APPENDIX 5
QIKIQTANI INUIT ASSOCIATION ATTACHMENTS

QIA 13 ATTACHMENT 1: FULL RESPONSE

QIA IR #13:

The Proponent's primary data source for IQ data collection and Inuit land and marine use is through community workshops.

A. Please identify whether and how this approach to data collection is appropriate in an Inuit context and identify other methods considered to gather cultural, traditional use, and IQ-related information and knowledge.
B. Please also provide more information on how it engaged with communities in the identification of the process used to gather IQ and Inuit Land and Marine Use information.
C. Identify what opportunities were Inuit given to determine what questions were asked and what topics were focused on in the workshops.

Baffinland Response:

Baffinland has utilized several methods to collect IQ during the environmental assessment phases of the Mary River Project (not only community workshops). IQ and other information obtained through these methods has continued to inform the assessment of Project-related effects and the design of Project components. Key sources of IQ that have been utilized by Baffinland to-date are summarized in Table 1.

Baffinland conducted community workshops for the Phase 2 Proposal as an additional supplement to IQ that had already been collected for the Project. More specifically, Baffinland identified the need for additional information to be collected regarding contemporary Inuit land uses in areas potentially affected by the Phase 2 Proposal. In addition to this land use information, there was a need for community concerns specific to the Phase 2 Proposal to be discussed and for potential mitigation measures to be reviewed and assessed. With these information gaps in mind, planning and execution of the community workshops commenced. QIA was involved the planning of these workshops and had representatives participate in each of them. Specific responses to the QIA's information requests are found in the sections that follow.

Table 1: Key Sources of IQ Utilized by Baffinland

Project Phase	IQ Source	Description	References
FEIS	Land Use Report	Provides an overview of the history of the region, information on land use during the contact-traditional period, and a more detailed overview of current land use activities in the North Baffin and Foxe Basin areas. Published sources, personal communications, and the Project's own IQ study were used to describe land use in the Mary River Project and surrounding areas.	**FEIS Appendix 4C (Baffinland 2012): Land Use Report** **FEIS Appendix 2B (Baffinland 2012): Summary of Community-Based Research Undertaken for the Mary River Project 2006 to 2010** **FEIS Addendum for the Phase 2 Proposal, TSD-05: Mary River Inuit**
	Individual Interviews	IQ interviews were conducted with individuals in Arctic Bay, Igloolik and Pond Inlet from 2006 to 2008. Working groups identified key knowledge holders in the community. Interview questions focused on Inuit use and understanding of the land, caribou, marine mammals, fish, birds, and other land mammals. A total of 45 interviews were completed.	
	Topic-Specific Workshops	From 2007 to 2010, workshops on caribou, marine mammals and Inuit land use were conducted in the North Baffin and South Baffin communities to identify areas of importance and use to	

Project Phase	IQ Source	Description	References
		Inuit and to identify potential Project interactions with these topics. A total of 15 workshops were completed.	**Knowledge Study Mapbook** **IQ Database for the FEIS (Baffinland and QIA internal database)**: Interview transcripts, workshop notes, and working group meeting minutes were incorporated into a central database and coded to sort by topic. Coding was completed using the NVivo software package, a commonly used application for analyzing qualitative data. The IQ database contains over 500 topic directories. Topic reports were made available for scientists and specialists involved in the Project to use.
	Individual Working Group Meetings	Information was collected through the establishment of, and meetings with, individual working groups in each North Baffin community. A total of five working groups were established: • Pisiksik Working Group (Pond Inlet - 2006) • Qaatiliit Working Group (Igloolik - 2007) • Inuksuligarjuk Working Group (Arctic Bay - 2007) • Tikkuu Working Group (Hall Beach - 2008) • Ukkakkut Working Group (Clyde River - 2008)	
	'Kajjuqtikkut' - Arctic Bay Working Group Meeting	In March 2008, Baffinland sponsored a 5-day workshop in Arctic Bay that brought together the working groups from each of the five North Baffin communities to discuss socio-economic issues, caribou, marine mammals, transportation, and the future of the working groups. Participants named the meeting 'Kajjuqtikkut', which means 'the place where everyone comes to meet after traveling', in reference to a place where Inuit traditionally used to meet near Nuluujaak (Mary River).	
	Baffinland - QIA Thematic Workshop	On September 8-12, 2010 Baffinland and the QIA jointly hosted a thematic workshop at Mary River. The QIA selected 3 to 4 representatives from each participating North Baffin and South Baffin community. The agenda included five days of thematic discussions on the EA process for the Project, caribou, marine mammals and shipping, the proposed 3 Mtpa road operation, consisting of year-round haulage of ore over the Milne Inlet Tote Road and open water shipping of ore out of Milne Inlet.	
FEIS Addendum for the Phase 2 Proposal	Community Workshops (2015 to 2016)	Baffinland organized a series of 'invited persons' workshops and public open houses in Pond Inlet and Arctic Bay, Nunavut between March 2015 and May 2016 to discuss the Phase 2 Proposal. These workshops were focused on five main themes: • Contemporary Inuit land use in the Eclipse Sound and Navy Board Inlet areas • Shipping through ice • Open water shipping • Caribou • Phase 2 and Arctic Bay In addition, a verification meeting was held in Pond Inlet in May 2016.	**FEIS Addendum for the Phase 2 Proposal, TSD-03: Phase 2 Workshop Report** **FEIS Addendum for the Phase 2 Proposal, TSD-04: Public Consultation Report**
	Community and Stakeholder Engagement	Baffinland has created several opportunities for Inuit community members and other stakeholders to share information with the Company (including IQ, if desired), provide feedback and/or suggestions regarding the Phase 2 Proposal. Baffinland has organized or participated in various meetings with the public, and community and stakeholder groups (including working groups), as described further in TSD-04.	
	Community Workshops (Planned to Occur in 2019)	The anticipated goal of these workshops is to establish community-informed terrestrial and marine environmental protection measures for the Phase 2 Proposal. For greater clarity, 'protection measures' may include management, mitigation, and/or related monitoring initiatives. Caribou are expected to be the primary focus of the terrestrial environment workshop discussions, while narwhal and aquatic invasive species/ballast water discharge are expected to be the primary focus of the marine environment workshop discussions.	

Project Phase	IQ Source	Description	References
		Additional topics will be discussed if time allows. QIA continues to be engaged on the design and execution of these workshops.	

A. Please identify whether and how this approach to data collection is appropriate in an Inuit context and identify other methods considered to gather cultural, traditional use, and IQ-related information and knowledge.

Workshops (also known as 'focus groups' or 'group interviews') are a commonly used qualitative research method (Patton 2002) that may be used in IQ-related work (e.g. Armitage and Kilburn 2015, Huntington 2000, Huntington et al. 2002). They have been successfully used by Baffinland in the past to collect IQ (see Table 1), to collect IQ for other recently approved mining projects in Nunavut (e.g. Kitikmeot Inuit Association 2014; ERM 2016), and for other mining-community focused research in Nunavut (e.g. Pauktuutit Inuit Women of Canada et al. 2014). Workshops typically encourage cooperation between participants, which is an important theme encouraged by Inuit Societal Values (e.g. 'fostering good spirits by being, open, welcoming and inclusive'; 'working together for a common cause'; and 'decision-making through discussion and consensus') (Government of Nunavut 2018).

Baffinland acknowledges different data collection methods have their individual strengths and weaknesses, and that no one method is necessarily without limitations (see for example: Armitage and Kilburn 2015, Huntington 2000, Huntington et al. 2002, and Patton 2002). Baffinland has employed several data collection and engagement methods during Project development to ensure a range of community perspectives have been shared and captured. These methods have included desktop studies, interviews, workshops, and meetings with the public and various stakeholder groups. Considering Baffinland's success with the workshop model in the past and the considerable amount of IQ that had already been collected for the Project, invited persons workshops were selected to help fill data gaps for the Phase 2 Proposal and supplement existing sources of information. Several open houses were also organized in association with each workshop to provide opportunities for the general public to participate. Likewise, follow-up meetings to each of the workshops were held with the Mary River Community Group (MRCG) where workshop summaries were presented for further input and feedback.

Baffinland developed and consulted with the QIA on its Phase 2 community workshop methodology proposals before commencing the workshops, in order to provide the QIA with an opportunity to comment on and approve all community workshop plans. Various suggestions were made by the QIA during their review of these documents, which were considered and incorporated by Baffinland as appropriate. QIA provided formal approval of the final workshop methodology proposals before any of the workshops proceeded. The QIA also had several representatives attend these workshops in-person.

Considering the above, it is Baffinland's view that properly designed workshops can indeed be appropriate in an Inuit context and particularly when matched with other data collection methods. For greater clarity, Baffinland has not suggested its workshops necessarily enabled communal decision making, consensus on all issues raised, or represented the views of every community member. Rather, Section 2.5 (Data Limitations) of TSD-03 notes:

> *"…this report provides a review of key issues raised by community representatives*

with regards to Phase 2. It does not present a complete representation of all viewpoints held by community representatives. The information presented in this report also isn't intended to reflect workshop and/or open house participant consensus on particular issues. Rather, it summarizes a range of discussions (organized by theme, where possible) that were held.

While this report strives to present land use information shared by workshop participants in a comprehensive manner, some discrepancies and data gaps may nevertheless exist. For example, workshop participants were not always in full agreement on the details of some land use activities (e.g. their exact timing and location) and not every knowledgeable land user participated in these workshops. As such, it is possible the land use information presented in this report could be added to and improved on in the future. Report maps, graphs, and summaries should be used for general information purposes only, with the understanding they may not fully represent all land use activities that occur."

B. Please also provide more information on how it engaged with communities in the identification of the process used to gather IQ and Inuit Land and Marine Use information.

&

C. Identify what opportunities were Inuit given to determine what questions were asked and what topics were focused on in the workshops.

Community-based research in support of the Mary River Project began in 2006 and focused on five North Baffin communities which have a traditional land and/or marine use tie to the Project development area (Arctic Bay, Clyde River, Hall Beach, Igloolik, Pond Inlet). In 2010, IQ research was also undertaken in two South Baffin communities (Cape Dorset, Kimmirut).

In the North Baffin, initial information and IQ was collected through the establishment of working groups in each community. Working groups were typically selected to represent a cross-section of people in the community with respect to sex, age, lifestyles and occupation. Baffinland approached Elders committees, hamlet leadership, HTO/HTA, and women's committees, and requested participation by nomination of a representative to the working group. Youth representatives and other recognized community experts such as people familiar with Project areas were later identified by the newly established working group to round out representation in the group. Knowledge was recorded through the course of discussion in working group meetings. Working groups were not established in the South Baffin communities due to the more limited focus of IQ studies there. Research agreements were negotiated between each of the five North Baffin community working groups and Baffinland and outlined roles and responsibilities of the parties; purpose and methods of the IQ study; and clarification on matters of privacy, informed consent and ownership of data.

Subsequently, several methods were used to collect IQ and further engage communities for the FEIS, as described in Table 1. FEIS Appendix 2B: Summary of Community-Based Research Undertaken for the Mary River Project 2006 to 2010 (Baffinland 2012) should be consulted for additional details. Information collected during the FEIS has provided an important foundation of IQ on which Baffinland continues to build upon. While the community working groups no longer exist, a Mary River Community Group (MRCG) has since been formed in Pond Inlet in cooperation with the QIA.

For the Phase 2 Proposal, workshop topics were identified by Baffinland after an internal review of data gaps in the existing IQ study, through discussions with the QIA, and after soliciting community feedback during an open house hosted by Baffinland on the Phase 2 Proposal in Pond Inlet on January 22, 2015. Preliminary workshop topics were also discussed with the Pisiksik Community Advisory Group (CAG; later dissolved in favour of creating the MRCG) in Pond Inlet on January 23, 2015. CAG members provided valuable feedback on potential workshop topics and structure, while also suggesting a number of potential workshop participants. The CAG members agreed that an invited persons workshop format would be useful for collecting information, but also felt that various members of the public and the CAG themselves should be provided the opportunity to participate. Workshop participants were also generally free to discuss any sub-topics/issues they pleased, within the overarching workshop topic.

As noted previously, Baffinland developed and consulted with the QIA on its Phase 2 community workshop methodology proposals before commencing the workshops, in order to provide the QIA with an opportunity to comment on and approve all community workshop plans. Baffinland is committed to working with QIA on future IQ studies and welcomes their feedback on alternative data collection methods that may be employed. Baffinland also acknowledges IIBA Article 16.3 (Collection and Use of IQ) will continue to guide the Company's actions in this area.

References:

Armitage, P. and S. Kilburn. 2015. *Conduct of Traditional Knowledge Research – A Reference Guide.* Whitehorse, YT: Wildlife Management Advisory Council North Slope.

Baffinland Iron Mines Corporation (Baffinland). 2012. *Mary River Project - Final Environmental Impact Statement*. February 2012.

ERM. 2016. *Phase 2 of the Hope Bay Project: Caribou Workshop*. Prepared for TMAC Resources Inc. by ERM Consultants Canada Ltd.: Vancouver, British Columbia.

Government of Nunavut. 2018. *Turaaqtavut*. 50 pages. Found at https://www.gov.nu.ca/information/turaaqtavut-0. Accessed on April 5, 2018.

Huntington, H.P. 2000. Using Traditional Ecological Knowledge in science: Methods and applications. *Ecological Applications*, 10(5): 1270-1274.

Huntington, H.P., P.K. Brown-Schwalenberg, K.J. Frost, M.E. Fernandez-Gimenez, D.W. Norton, and D.H. Rosenberg. 2002. Observations of the workshop as a means of improving communication between holders of traditional and scientific knowledge. *Environmental Management*, 30(6): 778-792.

Kitikmeot Inuit Association (KIA). 2014. *Naonaiyaotit Traditional Knowledge Project-Hannigayok (Sabina Gold & Silver Corp. Proposed Back River Project). Results from Data Gaps Workshops, Final Report (June 2014)*. Kitikmeot Inuit Association, Kugluktuk NU.

Patton, M.Q. 2002. *Qualitative Research and Evaluation Methods*. 3rd ed. Thousand Oaks, California: SAGE.

Pauktuutit Inuit Women of Canada, K. Czyzewski, F. Tester, N. Aaruaq, and S. Blangy. 2014. *The Impact of Resource Extraction on Inuit Women and Families in Qamani'tuaq, Nunavut Territory: A Qualitative Assessment.* Report for the Canadian Women's Foundation, January 2014. Pauktuutit, Inuit Women of Canada and School of Social Work, University of British Columbia.

QIA 14 ATTACHMENT 1: FULL RESPONSE

Table 1: IQ Recording Methods Used for the Phase 2 Community Workshops

Workshop Component	IQ Recording Methods	Additional Information
Invited Persons Workshops & Verification Meeting	• Consent forms • Workshop notes • Audio recordings • Poster-sized base maps • GIS database	• Consent forms were developed in cooperation with the QIA and were provided to workshop participants in both English and Inuktitut. Participants signed these for each workshop. • Appendix D of TSD-03 contains detailed workshop notes, including the name(s) of individuals who shared IQ and other information. • Audio recordings were made of each workshop and have been retained for archival purposes. • A GIS database was developed to store workshop spatial information and will be provided to the QIA. • Summarized results from workshops and open houses were reviewed during a final verification meeting.
Public Open Houses	• Consent forms • Open house notes • Poster-sized base maps • GIS database	• The same consent form utilized in the workshops was used in the open houses and served the dual purpose of being a sign-in sheet. • Appendix D of TSD-03 contains open house notes, although the names of individuals who shared IQ and other information were not recorded as these events were intended to be more informal and some individuals indicated on the sign-in sheets/consent forms they didn't want their names used. • The open houses were not audio recorded, due to the difficulties associated with audio recording in large group settings where multiple conversations could be occurring simultaneously. • A GIS database was developed to store open house spatial information and will be provided to the QIA.
Mary River Community Group Meetings	• Meeting notes	• Notes from MRCG meetings were not integrated into the analysis of workshop data, although feedback obtained during these meetings was used to identify potential issues or discrepancies that could be brought back to workshop participants at a later date.

QIA 20 ATTACHMENT 1: TABLE 1

Table 1

Inuit Law and Norm as Described by QIA	**Baffinland Sustainable Development Policy**
Working for the common good and not being motivated by personal interest or gain	"People are out greatest asset. Nothing is as important as their health and safety. Our motto is "Safety First, Always". "Baffinland applies the principles of pollution prevention, waste reduction and continuous improvement to minimize ecosystem impacts, and facilitate biodiversity conservation". "Baffinland contributes to the social, cultural and economic development of sustainable communities in the North Baffin Region." "Baffinland will take steps to understand, evaluate and manage risks on a continuing basis, including those that may impact the environment, employees, contractors, local communities, customers and shareholders".
Living in respectful relationships with every person and thing that one encounters	"We foster and maintain a positive culture of shared responsibility based on participation, behaviour, awareness and promoting courageous leadership. We allow our employees and contractors the right to stop any work if any when they see something that is not safe." "We respect human rights, the dignity of others and diversity in our workforce. Baffinland honours and respects the unique cultural values and traditions of Inuit". "Baffinland does not tolerate discrimination against individuals on the basis of race, colour, gender, religion, political opinion, nationality or social origin, or harassment of individuals freely employed".
Maintaining harmony and balance	"Baffinland employs a balance of the best scientific and traditional Inuit knowledge to safeguard the environment". "We honour our commitments by being sensitive to local needs and priorities through engagement with local communities, governments, employees and the public. We work in active partnership to create a shared understanding of relevant social, economic and environmental issues, and take their views into consideration when making decisions". "Baffinland is committed to undertaking a thorough public engagement process to create a shared understanding of relevant social, economic and environmental concerns and opportunities

Inuit Law and Norm as Described by QIA	Baffinland Sustainable Development Policy
	with its communities, regulators, stakeholder organizations and the public”.
Planning and preparing for the future	“We report, manage and learn from injuries, illnesses and high potential incidents to foster a workplace culture focused on safety and the prevention of incidents”. “We continuously seek to use energy, raw materials and natural resources more efficiently and effectively. We strive to develop more sustainable practices”. “Baffinland ensures that an effective closure strategy is in place at all stages of project development to ensure reclamation objectives are met.” “Baffinland endeavours to ensure that adequate resources are available and that systems are in place to implement risk-based management systems, including defined standards and objectives for continuous improvement”.

QIA 37 ATTACHMENT 1: TSS EXCEEDANCES

Baffinland

Supporting Documentation for Baffinland Response to Phase 2 Proposal - QIA 37 Technical Comment

Table A-1 - Summary of TSS Exceedances at Water Crossings along Tote Road (2016 - 2018; 30 mg/L Grab Sample)

Description of Metric	2016	2017	2018
Number of grab sample TSS exceedances observed downstream of water crossings.	1	27	5
Number of grab sample TSS exceedances observed upstream of water crossings.	1	15	3

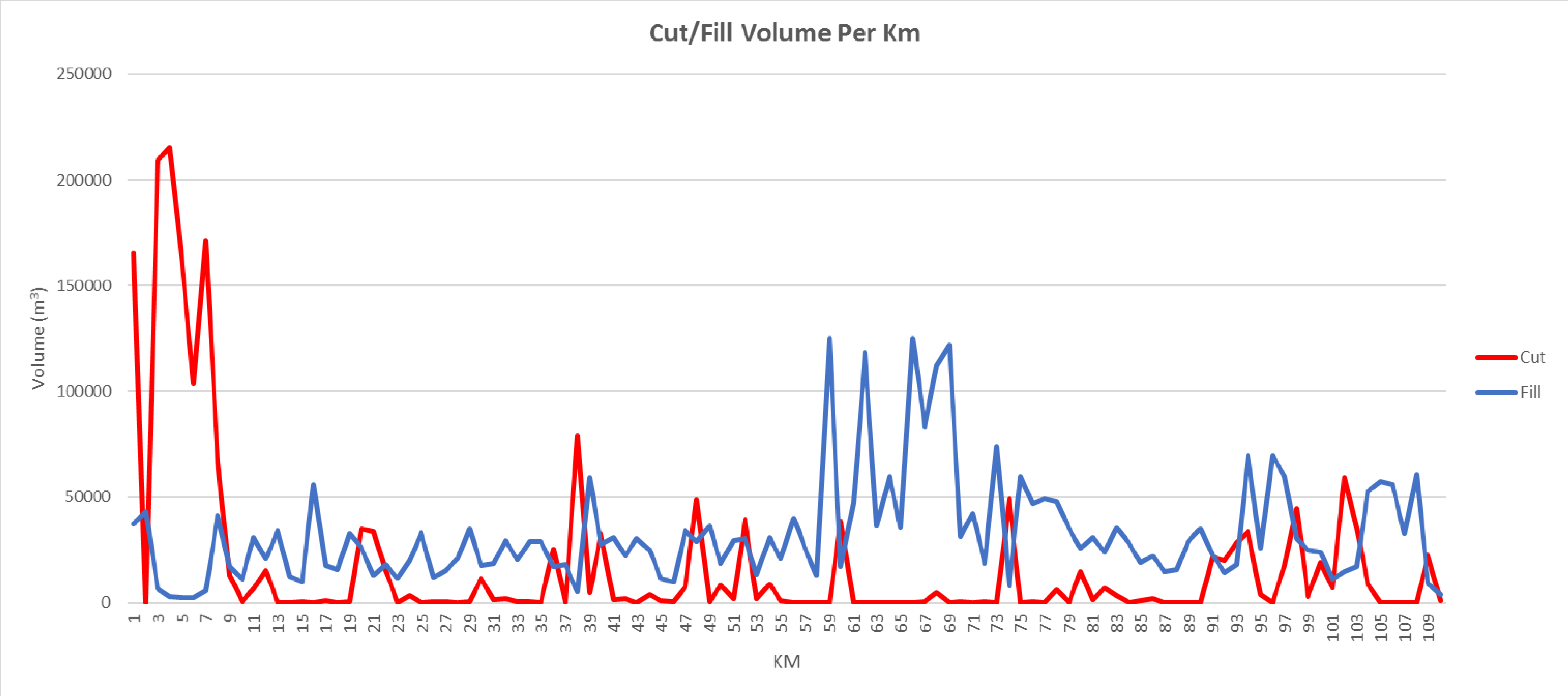
Table A-2 - Summary of TSS Exceedances at Water Crossings along Tote Road (2016 - 2018; 15 mg/L Monthly Average)

Description of Metric	2016	2017	2018
Number of monthly average TSS exceedances observed downstream of water crossings.	2	27	5
Number of monthly average TSS exceedances observed upstream of water crossings.	2	13	3

Notes:

Tables A-1 and A-2 compare the TSS results at Tote Road water crossings (2016 - 2018) with the applicable Type 'A' Water Licence criteria (2AM-MRY1325 - Amend. No. 1; Table 11)

QIA 41 ATTACHMENT 1: CUT/FILL VOLUME PER KM GRAPH


Cut/Fill Volume Per Km
250000
200000
150000
100000
50000
0
Volume (m3)
KM
Cut
Fill

QIA 59 ATTACHMENT 1: FULL RESPONSE

Comparison of Modelled and Monitored Dustfall

As discussed in detail below, a comparison was made between modelled dust deposition and measured dust deposition. In general, the comparison shows that the model is providing realistic results, but at locations that are in very close proximity to operations at the port, measured levels were influenced by dust from miscellaneous minor activities at the site (on-site vehicle traffic associated with pick-up trucks, etc.) that were not included in the modelling.

Tables 1 to 3 show a comparison of monitored and modelled dustfall at the Mine Site, Milne Port and the Northern Corridor, respectively. The monitored data available for this comparison were for a period extending from 2013 into 2018. Further details on the monitoring program and specific locations of each monitoring station can be found in Environment Annual Monitoring Reports for 2015, 2016, and 2017, prepared by EDI.

The red values in the tables below exceed either the annual average criterion (55 g/m^2/year) or the 30-day criterion (5.3 g/m^2-30d) at the specific location.

Mine Site

At the Mine site, the current operations are approximately 4.2 million tonnes per annum (Mtpa) of iron ore, transported to Milne Port by truck along the existing Tote Road for open water shipping. For the modelling, a mining production of 30 Mtpa was considered (the eventual maximum production rate, with 18 Mtpa eventually being shipped by rail to Steensby).

Table 1: Comparison of Monitored and Modelled Dustfall at the Mine Site

Station ID	Monitored			Modelled	
	Annual Average dustfall (g/m^2/day)	Annual Average dustfall (g/m^2/year)	30-Day Maximum Dustfall (g/m^2-30d)	Annual Average dustfall (g/m^2/year)	30-Day Maximum Dustfall (g/m^2-30d)
DF-M-01	0.29	106	125	25	4.0
DF-M-02	0.20	73	41	286	48.6
DF-M-03	0.16	60	24	261	32.3
DF-M-04	0.01	4	0.3	1	0.2
DF-M-05	0.01	4	0.3	0	0.1
DF-M-06	0.01	4	0.6	11	2.4
DF-M-07	0.01	5	1.4	14	1.8
DF-M-08	0.01	4	0.3	2	0.4
DF-M-09	0.02	7	2.5	2	0.3

The difference in the actual vs modelled production rate would explain why the modelled annual dustfall levels at stations DF-M-02 and DF-M-03 are much higher than the monitoring data, as actual production was 4.2 Mtpa, compared to 30 Mtpa that was modelled. Those two stations are the closest to mining and hauling operations modelled in the 30 million Mtpa scenario. Modelled annual dustfall levels at DF-M-06 and 07 are also much higher than measured values.

Station DF-M-01, which is a station close to the Tote Road, shows the highest measured annual dustfall levels. The modelled results for that station are much lower than the measured values, most likely due to the fact that no ore transport by truck was modelled within the Mine Site model domain in the 30 mtpa scenario, as that scenario represents ore transport by rail.

Monitoring sites DF-M-04, 05, 08 and 09 are all relatively remote sites, more than 3 km away from the disturbed area, and are not significantly impacted by the operations. The measured dustfall levels at these locations are relatively low. The model appears to underestimate at these locations compared to the measurements, but the reality is that the measurements are overestimating the dustfall. At these locations the measured values were below the method detection limit but were reported as equal to the method detection limit. Thus, the model is likely providing realistic values at these locations.

Milne Port

At Milne Port the current operations involve handling of 4.2 Mtpa of iron ore. For the more recent modelling scenario, we considered 12 Mtpa of iron ore transiting at this facility. Results from a previous modelling scenario of 3.5 Mtpa were also included in the table for comparison.

Table 2: Comparison of monitored and modelled dustfall at Milne Port

Station ID	Monitored			Modelled at 12 Mtpa		Modelled at 3.5 Mtpa
	Annual Average dustfall ($g/m^2/day$)	Annual Average dustfall ($g/m^2/year$)	30-Day Maximum Dustfall (g/m^2-30d)	Annual Average dustfall ($g/m^2/year$)	30-Day Maximum Dustfall (g/m^2-30d)	Annual Average Dustfall ($g/m^2/year$)
DF-P-01	0.43	155	84.3	273	35.8	25
DF-P-02	0.34	125	18.8	24	3.1	28
DF-P-03	0.01	4	0.4	0	0.0	0
DF-P-04	0.04	13	5.4	1	0.5	2
DF-P-05	0.27	100	32.1	16	2.0	21
DF-P-06	0.03	11	5.5	4	0.6	2
DF-P-07	0.06	23	22.2	206	32.8	92

It should be noted that the 3.5 and especially the 12 Mtpa modelling scenarios involved different layouts of dust sources compared to what is actually in place on site at present. Therefore, discrepancies between monitored and modelled dustfall are expected. For example, the most significant difference is with the stockpiles. The 3.5 Mtpa modelling scenario appears to be relatively similar to the layout for current operations, with relatively short stockpiles oriented north-south, whereas the design for the modelled 12 Mtpa scenario has two much longer stockpiles that are oriented south-west to north-east, and they are located further to the east.

DF-P-01 and DF-P-07 are located on the southwest side of the current port operations. In the 12 Mtpa model scenario, they are located close to the proposed Fine Ore Road, which is the likely explanation for why the model results are high at both these locations in that modelled scenario. In the 3.5 Mtpa modelling scenario, DF-P-07 is closer to modelled stockpile operations than DF-P-01, which may explain why the modelled annual dustfall at DF-P-07 is higher than that at DF-P-01 in that scenario. The monitoring data suggests that, in reality, the situation may be the other way around, i.e., DF-P-01 may actually be closer to stockpile operations than DF-P-07, as the measured values at DF-P-01 are significantly higher than those at DF-P-07, and comparable to the modelled levels at DF-P-07. Thus the actual layout of the operations may differ somewhat from what was modelled in the 3.5 Mtpa scenario.

At DF-P-02 and DF-P-05, measured annual dustfall levels are much higher than the modelled levels. This suggests that these monitoring sites are adjacent to a dust source that was not accounted for in the model scenarios (most likely miscellaneous on-site vehicle traffic other than the Tote Road ore transport trucks).

