7	2.0	3.2	2.4	3.8
10m leg	6.4	9.9	7.8	11.3	9.0	12.7
20m leg	25	38	30	42	35	46

Figure 4-1: Force on 5 meter diameter cylinder with significant wave

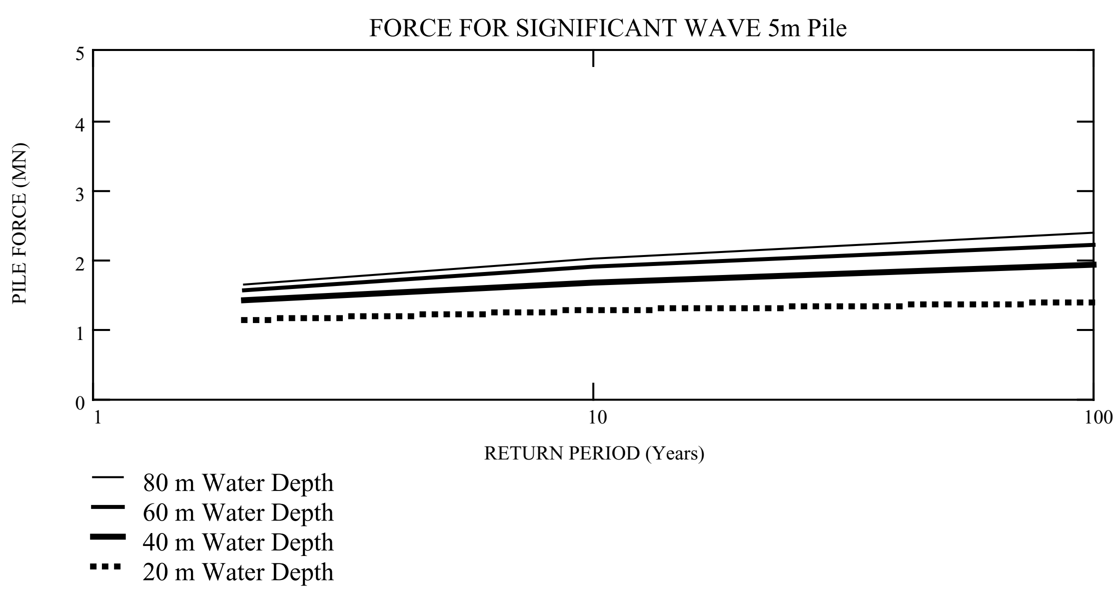


Figure 4-2: Force on 5 meter diameter cylinder with extreme wave

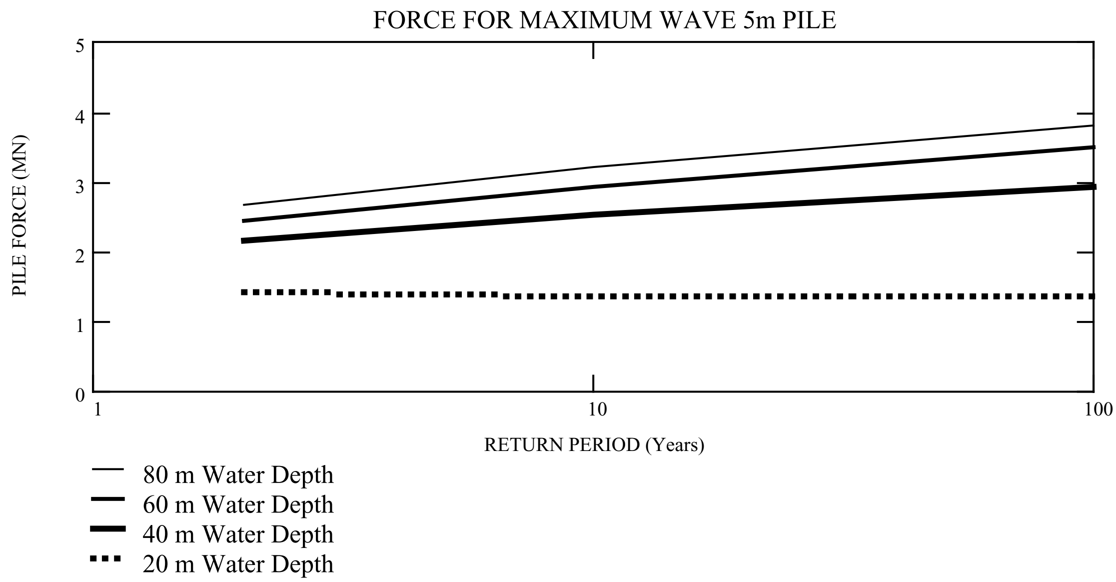


Figure 4-3: Force on 10 meter diameter cylinder with significant wave