Locations DF-P-03, 04 and 06 are relatively distant from the port operations and the measured levels at these sites are relatively low. The model underestimates at these locations. At DF-P-03, which is on the order of 4 km from the port operations, the problem is actually measurement overestimation rather than model underestimation. It is due to the measured values being below the method detection limit but reported as equal to the detection limit. At DF-P-04 and 06, the measurement detection limit is partly a factor, but other factors may come into play as well, such as background dustfall and dust from miscellaneous on-site traffic not included in the model.

Northern Transport Corridor

For the Northern Transportation Corridor, an Early Revenue Phase scenario was modelled involving 6 Mtpa of ore transported by truck along the Tote Road, in combination with construction of a railroad parallel to the Tote Road.

For the Northern Transport Corridor scenario, current operations include hauling of 4.2 Mtpa of iron ore on the Tote Road. For the modelled scenario we considered 6 Mpta of iron ore being hauled on the Tote Road.

None of the actual monitoring stations reside within the modelled domain. Therefore, it was not possible to do a direct comparison of observed and model results at specific monitoring locations. However, a general comparison can be made of averaged dustfall at various distances from the Tote road. Table 3 shows average monitored and modelled dustfall as a function of distances.

Table 3: Average monitored and modelled dustfall at various distances from the Tote Road

Approximative Distance from the road (m)	Monitored		Modelled	
	Annual dustfall ($g/m^2/year$)	30-Day Maximum Dustfall ($g/m^2/30d$)	Annual Dustfall ($g/m^2/year$)	30-day Maximum Dustfall ($g/m^2/30d$)
5000	7	7	0.4	0.1
1000	7	3	4	1
100	48	31	93	12
30	229	175	189	23

Overall, there is relatively good agreement between monitoring data and modelling results. The model slightly underpredicts annual dustfall at 30 m from the Tote road but overestimates it at 100 m. At distances of 1000 and 5000m, the measurement values are affected by the method detection limit and are also based on more limited sampling than at the shorter distances. Therefore, the measurements at these locations are overestimating the annual dustfall. The modelled annual dustfall levels are considered to be realistic at these distances.

Conclusion

Overall, based on the information and analysis presented in this response, no significant discrepancy exists between modelled and observed annual average results. It was found that the comparison of model results to monitoring data varies considerably from one monitoring site to another, but there is no clear trend showing that the model either under-predicts or overpredicts dustfall amounts. Therefore, no calibration of the model based on these monitoring results is recommended.

QIA 68 ATTACHMENT 1: ICE INTERACTION DESIGN CRITERIA FOR DETAILED DESIGN - ORE DOCK - BAFFINLAND IRON MINES: MARY RIVER EXPANSION STAGE 3, DECEMBER 13, 2018

HATCH Vendor/Contractor Document Review			
Doc Number	E353004-CG001-100-210-0002	SUB	03
Date Received			
Review Grade		Next Submittal Status	
☐ C1 - Proceed to next submission & status ☐ C2 - Proceed with exceptions as noted to next submission & status		☐Certified ☐As-Built	
☐ C3 - Do not proceed. Revise as noted & resubmit		☐Internal Review	
Next Submittal Date:		☐Certified/☐As-Built	
☐ C4 - No further submission required - Complete (select status below) ☐Certified Final ☐ Final ☐ Cancelled ☐ Superseded			
Package Engineer: Name, signature and date			
Reviewed only for general conformity with the specifications. Acceptance by the engineer does not warrant or represent that the information contained on this drawing/document is either accurate or complete. The sole responsibilty for correct design, details & dimensions shall remain with the party submitting the drawing/document.			

BESIX VanPile Joint Venture

BIM | Expansion Stage 3 | Mary River Project

Ore Dock No.2 | CG001

DESIGN NOTE

Ice Interaction Design Criteria

Baffinland Iron Mines Corporation Hatch Ltd Contract No : H353004-CG001		Client Document No.: E353004-CG001-225-230-0002			Rev No: 00
BESIX-VanPile JV		BV JV Document/Rev No. : CG1-BD-DN-108011_Rev 00			
Date	Description	Prep'd	Chck'd	Appr'd	Status Code
14-12-2018	First Issue	KEN	MZE	HVM	For Approval

Previous Rev	Date	Description	Prep'd	Chck'd	App'd	Status
00	14/12/2018	1st Issue	KEN	MZE	HVM	For Approval

Ice Interaction Design Criteria for Detailed Design

Ore Dock – Baffinland Iron Mines: Mary River Expansion Stage 3

Submitted to BESIX

Avenue des Communautés, 100 – 1200 Brussels – Belgium

Final V1

December 13, 2018

Authors

Norman Allyn

CMO, Vancouver, Canada

Ken Croasdale

KRCA, Calgary, Canada

Executive Summary

The purpose of this document is to define the ice design criteria and assumptions to be used by BESIX VANPILE JV for the detailed civil design for the new dock at Mary River, Baffin Island. A prior document provided ice design criteria for preliminary design.

The Project is located at Mary River and Milne Inlet on Baffin Island in the Territory of Nunavut, Canada. It is located on the northern half of Baffin Island, approximately 160 km south of Mittimatalik (Pond Inlet) and 1000 km northwest of Iqaluit, the capital of Nunavut Territory. The latitude is 71 degrees, placing it well into the Arctic region.

From about mid-November to the end of January the sun does not rise above the horizon; whereas during the months of May, June and July the sun never sets. In winter, the mean maximum temperature is -28c and the mean minimum is -35C. In summer the mean maximum is about +10C and mean minimum about +3C.

At the high latitudes of this location, ice is present for about 9 months of the year with annual ice growth up to about 2m. Additional thickening can occur due to rafting, ridging and over-ice flooding due to tidal effects which can also create ice bustles on dock structures. Multi-year and glacial ice may be driven into the northern end of Milne Inlet but is not likely to penetrate all the way to the dock.

Climate change may reduce the length of the ice season and annual ice growth thickness. However, because of uncertainties in climate change, engineering practice requires designs to be based on the severest conditions. For ice this will be historical data, but for waves the potential for more open-water may need to be assessed. Water level rise may increase the risk of over-topping by ice and waves.

The current accepted code for offshore structures in ice environments is ISO 19906. The current issue is ISO 19906 (2010); it is in the process of being updated and has been issued as a FDIS (final draft international standard). The development of the ice criteria for this project is based on this version.

The Code allows structures to be classified into Safety Classes according to consequences of failure. The selection of Safety Class leads to the selection of load factors and return periods.

Because the structure is not manned (during the period that ice loads occur), it can be at the lowest of three safety classes which is L3. The highest, L1, is for a manned offshore platform with high consequences of failure – both in term of loss of life and pollution. L2 is one where people can be evacuated and with lower consequences of failure. As would be expected, the L1 category requires the highest load factor of 1.35 to be applied to the load at the 10^{-2} annual probability level (100 year load). L1 structures are also required to consider rarer loads (in the 10^{-3} to 10^{-4} probability range).

For L2 structures, the required load factor is 1.1 on the 10^{-2} load; and rare loads are to be considered at the 10^{-3} level.

For L3 Structures, loads beyond the 100 year return period are not required to be considered, and load factors are not specified, but are left to the owner to decide, presumably based on potential economic loss. We suggest a load factor of 1.1 be the minimum – but also consistent with other load factors which may be relevant to other types of loading on the structure.

The ice loads and other ice interaction issues have been calculated based on three distinct periods of the winter which represent different ice conditions. The first period is from the time of ice formation to the end of freeze-up when the ice stabilizes and becomes landfast. This is defined as **Scenario 1** during which the ice is mobile and cold and is estimated to grow to 0.6m thick before it stabilizes into landfast ice

The global loads calculated as a function of loading width are shown in Table E.1

Note the widths chosen for calculation are based on the current design of the “combi-wall”. spacing of the large cylindrical piles in the combi-wall system (3.369m and multiples thereof). The loads are based on the methodology in ISO 19906 and details of the calculation of these loads are given in the report.

Table E.1: “100 year” global ice loads calculated for Scenario 1 (Freeze-up)

No of piles	Loading width (m)	Ice Thickness (m)	Line load (MN/m)	Maximum per pile (MN)	Potential global load (MN)	Max. wind & current drag 2km fetch (MN)
sheet pile	1.540	0.6	1.07	1.65	1.65	6
1 pile tributary	3.369	0.6	0.94	3.18	3.18	6
2	6.738	0.6	0.85	2.85	5.69	6
3	10.107	0.6	0.79	2.67	8.00	6
4	13.476	0.6	0.76	2.55	10.19	6
5	16.845	0.6	0.73	2.46	12.29	6
6	20.214	0.6	0.71	2.39	14.33	6
7	23.583	0.6	0.69	2.33	16.31	6
8	26.952	0.6	0.68	2.28	18.24	6
9	30.321	0.6	0.66	2.24	20.14	6
10	33.690	0.6	0.65	2.20	22.01	6
11	37.059	0.6	0.64	2.17	23.84	6
56	190.494	0.6	0.50	1.68	94.31	6

It can be seen that the line load diminishes with loading width and that the potential global load could be as high as 94MN if the ice crushes over the full structure width. However, because this dock is at the end of a narrow inlet the driving forces on the ice are limited. In fact even for a typical maximum sustained winds and currents the “static” driving force available is derived in this report as being about 6MN. However, this is not the design ice load because of another load case involving freely moving ice sheets that can be accelerated by the winds and currents. If a large ice sheet collides with the dock at its maximum speed then the kinetic energy of the floe can impose larger forces than the static wind and current drag. These impact cases have also been analyzed for this initial freeze-up period. Typical results are given in Table E2.

Table E2: loads due to impact of feely moving floes just prior to ice becoming landfast

ice thickness 0.6m		1500m floe	
wind speed (m/s)	20	15	10
floe speed (m/s)	0.45	0.337	0.225
force (MN)	25.4	22.6	18.3
loading width (m)	40	34.7	27.1
line load (MN/m)	0.64	0.65	0.68

The results in Table E2 indicate that the maximum global load is 25.4MN with a corresponding line load of 0.64MN/m.

Scenario 2 is the period when the ice is landfast and will be essentially stationary (small slow movements only). The ice is cold and attains it maximum thickness of about 2m. However, because of the relief of ice pressure due to creep, the loads will be lower than in other periods. The “100 year” load for this period is estimated at 0.3MN/m for all loading widths.

Scenario 3 is the break up period when the ice has deteriorated in strength prior to it becoming mobile again. It is also calculated to be thinner than the winter maximum and a value of 1.4m is used. The “100 year” line loads for this period are shown in Table E.3. (Line loads for Scenario 1 are also shown for comparison and it can be seen that they govern).

Table E.3: “100 year” global ice loads for Scenario 3 (Break up).

	SCENARIO 3					
	Break up	Warm weak ice	Strength Factor	0.32		
			Cr	0.896		
			n	-0.3		
			m	-0.16		0.6m
	Crushing	ISO 19906			With exposure	For comparison
					0.61	With exposure
Piles	Loading		Arctic Warm	Arctic Warm	Effect of exposure	Scenario 1
	width (m)	Thickness (m)	p (MPa)	Line Load (MN/m)	Load (MN/m)	Load (MN/m)
sheet pile	1.54	1.4	0.80	1.12	0.68	1.07
1 circular pile	1.83	1.4	0.78	1.09	0.66	1.04
1 pile tributory	3.369	1.4	0.70	0.99	0.60	0.94
2	6.738	1.4	0.63	0.88	0.54	0.85
3	10.107	1.4	0.59	0.83	0.50	0.79
4	13.476	1.4	0.56	0.79	0.48	0.76
5	16.845	1.4	0.54	0.76	0.46	0.73
6	20.214	1.4	0.53	0.74	0.45	0.71
8	26.952	1.4	0.50	0.71	0.43	0.68
10	33.690	1.4	0.49	0.68	0.42	0.65
12	40.428	1.4	0.47	0.66	0.40	0.63
14	47.166	1.4	0.46	0.65	0.39	0.62
16	53.904	1.4	0.45	0.63	0.39	0.61
21	70.749	1.4	0.43	0.61	0.37	0.58
56	190.494	1.4	0.37	0.52	0.32	0.50

In many ice interaction scenarios, local ice pressures can be higher than global ice pressures averaged over bigger widths or areas. The most severe local ice pressures in ISO have been developed based on high velocity ship impacts and on measurements on thick ice in confined situations; they are not considered suitable for this structure at this location.

In this case the ice is thin at freeze-up (0.6m) and at break up, although thicker (1.4m), it is very deteriorated. The method for thin ice in ISO has been investigated and it gives a line load very close to the highest global line load of 1.07MN/m. For this reason, a separate set of criteria for local ice loads is not considered necessary. It is also noted that in the experience of the authors and advisors, local damage from ice to sheet piles at existing Arctic docks (such as Nanisvik) has not been experienced. The same is true for many sheet pile docks in the North Caspian Sea.

Ice loads have also been calculated for the exposed rock slopes which form the outside causeways of the dock layout. The initial interaction of ice with rock slopes is by bending failure of the ice at lower loads than ice crushing against the vertical quay face. However, as the broken ice continues to ride up and build up on the slopes these forces have to be included. The loads in Table E4 account for that process.

Table E.4: Line loads on the exposed rock slopes (30m width)

Scenario	Bending failure and ride up	
	Horizontal MN/m	Vertical MN/m
Scenario 1: Freeze-up (0.6m strong ice)	0.16	0.09
Scenario 2: Break-up (1.4m weak ice)	0.30	0.07

Ice encroachment is the term for ice climbing or spilling onto the working surface of a platform. Experience in other areas has been reviewed. All ice encroachments in other areas have been in shallow water in which ice rubble can ground on the sea floor before building high enough to encroach onto the working surfaces. The encroachment events in other areas have also been at locations where the driving forces on the ice from winds and currents can build up over 100s of km and larger ice sheets than those possible in this enclosed inlet have been involved. These differences have been accounted for and are discussed in the report.

At the dock face the water depth is 23m and grounding of ice rubble would require significant ice movement and driving forces or momentum to push the ice past the dock any significant distance. As well the causeways and shoreline behind the dock would not allow more than about 100 to 200m of ice motion. It is possible that without grounding a floating ridge could build in front of the dock with a sufficiently high sail adjacent to the dock that some ice may spill over onto the dock surface. This is very unlikely because the available driving forces are limited. In fact, the analysis comparing ridge building forces with driving forces suggest that a ridge can only be formed from ice about 24cm thick (or thinner). (Thicker ice requires forces greater than the driving forces available).

Ridge dimensions for ridges formed from 24cm ice in terms of keel depths and a range of sail heights are shown in Table E5. These are for a range of very conservative ice movements against the dock (noting the shoreline behind will stop the ice).

Table E5: Ridge keel depths and sail heights vs ice thickness

	Keel depths and sail heights based on ice movement					
				θ =	30	degrees
		f=	0.5	tanθ =	0.577	
	m	m		m	m	m
	Ice movt.	ice thickness	porosity	Ridge depth	Sail height 1	Sail height 2
	L	**hi**	γ	**Hk**	**Hs1**	**Hs2**
Freeze up	200	0.24	0.25	6.50	0.84	1.46
Freeze up	300	0.24	0.25	7.96	1.03	1.79
Freeze up	200	0.6	0.25	10.27	1.33	2.31
Freeze up	300	0.6	0.25	12.58	1.62	2.83

It can be seen that the maximum sail height is 1.8m and this is for 300m of movement. Even at high water, the dock freeboard is 2.1m, so the ridge sail will not get as high as the dock surface. The 0.6m thickness is included for reference even though there is not enough driving force to create a ridge from ice this thick. This shows the possibility of sail heights above the dock face at high water for the extreme sail, but the average sail would still be below the dock surface. Carrying the "what if" case one step further, the potential encroachment with a ridge sail adjacent to the dock is shown in Table E6. It is about 1m. It is emphasized that this is not the prediction, but to answer the question of "what if".

Table E6: Potential encroachment for an extreme ridge sail at the dock face

Sail height at edge of dock	Freeboard	Position of apex	Encroachment distance
hp	y	wt	wp
(m)	(m)	(m)	(m)
2.83	2.10	0.00	1.04
2.83	2.10	1.00	0.04

Ice encroachment is more likely and may occur on the exposed causeways or rock berms facing offshore. But this process is also subject to the constraints relating to driving forces and limited kinetic energy; and whether large floes can push past the dock face. The analysis in this report indicates that the smaller floes that can miss the dock face can have sufficient energy to fail and ride up about 8m. This situation is shown in Figure E1 when the water level is at MWL. If the incident occurred at HAT water level then some encroachment can be expected as depicted in Figure E2.

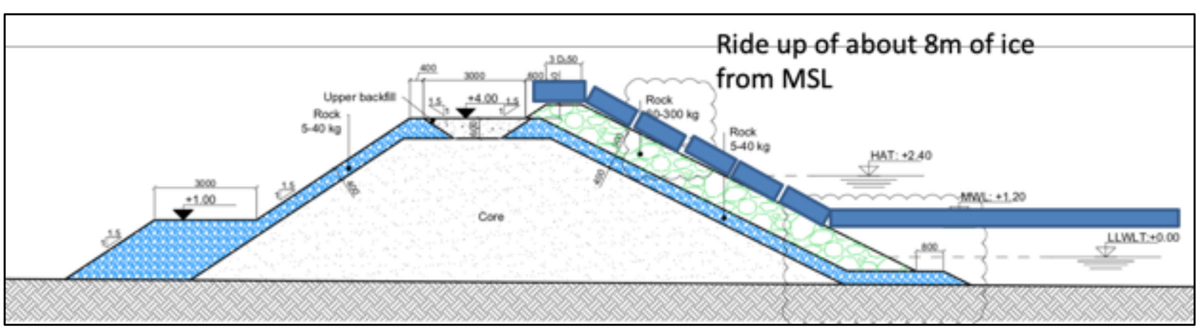


Figure E1: Maximum calculated ice ride up based on kinetic energy of interacting floe (over a 30m width) referenced to MSL

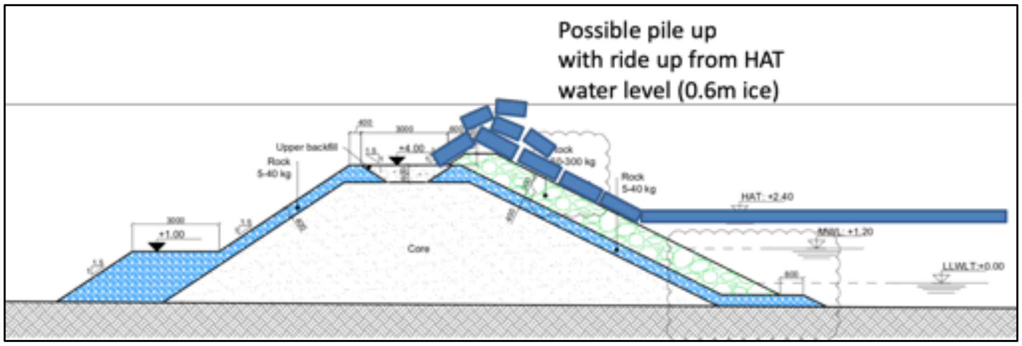


Figure E2: Encroachment possible at high water level on exposed causeway slopes

The consequences of this kind of encroachment are not considered high as the dock is not being used in the winter and sensitive equipment does not appear to be located where encroachment may occur. Note that the mooring piles could be subject to encroachment of rubble ice, but the loads will be due to loose ice blocks and not decisive. There is a slight risk of encroachment in a similar type scenario at break up, but again the floes would have to bypass the main dock face and act only on the causeways. It is mentioned because there may be operations underway at break up with equipment and personnel on the causeways. If ice floes are still in the area, it will be prudent to have some kind of alert system and operational procedures in place.

Final remarks

The suggestions for safety class and load factors should be reviewed carefully and discussed with the Owner's engineer (or regulator). The Owner may decide to use higher load factors to minimize the risk of damage (and subsequent maintenance).

Several assumptions have been made about the ice behaviour especially in terms of thickness and mobility (and tidal actions). These assumptions are considered sound, based on available data and the experience of the ice team. The available data for this location are sparse. Therefore, it would be prudent to take advantage of the coming winter to conduct some "ground-truthing" and collection of additional targeted satellite imagery. Although this report does not address winter construction issues, there is also much value in using the coming winter to assess ice access issues and safety relating to construction plans when the ice is present (and access onto the ice may be part of those plans). The specifics of these observations will become clearer once the proposed construction plans have been firmed up.

Contents

1.0 Introduction

1.1 Background

BESIX VANPILE JV is tendering for construction of new ore dock in Baffinland Iron Mines (BIM). BIM currently operates the Mary River iron mine in Nunavut, Canada. Ore is currently mined, crushed, screened and then trucked to Milne Port, where it is stockpiled until it can be shipped off site during the short arctic summer when the sea is free of ice. BIM plans to increase the production rate of the mine.

The purpose of this document is to define the ice design criteria and assumptions to be used for the preliminary civil design for the new dock. An earlier study on ice engineering for the new dock was commissioned by Hatch and conducted in 2014 by BMT Fleet Technology of Ottawa (BMT, 2014). That study is referred to in this document and parts of the prior developed criteria are referenced when appropriate. The proposed dock design is different from that assumed in the prior ice study and this changes how some of the criteria are developed. Furthermore, new insights into some of the ice interaction methods and ice data have resulted in changes in results from the 2014 work and in general the design ice loads are lower.

1.2 Location

The Project is located at Mary River and Milne Inlet on Baffin Island in the Territory of Nunavut, Canada. It is located on the northern half of Baffin Island, approximately 160 km south of Mittimatalik (Pond Inlet) and 1000 km northwest of Iqaluit, the capital of Nunavut Territory. (See Figure 1.1).

Figure 1.1: The Loading dock location

The latitude is 71 degrees placing it well into the Arctic region.

From about mid-November to the end of January the sun does not rise above the horizon; whereas during the months of May, June and July the sun never sets. In winter, the mean maximum temperature is -28c and the mean minimum is -35C. In summer the mean maximum is about +10C and mean minimum about +3C.

1.3 Overview of ice conditions

At the high latitudes of this location, ice is present for about 9 months of the year with annual ice growth up to about 2m. Additional thickening can occur due to rafting, ridging and over-ice flooding due to tidal effects which can also create ice bustles on dock structures. Multi-year and glacial ice may be driven into Pond Inlet and Milne Inlet but is not likely to penetrate all the way to the dock.

Climate change may reduce the length of the ice season and annual ice growth thickness. However, because of uncertainties in climate change, engineering practice requires designs to be based on the severest conditions. For ice this will be historical data, but for waves the potential for more open water may need to be assessed. Water level rise may increase the risk of over-topping by ice and waves.

As will be discussed in Section 3, the ice conditions through the winter can be divided into three periods. These are: (a) Freeze-up; (b) Mid-Winter; and (c) Spring Melt and Breakup. Ice interaction will be different during each of these periods.

The key reference for ice conditions and ice parameters used in the ice loads and ice encroachment assessments in this document is *"Milne Inlet Ice conditions: Volume 1: as relating to detailed design" by KRCA/CMO, November 30, 2018.*

1.4 Dock location and layout

The location of the dock at the head of the inlet is shown in Figure 1.2. Plan views, wall details and typical cross section at the dock face are shown in Figures 1.3 to 1.7. The current design has a combi-wall dock 189m wide with a water depth at the dock face of about 23m. The dock surface has a freeboard of 4.5m above MSL. The tidal range is 2.4m. The cross section of the causeways are given later in the context of ice interaction with the rock berms

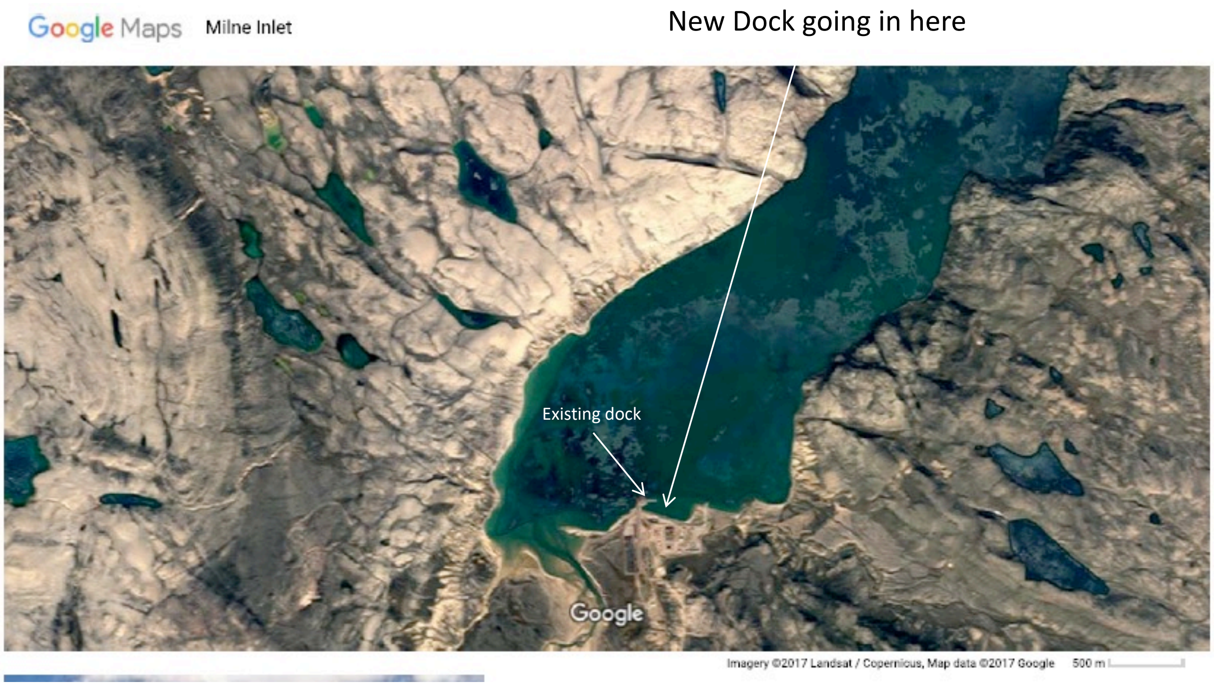


Figure 1.2: Dock location at head of inlet and to the East of existing dock

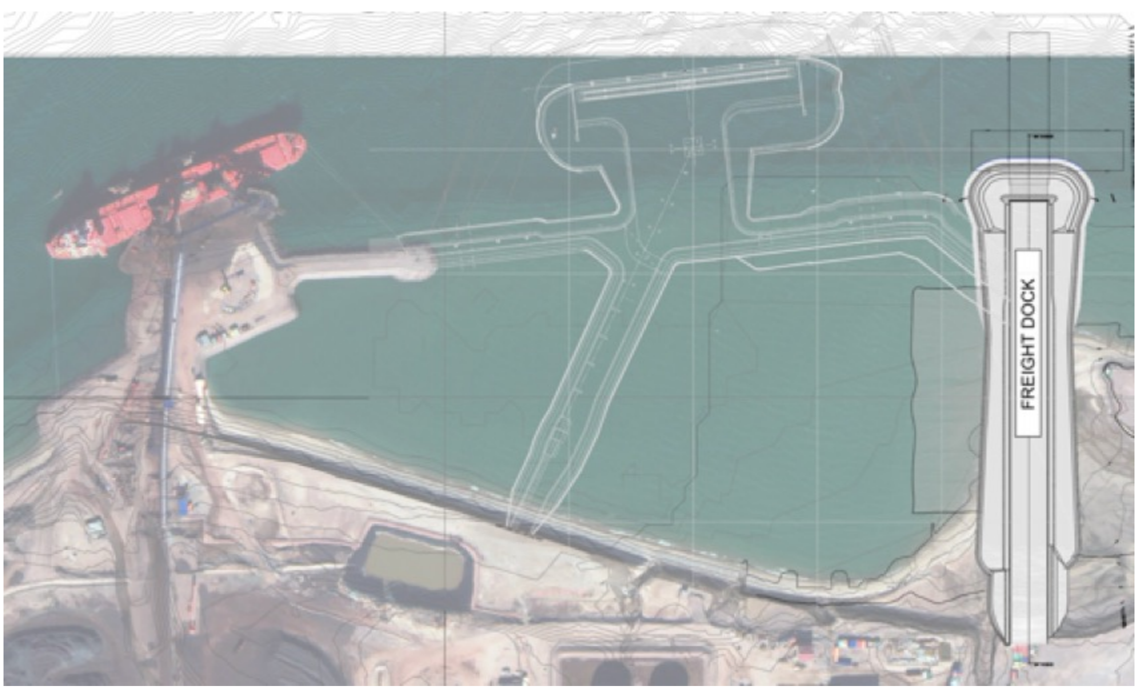


Figure 1.3: Plan view of new dock and causeways showing Dock 1 and new freight dock (to be built first)