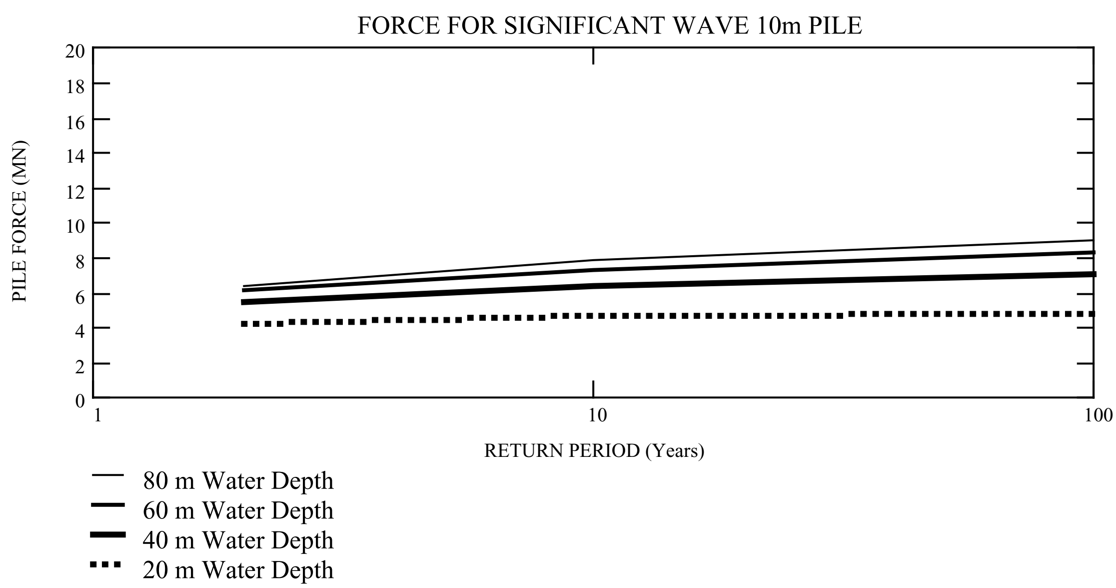


Figure 4-4: Force on 10 meter diameter cylinder with extreme wave

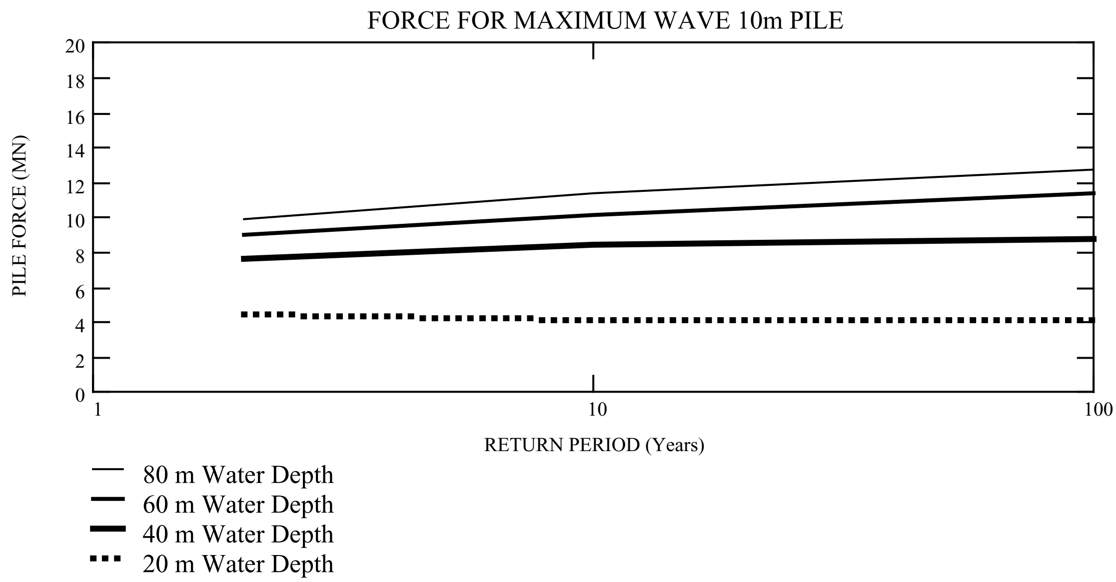


Figure 4-5: Force on 20 meter diameter cylinder with significant wave

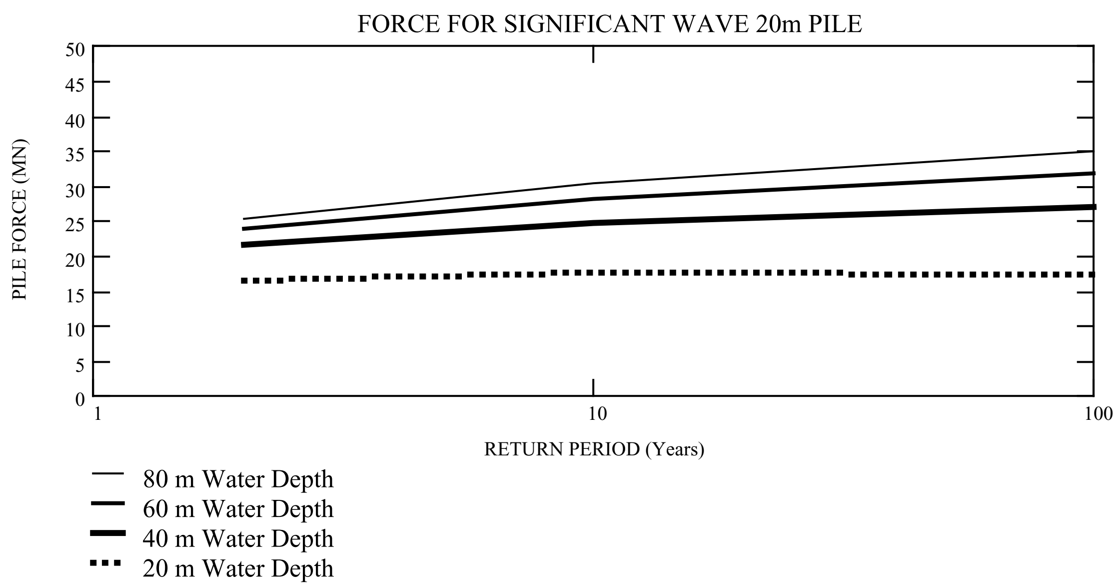
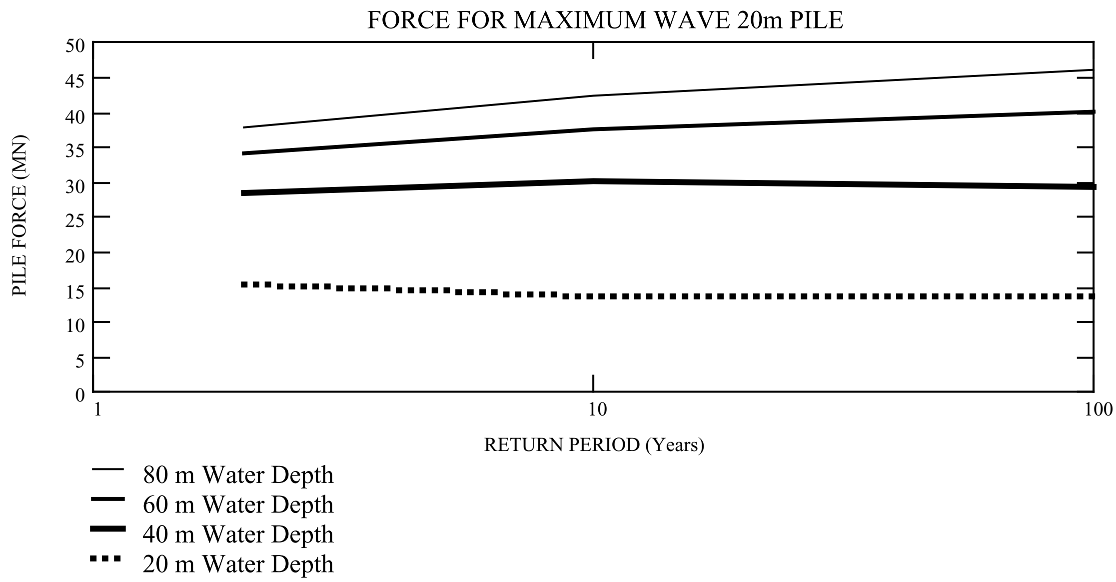


Figure 4-6: Force on 20 meter diameter cylinder with extreme wave



These data indicate that for the 100 meter wide vertical wall, the wave forces increase significantly with water depth. For the 100 meter wide vertical wall in 80 meter water depth the 100 year force for the extreme wave is approximately 1225 MN. For design purpose the 100 year extreme wave load should be multiplied by an appropriate factor of safety. Such a force level may present a significant challenge for providing the global resistance of the structure. For a 100 meter wide wall but with a sloping face the wave forces would be lower, see equation 4.1. For example if the angle of the face were 45 degree to the horizontal, the wave force could be half of that given in Tables 4.2 to 4.5. Note, however, that considerations of wave run-up would have to be addressed for a sloping face structure. The global wave load is only one of the many design aspects that would have to be addressed.

For cylindrical piles, the wave force is also a function of water depth and of return period with the dependency on water depth being stronger than on the return period. For square piles, the wave forces would be 25% larger (see equation 4.8) than those given in figures 4.1 to 4.5. However as can be seen from figures 4.1 to 4.5, cylindrical piles have wave forces much lower than for the 100 meter wide vertical wall. This is a result of the smaller waterline cross section plus the fact that the structure is circular. The circular nature allows for lower wave forces because there is a phase difference of the load on different parts of the structure. For a multi-legged structure the maximum wave force is not necessarily the single leg load times the number of legs. This is due to a phase difference between the wave load occurring at the different legs. For the 100 year extreme wave the wavelength using Table 4.1 about 1500 meter, much larger than the size of a structure and much larger the spacing of the legs. For such a wave the phase difference between the loads on different legs would be small and thus the structure load would on a conservative basis be the number of legs times the individual leg load.

The data in figures 4.1 to 4.6 indicate that as far as wave forces are concerned, particularly in the deeper water depths, there are advantages to using narrow waterplane structures. These types of structures reduce the wave forces.

In the estimation of the wave force there are a number of important factors that have not been taken into account. The major factor is that the real geometry of the structure has not been considered. For multi-legged structures there may be a base that goes from the seabed to some point below the water level. The presence of this design feature can significantly modify the magnitude of the wave force. In addition aspects such as uplift of the structure, run-up of waves on the legs and or vertical wall and slamming are important and should be considered in any design effort. The data presented here on wave force should be considered as preliminary only and should be considered to have a large degree of uncertainty. For conceptual or preliminary design, other sources of data, for example, pressure measurements from Model Scale tests should also be used. Note also that the wave heights used in these calculations are the mean values given in Table 4.1. There is a significant degree of uncertainty in the 10-year and 100-year wave height estimates. Using the lower bound of the 90% confidence limit or the upper bound on the 90% confidence limit would alter the wave forces calculated in this section. In addition, the 10 and 100-year significant wave height estimates may be uncertain due to them being based on a relatively few years of detailed wave data.

5 ICE FORCES

5.1 LOCAL ICE PRESSURES

The calculation of ice loads is a large subject that has been the subject of a much investigation. There are theoretical formulations for estimating such loads. Procedures that have gained a measure of acceptance in the ice community, for example API RP2N, are based principally on data gathered from offshore structures, ice going vessels and dedicated field projects.

The API relationship is between contact area in square meters and pressure. The expression is limited to 20 MPa for small areas and to 1.5 MPa for large areas. The relationship assumes that the ice fails in crushing and is principally applicable to local contact pressures. These data would be used for sizing the smaller scale structural elements.

$$
\begin{aligned}
&p = 8.1 \cdot Area^{-0.57} \quad (MPa) \\
&20\,(MPa) \le p \le 1.5\,(MPa)
\end{aligned}
\qquad 5.1
$$

The data from which the API relationship was obtained is dominated data collected on cold arctic ice. For the West Coast of Newfoundland, the ice will be mainly first-year with a high salinity and close to the melting temperature. Crushing pressures will thus expected to be significantly less than the API relationship. The API relationship was derived from a statistical analysis of the available data and equation 5.1 represents the mean plus two standard deviations curve. For the current geographical location ice pressure more in line with the mean of the data may be more appropriate. Under such circumstances the pressure can be represented by equation 5.2 in which the pressure is limited to 1.0 MPa at large areas and 20 MPa at small areas:

$$
\begin{aligned}
&p = 3.8 \cdot Area^{-0.57} \quad (MPa) \\
&20\,(MPa) \le p \le 1.0\,(MPa)
\end{aligned}
\qquad 5.2
$$

5.2 GLOBAL ICE PRESSURES

For estimating global ice forces on a wide structure, there are two main components to the ice force. The first component is that force required in failing the competent ice. For the West Coast of Newfoundland this ice would be either the solid first-year ice or solid rafted ice or the consolidated layer in a pressure ridge. The second component is the clearing force from the keel and the ridge of the first-year pressure ridge.