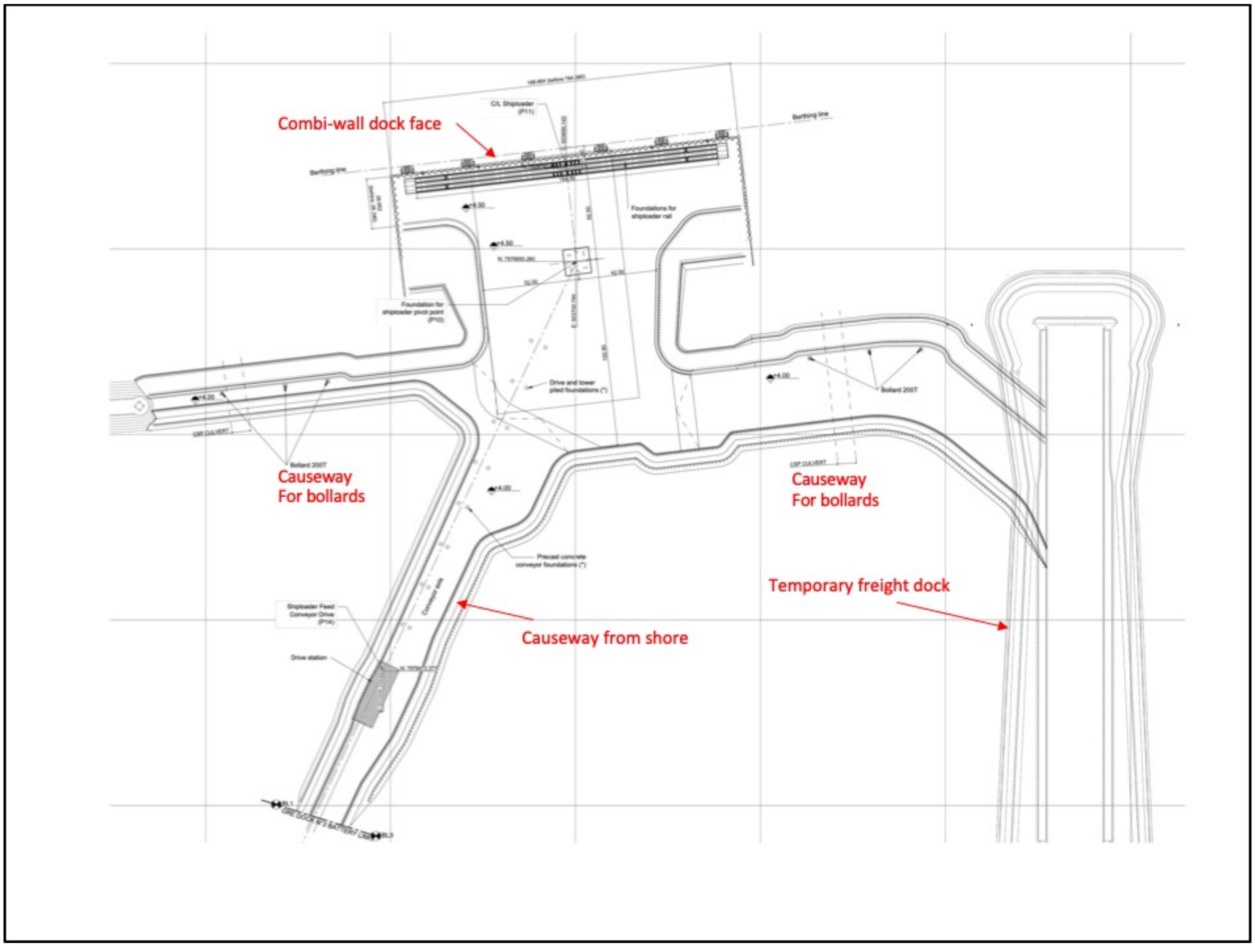


Figure 1.4: Drawing – plan view (current design total dock width is 188.7m (end piles CL – CL)

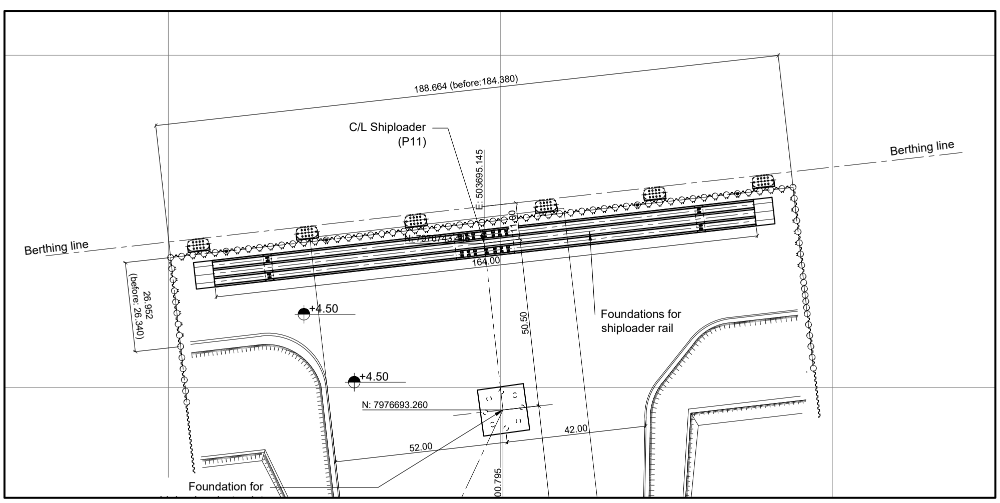


Figure 1.5: Closer view of dock

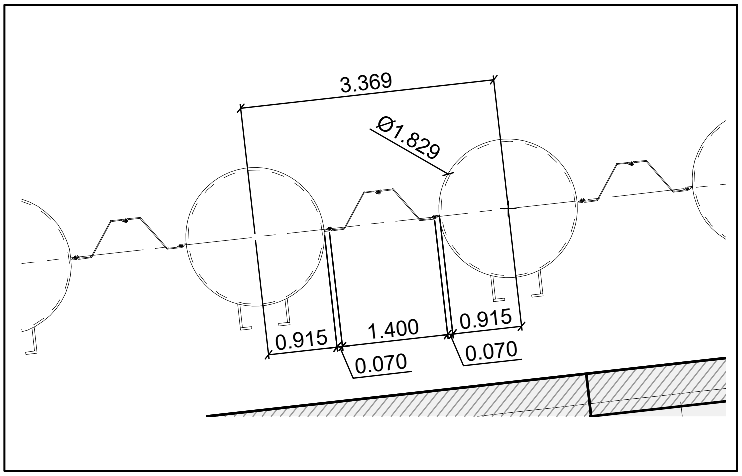


Figure 1.6. Details of combi wall

Revised dimensions (November 27, 2018)

Pile centre to centre 3.369m. Circular pile diameter 1.829m
Sheet pile width 1.40m + 2x0.07 = 1.54 (between circular piles)
(So basically : clutches = 2 x 0.070m)

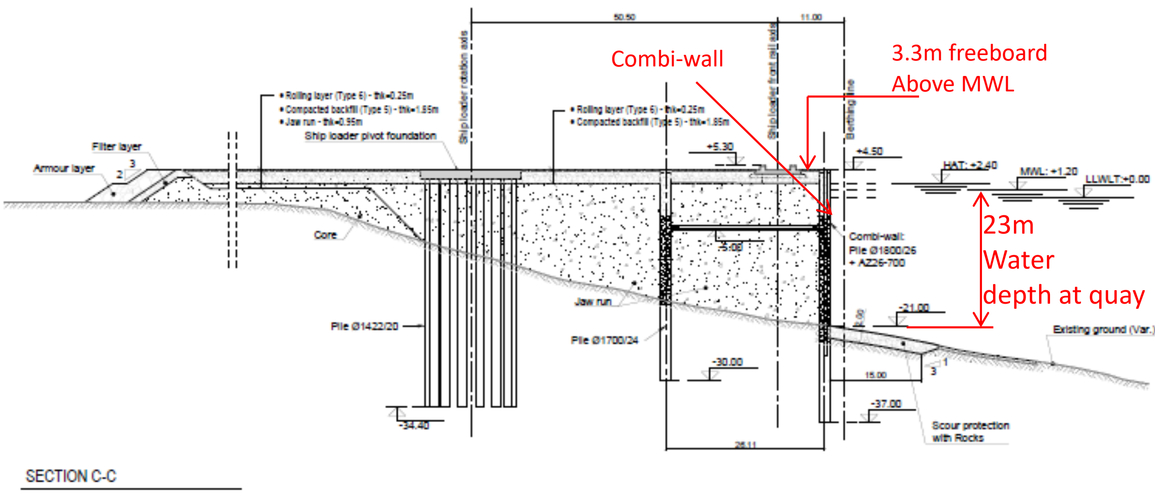


Figure 1.7: Section at quay

2.0 Ice Interaction Issues and design Criteria

2.1Overview

Structures in ice environments are subject to ice loads and these need to be determined at the appropriate levels of probability for safe design. Both global and local ice loads need to be derived. Global loads are those which act over the total structure (or major portions of it) and which can affect overall global stability and/or major structural deformations and failure. Local loads also have to be evaluated because ice pressures are higher over smaller widths and areas than the global average. This phenomenon can lead to local damage if not recognized and so local loads need to be specified and designed against.

Another ice interaction issue for low freeboard structures is that of ice encroachment. When ice moves against low freeboard structures there have been incidents of ice encroachment onto the working surface. The process has been observed and studied in the Arctic, the Great Lakes, Russia and the Caspian Sea. Damage to shoreline structures has been reported and is part of the oral tradition of some indigenous peoples.

The physics of the process is one of ice movement against a wide face (or shoreline) which leads to a build-up of ice rubble. The rubble pile may become grounded and builds higher. This rubble can spill onto the working surface of a structure and in the worst-case can provide a ramp for the ice to continue to ride up. A more detailed discussion of ice encroachment with estimated values is given in Section 4.

Year-round shipping operations at docks and harbours in ice areas can lead to additional ice issues and potential ice loading scenarios. The biggest ice operational concern is the build-up of brash ice due to repeated ice breaking and ship transits. Special ice management methods are often devised to reduce brash ice and even the use of heat may be required. Air bubblers have been used in fresh and brackish water such as the Baltic to reduce the ice cover and brash ice. This relies on stratification due to fresh water at 4C being denser than water at 0C. However, this may not be possible here because of the sea water environment in which stratification does not occur. It is understood that these dock facilities are not for year-round use, therefore the build-up of brash ice and possible design features to accommodate it will not be addressed.

There is the possibility however of ice breaking ship operations close to the dock structures if remnant ice persists and there is an incentive to open the port as early as possible. Scenarios for how ice can be pushed against the dock structures can be developed and ice loads specified. In the limit, the ice loads will be limited by ship momentum, so vessel sizes and speeds will also be an input. This topic has not been addressed in this report as it is assumed that ship operations will be specified to limit any dock damage.

Ice Issues during construction include forecasting the open water period available for marine transportation and construction activities. Historical records and local knowledge can be used. The opposite aspect will define the period available for working off the ice including ice thickness for safe loads and when that thickness would be expected. In the shoulder periods there may be a need for tactical ice forecasting with some possible ice management to extend any floating construction operations.

Again, this report addresses ice design criteria, ice issues relating to construction are reviewed separately.

Precedents and knowledge gained: In any project, it is important not to re-invent the wheel and to benefit from prior designs and their operational performance. The ice specialists on this team have good awareness of how similar facilities and structures in ice covered waters have been designed and have performed. Important precedents and similar structures on which experience has been developed are:

Ore loading dock at Nanisivik, Strathcona Sound, NU: This dock's performance in ice has been studied and reported widely in the literature. The ice specialists on this team have been involved in this, including visits to the dock, and the measurement of winter ice pressures on it. The ice specialists on this team have used the experience from it as an aid for evaluating ice issues for other arctic docks in ice.

Ore loading dock at Polaris Mines, Little Cornwallis Island, NWT: The ice team for this project has done ice engineering for it. The work included field observations at the site in the design stage, as well as defining the ice conditions from ice data and remotely-sensed images. Input was provided at the design stage that included selecting the dock site from various alternatives; and specifying the ice loads for it.

Ore loading dock at Red Dog Mines, Kotzebue, Alaska: The ice specialists on this team include the ice engineers for the dock. They made field observations at the site in the design stage, as well as defining the ice conditions from ice data and remotely-sensed images. Their input at the design stage included selecting the dock configuration from various alternatives; and specifying the ice loads for it.

Ore loading dock at Mary River Mines, Baffin Island, NU: This is the existing dock at the site (Dock 1). Experience with this dock since its construction should be examined to assess any ice damage and especially in the context of rock armour. This is vital site-specific experience which can be examined to help in design criteria for the new dock.

Low freeboard rock islands and causeways and gravity founded drilling barges in the Caspian Sea: (for ice rubble and ice encroachment design). The ice specialist's team includes those who spent over a decade addressing ice issues for the Kashagan Development.

Yamal LNG Harbor Russia: Developed the ice design criteria for the harbor protection barriers which extended about 1.5km offshore as well as the various pile supported loading facilities and berthing docks inside the harbour. The development of ice pressure reductions during break up was first incorporated on this project by the ice specialists

2.2 Codes and Standards

It is usual and often mandatory that marine facilities are designed to comply with relevant codes or standards. For a dock structure at an Arctic location in Canada there are a range of possible standards (Frederking, 2012). However, the two which would be most relevant (and with which we are familiar) are CSA 471- 04 and ISO 19906 (2010). Both relate to Arctic offshore structures with a focus on oil and gas platforms. Since adoption of ISO 19906 in Canada the Canadian Standards Association has withdrawn CSA 471-04 as a current standard. ISO 19906 is being updated and it at the DIS (Draft International Standard) stage. As such it can be utilized, and this is the case in this work. All references to ISO 19906 relate to the updated version ISO 19906 DIS (2018).

2.3 Safety Classes

ISO 19906 allows three classes as shown in Table 2.1. According to ISO this facility should be L3. This has implications for target reliability load factors which are reviewed next.

Table 2.1: ISO 19906 Safety Classes

Table 7-1 — Determination of exposure level

Life-safety category		Consequence category		
		C1 High consequence	C2 Medium consequence	C3 Low consequence
S1	Manned non-evacuated	L1	L1	L1
S2	Manned evacuated	L1	L2	L2
S3	Unmanned	L1	L2	L3

2.4 Event probability levels and partial factors

In ISO 19906 a discussion is provided of reliability targets as shown in Table 2.2 (Table A.7.1 in ISO)

Table 2.2: ISO 19906 reliability targets

Table A.7-1 — Reliability targets for each limit state action combination

Exposure level	Reliability target
L1	$1 - 10^{-5}$
L2	$1 - 10^{-4}$
L3	$1 - 10^{-3}$

It can be seen that L3 allows an order of magnitude lower reliability than L2. However ISO 19906 also states:

> For the lower exposure levels L2 and L3, the annual reliability target is reduced to compensate for the reduced exposure of personnel and the environment. For exposure level L2, it is reasoned that such exposure can be roughly 10 % of that for L1, on the basis that a precautionary evacuation and shutdown is 90 % likely to occur without casualties or damage, so that the annual reliability target can be reduced to $1 - 10^{-4}$. Similarly for L3, a further tenfold increase in exposure is considered appropriate, giving an annual reliability target of $1 - 10^{-3}$. The value for ISO 19906 "exposure level L3" is consistent with the "annual reliability target level" of $1 - 10^{-3}$ for CAN/CSA S471 "Safety Class 2".
>
> In view of the track record of successful application of these reliability targets for actual designs, an attempt at greater accuracy has been avoided, since an overly precise definition involving smaller subdivisions of orders of magnitude of the probabilities is not considered justifiable. The resulting calibrated partial action factors are consistent with other documents for offshore structures.

The ISO 19906 code gives load factors as shown in Table 2.3 (Table 7.3 in ISO 19906)

Table 2.3: ISO 19906 load factors for L1 and L2 (Note that an EL Action has a 10^{-2} annual probability).

Table 7-3 — ULS and ALS partial action factors and action combinations

Action combination		Limit state partial action factors						
		Permanent action G		Variable action Q		Environmental action E		Accident-al action A
		Gravity and hydrostatic (Dead) G_1	Defor-mation G_2	Long duration Q_1	Short duration Q_2	EL	AL	
Ultimate limit states								
1	Gravity and deformation – long and short duration	1,30[a] or 0,90[b]	1,00	1,50[a]	1,50[a]	0,70[c] or 0[b]	—	—
2	Extreme environmental	1,10 or 0,90[b]	1,00	1,10 or 0,80[b]	—	1,35 (L1)[c,d] 1,10 (L2)[c,d,e]	—	—
3	Damaged condition[f]	1,10 or 0,90[b]	1,00	1,10 or 0,80[b]	—	1,00	—	—
Abnormal (accidental) limit states								
4	Abnormal environmental	1,00	1,00	1,00	—	—	1,00[c]	—
5	Accidental	1,00	1,00	1,00	—	—	—	1,00

It can be seen from ISO Table 7.3 that an L2 event would have a load factor of 1.1 on the 100 year load (compared to 1.35 for an L1 structure). Abnormal events are not to be considered for L3 structures. This is the main point of logic in not considering multi-year ice and icebergs in the design of the dock.

Looking at the two codes it would seem that if classed as ISO L2, the design event should be that with an annual probability of 10^{-2} and a load factor of 1.1. Load factors for L3 are not given, it is an Owner's choice based on potential economic loss.

Therefore, to comply with codes for Arctic Structures requires designing for loads with annual probability levels of 10^{-2} and a load factor of 1.1. However, it is the prerogative of the Owner to design with a higher load factor to protect the investment. We suggest deriving loads with as little bias as possible and then the Owner consider whether a load factor higher the minimum required by the codes be used. It would seem that consistency of load factors for other types of loading should be aimed for.

Abnormal level ice loading events (same as Accidental) do not need to be included when a structure is classed as L3. We therefore agree with the conclusion in BMT Fleet (2014), that multi-year ice need not be considered as a loading scenario.

3.0 Ice Loads

3.1 The nature of ice loads

Any structure in an area with ice will be subject to ice loads. Ice loads occur when ice moves (or is driven by environmental forces) against a structure. An example of a drilling platform being subject to ice loading is shown in Figure 3.1.

Figure 3.1: The Molikpaq drilling platform in the Beaufort Sea in 1986

The structure in Figure 3.1 is in a water depth of about 35m and the ice being broken clears around it without building up and grounding in front of the platform. In contrast, a wide structure in shallow water such as the artificial drilling island shown in Figure 3.2 will usually generate an ice rubble field in front of it. This may be grounded, depending on water depth.

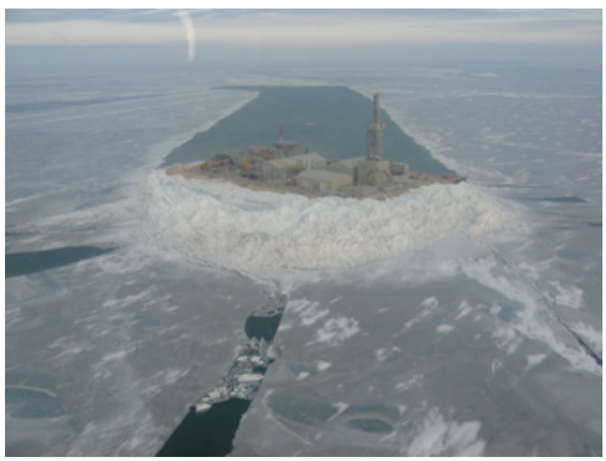


Figure 3.2: Drilling Island in North Caspian Sea subject to ice movement with grounded ice rubble being generated

Concepts for ice loads are quite simple. The ice load can be created and limited by one of three “Limits”:

1) Limit Stress: This defines the maximum ice load which can occur; there is sufficient driving force such that the load is limited by local failure of the ice in front of the structure and the ice fully envelops the full width of the structure. The ice stress at failure governs the load (as well as ice thickness and structure width).
2) Limit Force: If there is not enough driving force on the ice feature from winds, currents and pack ice pressures acting on it, to create the Limit Stress load, then this force controls
3) Limit Energy: If the ice is in the form of a discrete ice mass such as an isolated floe (or iceberg), the force can be limited by its kinetic energy as it is brought to rest.

The ice load cannot be any higher than that calculated as the Limit Stress load, so in preliminary design work, this is usually the only one calculated (unless it is an ice feature such as an iceberg which is controlled by limited energy available). In this document all loads estimated are based on ice failure in front of the quay (Limit Stress). Where a limited fetch may control the driving force, this is discussed. If loads are due to isolated floes, the Limit Energy loading is also checked.

Ice is a brittle material at high strain rates, so when ice fails against a structure, the ice pressures generated will fluctuate across the contact zone. Over small areas (or widths) the peak ice failure pressure will be higher than the average maximum over greater areas (or widths). This requires the designer to look at ice loads over a range of areas and widths. Generally, the ice loads are segregated into; global ice loads - which control the overall global stability of a structure; and local ice loads – which are used to design or check against local damage at the interface between ice and structure. Depending on the type of structure or vessel some local structural deformation or damage may be tolerated (if it is not very frequent and can be repaired if necessary).

As mentioned already, the main ingredients to ice loads are ice thickness and ice strength. Ice engineering practice has methods to combine these into design ice loads. The most current standard of practice is ISO 19906. There are other more detailed texts relating to ice action such as Palmer and Croasdale (2013) and this is also referenced. The ice load report commissioned by Hatch (BMT Fleet, 2014) has been made available to this team. It is very comprehensive; references will be made to it as appropriate and to avoid duplication. It should be noted however, that this document is an independent assessment and the current proposed dock structure is different than the one assessed in the Hatch Report.

3.2 Deterministic vs probabilistic loads

All codes and standards aim at a certain level of reliability and recognize the variability of many input parameters which should be treated statistically. This allows ice loads to be calculated within a probabilistic framework and this is encouraged by the codes and is demanded by some regulators.

Usually in preliminary design work, the ice loads are deterministic. In final design for high profile structures such as offshore drilling and production platforms it is usual practice to exercise a probabilistic model. The 100 year return period is often used for offshore platforms, which translates into events with annual probabilities of 0.01. Partial load and material factors are also applied. The load factor for manned Arctic platforms in ISO 19906 is 1.35 (For L1 structures). A discussion of annual probability levels, load factors and safety classes (which can affect these), has been given in Section 2).

The overall logic for ice loads is illustrated in Figure 3.3.

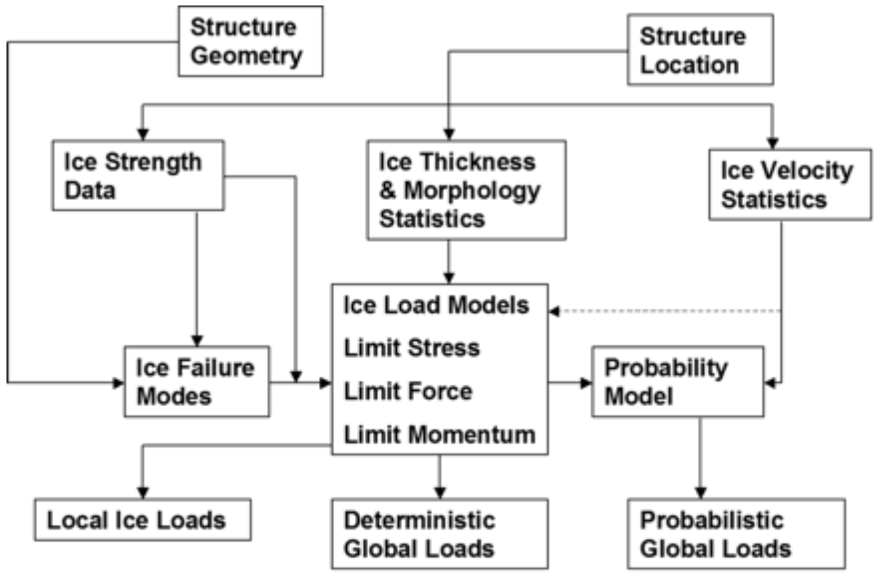


Figure 3.3: Logic for ice loads

Even within a deterministic approach, the load can be specified at a certain return period if statistics on the key ingredients such as ice thickness can be described statistically. This can be thought of as using deterministic methods to derive a nominal "100 year" load by using the 100 year ice thickness. Care is needed however, if there are several ingredients, not to combine extreme values for each because then the load will be more conservative than a true "100 year load" based on a full probabilistic analysis. As will be discussed later, some concepts which are inherent in probabilistic modelling such as the number of events in a year (this is called "exposure") can be approximated in the semi-deterministic approach which we will be using.

3.3 Global ice loads on the Quay

Global ice loads are those which act on sufficient widths of the structure to threaten global stability (not local damage – which is discussed later). The most important initial exercise in developing ice load criteria is to develop the range of ice interaction scenarios which apply for the proposed structure at the proposed location.

Experience of this ice team in working on harbours and shallow water platforms in the Arctic and Caspian Seas (where the ice can become landfast) suggests that the relevant ice scenarios are: (a) Freeze-up; (b) Mid-Winter when the ice is landfast; and (c) Spring Melt and Breakup.

3.3.1 Scenario 1): Early winter (Period 1) when the ice is mobile and has not yet become "landfast".

Even in sheltered inlets, the ice cover can be dynamic during this period as ice movements are to be expected when high winds occur and it has not been fully stabilized by the shoreline. The ice cover consists of floes of various thicknesses and sizes, although it is thin compared to conditions in late-winter. Ridging and rafting may occur during this time period.

The moving ice is cold (hence strong) and is moving at sufficient speed to impose maximum ice pressures. However, this load may not govern because the ice thickness will be less than the maximum ice thickness later in the winter. The key issue here is to determine the maximum thickness just before the ice becomes landfast. This can be done by looking at historical data and also based on experience from other Arctic areas. Historical temperature data can be used to predict the ice thickness but observations and experience are important in establishing when the ice becomes stable. The fact that the location is at the end of a relatively narrow inlet favours early stabilization as a function of thickness compared to other more exposed locations.

Ridges can be formed during this period. Mostly these will form further offshore from the facility; however, if ice motion is towards the dock very early in the period, then ice rubbling and possible ice encroachment may occur right at the dock and this will be discussed later under the topic of ice encroachment.

The worst case for ice loads in Period 1 is illustrated in Figure 3.4. On a vertical face the ice will fail in crushing. Line loads will be calculated using ISO 19906 using the maximum derived ice thickness for early winter mobile ice.

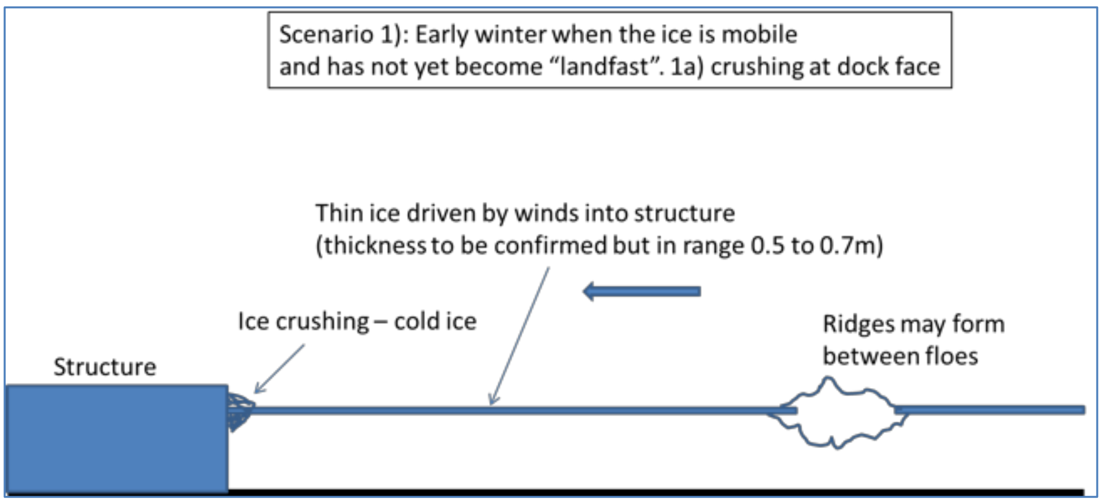


Figure 3.4: Ice action at the beginning of Scenario 1 (Just after freeze –up)

3.3.1.1 Ice thickness for Scenario 1 ice loads

In the Arctic, the ice grows quickly in early winter. Once the water temperature has been cooled to its freezing temperature, up to about 8cm of ice can grow in 24 hrs. The thickness at which the ice cover becomes landfast will vary from year to year according to the pattern of winds, currents and possibly the creation of grounded ridges further offshore. In the experience of the ice team, a rule of thumb based on other areas would be about 0.5m. This was a value used recently in a study of harbour design for an Arctic LNG Project in Russia. This location is more sheltered, and it is quite likely that the ice will stabilize at a lower thickness.

The "100 year" ice thickness is generally derived from statistics of ice measurements taken at a site over a number of years. Unfortunately, no ice thickness measurements have been taken at this dock site (to our knowledge – and this has been requested). In lieu of site-specific measurements it is generally acceptable to look at a similar location that has data and use that data as proxy. Furthermore, ice thickness can be predicted using air temperatures, but again these methods also rely on calibration at a particular site. Pond Inlet is the closet location with similar conditions as this site, so this has been used as proxy. Ice charts produced by the Canadian Ice Service as well as satellite imagery can also provide inputs as to when the ice becomes landfast and stabilizes.