For a multi-legged structure, the ice force is calculated on the assumption that the space between the legs will block. While blocking will not occur for all ice-structure interactions it is a conservative assumption to assume that it will. Blocking is more likely to occur during the interaction with a ridge rather than with a level ice sheet. The ice in the region is deformed with a significant amount of ridging. The ice force is then calculated for the total swath width of the structure rather than the total width of the legs.

The solid ice is treated as a material with a constant shear or crushing strength and the rubble in the keel is treated as a friction material with a small cohesion. The parameters used in the estimation of the global ice loads for vertically sided structures are given in Table 5.1. The parameters are now discussed. The global ice pressure is the crushing failure pressure that is applicable over large areas. As the typical structure width is 100 meter and a typical level ice thickness is 1.0 m for the West Coast of Newfoundland the contact area is about 100 m^2 (Masterson & Frederking, 1993). The friction angle is a conservative estimate for the range. The data on rubble cohesion value for large ridges is sparse but the range given is believed to be a reasonable estimate. The sub-arctic factor takes into account a number of aspects; namely that the ice is warm and saline. In addition the thickness of the consolidated layer will not likely reach its maximum value at the same part of he ridge as the location when the keel is the maximum depth. Consolidated layer growth is a thermal process that will be impeded by the presence of a large keel. The factor of safety is the appropriate factor for the 1 in 100 year load.

Table 5-1: Parameters used in global load estimation

Parameter	Value	Symbol
Water unit weight	0.0100 (MN/m^3)	g_w
Ice unit weight	0.0089 (MN/m^3)	g_i
Ice global pressure	1.0 to 1.5 (MPa)	p_i
Sub-arctic factor	0.75	f_s
Ice rubble friction angle	20 to 45 (deg.)	ϕ
Ice rubble cohesion	0.004 to 0.020 MPa	c
Factor of safety	1.35	F_s

The global load from the consolidated layer plus the keel of the ridge for a wide structure is given by equation 5.3. For narrow structures an additional clearing force from the edges of the interaction zone would have to be included. For the dimensions assumed here this contribution is ignored. In addition the clearing force from the sail of the ridge has been ignored. The clearing loads are based on a soil mechanics approach for the keel rubble (Terzhagi et al 1996).

$$F_{ice} = \left[h_i p_i + 0.5 \gamma h_k^2 K_p + 2 c h_k \sqrt{K_p} \right] \cdot W \cdot f_s \cdot F_s \qquad 5.3$$

where:

$h_i = thickness\ of\ consolidated\ layer\ (m)$
$h_k = keel\ depth\ (m)$
$W = width\ of\ structure\ (m)$
$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$
$\gamma = g_w - g_i\ (N/m^3)$

To use equation 5.3 appropriate thickness for the consolidated and for the keel depth are required. From the Canatec Report Table 2-6 are assumed to represent the 100 year event. It is unlikely that the maximum consolidated thickness occurs in the ridge with the maximum keel depth. This is because the consolidation may be a thermally driven process and the greater sail thickness would reduce heat transfer from the core of the ridge to the atmosphere. In addition, the maximum keel thickness is, in general the thickness at a point rather than the thickness over a distance comparable to the width of the structure

Table 5-2: Extreme first-year pressure ridge dimensions (1 in 100-Year)

Parameter	Value
Consolidated Layer	2.0 (m)
Keel Depth	23 (m)

The uncertainty in material parameters for the consolidated layer strength and the keel parameters was taken into account, as follows. The three parameters were treated uniformly distributed random variables between the limits indicated in Table 5.1. For the 100 m wide structure, the forces are as follows for the dimensions in Table 5-2:

Minimum force	=	264 MN	
Mean force	=	357 MN	
Maximum force	=	474 MN	5.4
10% probability force	=	415 MN	
90% probability force	=	300 MN	

A complete Monte Carlo calculation would also take into account the keel and the consolidated thickness distributions. What the values in 5.4 represent is the range in ice interaction force levels given the ridge dimensions in Table 5.2. The 10% and 90% force levels indicate that the force will likely be between approximately 300 and 415 MN.

For structures with a series of cylindrical legs, the ice load will in general depend upon whether the ice jams between the legs. If this does happen then the ice force will be calculated using the swath width of the structure rather than the leg width at the waterline. While the ice may not jam between the legs, the state of knowledge does not appear to be adequate to reliably indicate under what conditions this may be true. For the 100-year ice forces the question is if the ice will jam when a pressure ridge is interacting with the structure. Thus to be conservative, we will assume that jamming will occur. The ice forces will then be determined using equation 5.3 where W is now the swath width of the structure rather than the 100 meter width.

5.3 ICEBERG IMPACT FORCES

From the data presented in the Canatec report, there are icebergs that pass through the area. In this section we estimate the 100 year return period interaction force.

The interaction force model used here is that presented by Nevel (IAHR 1986). In that model the floe interacts with the structure crushing ice at the contact zone. The volume of crushed ice is calculated from the decrease in kinetic energy of the floe. From the geometry of the floe and the structure, the contact area at the end of the interaction is calculated. From the maximum contact area the interaction force is then calculated. The various parameters in the model were treated as random variables from which the probability of exceedence of the force was calculated. The flux of icebergs was then used to calculate the probability for the 100 year event.