The accompanying report on ice conditions (KRCA 2018a) analyses ice thickness data, ice charts and imagery to assess the ice thickness for this scenario. The results are as follows:

- Mean ice thickness is 0.32m and standard deviation is 0.11m
- From these, the value at a 0.01 probability level (100 year) is about 0.57m. This is nominally the "100 year value" and is rounded up to 0.6m. In ice load calculations, if other parameters are included which have a statistical distribution, care is needed to avoid being overly conservative by choosing extremes for these values also.

3.3.1.2 Global ice loads – Scenario 1

The physics of ice crushing is complex with non-simultaneous failures occurring in various micro-modes such as spalling to the upper and lower surfaces with small high pressure zones rapidly growing and then decaying across the contact zone. No theoretical models based on pure physics are proven, so the current methods are based on the empirical treatment of measured data combined with plausible adjustments for different geometries and conditions. The accepted methodology is outlined in ISO 19906 and can be summarized as follows.

$$p_G = C_R \left(\frac{h}{h_1}\right)^n \left(\frac{w}{h}\right)^m$$

Equation (3.1)

Where;

p_G is the global average ice pressure, expressed in megapascals;

w is the projected width of the structure, expressed in metres;

h is the thickness of the ice sheet, expressed in metres;

h_1 is a reference thickness of 1 m;

m is an empirical coefficient equal to –0,16;

n is an empirical coefficient, equal to –0,50 + h/5 for h < 1,0 m, and to –0,30 for $h \geqslant$ 1,0 m;

C_R is the ice strength coefficient, expressed in megapascals.

In ISO 19906 it is suggested that C_R = 2.8 for Arctic regions (= 2.4 for sub-Arctic regions: = 1.9 for temperate regions).

For design of the quay it is necessary for the structural engineer to have a range of loads as a function of widths of interest. The current design of the dock and combi wall have been shown in Figures 1.2 to 1.5 and are repeated here as Figures 3.5 and 3.6.

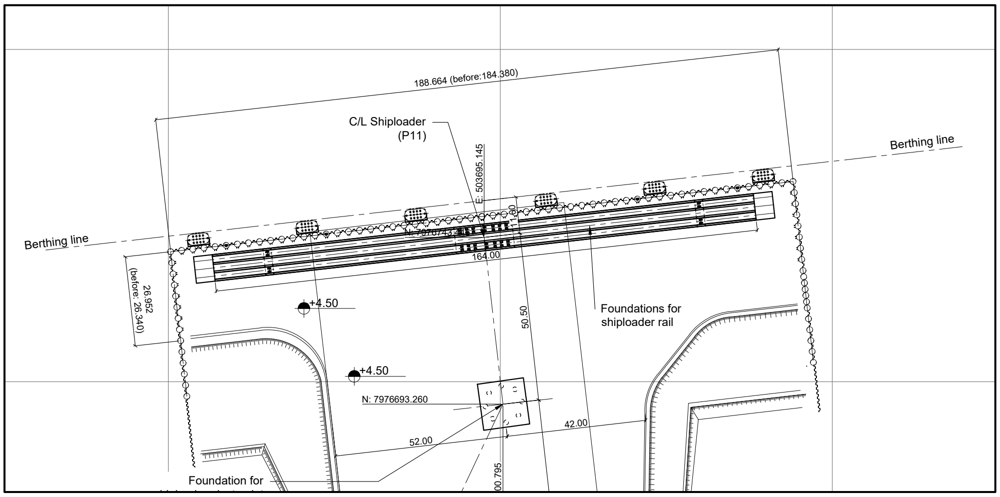


Figure 3.5: Drawing – plan view (current design total dock width is 188.7m (end piles CL to CL)

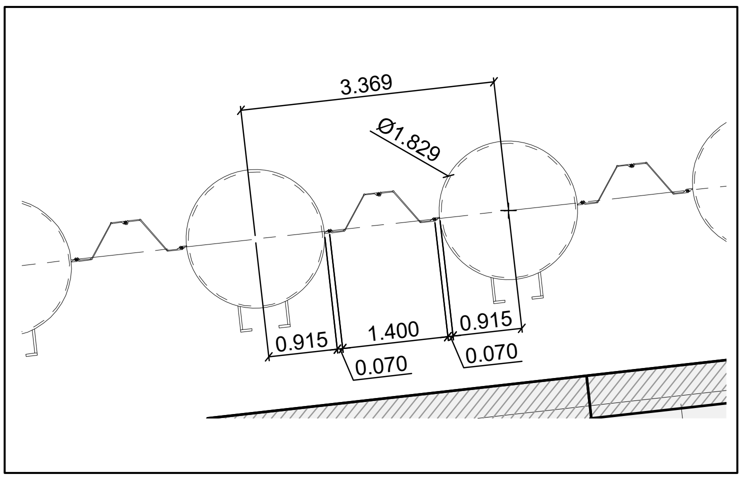


Figure 3.6. Details of combi wall (Revised dimensions (November 27, 2018))

The precise dimensions are subject to change as the detailed design proceeds, however the same principle in load derivation will apply and load tables can be adjusted accordingly.

Using the ISO 19906 methodology for this scenario with an end of period 1 ice thickness of 0.6m gives the range of line loads for the various widths of interest as shown in Table 3.1. It can be seen from Table 3.1 that the ice load varies with loading width. The smallest loading width used is that of the width of the sheet pile in the combi-wall (1.54m). The next width is a single circular pile (1.83m). The subsequent widths increase based on the number of piles engaged in the width of loading.

For structural analysis, depending on the load paths expected, the structural engineer should check various loading width cases. Over the full dock width of 187m this calculation gives an average ice line load of 0.77MN/m. Guidance is provided later on how the variability of line load with width can be accounted for but staying within the global load envelope.

Table 3.1: Global line loads for Scenario 1 (end of freeze-up period) – no correction for limited exposure

	Scenario 1	ISO 19906			Cr
					2.8
				m	-0.16
No of piles	Loading	Ice		Arctic cold	Arctic cold
	width (m)	Thickness (m)	n	p (MPa)	Line Load (MN/m)
sheet pile	1.540	0.6	-0.38	2.92	1.75
1 circular pile	1.830	0.6	-0.38	2.84	1.71
1 pile tributary	3.369	0.6	-0.38	2.58	1.55
2	6.738	0.6	-0.38	2.31	1.39
3	10.107	0.6	-0.38	2.16	1.30
4	13.476	0.6	-0.38	2.07	1.24
5	16.845	0.6	-0.38	1.99	1.20
6	20.214	0.6	-0.38	1.94	1.16
8	26.952	0.6	-0.38	1.85	1.11
10	33.690	0.6	-0.38	1.78	1.07
12	40.428	0.6	-0.38	1.73	1.04
14	47.166	0.6	-0.38	1.69	1.01
16	53.904	0.6	-0.38	1.66	0.99
21	70.749	0.6	-0.38	1.58	0.95
56	190.494	0.6	-0.38	1.35	0.81

Effects of "exposure"

These loads would apply to a structure such as an offshore platform with many 100s of km of ice motion moving past it each winter. This dock face will have limited ice movement against it, so the values in Table 3.2 are conservative. In the latest ISO 19906 (DIS), a method is included to account for the amount of ice motion ("exposure" is the term used)

The main reason why ice loads can vary with the amount of ice movement is because of the nature of ice as a brittle natural material at high strain rates. As the ice moves a series of load cycles occur (events). For each load cycle, the load builds up creates a load peak and then fails. These peaks are not equal because the ice contains many cracks and flaws so that effective ice strength is very variable. The more load peaks that are included in a series, the probability of a higher value increases. Therefore, the more the ice movement the higher the load peak that will be captured.

The empirical methods which are based on measured data tend to use the upper envelopes of the measured data implying that many events will not attain the ice loads predicted by them. This does not matter if there are thousands of ice events per year because at least a few of those events will correspond to the upper envelope of the measured data. However, if the structure is in a location with only a few ice events per year, then the probability of attaining an ice load equivalent to the upper envelope defined by the empirical method is reduced. In other words, the ice load is a function of "exposure" which is a function of the number of ice events per year. The number of ice events per year is a function of the amount ice movement past the structure.

To account for this phenomenon, a full probabilistic analysis of ice loads can be undertaken (using variable strength as a statistical input). In ISO 19906 (DIS) the issue of exposure is discussed, and examples given. Results for the Baltic region are shown in Table 3.2.

Table 3.2: Effect of exposure on strength coefficient in ice load calculations (from ISO 19906 DIS)

Total distance Moved by ice (km)	Return period	Probability	C_R (MPa)
6	1 year	0.5	0.99
6	100 years	0.99	1.45
135	1 year	0.5	1.34
135	100 years	0.99	1.8
135	10,000 years	0.999	2.8
563	1 year	0.5	1.49
563	100 years	0.99	1.96

Quote from ISO 19906 is follows.

"In Reference [130], a probabilistic method is proposed for including the effect of exposure on CR using temperate data from the Baltic Sea. Exposure was represented in terms of the distance moved by the ice past the structure. Using this approach, C_R values can potentially be scaled based on exposure and applied to different regions of interest."

The values above relate to the Baltic for which the nominal value of C_R is stated as 1.80 for a 100 year event. It can be seen that this is attained with an exposure equivalent to a distance moved by the ice annually of 135km. If the ice only moved 6km, the C_R value would be reduced to 1.45 which is a reduction in the expected load of 20%. (Factor of 0.8)

When considering the Mary River dock (or almost any dock) on the shoreline, it can be appreciated that the amount of ice motion against the dock from offshore is limited by the shoreline behind and adjacent to the dock. (This would nominally be about 200m; to be conservative a value of distance moved of 500m is assumed)

The information given in ISO 19906 on reduction in C_R with lower exposure can be analyzed to look at more limited interaction distances. The 100 year values of C_R are plotted against distance in Figure 3.12.

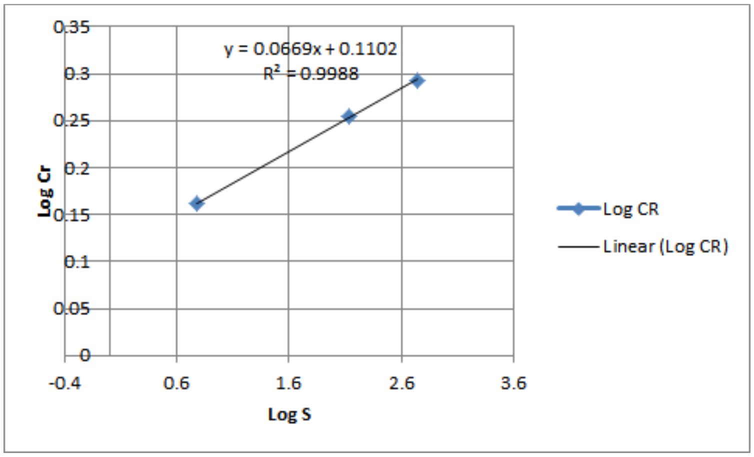


Figure 3.7: Ice strength coefficient (C_R) vs "exposure" distance moved during the interaction (s)

The equation for the log – log plot is

$$Log\ C_R = 0.0669\ (Log\ s) + 0.1102 \qquad \text{Eq. 3.2}$$

For a length of interaction of 500m (which is still conservative), the value of C_R is 1.09. This is compared to the 100 year value with extensive movement of 1.8 which implies a reduction factor of 0.61.

It is valid to apply this factor to the C_R value being used for this Arctic location (2.8). This will reduce the ice loads to those now shown in Table 3.3. The average line load over the full dock is now 0.5MN/m.

It can be seen that the global load if the ice is failing across the full structure width is 190.5x0.5= 94MN. One issue to also address is whether there can be sufficient driving force on the ice or enough kinetic energy in an individual ice floe to generate such global loads. This issue will be addressed once all the three ice load scenarios have been assessed so that the critical scenario can be examined further.

Table 3.3: Ice loads for Scenario 1 (accounting for limited exposure)

	Scenario 1	ISO 19906			Cr	
					2.8	0.61
				m	-0.16	
No of piles	Loading	Ice		Arctic cold	Arctic cold	With exposure factor of 0.61
	width (m)	Thickness (m)	n	p (MPa)	Line Load (MN/m)	Line load (MN/m)
sheet pile	1.540	0.6	-0.38	2.92	1.75	1.07
1 circular pile	1.830	0.6	-0.38	2.84	1.71	1.04
1 pile tributary	3.369	0.6	-0.38	2.58	1.55	0.94
2	6.738	0.6	-0.38	2.31	1.39	0.85
3	10.107	0.6	-0.38	2.16	1.30	0.79
4	13.476	0.6	-0.38	2.07	1.24	0.76
5	16.845	0.6	-0.38	1.99	1.20	0.73
6	20.214	0.6	-0.38	1.94	1.16	0.71
8	26.952	0.6	-0.38	1.85	1.11	0.68
10	33.690	0.6	-0.38	1.78	1.07	0.65
12	40.428	0.6	-0.38	1.73	1.04	0.63
14	47.166	0.6	-0.38	1.69	1.01	0.62
16	53.904	0.6	-0.38	1.66	0.99	0.61
21	70.749	0.6	-0.38	1.58	0.95	0.58
56	190.494	0.6	-0.38	1.35	0.81	0.50

3.3.2 Scenario 2): During mid-winter until break up the ice.

Close to shore and in sheltered inlets the ice becomes "landfast".

However, so-called "landfast ice" can move under thermal strains, ice jacking (as cracks refreeze) and under wind stress. These ice motions are however at low strain rates. Because ice creeps at low strain rates, slow motions give much lower ice pressures than when ice moves faster during freeze-up and break up. Therefore, this period will not give the design ice load.

In ISO 19906 it is suggested that *"For a preliminary assessment of thermal actions, indicative values in the range of 150 kN/m to 300 kN/m can be used regardless of the ice thickness. Thermal actions in freshwater ice are larger in magnitude than those in sea ice".*

It is recommended for this project and this scenario that the mid-winter horizontal ice loads be taken as 0.3MN/m. Such a load level will not govern and is well below the load levels for the freeze-up and break-up periods that it needs no further refinement.

Note that during this period (and also during the freeze-up period), the ice cover will be moved up and down by the tides. This can create ice bustles which are a build-up of ice frozen to the dock face with a tidal crack on the outer side of the frozen ice. There are some nominal vertical loads associated with these ice bustles (due to their weight and buoyancy). The topic of ice bustles and associated effects will be reviewed in Section 5.

3.3.3 Scenario 3): Break up

3.3.3.1 Overview

This period starts with ice decay, which occurs at the shoreline because this area is the first to warm up, in response to solar radiation. As break-up progresses, the nearshore area becomes ice-free while ice is still present offshore. At this time, the ice cover becomes broken up into floes, some of which may be large. During this period, the ice can be quite mobile due to the actions of winds and tidal currents; although fetch is limited except along the axis of the inlet. A typical image of large ice floes at break up is shown in Figure 3.7. This is for ice in Frobisher Bay but a similar process is to be expected in Milne Inlet. In fact the ice chart shown in Figure 3.8 indicates that Milne inlet on July 18 2016 has a 7/10 ice cover, which is thick first year ice and is comprised of big floes, 500 m to 2000 m wide. Unfortunately, the resolution of the ice charts is rather coarse and these conditions are more likely to be those in the more northerly region of the inlet (not close to the dock).

Figure 3.9 is an image on June 30, 2017 and shows the ice near the dock site melting in place (probably due to inflow from the Mary River). This condition would impose zero loads on the dock. Figure 3.10 shows a later image on July 6th 2017; mobile floes are shown further north in the wider part of the inlet many km from the dock and with only small ice debris closer to the dock location. It can be appreciated that although there may be some probability of floes released at break up impacting the dock there is a greater chance that the scenario seen in 2017 occurs (with no ice after break up being driven back into the dock). Despite this less than 100% probability of occurrence every year, the scenario will be assessed as though it can, but also taking into account the other mitigating aspects on ice loads associated with melting ice, namely thickness and strength reduction.

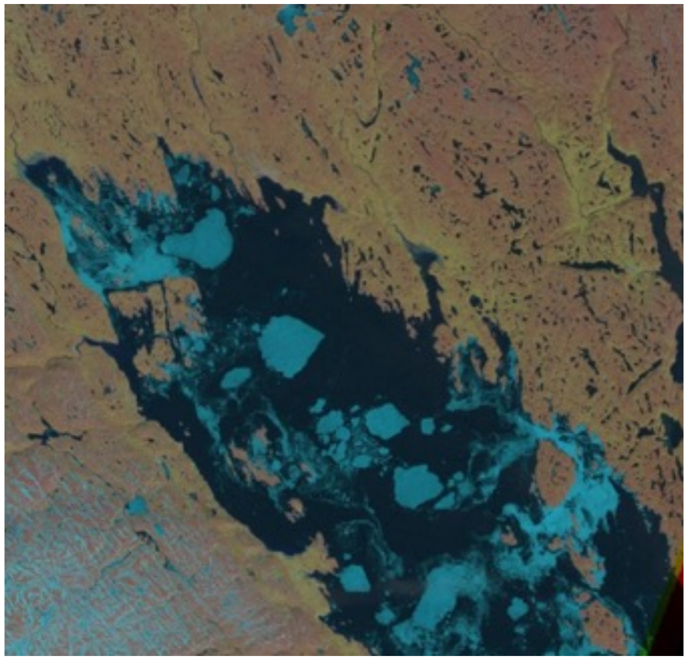


Figure 3.7: Satellite Image of Ice Conditions in the Northern end of Frobisher Bay on July 16, 1990

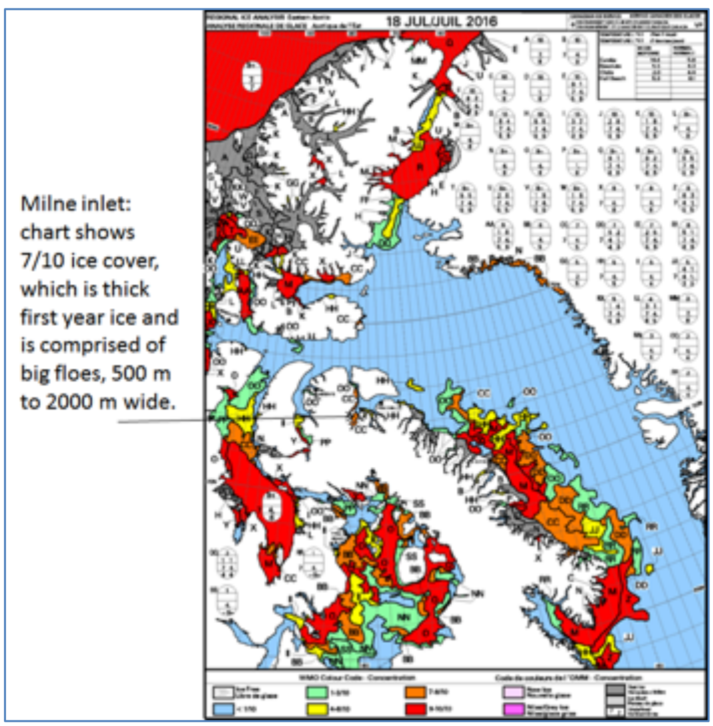


Figure 3.8: Ice chart for 18 July 2016

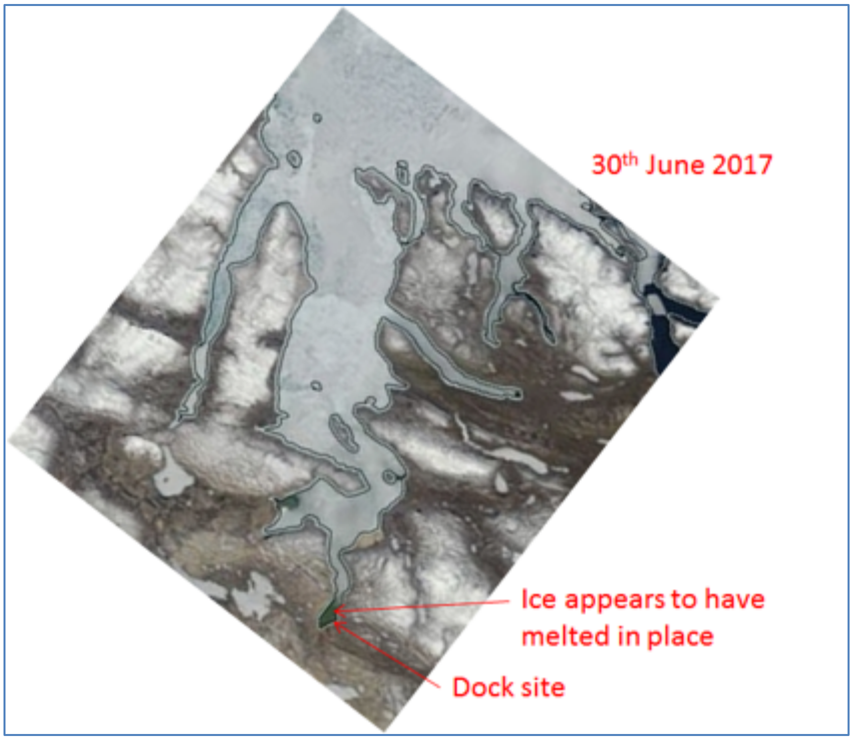


Figure 3.9: Initial ice melting at dock site; 30th June 2017

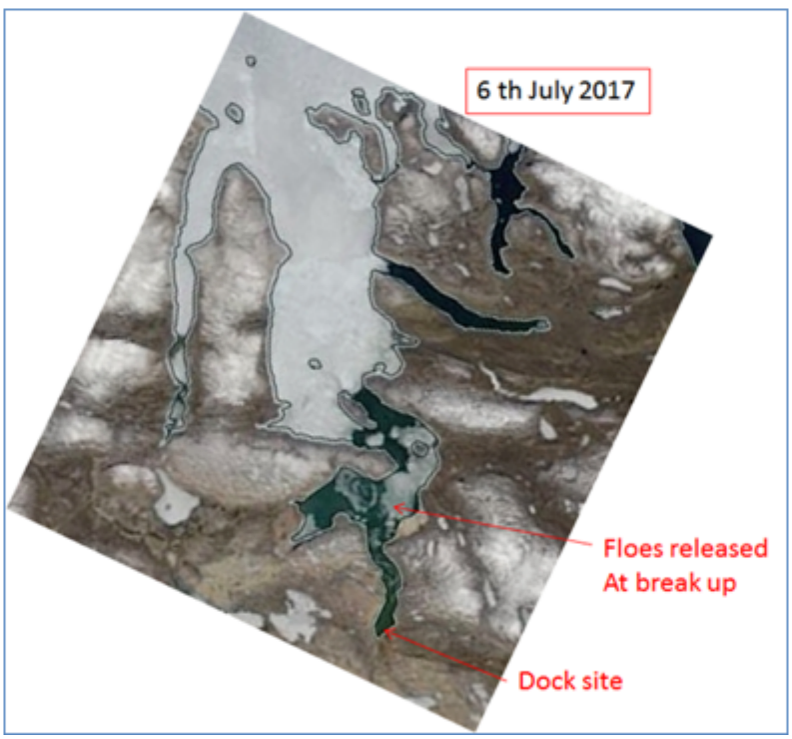


Figure 3.10: Ice break up by July 6, 2017

3.3.3.2 Ice thickness reduction

This is addressed in the accompanying "Ice Conditions report" (KRCA, 2018). Typical ablation of Arctic ice is shown in Figure 3.11. It can be seen that thickness at break up is less than the maximum winter value. These data are for Pond Inlet as there are no site-specific data available for Milne Inlet. As mentioned earlier, the inflow of the Mary River, the enclosed nature of the inlet with land mass on both sides will likely amplify this trend of reduced ice thickness at break up.

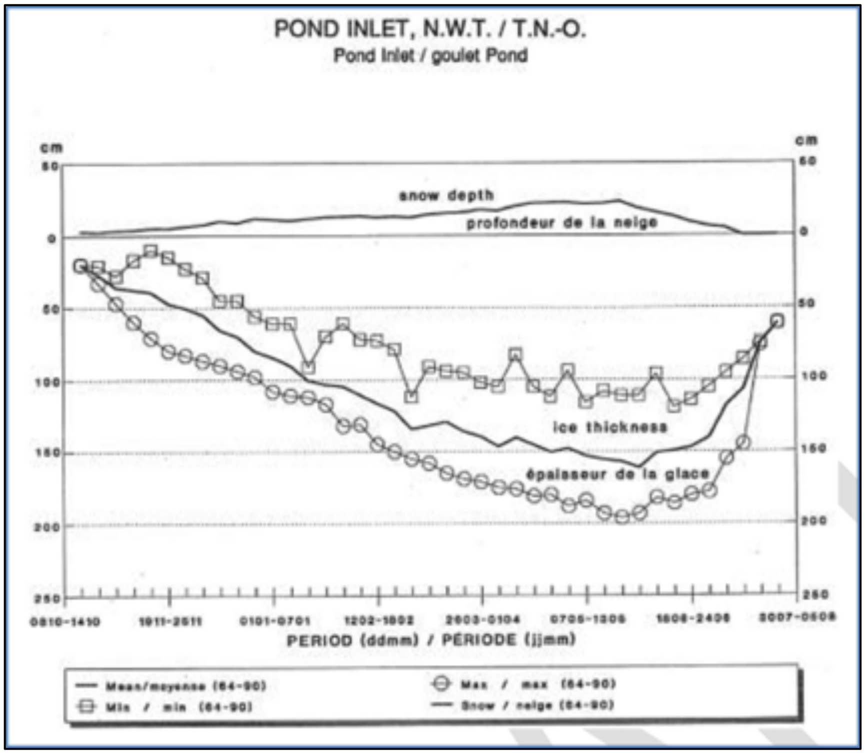


Figure 3.11: Maximum mean and minimum ice thickness data for Pond Inlet showing ice thickness reduction in the spring (Environment Canada (Canadian Ice Services (CIS)).

The detailed analysis of ice thickness data available for Pond Inlet in the accompanying report gives a "100 year" break up thickness of 1.4m. This will be used in the load calculations.

3.3.3.2 Ice strength reduction

The next key issue is the crushing pressure applied during this period. The same set of equations from ISO 19906 as used for the freeze-up period (for wide structures) will be used for the break up period, but with different inputs of strength and thickness; that is,

$$p_G = C_R \left(\frac{h}{h_1}\right)^n \left(\frac{w}{h}\right)^m \qquad \text{Repeat of Eq. 3.1}$$

Where,

p_G is the global average ice pressure, expressed in megapascals;

w is the projected width of the structure, expressed in metres;

h is the thickness of the ice sheet, expressed in metres;

h_1 is a reference thickness of 1 m;

m is an empirical coefficient equal to –0,16;

n is an empirical coefficient, equal to $-0,50 + h/5$ for $h < 1,0$ m, and to –0,30 for $h \geqslant 1,0$ m;

C_R is the ice strength coefficient, expressed in megapascals.

As noted earlier, C_R= 2.8 for Arctic regions (= 2.4 for sub-Arctic regions: = 1.8 for temperate regions).

ISO 19906 also allows an adjustment for C_R based on a comparison of strength for the scenario is question with the implied strength of cold Arctic ice (for which C_R=2.8 applies).

It is well proven that ice strength reduces with higher temperature and porosity. With sea ice, porosity is dominated by brine volume which for a given salinity also increases with higher temperature. ISO 19906 gives the following table for the adjustment of C

Table A.8-5 — Ice strength coefficient, *C*, as a function of the brine volume[a]

v_b	0,001	0,010	0,025	0,050	0,100	0,200
C MPa	8,4	6,0	3,4	1,6	1,0	0,8

[a] See Formula (A.6-7).

The following information with respect to brine volume is also given in ISO19906.

$$v_b = S_i\left(\frac{49,2}{|T_i|} + 0,532\right) \qquad \text{(A.6-7)}$$

where

v_b is the brine fraction (or total porosity, neglecting gas fraction), expressed as a volume fraction;

S_i is the ice salinity, expressed as a mass fraction in grams per kilogram (‰);

T_i is the ice temperature, expressed in degrees Celsius.

In growing cold FY sea ice, the gas fraction is often around 0,02 to 0,05, but in melting FY ice and old ice much of the brine is replaced by air and the gas fraction can be substantially higher, up to at least 0,2.

At break up when the ice becomes mobile it can be assumed that the ice has become isothermal through its thickness with a typical temperature of -2C. A typical salinity for sea ice is 6 parts per thousand. Inserting these values in the above equation gives a brine volume of 0.15. Looking at Table A8.5 above indicates that the strength is about 0.9MPa. This implies a strength reduction factor from the Arctic cold case of about 0.3.

Strength reduction at break-up was also addressed by Timco and Johnston (2002). Their measurements at Resolute (further North) are shown in Figure 3.12. These results would justify an even lower strength factor than the 0.3 value. In the data plot, the borehole strength data was that actually measured and by early July it was down to about 0.1 of its mid-winter strength.

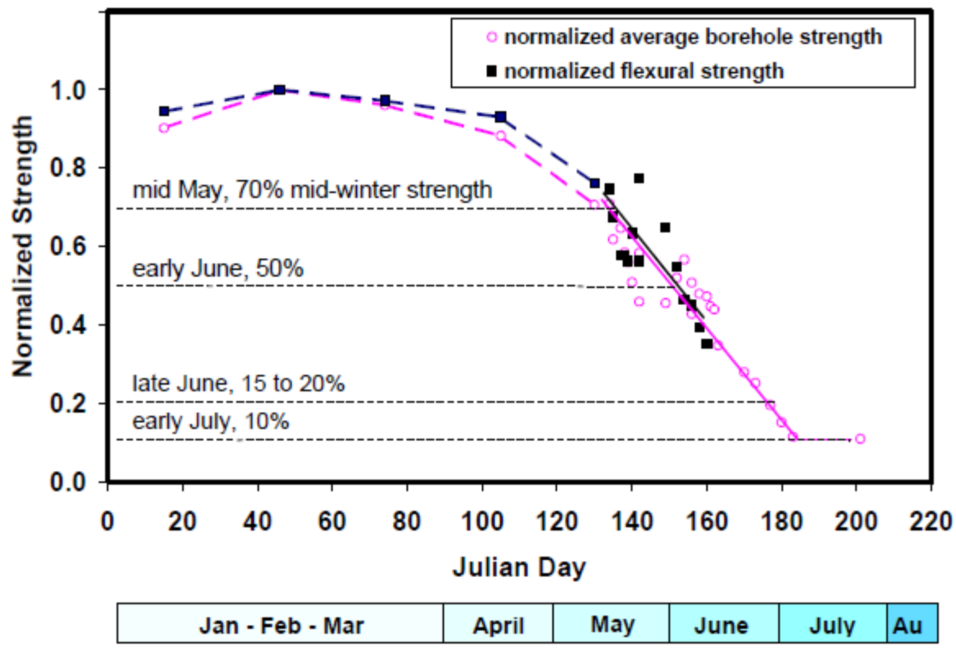


Figure 3: Comparison of normalized ice borehole strength and calculated flexural strength as a function of the Julian Day

Figure 3.12: Sea ice strength reduction at break up (Resolute – Canadian Arctic) (From Timco and Johnston, 2002)

The ice loads for this scenario based on the inputs discussed above are shown in Table 3.4. An exposure factor reduction is also incorporated because it is further shown in Section 3.3.1.2 that the discrete ice floes associate with this scenario are stopped in less than 100m (but to be conservative the exposure factor for 500m is used, that is 0.61. Also shown are the line load values for Scenario 1. It can be seen that the Scenario 1 loads will govern.

Table 3.4: Global line loads for Scenario 3 (and Scenario 1)

	SCENARIO 3					
	Break up	Warm weak ice	Strength Factor	0.32		
			Cr	0.896		
			n	-0.3		
			m	-0.16		0.6m
	Crushing	ISO 19906			With exposure	For comparison
					0.61	With exposure
Piles	Loading		Arctic Warm	Arctic Warm	Effect of exposure	Scenario 1
	width (m)	Thickness (m)	p (MPa)	Line Load (MN/m)	Load (MN/m)	Load (MN/m)
sheet pile	1.54	1.4	0.80	1.12	0.68	1.07
1 circular pile	1.83	1.4	0.78	1.09	0.66	1.04
1 pile tributory	3.369	1.4	0.70	0.99	0.60	0.94
2	6.738	1.4	0.63	0.88	0.54	0.85
3	10.107	1.4	0.59	0.83	0.50	0.79
4	13.476	1.4	0.56	0.79	0.48	0.76
5	16.845	1.4	0.54	0.76	0.46	0.73
6	20.214	1.4	0.53	0.74	0.45	0.71
8	26.952	1.4	0.50	0.71	0.43	0.68
10	33.690	1.4	0.49	0.68	0.42	0.65
12	40.428	1.4	0.47	0.66	0.40	0.63
14	47.166	1.4	0.46	0.65	0.39	0.62
16	53.904	1.4	0.45	0.63	0.39	0.61
21	70.749	1.4	0.43	0.61	0.37	0.58
56	190.494	1.4	0.37	0.52	0.32	0.50

3.3.4 Other limits to the ice loads

In Section 3.1, the three limits to ice loads were mentioned as

1) Limit Stress: This defines the maximum ice load which can occur; there is sufficient driving force such that the load is limited by local failure of the ice in front of the structure and the ice fully envelops the full width of the structure. The ice stress at failure governs the load (as well as ice thickness and structure width).

2) Limit Force: If there is not enough driving force on the ice feature from winds, currents and pack ice pressures acting on it, to create the Limit Stress load, then this force controls.

3) Limit Energy: If the ice is in the form of a discrete ice mass such as an isolated floe (or iceberg), the force can be limited by its kinetic energy as it is brought to rest.

The ice loads in the prior subsections are based on the “Limit Stress” concept and the controlling set of values of loads on various widths was established as that at freeze-up just before landfast ice is established. The table of those line loads but with the corresponding global loads added is repeated below as Table 3.5.

For each width of loading, there is a corresponding global load. In the extreme, for the ice acting and failing across the whole dock width, it can be seen that the global load is 88MN. If this was an isolated structure in the far offshore (such as the structure shown in Figure 3.1), such a load may be possible (and indeed such high loads and higher have been measured) on such exposed structures in the Arctic.

However, this dock is in a very sheltered inlet and ice forces are generated by the action of winds and currents on the ice. If fetch is limited, then the driving forces are limited. This is the “Limit Force” situation in the above list.

Furthermore, the size of the inlet can also limit the size of isolated floes and their speeds which could impact on the structure. This limits the available energy in the ice and which is dissipated by the ice force as work done. This process can also limit the total global load.

These two limits are now examined in the context of the freeze-up period.

Table 3.5: Ice line loads for Scenario 1 (at end of freeze -up) (repeat of Table 3.3) with corresponding global loads added.

	SCENARIO 1	Level ice crushing						
		ISO 19906			Cr			
					2.8	With exposure factor		
				m	-0.16	0.61	Max.	
No of piles	Loading width (m)	Ice Thickness (m)	n	Arctic cold p (MPa)	Arctic cold Line Load (MN/m)	Effect of exposure Line load (MN/m)	Per pile Case A (MN)	Global load (MN)
sheet pile	1.540	0.6	-0.38	2.92	1.75	1.07	1.65	1.65
1 pile	1.830	0.6	-0.38	2.84	1.71	1.04	1.91	1.91
2	5.198	0.6	-0.38	2.41	1.44	0.88	2.29	4.58
3	8.567	0.6	-0.38	2.22	1.33	0.81	2.32	6.97
4	11.936	0.6	-0.38	2.11	1.26	0.77	2.30	9.20
5	15.305	0.6	-0.38	2.02	1.21	0.74	2.27	11.34
6	18.674	0.6	-0.38	1.96	1.18	0.72	2.23	13.41
7	22.043	0.6	-0.38	1.91	1.15	0.70	2.20	15.41
8	25.412	0.6	-0.38	1.87	1.12	0.68	2.17	17.37
9	28.781	0.6	-0.38	1.83	1.10	0.67	2.14	19.28
10	32.150	0.6	-0.38	1.80	1.08	0.66	2.12	21.16
11	35.519	0.6	-0.38	1.77	1.06	0.65	2.09	23.01
57	190.493	0.6	-0.38	1.35	0.81	0.50	1.65	94.31

Limiting Driving force

This limit has been examined in terms of winds and currents. It can be seen that at the head of the inlet there is limited fetch available to create wind and current drag forces on the ice; it is less than 2km.

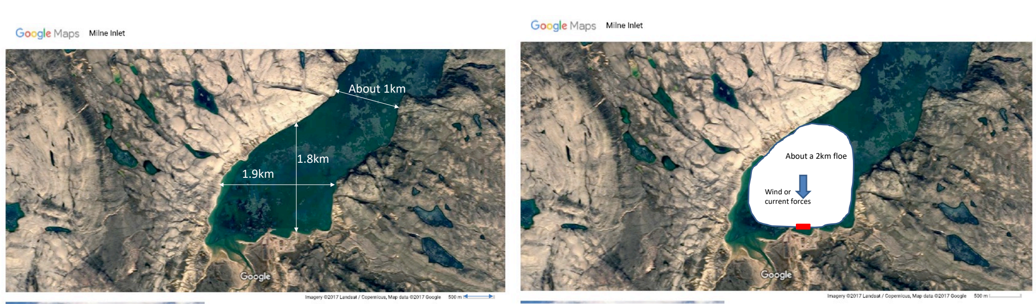


Figure 3.13: Approximate dimensions of head of Milne Inlet and floe size that can exert forces on the dock

In reality the ice at freeze-up is not as simple as that shown in Figure 3.13. Ice grows from shore and limits the size of freely moving ice and/or driving forces are resisted by shorelines as well as acting on a structure such as the dock.

The satellite photo in Figure 3.14 shows the complexity of freeze-up in 2018. It can be appreciated that the areas of ice available to concentrate wind and current loads on the dock are limited because of how the ice is also attached to the shore. In the situation shown in Figure 3.14, the ice tongue growing out from the dock is limited to about 300 – 500m in size. The already fast ice to the left will limit the size of still free ice which could concentrate wind and current drag onto the dock

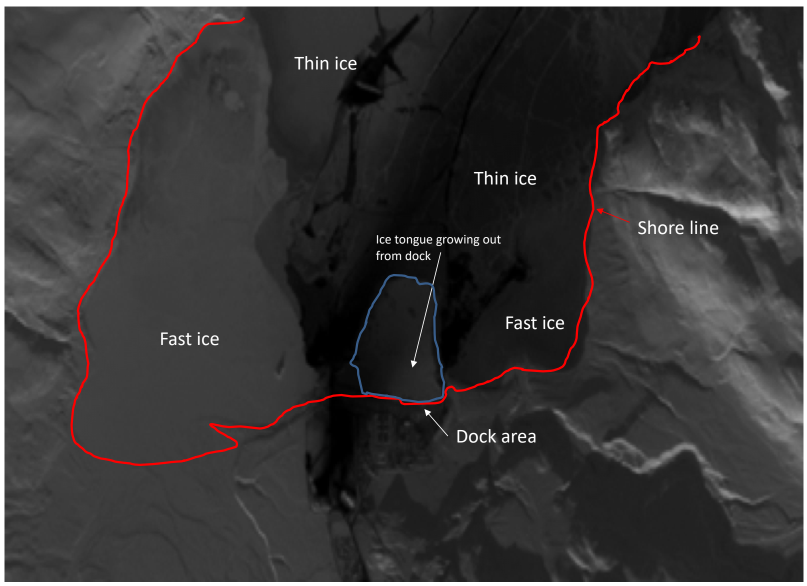


Figure 3.14: Ice freeze-up, 2018 (Image from A. Strandberg)

Wind and current drag can be estimated using accepted drag coefficients between ice and air and water (Sanderson, 1988; Palmer and Croasdale, 2013). A spreadsheet has been developed to calculate these forces for typical ice areas. Some example results for loads due to wind drag are shown in Table 3.6. Loads are calculated for sustained wind speeds up to 20m/s (72km/hr). This is not the 100 year wind but (as discussed earlier) one should not combine the 100 year thickness with other 100 year values. For a wind of 20m/s acting on the largest ice sheet possible at the upper end of the inlet (2km), the load on the dock is quite small at about 2MN. On the full width of the dock is 189m, then the line load is only 0.01MN/m. This compares with the line load for the “Limit Stress” condition of 0.46MN/m. The value of 0.01MN/m is based on the assumption that a large ice sheet (2km x2km) will also be acting on the shoreline so a conservative factor of 0.5 is used (this implies that the dock face sees 50% of the total applied drag force on the ice). The corresponding stresses in an ice sheet (0.6m ice) are also low (0.02MPa), so ice failure against the full dock width will not occur.

An ice sheet size which might apply the full drag force to only the dock might be limited to about 500m, such an ice sheet size is also shown in Table 3.6. This situation results in very small drag loads and line loads on the dock).(001MN/m).

Table 3.6: Ice forces due to wind drag over various ice sheet sizes and assumptions

Wind Drag								
						ice thickness	Width	
air density	1.275	kg/m3	Dock width	189	m	0.6	10	m
Ca	0.002							
m/s	m	MN		MN	MN/m	MPa	MN/m	MPa
wind	Equiv.Floe size (W)	Drag	Fraction taken by shore	Load on Dock	Line load across full dock	Ice stress	Line load on lesser width	Ice stress
5	2000	0.26	0.5	0.13	0.001	0.0011	0.013	0.021
10	2000	1.02	0.5	0.51	0.003	0.0045	0.051	0.085
20	2000	4.08	0.5	2.04	0.011	0.0180	0.204	0.340
30	2000	9.18	0.5	4.59	0.024	0.0405	0.459	0.765
5	500	0.02	0	0.02	0.000	0.0001	0.002	0.003
10	500	0.06	0	0.06	0.000	0.0006	0.006	0.011
20	500	0.26	0	0.26	0.001	0.0022	0.026	0.043
30	500	0.57	0	0.57	0.003	0.0051	0.057	0.096

If the ice sheet has a random shaped edge or approaches the dock at an angle, the drag force could be acting on a narrower width than the full dock width of 189m; Table 3.6 shows the line load in the case of a 10m width. This value is about 0.02MN/m, again insufficient to actually fail the ice so that the line load for a width of 10m shown in Table 3.5 of 0.79MN/m cannot occur. Clearly, if the width of the dock over which the drag force is applied becomes less and less, the line load will increase to the value based on ice failure at the dock. The realism of such a case, is open for discussion, but maybe there is piece of ice debris between the floe and the dock. If this was only about 2m, then the line load would approach 1MN/m this would be a one pile loading.

Current drag has also been evaluated and is shown in Table 3.7. The values are lower than for wind drag when typical currents to be associated with a 100 year thickness are used. Whether currents should be added to the winds is uncertain but even if this was the case, the line loads are still very low and similar conclusions can be reached. As a reference point for combined wind and current loads, which will be quoted in the summary, and which is conservative, the wind drag on a 200m ice sheet for a wind of 30m/s (4.59MN) is added to a current drag of 0.74MN to give 5.73MN (6MN rounded).

The loads discussed so far are “static loads” assuming the ice is not accelerated by the drag forces and impacts the dock with some speed. This situation is examined next as a “Limit Energy” situation

Table 3.7: Driving forces due to current.

Current Drag								
						ice thickness	Width	
water density	1028	kg/m3	Dock width	189	m	0.6	10	m
Cw	0.004							
m/s	m	MN		MN	MN/m	MPa	MN/m	MPa
Current speed	Equiv floe size (W)	Current Drag	Fraction taken by shore	Load on Dock	Line load across full dock	Ice stress	Line load on lesser width	Ice stress
0.1	2000	0.16448	0.5	0.082	0.000	0.001	0.008	0.014
0.2	2000	0.65792	0.5	0.329	0.002	0.003	0.033	0.055
0.3	2000	1.48032	0.5	0.740	0.004	0.007	0.074	0.123
0.1	500	0.01028	0	0.010	0.000	0.000	0.001	0.002
0.2	500	0.04112	0	0.041	0.000	0.000	0.004	0.007
0.3	500	0.09252	0	0.093	0.000	0.001	0.009	0.015

Limit energy loads

This loading condition applies when isolated floes are driven by winds or currents into a structure at a certain impact speed. In the authors' opinions this situation is more likely to be a loading scenario for break up, when isolated floes some km offshore can be accelerated to a terminal speed of about 0.3m/s.

At freeze-up the scenario is considered to be one of large ice sheets forming over much of the inlet at about the same time and there not being much space for them to move at their terminal speed. In fact ice charts always shows ice at about 9/10th coverage before it is landfast. If only 1/10 is open water, the ability for the moving floes to reach terminal speeds is limited.

The ice floe is brought to a stop when all the kinetic energy is dissipated by the "work done" by the ice force. It is simple mechanics. A spreadsheet has been developed for the situation shown in Figure 3.15.

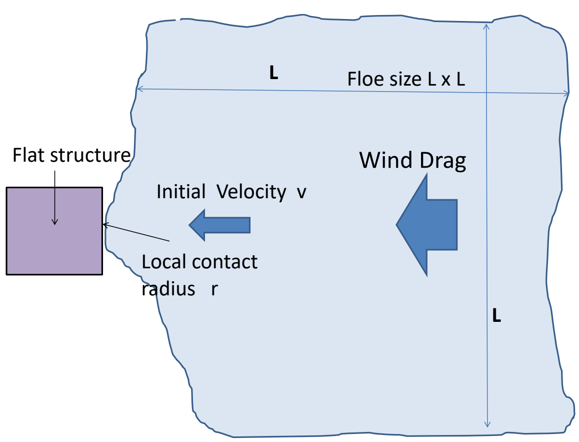


Figure 3.15: Impact of an ice floe into a flat structure

The calculation recognizes that ice in nature will have an uneven edge so a local radius is also specified in addition the nominal floe size. Typical results are shown in Table 3.8 for a 1000m size ice sheet moving into the dock at 0.3m/s. In this case, there is an effective penetration of the floe of only about 3m and the effective width of loading on the dock is about 22m. The peak load just before the floe stops is 15.3MN and the equivalent line load is 0.7MN/m

Table 3.8: Results for a typical floe at freeze-up moving against the dock at a speed of 0.3m/s

Input Data:		
Structure width (D) (m)		189
IceThickness (m)		0.6
Ice thickness at edge (m)		0.6
Floe size (L) (m)		1,000
Ice Density (kg/m^3)		900
Initial floe speed (v) (m/s)		0.3
Added mass factor		1.2
Local radius of floe at contact - r		20
m		-0.16
n		-0.38
CR		2.8
Exposure factor		0.61
Strength factor		1
Wind speed (m/s)		20
Drag coeff		0.002
Drag force (MN)		1.02
Initial 'KE (MN-m)		29.16
Pen. Incr. (m)		0.25
Mass + added mass (mln. kg.)		648.00
Maximum Impact Load (MN)		**15.3**
Maximum contact width (m)		**21.9**
Distance to stop (m)		**3.3**
Time to stop (s)		**18.3**
Line load (MN/m)		**0.70**

These results tell us that if the critical thickness ice sheet for Scenario 1 has had wind drag or current drag forces acting on it and has had some freedom to move and attain some speed, then the ice loads will be greater than the very low static drag forces developed previously.

On the other hand, there is still a limit to the total global ice load and loading width. In the case shown, in Table 3.8, the ice speed is 0.3m/s and the floe 1000m in size and its thickness the critical maximum thickness at the end of the freeze-up period (0.6m).

The line load over the maximum contact width of 22m is 0.7MN/m which is exactly the same line load as calculated in the "limit stress" calculations for that width (shown in Table 3.5) – this is to be expected because the formula for the ice crushing load is the same in all limiting cases (as given by Eq. 3.1 and with the same adjustment for limited exposure Eq. 3.2).

It can be seen however that the distance to stop is only a few metres (3m). This could lead to justification to reduce the 0.6 exposure factor used; (although the value becomes asymptotic to about 0.54).

Calculations have been performed for various plausible floe sizes at freeze-up advancing into the dock at various speeds. The results are shown in Table 3.9 (for reference the same scenario at break up is also shown – but the ice is weaker, so the line loads are less). In these cases the floe speed is calculated from the acceleration of the floe over 200m which is considered a reasonable open water space at freeze-up with 9/10 ice coverage over a 2km width.

Table 3.9: Impact of a 1500m floe at various speeds against the dock showing maximum contact widths

ice thickness 0.6m		1500m floe	
wind speed	20	15	10
floe speed	0.45	0.337	0.225
force	25.4	22.6	18.3
width	40	34.7	27.1
line load	0.64	0.65	0.68
ice thickness 1.4m		1500m floe	
wind speed	20	15	10
floe speed	0.336	0.25	0.17
force	18.9	16.6	14
width	46.6	40	32.5
line load	0.41	0.42	0.43

Overall, these calculations indicate that the global ice loads on the dock over the full structure width cannot be realized because of limited driving forces and limited kinetic energy of impact.

Because overall global stability of the dock is likely not an issue, these lower global loads will probably not affect the design to resist ice because lines loads over the shorter widths are not reduced.

However, this limit on the ability of driving forces to push ice into and past the dock will reduce the amount of potential ice encroachment; this is reviewed in Section 4 of this report.

3.5 Loading situations for design

3.5.1 Loading at 90 degrees to the dock face

As already reviewed the loading width over which the ice is crushing and generating its largest line loads is limited to floe impacts because in a static loading situation the wind and current drag on the ice sheet has limited fetch and cannot generate enough force to crush the ice over realistic widths.

In the situation examined for extreme- size mobile floes at freeze-up, typical loading widths are in the range 15 – 40m (see prior Table 3.9). The largest width is for the highest impact speed.

The line loads as a function of width are governed by the ISO crushing formula and have been tabulated over a range of widths in the prior tables. For example, at freeze-up with 0.6m thick ice, the line loads per width are shown in Table 3.3 and 3.5. They vary from 0.94MN/m over the width of a single pile “tributary” of 3.37m to about 0.66MN/m over a width of about 30m which is equal to the “tributary” for 9 piles.

The structural designer will need to check the design for various widths up to the typical maximums in Table 4.10. These loading widths can act anywhere along the dock face. It should also be recognized that line loads can vary within the average line load value for a given width. This is discussed next.

3.5.2 Global load variability across width

The preceding tables have shown how the line loads can be higher over smaller widths. In conducting structural analyses this should be recognized; however, in using a higher load over a smaller width, the average over the width being analyzed will need to be maintained at the values given. An example is given below (note the widths are chosen by a whole number of piles in the combi-wall which are at 3.37m spacing):

- Total width being analyzed is say 9x3.37 = 30.3m
- For the controlling Scenario 1, the average line load over 30.3m = 0.66MN/m (see Tables 3.4 and 3.6)
- At a width of 3.37m (tributary for one pile) the line load is 0.94MN/m, therefore, the line load over the remainder of the 30.3m is = (30.3*0.66 – 3.37*0.94)/ (30.3-3.37) = 0.625MN/m.

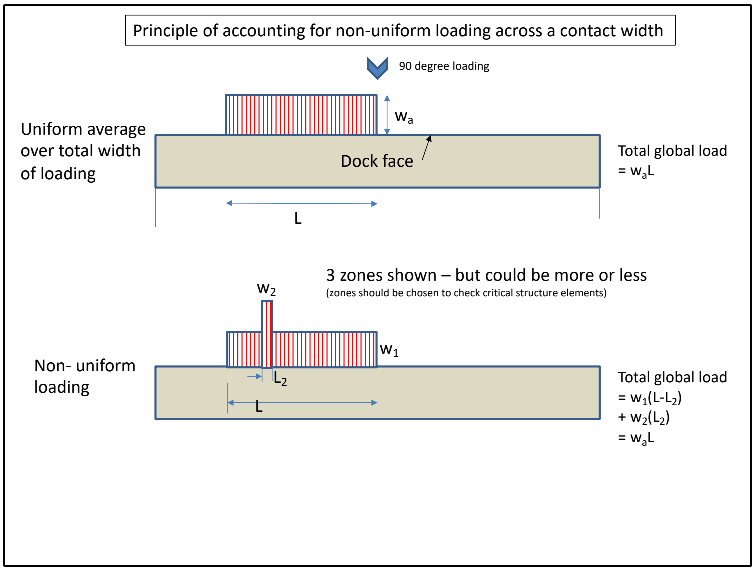


Figure 3.16: Non-uniform loading – principles of calculation

3.3.6 Load paths

The ice will act across the selected width to be analyzed in the manner shown in Figure 3.17. Crushing will occur across the whole width. There may be some process whereby the circular piles which are leading create fractures and spalls in the ice which lead to lower line loads on the sheet piles, but this cannot be taken for granted.

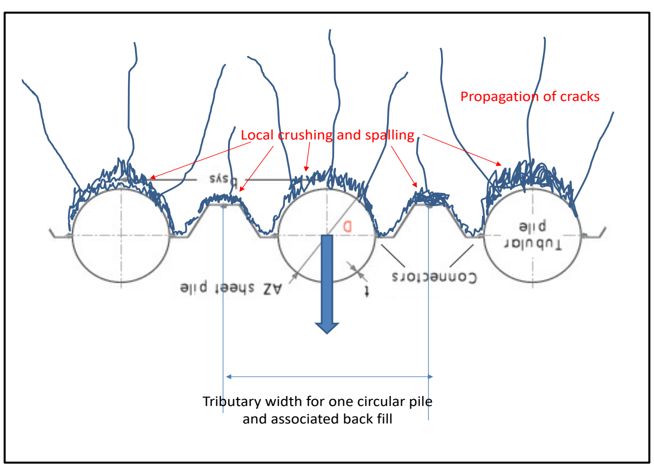


Figure 3.17: the nature of ice action on the combi-wall

Critical widths and analysis of structural integrity will depend on load paths from the ice line into the major resisting components of the structure. This cannot be determined “a priori” because it depends on relative stiffnesses of the various load paths. But the load paths may be bounded. The issues are shown in Figure 3.18:

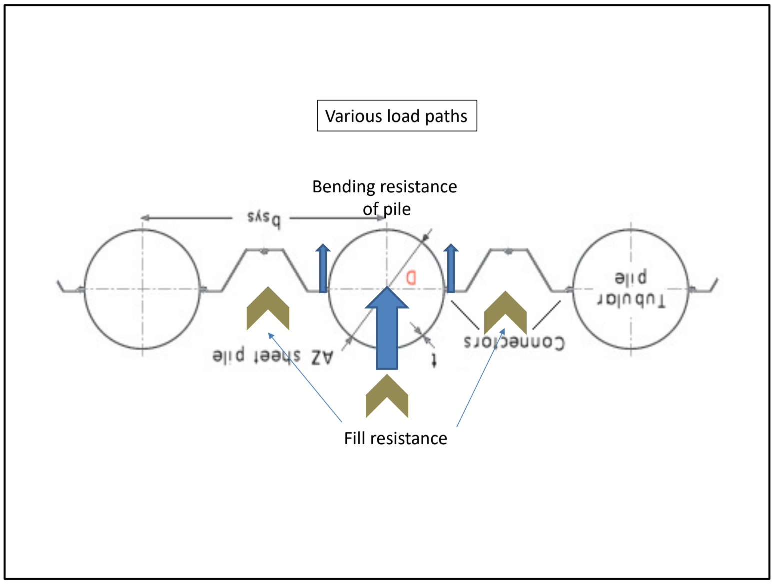


Figure 3.18: Issues relating to load paths

The main issue is how much the backfill can carry in shear through the fill versus the bending capacity of the piles. This may depend on whether the back fill is frozen. It is beyond the ice engineering aspects of developing the ice load criteria to answer this.

A key issue is whether the ice loads on the sheet piles have to be reacted by their connection to the circular piles or whether the backfill can absorb the load (or a proportion of the load). For ice loading, it has to be assumed that the sheet piles see the same line loading as the rest of the combi-wall. Relative stiffness does not affect the ice loading (as there is a continuous failure of the ice – creating more like a hydrostatic pressure) but it may affect the load paths. It is up to the structural engineer to figure that out. One bound is that all the ice loading on the sheet piles is transmitted sideways to the circular piles.

3.3.6 Corner loads

An ice sheet or floe can drift towards the dock at various angles of attack. A case which may require some special structural analysis is a corner loading. The type of scenario creating a corner loading is shown in Figure 3.19.