The interaction force between a spherical floe and the structure is given by:

$$M = \rho_{ice} \frac{4}{3} \pi r^3 \left(1 + M_{add}\right)$$

$$Vol = \frac{M V^2}{2} \left(1 - \frac{e^2}{1 + \frac{2}{5}}\right) \frac{1}{\sigma\left(1 + \mu \tan(\beta)\right)} \qquad \text{5.5 a,b,c}$$

$$F = \sigma \sqrt{4 \pi r \, Vol}$$

where:

$M = Floe\ mass\ (kg)$

$Vol = Volume\ of\ crushed\ ice\ \left(m^3\right)$

$F = Maximum\ force\ (N)$

$\rho_{ice} = Ice\ density\ \left(kg/m^3\right)$

$r = Floe\ radius\ (m)$

$M_{add} = Added\ mass\ coefficient$

$V = Floe\ velocity\ (m/s)$

$e = Eccentricity$

$\sigma = Ice\ strength\ (Pa)$

$\mu = Coeficient\ of\ friction$

$\beta = Contact\ angle$

5.3.1 Random Variables

The parameters describing the interaction are treated as uncorrelated random variables. The form and characteristics of the distribution is guided by the available field data. We now outline the limitations and approximations for the random variables.

The icebergs were treated as spherical and the effective mass calculated assuming that there was a 40% added mass contribution. The field data in Table 2.9 of Canatec from 1960 to 1977 was used to select a suitable distribution function. Processing the data indicated that the data on iceberg length could be represented by an exponential distribution. The floe radius was taken as half of the length. The mean iceberg radius used in the distribution was 28.5 meters.

The floe velocity distribution used the data in Figure 2.17 of Canatec, a log-normal distribution function was found to be a suitable representation of the data. The mean value used was 0.18 m/s and the standard deviation was 0.085 m/s. Note that the velocity is the average velocity over a period of one day rather than the instantaneous velocity. The instantaneous velocity will be larger than the daily average velocity because of the irregular motion of the floes.

The ice strength was based on Masterson et al (1992) who presented iceberg strength data at medium scale. The data given for area between 1 and 3 m^2 were used. A scaled beta distribution was used with a mean value of 3 MPa, a standard deviation of 1 MPa and a range between 0 and 6 MPa. The contact pressure is likely to be lower than the values used for the contact area at the end of the interaction. This would then lead to lower force levels.

The eccentricity was treated as a uniform distribution between 0 and 1.

The impact angle was treated as a uniform distribution between 0 and 90 degree.

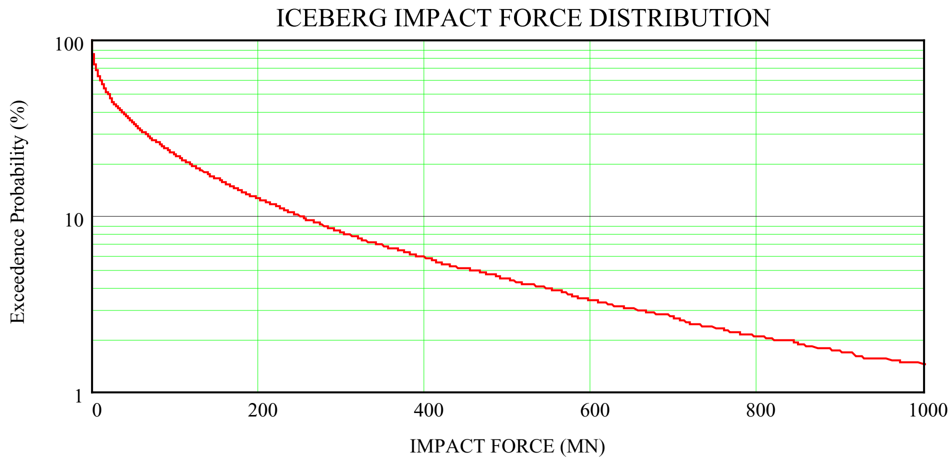
The ice-structure friction coefficient was treated as a uniform distribution between 0.15 and 0.45.

The flux of the icebergs used data from Table 2.10 of Canatec. No allowance was made for the grounding of icebergs in shallow water. The grounding of the icebergs will reduce their flux and introduce a dependence on water depth into the 100 year force levels. The iceberg flux was calculated for a structure 100m in diameter. Since the impact rate linearly depends on the structure diameter, different structures would have different impact rates and hence different 100 year force levels.

5.3.2 Calculated Results

The impact force distribution is illustrated in Figure 5.1 for a 100m diameter structure. Note that the safety factor of 1.35 has been incorporated into the force levels. These impact forces should be considered for conceptual or pre-conceptual purpose only. For preliminary or detailed structure engineering, the approximations and idealizations in the model and the data would have to be addressed.

Figure 5-1: Iceberg Impact Force Distribution



The 100 year impact force is provided in Table 5.3 using the impact rates from Table 2.10 of Canatec. As may be seen the 100 year return period iceberg impact force is strongly dependent upon the zone under consideration. The 100 year force reflects the variation in annual impact probability. Note that in Table 5.3 the 100 year forces have incorporated the 1.35 factor of safety.

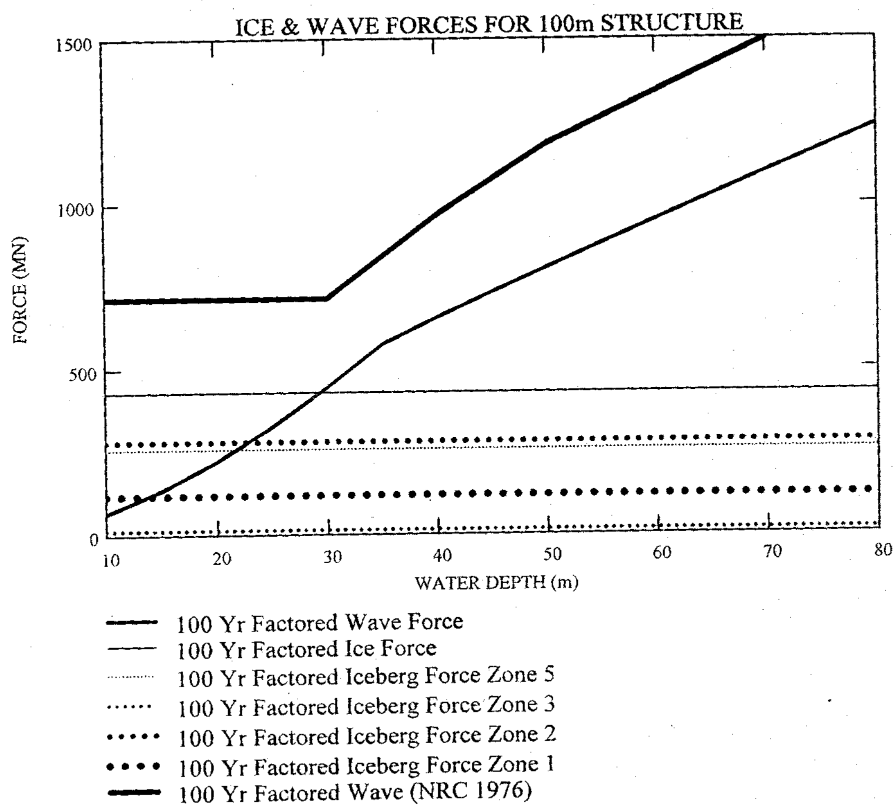
Table 5.3: 100 year Iceberg Factored Impact Forces by Zone

Zone	Annual Impact Probability all floes	Impact Probability in 100 Years	100 Year Return Period Force (MN)
5	0.023	90.2%	256
3	0.006	45.2%	14
2	0.024	91.2%	279
1	0.016	80.1%	117

5.4 COMPARISON BETWEEN WAVE AND ICE FORCES.

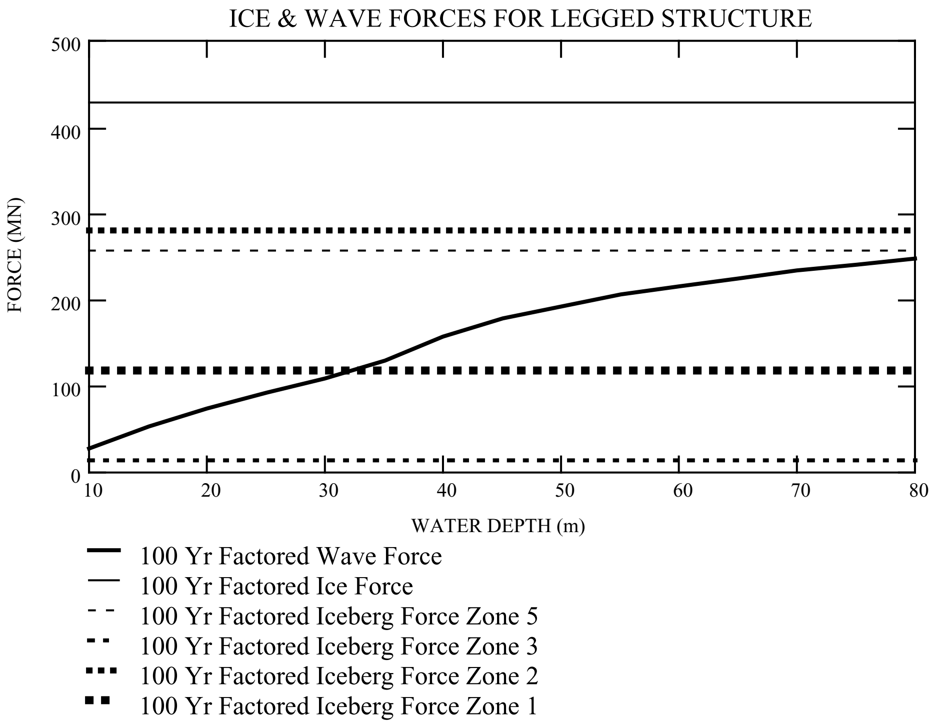
We now compare the wave forces and ice forces for various structural elements. The comparison will be done as a function of water depth. The results of the wave, ice force and iceberg impact calculations for a 100 wide vertical structure are presented in Figure 5.2. The wave force has been multiplied by a factor of safety of 1.35 to make it directly comparable with the ice and iceberg impact load. In Figure 5.2 there are two lines for the 1 in 100 year wave force. The lower line is calculated using Morris and the upper line is calculated from the NRC methodology. The NRC curve is considered to be more reliable. As may be inspected, the wave force dominates over the ridge interaction force and also over the iceberg impact force. The data presented suggests that shapes which reduce or minimize the wave force should be considered.

Figure 5.2 : Factored 100 Year Ice and Wave forces for 100m wide vertical structure.



Similar calculations as presented in Figure 5.2 were done for legged structures and are presented in figure 5.3. The example is for a structure with four 20 meter diameter cylindrical legs. The structure had an overall swath width of 100m. As may be seen from figure 5.3 the ice force dominates over the wave force for all water depths. Note that for the legged structure the ice force is proportional to the swath width. If the swath width were reduced from the 100 meter value assumed, then the overall force would also be reduced. The iceberg force depends upon which zone is considered and also on the diameter of the structure. The iceberg force is comparable to the wave force but less than the ridge ice force.