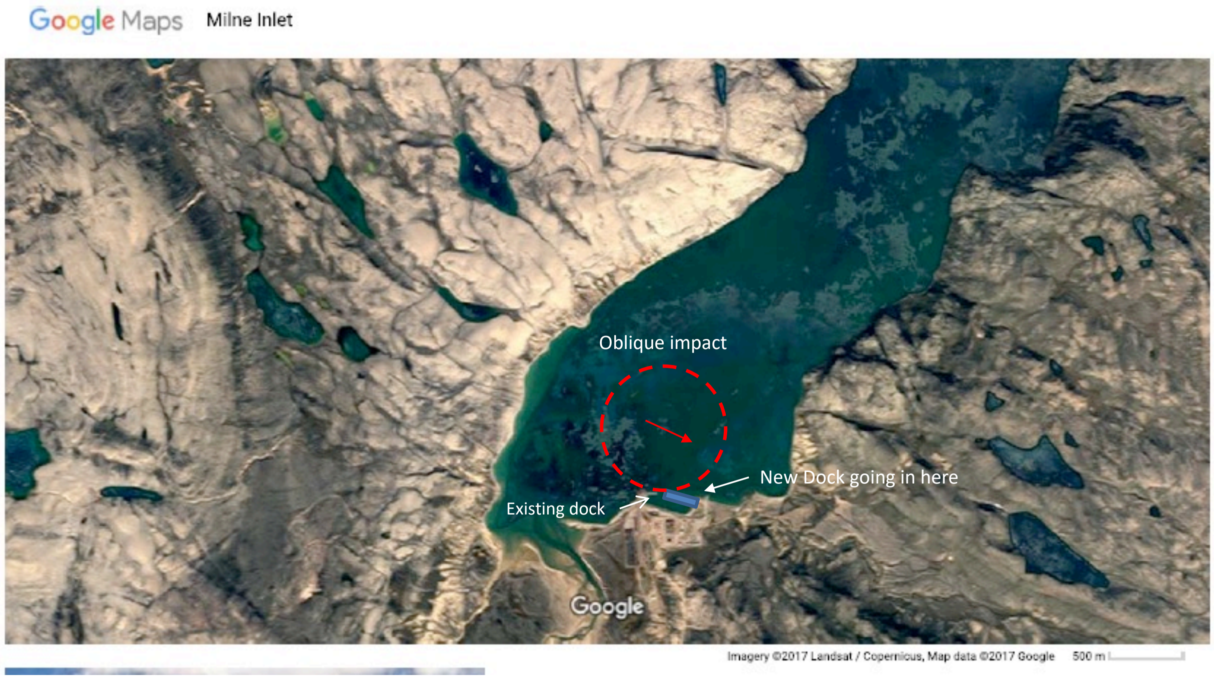


Figure 3.19: Oblique impact and corner loading

As penetration of the ice floe by the structure proceeds, the width of loading increases and involves more circular piles. The geometry for calculation is shown in Figure 3.20. To get the maximum width of loading an energy calculation is performed. The base-case floe is a maximum of 1500m size drifting at 0.3m/s; the energy calculation indicates a maximum width of loading when the floe stops of 11m. The loads associated with such an interaction are shown in Table 3.10.

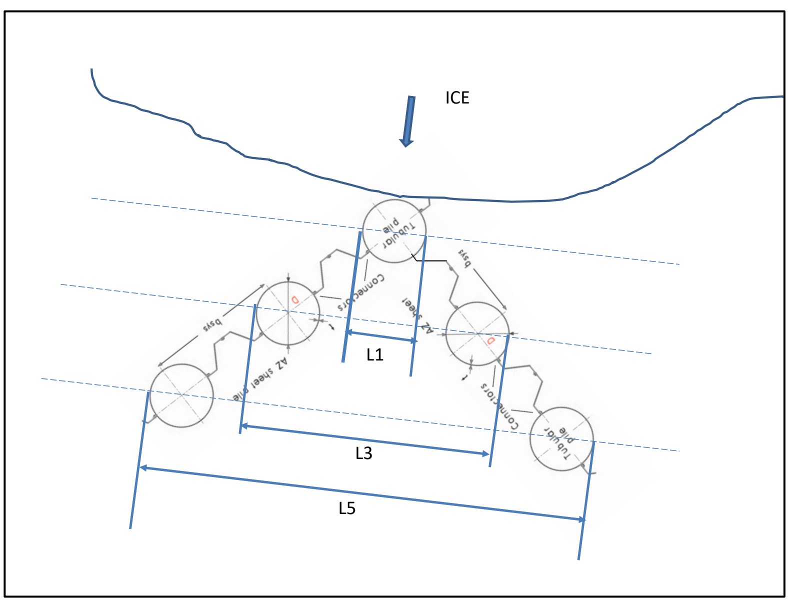


Figure 3.20: Geometry of corner loading and loading widths

Table 3.10: Corner loading

Corner loading	45 degrees	ISO 19906			Cr			
					2.8	With exposure factor		
				m	-0.16	0.80	Max.	
No of	Loading	Ice		Arctic cold	Arctic cold	Effect of exposure	Per pile Case A	Global load
piles	width (m)	Thickness (m)	n	p (MPa)	Line Load (MN/m)	Line load (MN/m)	(MN)	(MN)
corner pile	1.830	0.6	-0.38	2.84	1.71	1.37	2.50	2.50
3	6.600	0.6	-0.38	2.32	1.39	1.11	2.45	7.34
5	11.360	0.6	-0.38	2.12	1.27	1.02	2.32	11.58

3.3.7 Oblique loads

Oblique loads on the front face can be handled in a similar manner. In this case, the effective width of interaction can be determined from the principles of geometry shown in Figure 3.21. The loading widths are those involved in generating the ice force and determining the line loads. The length of the quay (*wq*) which is loaded is then determined from the angle of attack *a* using the expression;

$$wq = wl/sina \qquad (3.3)$$

Based on engineering judgement, considering the shape of the inlet and dock location, it is recommended that the minimum angle of attack considered should be 40 degrees. Table 3.11 shows how the length of loaded dock relates to the loading width for the energy dissipation calculation and for the critical cases given earlier in Table 3.9. For design, also note that the loading on the piles and dock face is in the direction of ice movement.

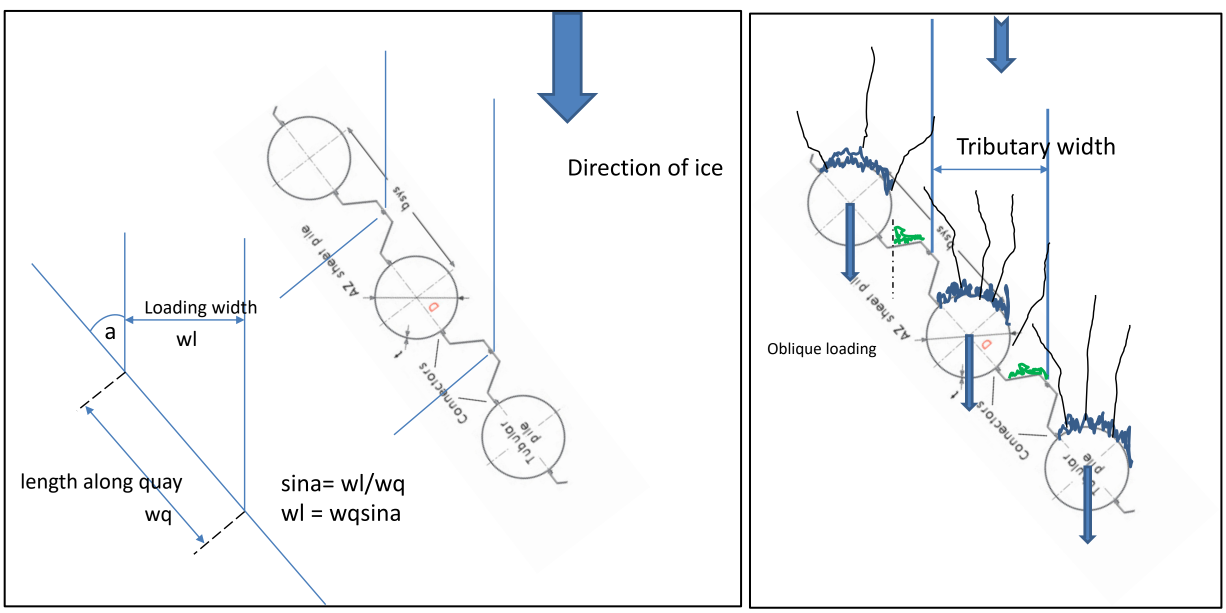


Figure 3.21: Oblique loading

Table 3.11: Oblique loading (40 degree angle of attack)

ice thickness 0.6m		1500m floe	
wind speed (m/s)	20	15	10
floe speed (m/s)	0.45	0.337	0.225
force (MN)	25.4	22.6	18.3
loading width (m)	40	34.7	27.1
line load (MN/m)	0.64	0.65	0.68
Ange of attack (deg)	40	40	40
loaded width on dock (m)	62.2	54.0	42.2

3.3.4Global load application levels

The ice loads can be assumed to act at the mid-point of the ice thickness over the full range of expected water levels. It can also be assumed that the top of the ice surface is at the water level.

3.4 Local ice loads on the quay wall

3.4.1 Overview

All test data and measurements of ice pressures on structures and ships indicate that ice crushing pressures can be very high on small areas; (but much lower on larger areas). This is due to the brittle nature of ice failure at high strain rates. So although about 1.0 MPa may be the right value for global loads, the same ice can cause ice pressures of about 4 MPa on small areas of about say 1m^2.These quoted pressures are relevant to high energy impacts such as encountered by ice breaking vessels, but such a scenario will not apply for the face of docks in sheltered areas.

The most convincing information is evidence from existing sheet pile docks in the Arctic and also in other areas with ice such as the Caspian Sea. Experience indicates that damage to sheet pile docks from ice has not occurred.

In the Arctic this could be because frozen fill will exist behind any sheet pile periphery when ice loading occurs (certainly in the winter). Frozen saturated fill has been shown to be stronger than pure ice. This enables sheet pile walls to generally avoid local damage due to high local ice pressures. Hollow piles may or may not be able to resist high local ice pressures without denting – this depends on wall thickness. One method to avoid local denting is to place concrete in the piles where local ice pressures may occur.

It can be argued therefore, that local ice pressures need not be considered for this structure. Nevertheless, it would be imprudent not to review typical potential values based on methodologies available, and then decide on a way forward.

3.4.2 Nature of local ice loads

The physics of ice crushing is complex. Figure 3.22 shows various processes within a contact zone

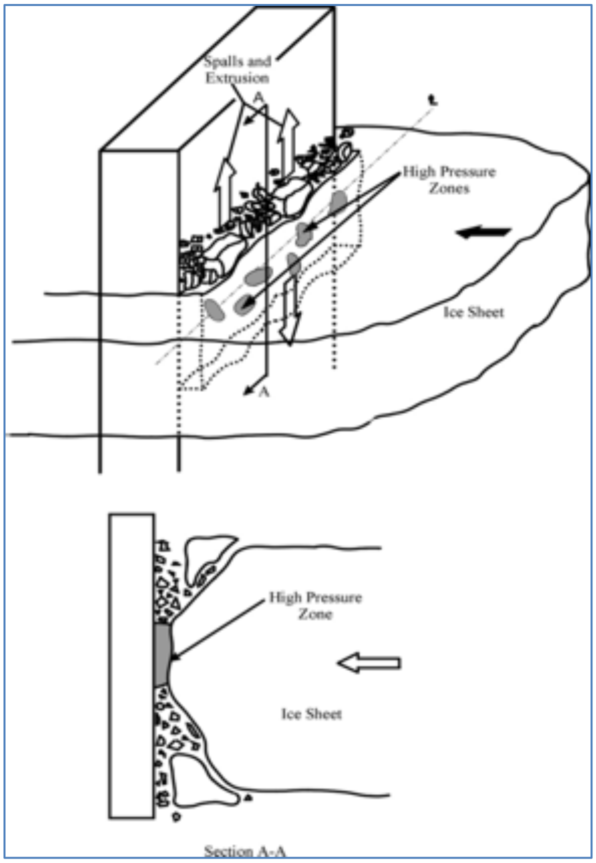


Figure 3.22: Detailed failure processes in ice crushing (Jordaan et al, 2009)

The high pressure zones grow and decay in very short time periods so they are not relevant for design. It is the average of the pressures over relevant time periods, areas and widths which are needed for design. These have been developed empirically from measurements during experiments and measurements on vessels and full scale structures. For local design the pressure is often specified as a function of area. It is important to understand the location of specified areas in relation to the ice. Figure 3.23 shows one situation. The specified area is a long way from the free boundaries of the ice feature. In other words, the design area of interest is within a much thicker ice contact zone and the ice failure is confined giving the highest pressures. This could be the case for interaction with thick multi-year ice or an iceberg.

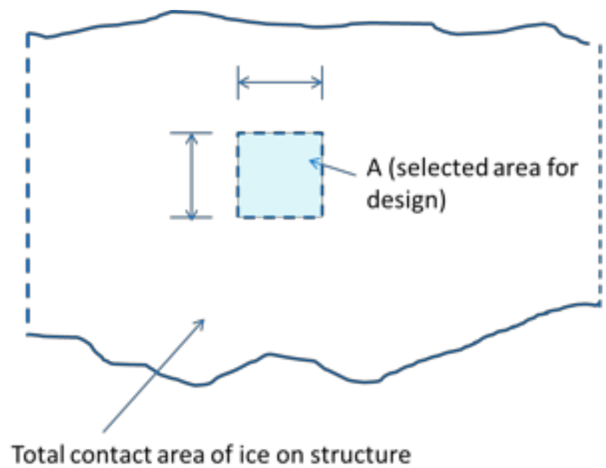


Figure 3.23: A design area confined within a larger contact area for local ice load design

For this situation, ISO 19906 suggests (for cold Arctic ice):

$$p = 7.4\ A^{-0.7}\ \text{MPa} \qquad \text{Eq. 3.4}$$

This would give a local ice pressure of 7.4 MPa on a $1m^2$.

Because this approach is for confined areas within a larger area and for high impact interactions it is considered far too conservative for this application.

The other local ice pressure situation is more suitable for level ice and is depicted in Figure 3.24. The design area of interest is close to the ice surface (even full thickness). This is a much less confined situation.

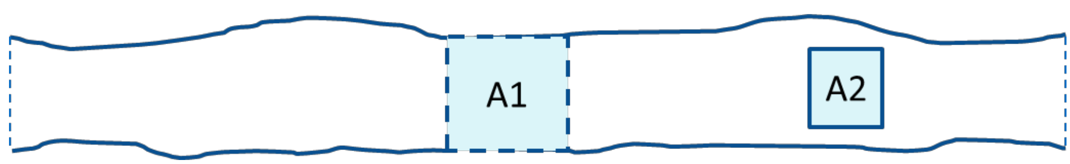


Figure 3.24: Design areas close to the free edges of the ice sheet (less confined)

A method for this situation is given in ISO 19906 based on data gathered in the Gulf of Bothnia. It gives the average full thickness ice pressure as;

$$p = 2.35\ h^{-0.50}\ \text{(MPa) (for } h > 0.35\text{m)} \qquad \text{Eq. 3.5}$$

$$p = 4\ \text{(MPa) (for } h < 0.35\text{m)}$$

where: p = the full thickness ice failure pressure (MPa) and h = the ice thickness, in m

There may be some rationale in increasing the 2.35 value when looking at Arctic structures as the data was gathered in the Gulf of Bothnia which might be considered Sub-Arctic. However, the Baltic location is almost Arctic (66 degrees north) and the ice has low salinity therefore this value will be retained. In fact, a simple approximate comparison based on brine volume suggests that the Arctic ice has a slightly higher brine volume (see the table below) and hence it is nominally slightly weaker.

Table 3.12: Comparison of typical brine volumes

	Arctic	Baltic
Ice temperature (average through thickness ^{o}C)	-15	-8
Salinity	7ppt	3ppt
Brine volume	0.02677	0.021

3.4.3 Local ice loads by scenario
The same scenarios as used for global ice loads will be used for local ice loads.

3.4.3.1 Scenario1: Freeze- up
As discussed in section 3.3.1 the critical interaction during this period will be due to the thickest ice which can occur before the ice becoming landfast. As also discussed, this was assessed as 0.5m in preliminary design, but further analysis leads to a proposed value of 0.6m. Applying Eq. 3.5 gives;

$$p = 2.35^{*} (0.6)^{-0.5} = 3.03 \text{ MPa}$$

This is due to 0.6m thick ice and can be regarded as a line load of 0.6*3.03 = 1.82MN/m

Because the empirical data came from ice pressure panels 1.2m wide, this line load can be applied to a width of 1.2m. This is close to the smallest width of interest in the design of the combi-wall.

The logic of “exposure “ should also be applied to local loads (as it has been applied to global loads). If the exposure factor of 0.61 is applied, the local line load then becomes 1.1MN/m. This compares with the global line load over the smallest width of relevance of 1.0 MN/m (see Table 3.3).

In our opinion this is close enough to suggest that the global ice load methodology can also apply to local ice loads in terms of line loads for design.

3.4.3.2 Scenarios 2 and 3; Midwinter and break up
Because Scenario 1 was found to be the most critical for global loads, it is proposed that it will also control local loads.

For the mid-winter Scenario 2, the ice is landfast and will have limited slow horizontal movements. Ice creeps at slow strain rates and high local ice loads will not develop. The global load value of 0.3MN/m can also be used for local loads. As well, an ice bustle will further spread load variations.

In Scenario 3, the ice, although thicker, will be very weak, and as shown, does not control global loads. Local line loads will also be assumed to be lower than in Scenario 1.

3.6 Ice interaction with the causeways (and rock berms)

3.6.1 Global loads on rock berms

The topic of ice action on rock protection barriers and islands has been studied extensively for the North Caspian Sea where such facilities are common. A good overview of ice interaction scenarios and potential rock slope failure modes is given in Lengkeek et al (2003). The same general approach can be used to examine global loads on the rock slopes and causeways of the proposed offloading dock.

As an overall comment, most of the rock slopes (especially the causeway to shore) will not be exposed to significant ice push because of the sheltering effects of the outer quay and the shoreline itself. Furthermore, apart from the potential for armour stones to be pushed around (see next section), past experience suggests that global failures of rock berms is unlikely because of their high resistance to lateral ice loads. Therefore the scenarios which follow are unlikely to be critical.

The potential critical failure modes of a rock berm are shown in Figure 3.25. This figure is taken from early Caspian Sea work and some of the comments on the figure are not strictly relevant to this design. The message is that global sliding and decapitation resistance need to be higher than the ice loads; but some local edge failures may be acceptable (but should also be avoided).

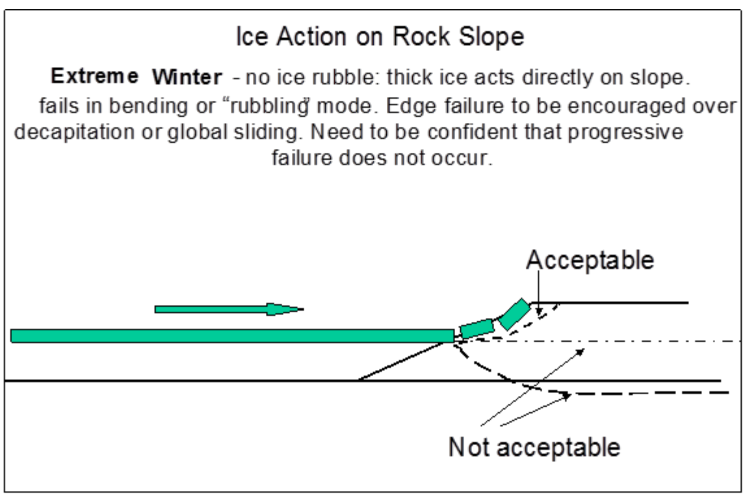


Figure 3.25: General potential failure modes of a rock berm (Croasdale, 2003)

In reality, the interaction mode depicted of thick winter ice acting directly on the slope is unlikely once we use the same periods of ice loading as previously used for ice loads on the quay. These now follow. The approach for global loads will be to define some bounding interaction cases for which the rock slopes and causeway can be checked for overall stability and potential slope failures.

3.6.2 Scenario 1: Early winter with mobile ice up to 0.6m thick

The first bounding case will be to assume the 0.6m thick ice acts directly on the rock slope and fails in bending with some ride up and rubble creation. This is shown conceptually in Figure 3.26.

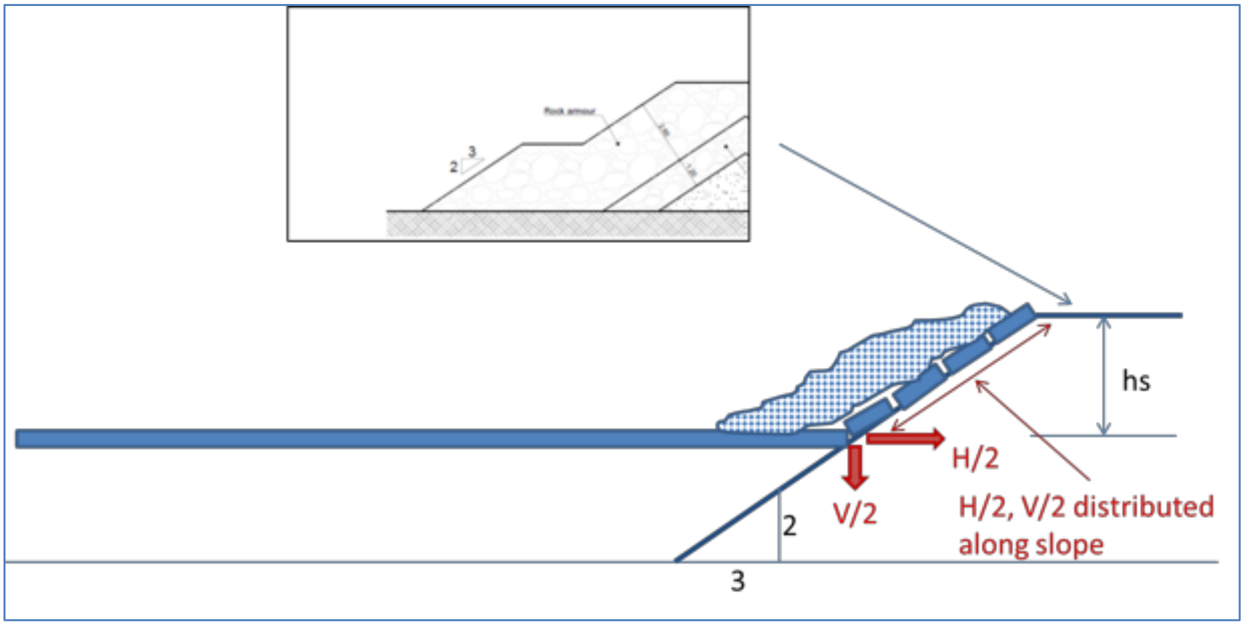


Figure 3.26: Interaction of level ice on a rock slope – idealized to a uniform slope with ice failing in bending with ride up and rubble formation

The ice loads can be calculated using the method for ice action on a sloping face (ISO 19906 and Croasdale et al, 1994, 2016). In this case the slope angle is 34^0 and an ice thickness of 0.6m will be used. Table 3.13 gives the other detailed inputs and outputs (from a KRCA spreadsheet). The calculated value of H is 0.16 MN/m. It should be noted that if the rubble angle of repose is chosen to equal the slope angle it implies a single layer of ice ride up. If as in many situations ice rubble after riding up the slope tumbles back on the ice riding up, then the total global load is higher because ice has to be pushed through the ice rubble. In many cases the rubble is not a linear slope but can be flat near the top with a steeper slope lower down. The angle of repose in the spreadsheet is for an idealized linear slope and is chosen to give a plausible volume of ice rubble on the slope. Later where we are examining encroachment distances for ice rubble, linear slopes are again chosen but based on observations will be steeper than the 24 degrees used in Tables 3.13 and 3.14.

Table 3.13: Early winter level ice (0.6m) acting on rock slope and failing in bending; slope height is 4m

Input Data in blue: Derived values in black and results in green		**rock slope base case - winter 0.6m ice**
Flexural strength of ice	(kPa)	**500**
Specific weight of ice	(kN/m^3)	**8.98**
Specific weight of water	(kN/m^3)	**10.20**
Young's modulus	(kPa)	**5.00E+06**
Poisson's ratio		**0.3**
Slope Angle	(deg)	**34**
Rubble angle of repose	(deg)	**24.0**
Rubble friction angle	(deg)	**0**
Waterline width (D)	(m)	**30**
Slope height (hs)	(m)	**4**
Width for HT (Dt)	(m)	**30.00**
Width for centre ride up	(m)	**30.00**
Ice-slope friction		**0.5**
Ice-ice friction		**0.25**
Thickness for ride up	(m)	**0.6**
Thickness for breaking (hb)	(m)	**0.6**
Non simultaneous factor for HB		**1**
Non - sim factor for breaking in ride up limit		**1**
Angle difference for ride up load	(deg)	**5**
Rubble porosity		**0.15**
Cohesion of rubble	(kPa)	**5**
m - fraction of h to be added to slope ride up (abnormal)		**0.1**
Characteristic length (Lc)	(m)	9.92
Ratio of structure width to characteristic length		3.02
First break length (R)	(m)	7.74
wB	(m)	54.32
Limit avg ride up height + hb/2	(m)	**3.00**
Second break length (R2)	(m)	3.4
Third break length	(m)	1.5
Block length ratio to thickness (2nd break) (n)		5.6
y (second break) - has to be less than h	(m)	0.77
Results		
Predicted Horizontal Load (MN)		6.37
Predicted Horizontal Load less HT (MN)		4.82
Horizontal line load (MN/m)		**0.16**
Vertical line load (MN/m)		**0.09**

The next bounding case is to assume that ice rubble has built up significantly on the slope and the ice failure process shifts to the outer face of the rubble. The failure process there is independent of the slope angle however the full ice force is transmitted to the slope face. This is shown in Figure 3.27.

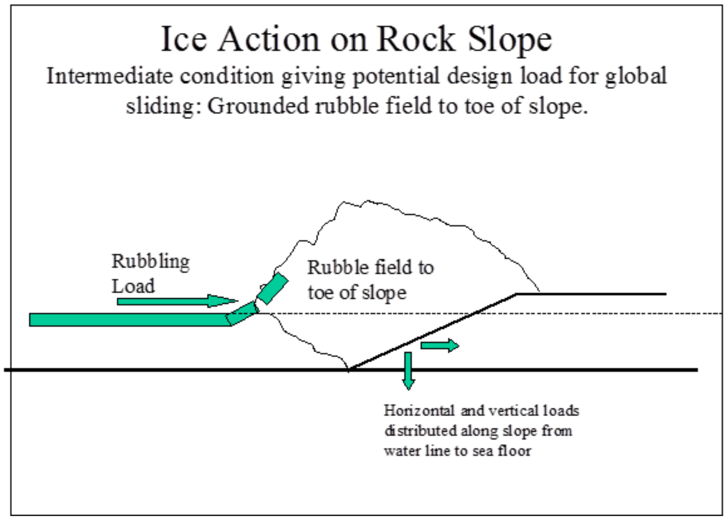


Figure 3.27: Bounding case for ice loads on rock slope with significant ice rubble

In this case the ice rubbling force is calculated using ice load formula given in Palmer and Croasdale, (2013). That is;

$$w=4.8(L^{-0.5})(h^{1.1}) \qquad \text{Eq. 3.6}$$

Where w is the horizontal line load, L is the width and h is the ice thickness. In this case, we will use a nominal width of 30m (the rubble is assumed to spread the load over this width) and h is 0.6m. Hence w is calculated at 0.5MN/m. This load is accompanied by a vertical load which is the weight of the rubble and is given by:

$$V = w\frac{\cos\alpha - \mu\sin\alpha}{\sin\alpha + \mu\cos\alpha} \qquad \text{Eq. 3.7}$$

Where V is the vertical line load, is the slope angle and is the friction on the slope.

In this case V is calculated as 0.28MN/m.

Both w and V can be assumed to be evenly distributed along the slope from the water line to the top of slope.

This scenario requires a significant ice rubble build up such that the weight of ice rubble on the slope resists the pushing forces. As will be discussed later under the topic of ice encroachment, there are not sufficient available driving forces to create this situation. It is therefore dismissed as a load case.

3.6.3 Scenario 2: Mid-winter with stable ice up to 2.05m thick.

In this period the ice will likely be frozen to the rock slope and there will low loading caused only by thermal and tidal jacking. It is likely that the vertical motion due to tides will occur a few m from the slope at a tidal crack. The loading should be assumed the same as for global loads on the dock of 0.3MN/m.

3.6.4 Scenario 3: Break -up

In most years the ice will melt in place with little ice action. However, it is possible (as with global loads on the dock face) that large offshore floes released during the break up period could be blown inshore and interact with some parts of the rock slopes. Therefore, the same loading situations are assumed in as Scenario 1 except that the ice is thicker but considerably weaker.

For ice acting directly on the rock slopes and failing in bending, the same spreadsheet is used and results added for this period with a thickness of 1.4m but weaker ice (50kPa flexural strength – from the Timco plot in Figure 3.8). The results are shown in Table 3.14. The loads are higher with values of 0.30MN/m for H and 0.17MN/m for V.

Table 3.14: Break up case added for level ice acting directly on a rock slope

Input Data in blue: Derived values in black and results in green		rock slope base case - winter 0.6m ice	break up 1.4m weak ice
Flexural strength of ice	(kPa)	500	50
Specific weight of ice	(kN/m^3)	8.98	8.98
Specific weight of water	(kN/m^3)	10.20	10.20
Young's modulus	(kPa)	5.00E+06	5.00E+05
Poisson's ratio		0.3	0.3
Slope Angle	(deg)	34	34
Rubble angle of repose	(deg)	24.0	24.0
Rubble friction angle	(deg)	0	0
Waterline width (D)	(m)	30	30
Slope height (hs)	(m)	3	3
Width for HT (Dt)	(m)	30.00	30.00
Width for centre ride up	(m)	30.00	30.00
Ice-slope friction		0.5	0.5
Ice-ice friction		0.25	0.25
Thickness for ride up	(m)	0.6	1.4
Thickness for breaking (hb)	(m)	0.6	1.4
Non simultaneous factor for HB		1	1
Non - sim factor for breaking in ride up limit		1	1
Angle difference for ride up load	(deg)	5	5
Rubble porosity		0.15	0.15
Cohesion of rubble	(kPa)	5	5
m - fraction of h to be added to slope ride up (abnormal)		0.1	0.1
Characteristic length (Lc)	(m)	9.92	10.53
Ratio of structure width to characteristic length		3.02	2.85
First break length (R)	(m)	7.74	8.22
wB	(m)	54.32	55.82
Limit avg ride up height + hb/2	(m)	3.00	3.00
Second break length (R2)	(m)	3.4	3.6
Third break length	(m)	1.5	1.5
Block length ratio to thickness (2nd break) (n)		5.6	2.5
y (second break) - has to be less than h	(m)	0.77	0.37
Results			
Predicted Horizontal Load (MN)		6.37	12.77
Predicted Horizontal Load less HT (MN)		4.82	8.92
Horizontal line load (MN/m)		0.16	0.30
Vertical line load (MN/m)		0.09	0.17

3.6.5 Summary loads on rock berms

A summary of these loads is given in Table 3.15. These are on a nominal width of 30m. It is assumed that stability calculations will be done on a linear basis. So limits to actual widths are not imposed. However when looking at limits to ice encroachment it is noted that limited kinetic energy will likely limit the widths of ice that can be driven to the top of the causeway to about 30m.

Table 3.15: Line loads on the exposed rock slopes (30m width)

Scenario	Bending failure and ride up	
	Horizontal MN/m	Vertical MN/m
Scenario 1: Freeze-up (0.6m strong ice)	0.16	0.09
Scenario 2: Break-up (1.4m weak ice)	0.30	0.07

3.6.2 Ice action on rock armour

3.6.2.1 Overview and background

Individual rocks may be expected to be dislodged by the ice from time to time as it is almost impossible to place large enough rocks to resist local ice loads due to their weight alone. Furthermore, the tidal action may result in rocks being lifted once they are surrounded by ice (although it is expected that the tidal crack will be some metres away – see Section 5). Annual monitoring is recommended for the first few years and some periodic maintenance may be required. Experience with shore protection and breakwaters in many ice regions shows that large rocks have been moved by ice; see the Caspian example in Figure 3.28. Experience with underwater rock berms to protect pipelines also indicates that rocks can be lost due to ice effects. (Especially if grounded rubble transmits the ice loads from the surface to the berm).

Figure 3.28: Armour stone displaced by ice (drilling island in the North Caspian Sea)

Guidelines were developed for the North Caspian Sea and are quoted in ISO 19906 (new version) as

Quote:

"Reference [255] provides three rules of thumb to design a rock-armoured breakwater for ice conditions similar to the North Caspian Sea.

a) The minimum crest freeboard should be twice the maximum ice thickness. A higher crest is not effective as a method to prevent local failure. In addition, to prevent progressive failure, the fill should not be exposed to the open water (waves) and ice, and an armour layer is required.

b) The thickness of the armour layer should be at least equal to the maximum ice thickness.

c) The rock size should be at least half the maximum ice thickness. "

Caution is expressed with regard to extrapolating these guidelines to other regions; they perhaps represent minimum requirement recommendations.

Furthermore, guideline c) is open to debate based on other work. For example see the quote below by Torum.

7.3.6 Design of ice protection rock rubble mound barriers in the Caspian Sea

Lengkeek et al. (2003) dealt with ice attack on ice protection barriers in the Caspian Sea. Their primary concern was the geotechnical stability of the rubble barrier. They state however that the cover layer should consist of two layers or more of armour rock and that the rock size diameter should be about half the ice thickness. This is somewhat contradictory to the results of Sodhi et al. (1996), Figure 47. The results of Sodhi et al. indicate that the rock size diameter should be 1 – 2 times the ice thickness. This is, may be, an indication of the poor knowledge that exists on the requirement to the rock size on rock slopes subjected to ice loads.

3.6.2.2 Recommended approach

The recommended approach is as follows:

Rock for Arctic applications is generally first sized for waves, then checked for resistance to damage from ice action, and finally increased in size to resist damage from ice action if needed. A number of steps are required in the sizing of riprap or armour rock, which are provided below, and finally an example is provided using the largest rock slope protection used at the existing dock. The design steps are as follows:

1. The first step is to size the rock using the Hudson's equation in the US Army Corps of Engineers "Shore Protection Manual (SPM)", 1984, or other more recent formulations such as by Van der Meer, for example. Hudson's equation is:

   $W_{50} = \gamma_r (H_S)^3/(K_D\Delta^3\cot\theta)$, in kg, where

γ_r = density of rock in kg/m^3

H_S = significant wave height, in m

K_D = dimensionless stability coefficient (approx. 2.2 for three layers of angular rock)

Δ = dimensionless relative buoyancy of rock, (ρ_r / ρ_w - 1)

ρ_r = density of rock in kg/m^3 (typically a minimum of 2,650 kg/m^3)

ρ_w = density of water in kg/m^3 (approx. 1,020 kg/m^3, depending on salinity)

cotθ = cotangent of the slope angle, θ, with respect to horizontal

2. The rock size is then checked for damage under ice action at the maximum thickness before freeze-up, using D.S. Sodhi, S.L. Borland and J.M. Stanley "Ice Action on Riprap", USACE, CRREL Report 96-12, September, 1996.

3. The rock size may be increased at this step to limit the damage to an acceptable amount, based on the design ice thickness.

4. Plucking of rocks off the slope from ice movement at break-up can be evaluated to first order only. There is very limited information on this, as given in A. Torum “Breakwaters in Arctic Areas, Review and Research Needs”, Norwegian University of Science and Technology, Report No. BAT/MB-R2/2004, December 14, 2004. Torum refers to other publications where it is stated that to avoid plucking, the stone size D_{50} should be at least be equal to the ice thickness, and that it takes a certain ice sheet size to bear the weight of a rock, and as an example “consider a stone diameter D_{50} = 1.5 m with a mass density of 2,700 kg/m^3, or W_{50} = 9,112 kg, and a 1.5 m thick ice with mass density 950 kg/m^3 - the required ice sheet area to carry this stone is approximately 186 m^2. Such an ice floe would necessarily be in contact with several stones and it is questionable if plucking may occur for such stones.”

   Plucking will be calculated using first order methods in terms of required shear, tensile and flexural strengths of a weak ice sheet at break-up, to lift the rocks.

5. For the final step, a detailed gradation of the rock is specified using riprap or armour gradations as per the SPM.

An example is the Type A Armour used at the existing dock, where the mean size, D_{50}, is 0.91 m cubic size as shown on the design drawings. This armour is placed on a 1.5H:1V slope, where H is the horizontal component of the slope for 1 unit of vertical rise V, and so cotθ = 1.5. The mass of the mean size of rock in this case is $W_{50} = \gamma_r * (D_{50})^3$, where γ_r mass is the dry density of rock, typically a minimum of about 2,650 kg/m^3 for good quality armour rock resistant to break down from freeze-thaw and abrasion from ice and wave action.

The 1 in 100 year significant wave height is 2.19 m as given in Attachment C - Metocean Data - H353004-CG001-200-242-0031 by Hatch, December 14, 2016, and the rock size calculated using Hudson's formula is D_{50} = 0.91 m, which agrees with the design drawings.

Mass governs in specifying a gradation for the rock, and for Type A Armour, the mean mass is:

W_{50} = 2,650 kg/m^3 * (0.91 m)^3 = 1,997 kg, or about 2 tonnes.

Damage to a rock slope for varying slope angle is given in the following figure from Sodhi et al, which is based on spherical rock size:

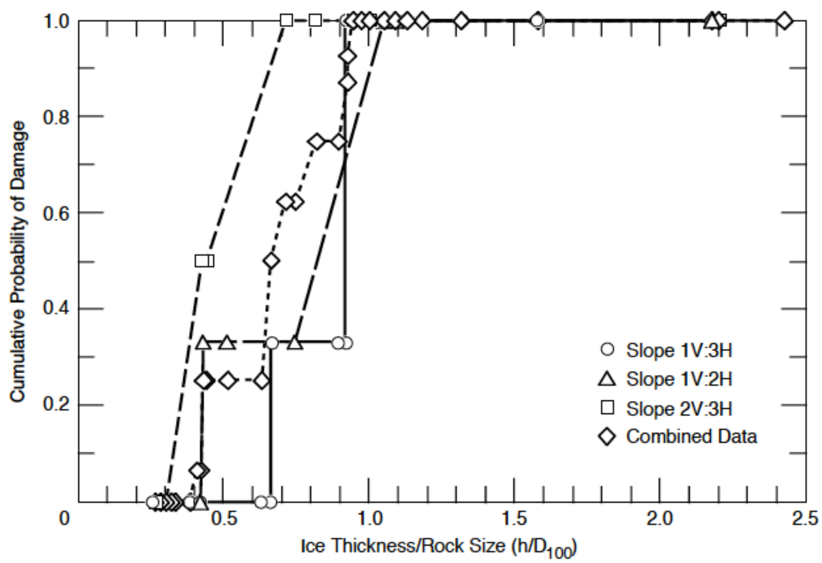


Figure 31. Plot of cumulative probability of damage vs. the ratio of ice thickness to D_{100} rock diameter.

Using the figure above, the following ice thicknesses have been determined for three levels of damage:

- No damage for $h_i < 542$ mm
- 50% damage for $h_i = 813$ mm
- 100% damage for $h_i > 1265$ mm

For this case there will be no damage for ice thickness of 542 mm. If, for example, the ice thickness of mobile ice at freeze-up is h_i = 600 mm, then a slightly flatter slope or a D_{50} rock size of 1000 mm may be considered.

However, it should also be recognized that the 600mm thickness is that associated with the 100 year event. The mean thickness just prior to freeze-up when the ice can be mobile is estimated to be 320mm, and this will cause no damage based on the above methodology

4.0 Ice encroachment

4.1 Nature of ice encroachment with examples

When ice moves against low freeboard structures there have been incidents of ice encroachment onto the working surface. The process has been observed and studied in the Arctic, the Great Lakes, Russia and the Caspian Sea. Damage to shoreline structures has been reported and is part of the oral tradition of some indigenous peoples. Ice encroachment is not treated in any quantitative way in ISO 19906. The following is based on Palmer and Croasdale, (2013).

Ice encroachment is the term given to ice moving onto the surface of a platform. The risk of ice encroachment is inversely proportional to the freeboard of the platform, but water depth also plays a role; the shallower the water in front of a structure, the more likely is ice encroachment. There are two kinds of ice encroachment; ice over-ride and ice pile-up.

An example of ice over-ride in the Caspian Sea is shown in Figure 4.1. During an ice movement event, ice about 0.5 m thick started to rubble directly against the vertical-faced quay. After a few minutes, advancing ice climbed over the ice rubble and rode across the perimeter of the island. It was stopped by some equipment which was not damaged but gave enough resistance to trigger instabilities in the ice blocks at the island perimeter.

Figure 4.1 Ice over-ride incident in the Caspian Sea

Steeper sloped islands (and those constructed from rock) favour ice pile up rather than ride up. An example of high ice piles up are shown in Figure 4.2. In these cases, an ice encroachment perimeter may be a reasonable solution. No permanent facilities are placed in this perimeter allowance, but it could be a perimeter road. The width of this perimeter depends on an assessment of risk. For example, in the Caspian Sea a distance of 10 -15 m is normally used. Figure 4.3shows the principles of how the width of the perimeter should be calculated for various island freeboards and slopes. An alternative to the perimeter strip is to raise the surface (or deck) above the height of natural pile-ups seen in the region, or the top of an ice deflector should be at this height.

Figure 4.2: Example of pile-ups at perimeters of Caspian Sea platforms

The equation for the width of the ice encroachment management strip is based on simple geometry. Using the parameters defined in Figure 4.4 the width of the perimeter strip (w_p) is given by.

$$w_p = \frac{h_p - y}{\tan\theta} - w_t \qquad [4.1]$$

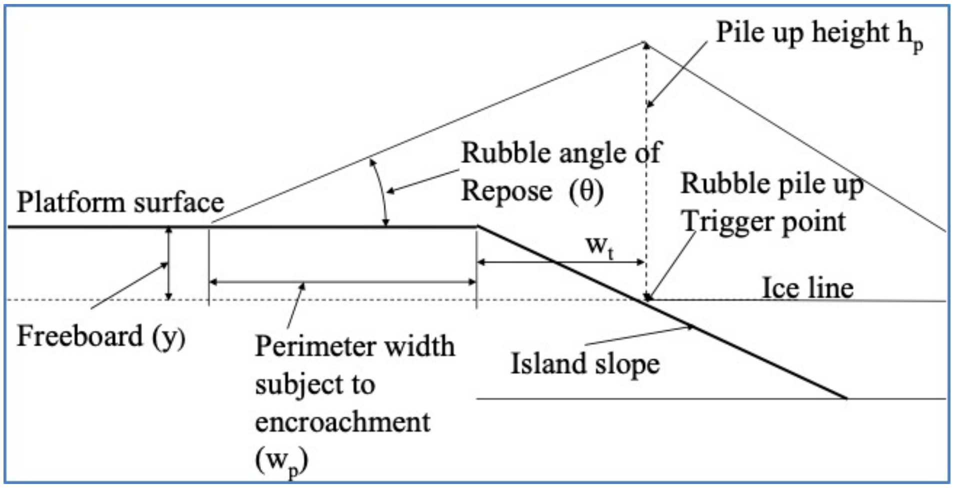


Figure 4.3: Scheme to specify edge perimeter allowance for ice encroachment due to ice pile-up.

4.2 Relationships between pile up heights and thickness (shallow water platforms)

The two main ingredients for an ice pile up are 1) an obstacle (such as a platform or a dock facility or a shoreline) and 2) an ice movement. The dock and causeways create an obstacle and ice movements can be expected both in the early winter and later at break-up.

Extreme ice pile ups will not occur with every ice motion event. As observed at several locations where pile ups occur, the ice pile up may decide to grow outwards in “waves” of ice rubble rather than continuing to build up the initial pile up to great heights. It is presumed that growth of waves of ice rubble requires less energy and driving force than a high pile up. Therefore extreme pile ups immediately adjacent to the structure will not always occur even with extensive ice motion. As will be discussed in later, water depth in front of an obstacle will also play a role; the deeper the water the less likely is encroachment. In shallow locations, experience in the Caspian Sea and other locations confirms that high pile ups can occur adjacent to structures with sufficient frequency to create potential ice encroachment – see McKenna et al (2011).

A typical relationship between pile up height in the Caspian and ice thickness is shown in Figure 4.4.

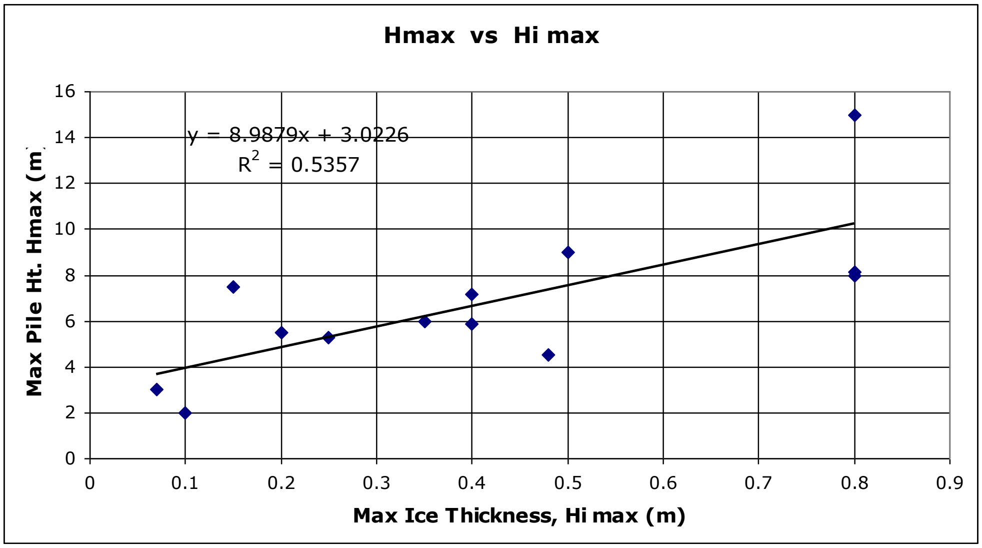


Figure 4.4: Example of data from the Caspian Sea

McKenna et al (2011) report that based on data from 27 rubble building events against structures in the north Caspian Sea, the maximum height in a rubble field can be sampled from the average ice thickness as

$$H_{max} = 4.32 + 9.80\, h + N(0,2.03) \quad [4.2]$$

Where, $N(0,2.03)$ is a normal distribution with a mean of zero and a standard deviation of 2.03. Equation 4.2 refers to maximum pile-up heights anywhere in a rubble field against a structure. That work also suggested a factor of 0.5 between maximum rubble height and the pile up heights at the structure.

Another good source of information for the effect of ice thickness on rubble height at structures is the compilation by Timco and Barker (2002). Some of the data are from the Caspian Sea, while the remainder are from the Baltic and the Beaufort.

A linear correlation from this data is

$$H_{max} = 2.86 + 8.86\, h \quad [4.3]$$

However, there appears to be a levelling off with thicker ice; on the other hand there is limited data for the thicker ice.

4.3 Predicted potential ice encroachment on the rock causeways

The regions of the facility exposed to ice encroachment are those facing offshore - as depicted in Figure 4.5. In terms of rock berms these are the causeways on each side of the dock face

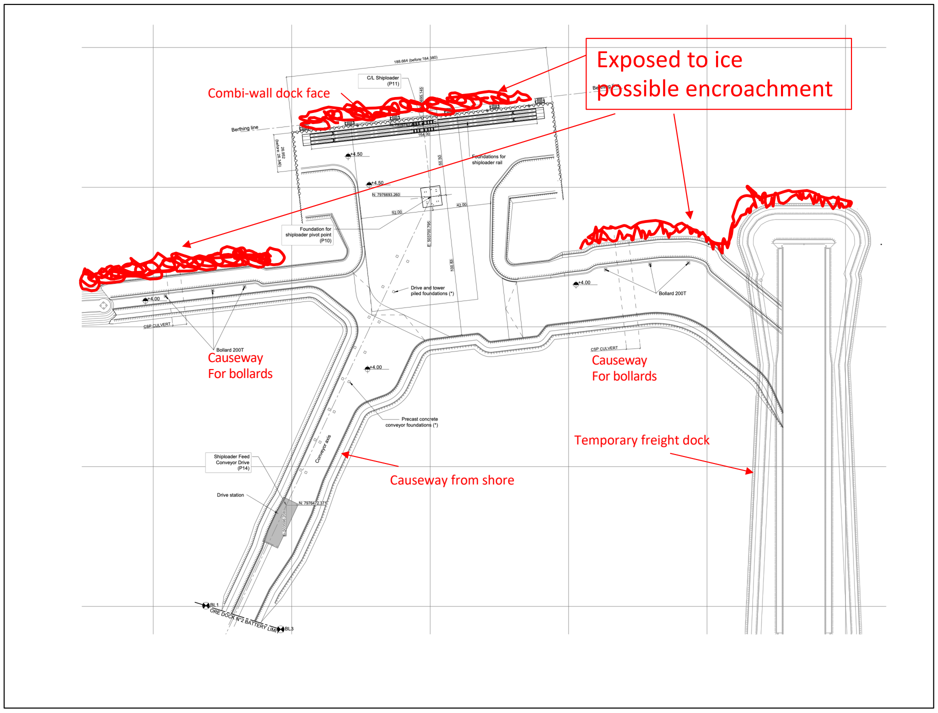


Figure 4.5: Faces/causeways exposed to potential ice encroachment

The experience and empirical data for shallow water structures can be applied directly as much of the data is for structures in 2 – 4m of water which is similar to the effective water depths on the exposed slopes of the rock berms.

During Period 1 (Scenario 1), maximum ice thickness prior to the ice becoming landfast is derived as 0.6m. Inserting this in Equation 4.3 gives a pile up height of about 8m. Using Eq 4.2 with the factor of 0.5 for pile up height at the structure gives about 7m.

To achieve such a rubble height, there have to be two other ingredients; a driving force which would push the ice past the dock face to contact the rock berm and continue with enough force and distance to create enough volume of ice to create the pile up to the heights suggested above (say 8m) and to the geometry indicated in Figure 4.3.

The driving forces from wind and current have already been reviewed in Section 3 when looking at ice forces. The situation shown in Figure 3.13 was examined. A 2km ice sheet (about the largest that can be fitted into the inlet) is being driven by wind or current (or both). A spreadsheet calculation is tabulated below for a very extreme wind of 30m/s acting on the 2km floe.

Table 4.1: Wind drag driving forces on large floe pushing against dock facility.

Wind Drag					
air density	1.275	kg/m3	Dock width	189	m
Ca	0.002				
m/s	m	MN		MN	MN/m
wind	Equiv.Floe size (W)	Drag	Fraction taken by shore	Load on Dock	Line load across full dock
5	2000	0.26	0.5	0.13	0.001
10	2000	1.02	0.5	0.51	0.003
20	2000	4.08	0.5	2.04	0.011
30	2000	9.18	0.5	4.59	0.024
5	2000	0.26	0	0.26	0.001
10	2000	1.02	0	1.02	0.005
20	2000	4.08	0	4.08	0.022
30	2000	9.18	0	9.18	0.049

It can be seen that the largest force in the extreme case of all the force acting on the dock face is 9.18MN. If we refer to the required force to fail the ice against the dock derived in Section 3 for scenario 1, we see that it is about 94MN (Table 3.6). Clearly there is not enough wind force to create such a load and push the ice past the dock face. Even if we add a current drag acting together with the wind, as shown in Table 3.8 the additional driving force will only be about 1 to 2 MN.

Even in a dynamic situation with a large floe moving into the dock with an impact speed of 0.3m/s, it has been shown in Section 3 (Table 3.9) that the distance to stop the floe is typically less than 10m.

Noting these numbers, it is impossible to get a situation leading to any encroachment onto the exposed rock berms from free floating large ice sheets because they will be stopped before reaching the rock causeways. If ice is already in contact with the rock slopes, the same logic applies except that the driving forces also have to overcome the resistance of the shoreline. There is simply is not enough driving force to move the ice up the shoreline and against all the facilities in which the ice is in contact.

The only possible situation would seem to be from smaller floes that can avoid contacting the dock face and act directly on the rock berms (as depicted in Figure 4.6).

Figure 4.6: Possible situation for ice sheets to directly contact the rock berms without being stopped by the dock face

Even in this case, there are facilities either side of the causeways (such as Dock 1) which will resist the ice from being driven too far up the causeway slopes.

For typical loads, the smaller floe areas are about say 1300 x 500m = equivalent to 800x800m square floe. This can be substituted in the drag calculation; see Table 4.2.

Table 4.2:

Wind Drag		
air density	1.275	kg/m3
Ca	0.002	
m/s	m	MN
wind	Equiv.Floe size (W)	Drag
5	800	0.04
10	800	0.16
20	800	0.65
30	800	1.47

The typical extreme maximum force is about 1.5MN. Is this enough to generate a pile up ?

Forces on the rock berm slopes have been investigated earlier. For the initial interaction with the ice failing in bending and riding to the top of the slope with some rubble on top of the ice requires about 0.16MN/m. Realistically, the width of interaction on a causeway in the situation shown in Figure 4.8 will be over 10 – 20m. At 20m the total force required is about 3.2MN. It can be seen that an 800m floe even with a wind speed of 30m/s acting on it, does not generate enough driving force. The interaction width would have to be less, perhaps a small floe in front of the larger one is a possible scenario.

The more mature situation with the ice failing in rubbling and building up a rubble pile requires a force of 0.5MN/m. Even over only a 10m width this requires a force of 5MN. Yet the available driving force for a 30m/s wind on the ice is only about 1.5MN.

The prior values were for ice 0.6m thick, what about thinner ice ? The driving forces are the same but the ice failure forces are less – although creating a pile up of a certain height requires roughly the same amount of energy. An ice thickness of 0.2m will be examined as representative of thinner ice during Scenario 1.

Using the crushing formula, the force to fail 0.2m ice across the whole dock width is still 45MN. Therefore, the same argument applies that there is insufficient driving force to fail the ice across the dock width which is a condition for a large floe to reach the causeways to create any encroachment.

For the smaller floes which may sidetrack the dock face the forces to fail the thinner ice in bending and push it to top of the slope will be lower for this thinner ice than for 0.6m of ice. The spreadsheet shown in Table 3.8 has been re-run for the 0.2m ice. Results are shown in Table 4.3 which also shows the 0.6m thicker ice for comparison. The line load is reduced from 0.16MN/m for 0.6m thick ice to 0.09MN/m for 0.2m thick ice. Over a possible 20m width of the causeway, the load to create the ice ride up is then about 2MN. The driving force on an 800 m floe which might bypass the dock face and act just on the causeway over this limited width was shown in Table 4.2 to be about 1.5MN, so again, even a 0.2m thick ice sheet may not reach the top of the rock slope

Table 4.3: Loads to fail ice against causeway slopes and ride up to top of slope (3m freeboard)

Input Data in blue: Derived values in black and results in green		rock slope base case - winter 0.6m ice	rock slope 0.2m ice
Flexural strength of ice	(kPa)	500	500
Specific weight of ice	(kN/m^3)	8.98	8.98
Specific weight of water	(kN/m^3)	10.20	10.20
Young's modulus	(kPa)	5.00E+06	2.00E+06
Poisson's ratio		0.3	0.3
Slope Angle	(deg)	34	34
Rubble angle of repose	(deg)	24.0	24.0
Rubble friction angle	(deg)	0	0
Waterline width (D)	(m)	30	30
Slope height (hs)	(m)	3	3
Width for HT (Dt)	(m)	30.00	30.00
Width for centre ride up	(m)	30.00	30.00
Ice-slope friction		0.5	0.5
Ice-ice friction		0.25	0.25
Thickness for ride up	(m)	0.6	0.2
Thickness for breaking (hb)	(m)	0.6	0.2
Non simultaneous factor for HB		1	1
Non - sim factor for breaking in ride up limit		1	1
Angle difference for ride up load	(deg)	5	5
Rubble porosity		0.15	0.15
Cohesion of rubble	(kPa)	5	5
m - fraction of h to be added to slope ride up (abnormal)		0.1	0.1
Characteristic length (Lc)	(m)	9.92	3.46
Ratio of structure width to characteristic length		3.02	8.67
First break length (R)	(m)	7.74	2.70
wB	(m)	54.32	38.48
Limit avg ride up height + hb/2	(m)	3.00	3.00
Second break length (R2)	(m)	3.4	1.2
Third break length	(m)	1.5	0.5
Block length ratio to thickness (2nd break) (n)		5.6	5.8
y (second break) - has to be less than h	(m)	0.77	0.70
Results			
Predicted Horizontal Load (MN)		6.37	2.80
Predicted Horizontal Load less HT (MN)		4.82	2.62
Horizontal line load (MN/m)		0.16	0.09
Vertical line load (MN/m)		0.09	0.05

The comparisons of driving forces and ride up forces so far have been for static equilibrium. It is also necessary to look at whether the kinetic energy of a moving floe that by-passes the dock and acts directly on the causeways can create any encroachment.

The 0.6m thick case shown in Table 4.3 will be examined as though it was a moving floe. In this case we will take the floe as being equivalent to one of those shown in Figure 4.8. That is equivalent to an 800m floe in terms of mass, but having a narrow tongue at the leading edge of about 30m in width (to match the forces shown in Table 4.3). The final load when the ice gets to the top of the slope is 4.8MN say about 5MN.

The initial KE of such a floe is given in the spreadsheet shown in Table 4.4. It is about 19MN-m. We can do a simple energy balance with these numbers to get the distance the ice moves up the slope (x) before it stops.

Work done by ice force = 5x/2 MN-m

Initial KE = 19 MN-m

Therefore, x= 2(19)/5 = 7.6m

We can also do an equivalent calculation with the spreadsheet set up for ice crushing on a vertical face. This is done in Table 4.4. The strength factor is adjusted to get a final load of about 5MN; the local radius at the front of then floe is adjusted to match the width of about 30m (for which the 5MN load was calculated in Table 4.3. This has been done and the results are shown in Table 4.4. In this pseudo calculation, the stopping distance is 6m – reasonably close to the 7.6m using a simple mechanics approach.