Figure 5.3 Factored 100 year Ice and Wave forces for a structure with 4 of 20 m diameter legs and a swath width of 100m



Combining the data illustrated in Figures 5.2 and 5.3 leads to the following conclusion. At water depths of less than about 30 m structure designs which are either vertically sided or a legged structure could be considered. At water depths of greater than about 30m the relative ice and wave forces suggest that legged structures would have benefits over vertically sided ones. For the legged structures in the deeper water there are advantages to reducing the swath width of the structure. For a caisson structure the wave force could be reduced by angling the faces away from the vertical. For example if the angle were 20 degree the wave force would be reduced by 11.7% (see equation 4.1). This feature would then shift the water depth at which the ice and wave forces were equal to deeper water depths. There are however, other considerations such as the dynamic response of the structure and floating stability during installation that would come into play.

5.5 SLOPING STRUCTURES

For structures with sloping faces at the waterline e.g. cones the interaction force would be calculated by using an appropriate flexural failure model for example Nevel (1992).

The considerations in the earlier sections suggested that narrow waterplane concepts could be considered at water depths greater than 30 m. A monopod with an icebreaking cone could be used as a narrow waterplane structure. Note that, in general, the ice breaking force may be less for a cone than for a vertically sited structure. Thus at water depths of less than 30 m cone type structures could also be considered.

5.6 GEOTECHNICAL CONSIDERATIONS

Data for the type and strength of the bottom sediments are presented in Sections 5.2 and 5.3 of the Canatec report. For much of the offshore area there is a thin surface layer of pelite that has a shear strength of between 1 and 5 KPa. The data however are based on only very few measurements. For a structure with a base of 100 meter by 100 meter the reported shear strength values would provide for a structure resistance of between 10 MN and 50 MN. This resistance is much less than the environmental forces presented above. Note that for a shear material the resistance would be independent of the total load on bottom and only dependent upon the area of the base. Other areas have a sand layer on the seabed. Typically the friction angle between the sand and the bottom of structures is between 25 and 35 degree. For this material the available sliding resistance would be a proportional function of the total bottom load from the structure but independent of the area of the base. With suitable ballast in the structure the available resistance can be adjusted. For example haematite is available as a high density aggregate. These considerations suggest that the weak surface layer should be removed and replaced with a sand or gravel material. If the thin surface layer cannot be avoided then a base employing skirts could be considered. These skirts would penetrate the surfaced layers. The shear strength used in design would then be the strength of the subsurface layer rather than the surface layer.

Some concepts use piles to provide for the lateral resistance to environmental forces. In many locations there is only a thin layer of deposited material before bedrock is reached. Thus the applicability of using piles would have to be assessed once a particular location for the structure has been selected.

A difficulty in resisting the ice and wave forces in a piled design is the difference in stiffness between the pile and the soil. In areas without ice the multi-legged structure can be made essentially transparent to wave forces. In an ice region the leg diameter will be larger to resist the ice forces leading to larger wave forces. A multi-legged structure in an ice region will have significant wave and ice force levels. Thus the question of a piled design is one of resisting the appropriate environmental force level.

Connection details between the piles and the structure are also important. The entire load is transferred to the piles through a relatively small part of the base. This can lead to difficulties in designing the connections, which are both sufficiently strong while installable in the field.

5.7 SEISMIC LOADS

The area is one of low seismic risk compared with many areas of the world or of Canada. From the data of Figure 5.5 of Canatec, the seismic acceleration for the 100 year event is less than about 0.05 g. However for the 10,000 year event the seismic accelerations can approach 0.9 g. It is surprizing however that the 10,000 year acceleration is much greater than the 100 year accelerations. Except for possibly structures with low lateral stiffness, the response to 200 year or 5,000 year seismic events is not likely to be a significant limitation on the design.

5.8 DYNAMIC CHARACTERISTICS

For bottom founded structures in ice environments the dynamic response characteristics should be considered. The dynamic response can effect aspects such as the global sliding resistance, fatigue design, acceleration levels for equipment and acceleration levels for personnel habitability. For the wave loading, the forcing function can be sinusoidal with an impulse for the wave slamming. The ice force waveforms are usually not sinusoidal in shape. Often the waveforms can be represented as a linear ramp and with a rapid load drop. The frequency of the waveform can cover a wide range depending upon the mode of failure of the ice. (See for example Langohr and Ghali, 1997). The range of input forcing function frequency will likely overlap the range of natural resonant frequencies of the structure. In structural design the dynamic response characteristics is often a major topic. Within the scope of this project no definite statements can be made about the structure dynamic characteristics except the following. Even caisson type structures e.g. the Molikpaq (Brown et al 1992) have dynamic characteristics in the frequency range of the ice forces. In addition legged structures (Tseng et al 1992) in the Bohai Gulf of China have collapsed due to the effects of ice loading dynamics.

6 COMPARISON OF WAVE AND ICE ENVIRONMENTS IN DIFFERENT GEOGRAPHICAL REGIONS

The wave and ice environment for the West Coast of Newfoundland is compared to the wave and ice environment in a few different geographical regions of the world. Regions which have had drilling structures designed or deployed will be used.

Table 6-1: Typical wave and ice environments

Location	Water Depth (m)	100 Year wave H_{sig} (m)	Level First-Year Ice	First-Year Ridge	Other Ice
W Coast Newfoundland	20 to 80	10.6	1.5 m	23m	Icebergs
Navarin Basin	100 to 200	13.5	1.0 m	22m	None
Sea of Okhotsk	30	11	1.5 m	22m	None
Canadian Beaufort	30	5	2.0 m	~30m	Multi-year
Grand Banks	90	15.5	Small floes	N/A	Icebergs

From Table 6.1 it can be seen that the ice environment off the West Coast of Newfoundland is less severe than the Canadian Beaufort Sea. The wave environment is less severe than at the Grand Banks. The ice and wave environments and water depths are very similar to the Sea of Okhotsk. Note that Sakhalin is considered to be a SEVERE wave environment for the Molikpaq. Given the limited fetch on the Gulf of St. Lawrence compared with the East Coast of Sakhalin, there are some doubts as if the 100 year significant wave height is 10.6 m. Further field data would be required to elucidate this point. The major differences between these two areas are that the Sakhalin area has greater seismic risk than for the West Coast of Newfoundland and there are occasional icebergs. It may be expected that designs for the Sakhalin region would, in general, be applicable to the West Coast of Newfoundland. Designs optimized for the Beaufort Sea may not be optimum for the current area as the wave and ice environments are significantly different. Beaufort Sea designs are applicable to locations where the ice force is the dominant environmental loading factor. The Navarin Basin is part of the Bearing Sea and a number of concepts have been proposed, see for example Wang et al 1985. The ice environment is similar to the West Coat of Newfoundland although the wave environment is more severe.

7 IMPLICATION OF ENVIRONMENTAL LOADS ON CONCEPTS

The various environmental load estimates were presented in Section 5. In this section we consider the effect of the loads on the selection of concepts.

7.1 FLOATING PRODUCTION SYSTEMS

The wave forces on a moored structure would be less than for the bottom founded structures used in section 5. The mooring could probably be made to function for the West Coast of Newfoundland wave climate. The ice forces however would also be less than for the examples in section 5. The major reason is that for the extreme load from the first year ridge, the keel depth would be greater than the draft of the vessel. However loads of about 300 to 400 MN could still be expected. Such loads would be difficult to accommodate. For example the Kulluk has a design global ice load of 7.4 MN (Masterson, 1991). Note that this circular drill ship was specifically designed for the Beaufort Sea. The global resistance is provided by a set of chains and anchors. In operation, an icebreaker was used to break up ice floes upstream of the Kulluk. In the Beaufort Sea, wave forces are less severe than the West Coast of Newfoundland. Even so, the Kulluk, being circular, had difficulties in high seas. A drilling/procedure vessel which is more “ship shaped” may be more appropriate particularly if the vessel can be weather-vaned. This would suggest that the structure may have to leave the location during such an event or that active management of the ice may be required from ice breaking ships. These aspects would have to be included in designs.

Floating production systems have been considered for the Hibernia structure rather than the gravity based structure currently deployed. Also floating production systems are being considered for Sable Island. Note that the ice environment at these locations is different from the West Coast of Newfoundland and different solutions are found.

The open water season for the area varies within the lease area. From data in Figure 2.11 of Canatec, the open water period is between 8 and 9 months and the iceberg impact rate is low. For a summer only system, the major environmental loading would be from waves. These suggest that a seasonal production system could be investigated. There were suggestions for a seasonal production system at Hibernia and also for the Terra-Nova field. The variables of a seasonal production will depend upon the reservoir and it’s product. Gas is difficult to shut in whereas oil may not be. For example, if the oil is waxy, then the pipeline may have to be purged prior to the shut-in.

The overall suggestion is that it may be difficult to make a year-round floating production structure work in the current geographical location although summer only systems should be considered.

7.2 BOTTOM FOUNDED SYSTEMS

The lack of specifications on the producing field means that definite statements on what concepts are suitable cannot be made. Assuming that the geotechnical considerations can be met then a wide range of structural concepts are possible.

The ridge force is likely to be larger than the iceberg impact force given the rarity of icebergs. The 100 year ridge has a 23 m keel and thus the grounding of the ridge is unlikely to be a significant concern in the nominal 20 to 100 m water depth range.

The wave forces increase with increasing water depths. At about a water depth of 30 m the ice force and the wave forces may be approximately equal. Thus the type of structure which may be appropriate depends upon water depth. At depths of greater than the 30 m water limit, designs which reduce the wave force could be considered. Note that the 30 m limit is dependent upon the 100 years significant wave height assumed and the characteristics of the pressure ridge assumed. There are designs for the Sakhalin region of the Sea of Okhotsk currently under way. The Molikpaq has been re-deployed as a production structure in the region and other designs are being considered.

Given the general nature of this study, definite statements about particular structures cannot be made.

8 RECOMMENDATIONS

The recommendations regarding the environmental data deficiencies have been made in the report by Canatec.

The major deficiencies for furthering conceptual analysis of structures, is a complete lack of data on the field characteristics and the water depth for the structure. Once these items are known then conceptual design work can be conducted. The ice environment data is reasonably detailed and does allow estimates of loading forces. Additional fieldwork to define the wave environment in the shallow water should be undertaken. In addition, numerical and physical modeling of the complex interaction between breaking waves in shallow water and the actual structural geometries are needed. Of critical importance is the very limited data on the strength and other characteristics of the seabed. These data are important in guiding what type of structure base is appropriate. For example if piles or gravity based structures are appropriate.

The wave data for large return periods may be improved by accessing long-term wind databases combined with appropriate modelling of the fetch and storm duration data. A program to record wave heights over a continuous period of 1 to 2 years located in the region of interest would improve the reliability of the wave height data.

The thickness of the consolidated layer in the large pressure ridges is a major factor in estimating ice forces. There is very little information on this parameter for the region of interest. Extrapolation from data collected in cold arctic regions may be unreliable. Field measurements of the consolidated layer would be advantageous along with numerical and physical modelling of the growth of the consolidated layer.

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APPENDIX A

Background Material

APPENDIX B

Project Data Sheets

PERD Research Project Deliverable/Document Data Sheet

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Notes or Remarks

Abstract

Sandwell Engineering Inc. and Canatec Consultants Ltd., were contracted to perform a study of Ice Regimes off the West Coast of Newfoundland. The study had two major aspects. The first was to access and document available ice and environmental data for the geographical area of investigation. The second aspect was to use this ice and environmental data to provide recommendations, where appropriate, on offshore oil/gas drilling structure concepts and structures. The study was to concentrate on data applicable to the offshore oil lease areas on the West Coast of Newfoundland. This report describes the environmental conditions in general. The wave and ice regimes data are used to estimate environmental forces on different representative structure geometries that could be utilized for offshore oil production.

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