Table 4.4: Simulation of equivalent interaction of a moving floe with a sloping berm: 0.6m ice

Input Data:		
Structure width (D) (m)		189
IceThickness (m)		0.6
Ice thickness at edge (m)		0.6
Floe size (L) (m)		800
Ice Density (kg/m^3)		900
Initial floe speed (v) (m/s)		0.3
Added mass factor		1.2
Local radius of floe at contact - r		25
m		-0.16
n		-0.38
CR		2.8
Exposure factor		0.61
Strength factor		0.25
Wind speed (m/s)		20
Drag coeff		0.002
Drag force (MN)		0.65
Initial 'KE (MN-m)		18.66
Pen. Incr. (m)		0.25
Mass + added mass (mln. kg.)		414.72
Maximum Impact Load (MN)		**5.3**
Maximum contact width (m)		**32.5**
Distance to stop (m)		**6.0**
Time to stop (s)		**30.6**
Line load (MN/m)		**0.16**

A typical cross section of the exposed causeways is shown in Figure 4.7. A ride up of about 8m of ice is shown from MSL. Figure 4.8 shows a possible scenario of a similar ice movement distance from HAT:

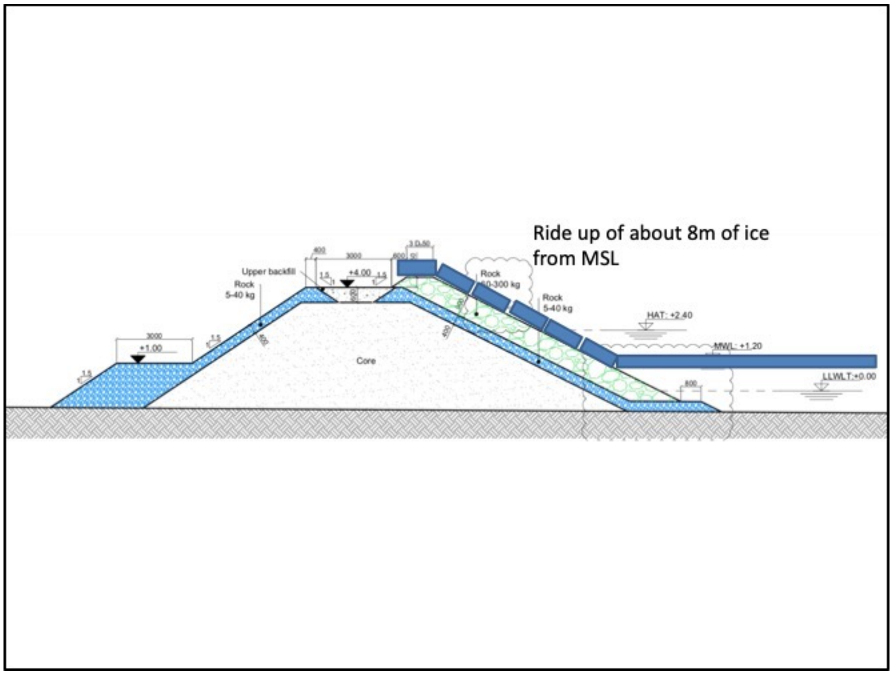


Figure 4.7: Cross section of the wing causeways with ride up of 0.6m ice from MSL (due to 800m floe moving at 0.3m/s acting over a 30m width of the berm)

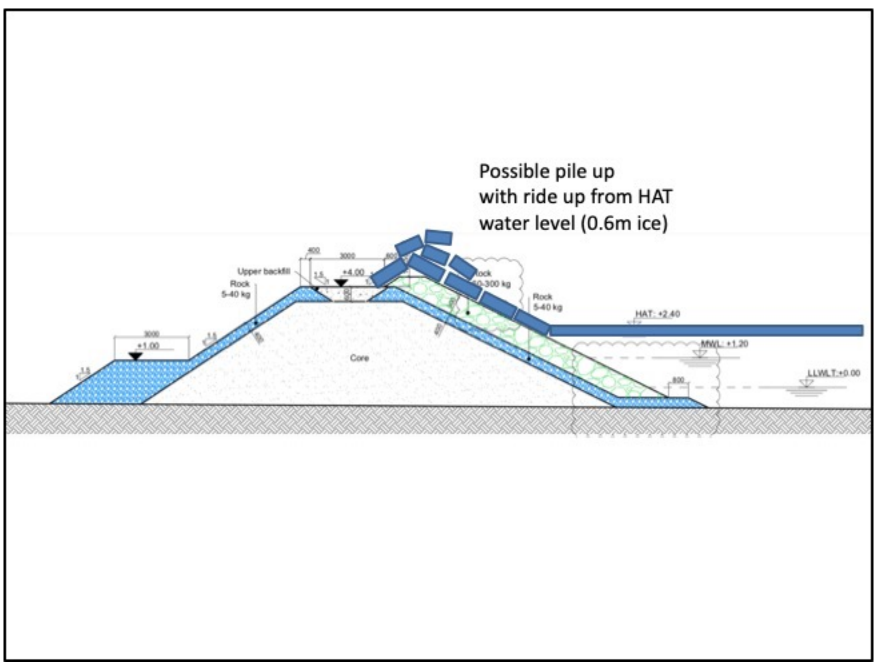


Figure 4.8: Ride up and possible pile up of 0.6m ice from HAT

Based on the above review, some limited ice ride up and pile up may occur, but only when a combination of ingredients occur. A freely moving floe is required (not a static ice sheet); this floe has to be of such a shape as to miss contacting the dock yet have a mass equivalent to an 800m square floe, 0.6m thick. It will be stopped in about 8m which from MSL means the ice will ride up to the top of the slope of the causeway (in this example over a 30m width). From a higher water level the ice may tumble onto the causeway surface where a pile up will likely start to build, but the volume of ice is limited to the equivalent of about a 10m ice movement (after which the floe is stopped).

The same approach can be examined for large ice floes at break up.

A typical calculation has been done for what would likely be the largest floe that could be driven against the dock (1.5km) and looking at the approximate penetration of past the dock face before it is brought to rest. The results are shown in Table 4.5.

The calculations show that a floe 1.5km in size (1.4m thick) moving into the dock at a speed of 0.3m/s with a wind of 20m/s driving it, will be brought to rest in about 40 seconds with a relative penetration on the 189m wide dock of only about 10m. This means that for a head-on collision the floe edge will not contact the rock berms.

Table 4.5: Simulation of equivalent interaction of a moving floe with a sloping berm: 1.4m ice

Structure width (D) (m)	189
IceThickness (m)	1.4
Ice thickness at edge (m)	1.4
Floe size (L) (m)	1,500
Ice Density (kg/m^3)	900
Initial floe speed (v) (m/s)	0.3
Added mass factor	1.2
Local radius of floe at contact - r	50
CA	-0.16
CR	2.8
Exposure factor	0.6
Strength factor	0.3
Wind speed (m/s)	20
Drag coeff	0.002
Drag force (MN)	2.30
Initial 'KE (MN-m)	153.09
Pen. Incr. (m)	0.25
Mass + added mass (mln. kg.)	3,402.00
Maximum Impact Load (MN)	**20.6**
Maximum contact width (m)	**58.6**
Distance to stop (m)	**9.5**
Time to stop (s)	**40.7**
Line load (MN/m)	**0.35**

As in the prior example for freeze-up, smaller floes which might clear the dock face can be examined. For the floe size used previously for the 0.6m ice, but now 1.4m thick, its kinetic energy is about 44MN-m. Also, from the analysis of such an ice thickness breaking on the causeway slope and riding up, the peak load was about 10MN (Table 3.14). The simple analysis can be repeated to give a distance to stop the floe of:

$$x = 44(2)/10 = 8.8m$$

This is of the same order as for the 0.6m thick floe. Therefore, similar ride ups and potential pile ups could occur in this very rare situation.

4.4 Encroachment consequences

The nature of ice encroachment is that any ice which does encroach is in the form of discrete ice blocks – already fractured during the encroachment process. The forces on the mooring piles will be negligible compared to the mooring loads

Even though the causeway “wings” are exposed to the offshore ice which can be driven into the dock facility, the prior calculations and reasoning suggest that any ice encroachment is actually very unlikely for several reasons.

First, for large ice sheets which have to interact with the dock prior to interacting with the causeways, there is insufficient driving forces on the ice to push the ice past the dock face. There is also insufficient kinetic energy in freely moving floes to penetrate past the dock nor than 10 -20m.

Second, even if smaller floes can bypass the dock face, they will have lower driving forces. Sample calculations indicate that these driving forces are only just sufficient to fail the ice and push it to the top of the slope over limited widths of typically 20 - 30m. The floe has to be a very strange shape for it to only interact with a 20m – 30m width.

In the event that some ice encroachment could occur (perhaps at high water levels), it will be very limited and can be removed using bulldozers after the event.

4.6 Ice encroachment at the quay

Although the quay has a relatively low freeboard, the correlations for rubble heights and encroachment will not apply because of the deeper water at the dock face. The dock face is certainly exposed to ice action from onshore (as discussed under ice loads), however after initial crushing failure, the ice debris will build downwards and outwards. The failure mode will change to one of rubbling similar to the process involved in ridge building when two ice sheets collide. We can use correlations obtained from pressure ridge measurements to predict the extreme keel depths of ridges formed at the quay. Then correlations of sail heights to keel depths will enable the height of rubble adjacent to the quay to be estimated. The scenario is as shown in Figure 4.9. An extreme situation with encroachment is shown in Figure 4.10.

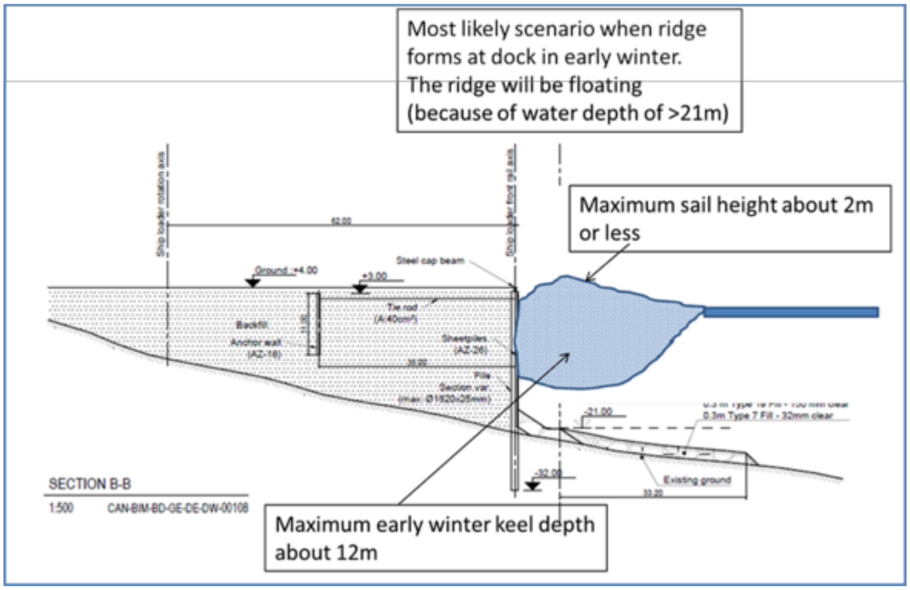


Figure 4.9: Extreme early winter floating ridge formed at dock face

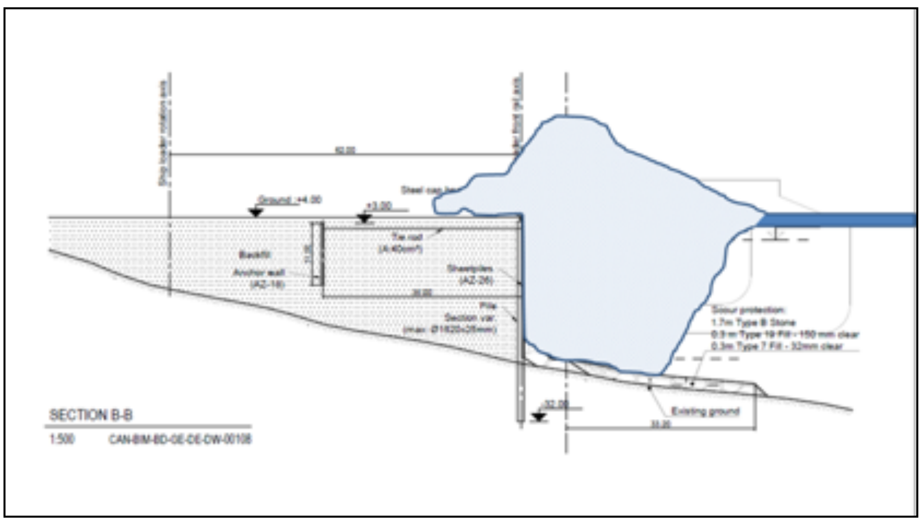


Figure 4.10: Extreme possibility for ice encroaching onto dock surface

The extreme situation shown in Figure 4.11 is very unlikely because it requires severe grounding which is not likely for several reasons:

- Insufficient sustained driving force to fail ice over full dock width and continue with ridge building to create a grounded ridge.
- Limit to keel depth (based on ice thickness and strength).
- Limited ice volume available to build ridge and rubble (based on amount of ice movement).
- Ice motion against dock is limited by advancing ice being stopped by the shoreline each side of dock (hence limiting ice volume available).
- At break-up ice motion is limited by floe momentum and driving force.
- After a certain ridge depth and rubble height, nature prefers to build outwards not upwards or downwards.

It may be sufficient to show that only one of the above can limit the encroachment.

Let us examine driving forces as we just did for the review of encroachment on rock berms. On a 2km large floe it was indicated that the maximum driving force under the action of rather severe 30m/s wind was about 9MN (see Table 4.1). For Scenario 1 (Freeze-up), ice acting on the dock was examined in Section 3. It was shown there that across its full width a load of about 90MN was required to fail the ice in crushing. This clearly would not occur with only 9MN driving force. Failure over smaller widths are of course possible with 9MN driving force, but this width would be limited to approximately 15m. So if an ice sheet only 15m wide initially interacted, the ice could fail in crushing and eventually create rubble which might build up into a ridge. However, the scenario of a sustained rubble building on only a 15m width is impossible to imagine as it requires the "floe width" to remain at about that width for 100s of metres. Such an ice feature does not exist.

It may be argued that with a switch to ridge building, the forces required would be smaller than in crushing – and that is the case. For floating ridge-building, the following equation to sustain the ridge building has been developed (Palmer and Croasdale, 2013).

$$w = 1.36(L^{-0.34})(h^{1.1}) \qquad \text{Eq. 4.4}$$

Using this equation, we can examine a range of ridge building line loads for various widths and ice thicknesses – see Table 4.6.

Table 4.6 Ridge building line loads for various ice thickness and global loads over various widths

Rubbling - floating ridge	w = 1.36*(L^-0.34)*(h^1.1)							
Width (m)	20		30		100		189	
h (m)	MN/m	MN	MN/m	MN	MN/m	MN	MN/m	MN
0.20	0.084	1.67	0.073	2.19	0.048	4.84	0.039	7.36
0.40	0.179	3.58	0.156	4.68	0.104	10.37	0.084	15.79
0.50	0.229	4.58	0.200	5.99	0.133	13.26	0.107	20.18
0.60	0.280	5.60	0.244	7.32	0.162	16.20	0.130	24.66

Even though widths of 20m and 30m are shown, these are unlikely to be sustained by an advancing floe (especially as the floe size to generate the driving force is 2km in size). It will be enlightening to look first at the full dock width of 189m. The last column in the Table indicates that a 9MN driving force could create a ridge with ice 20cm thick or so. As the ice gets thicker it would not be sufficient. A refinement of the calculation indicates that a driving force of 9MN will create a ridge in ice of 24cm. Would such a ridge ground in front of the dock and/or build a high enough sail to generate ice encroachment?

For floating ridges, extensive field measurements have developed relationships between ice thickness creating a ridge and the maximum sail heights and keel depths of the ridge. These relationships are shown in Figure 4.11.

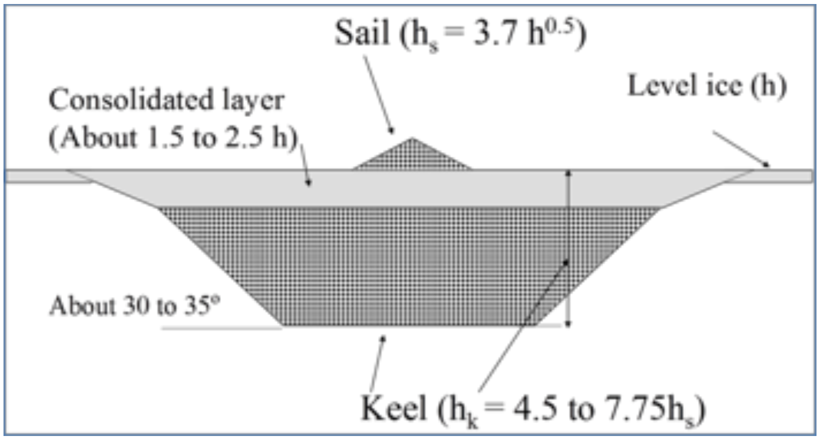


Figure 4.11: Idealized floating ridge cross section

The geometrical parameters of a ridge created by ice 24cm thick using the relationships given in Figure 4.11 are shown in Table 4.7. It can be seen that for ice 24cm thick, the ridge keel depth is in the range 8m to 13.6m so it will not ground. Furthermore, the associated sail height is 1.8m. The dock freeboard at MSL is 3.3m so at MSL the sail will not even reach the top of the dock wall and no encroachment is possible. Even at highest tide with a freeboard of 2.4m, the same is true.

Table 4.7: Ridge depths and sail heights created by various ice thickness

Keel depths and sail heights based on ridge geometry statistics			
Using ratios quoted by Timco			
m	m	m	m
Ice thickness	sail height	ridge depth	ridge depth
hi	hs	hk1	hk2
0.24	1.81	8.2	13.6
0.35	2.19	9.9	16.4
0.6	2.87	12.9	21.5

Other ice thicknesses are shown in Table 4.4, but as already demonstrated there is insufficient driving force to create ridges across the full dock width at these thicknesses. Even if there was, the sail height associated with 0.6m ice is still only about 0.4m above the dock surface level.

Another limit on ice encroachment can be assessed from the amount of ice motion available and whether that can generate sufficient volume of ice in a ridge to reach the top of the dock surface. The geometrical scheme for this is shown in Figure 4.12. In this case it assumed to be a half ridge – which is conservative in terms of calculating the height of the sail at the dock face.

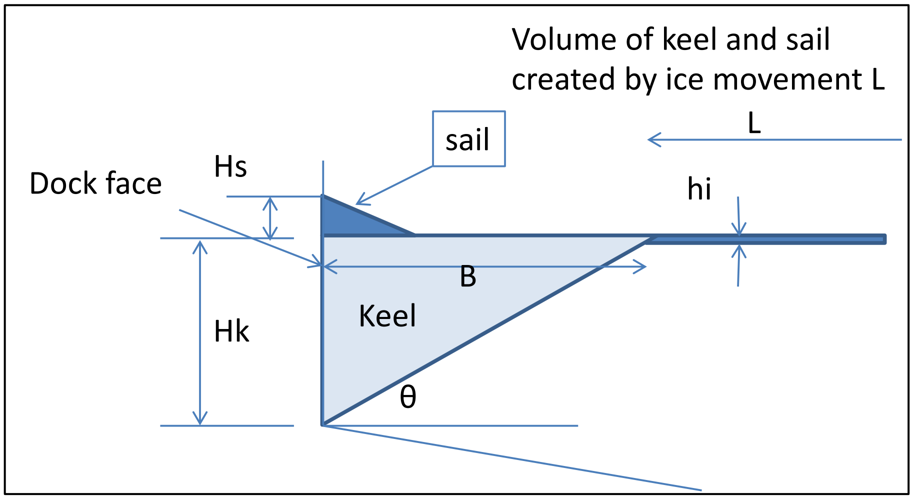


Figure 4.12: Geometry of a ridge in front of the dock face

For a given ice movement L, the keel depth (Hk) generated is

$$Hk = [Lhi/1.11(1-\gamma)(f+1/2\tan\theta)]^{0.5} \tag{4.4}$$

Sail height can be calculated using the range of relationships shown in Figure 4.4. Typical results are shown in Table 4.8. Remember that the available driving force will only generate a ridge in ice up to 24cm thick over the full dock width. The 0.6m ice is shown for reference as a "what if " case. It should not be used for design.

The ice movements shown of 200m and 300m are conservative because the distance from the dock face to the causeways and adjacent shorelines is less.

Table 4.8: Keel depths and sail heights based on ice movement

	Keel depths and sail heights based on ice movement					
				θ =	30	degrees
		f=	0.5	tanθ =	0.577	
	m	m		m	m	m
	Ice movt.	ice thickness	porosity	Ridge depth	Sail height 1	Sail height 2
	L	**hi**	γ	**Hk**	**Hs1**	**Hs2**
Freeze up	200	0.24	0.25	6.50	0.84	1.46
Freeze up	300	0.24	0.25	7.96	1.03	1.79
Feeze up	200	0.6	0.25	10.27	1.33	2.31
Freeze up	300	0.6	0.25	12.58	1.62	2.83

It should also be noted that no situations are known of ice encroachment onto dock faces with deep water in front of them in the Arctic, especially in sheltered locations.

All examples of ice encroachment onto dock surfaces have been with shallow water (e.g. 3 – 6m) at the dock.

If the owner wishes to be ultra conservative, then the highest ridge sail values in Table 4.7 and 4.8 might be considered (2.83m. If this coincided with high tide, the freeboard is 4.5- 2.4 = 2.1m. Table 4.9 shows the corresponding encroachment. It is 1m. A more realistic case (of this extreme) would be for the apex of the ridge to be further offshore than the dock face. The value for 1m is shown and then the encroachment is essentially zero.

Table 4.9: Encroachment distance for extreme case of a 2.83m ridge sail in front of dock

Sail height at edge of dock	Freeboard	Position of apex	Encroachment distance
hp	y	wt	wp
(m)	(m)	(m)	(m)
2.83	2.10	0.00	1.04
2.83	2.10	1.00	0.04

5.0 Ice bustle and ice foot due to tidal effects

5.1 Overview

Once formed, an ice sheet is subject to water level changes such as tides and surges. Far offshore the ice sheet will simply rise and fall with water level with no stresses induced into it.

Where the ice sheet is in contact with a fixed structure and/or the shoreline, stresses can be induced into the sheet forming cracks. At a gently sloping shoreline, the ice becomes frozen to the shore. In this situation, as depicted in Figure 5.1 there is an “ice foot” usually grounded extending from shore a certain distance to the tidal cracks. There can be two cracks with a short length of ice hinged between. At high tide the water floods over the ice foot and thickens it so that its elevation is about the same as the top of the floating ice at high tide. At low tide there is a difference equal to about the tidal range.

Because a crack is maintained, if ice pressure builds up, this is also where a shore ridge (or grounded ice rubble can occur) especially early in the winter.

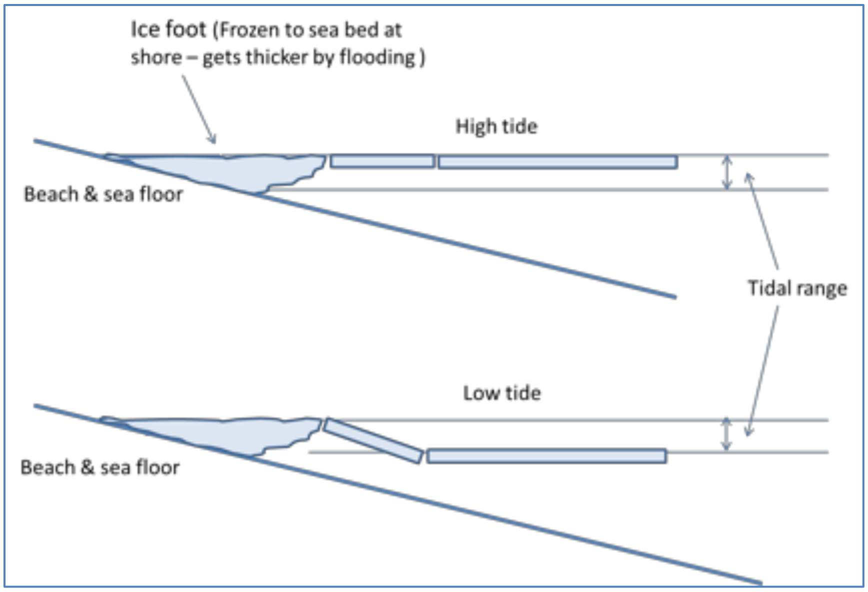


Figure 5.1: Ice foot at shore line (or on sloping rock slope)

In the case of a single vertical faced pile, the initial crack is often right at the pile and is in the form of a shear crack. At each tide the ice sheet goes up and down and the water in the crack between the ice and the pile freezes to the pile - gradually increasing its width with an "ice bustle". This process is shown conceptually in Figure 5.2. (Note the first part of this material is taken from the accompanying report on ice conditions - KRCA/CMO, 2018).

Some photos of ice bustles at a coal loading dock in Svalbard are shown in Figure 5.3. It can be seen that on an isolated pile, an ice bustle can be large compared to the pile diameter.

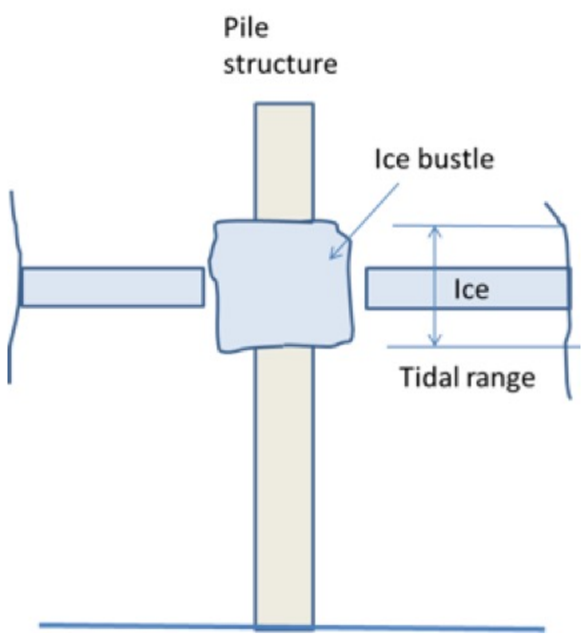


Figure 5.2: Ice bustle on a vertical pile

Figure 5.3: Images of ice bustles at coal loading dock in Svalbard, Norway (Loset and Marchenko, 2009)

On a vertical dock face ice can also form a solid ice bustle (but linear). This situation is shown in Figure 5.4.

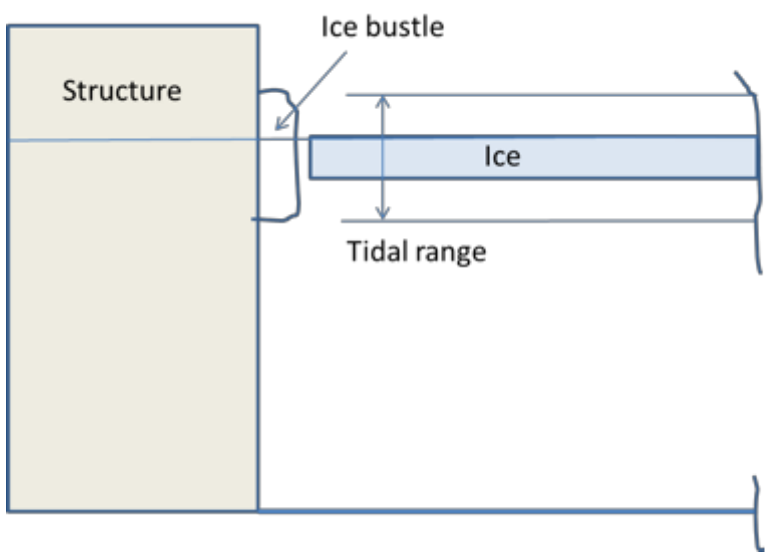


Figure 5.4: Linear ice bustle on dock face

At the Nanisivik dock it was reported by Frederking and Sinha (1977) that the ice bustle was 1.2m thick at the end of November. They also identified an active zone beyond the ice frozen to the structure similar to the hinged ice piece shown in Figure 5.1. At the dock this is shown conceptually in Figure 5.5.

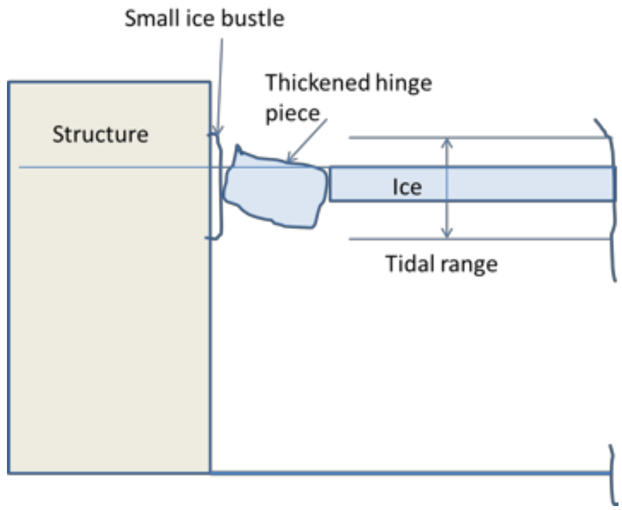


Figure 5.5: Thickened hinge piece reported by Frederking at the Nanisivik Dock (.

The hinge piece can be thicker than the offshore ice because of surface flooding and refreezing due to the tidal cycles. At Nanisivik, the geometry of the thickened hinge piece is shown in Figure 5.6. It was about 5m thick at its maximum on March 28 – although only about 2m thick at the dock face. Also shown in Figure 5.5 is that sometimes the adjacent ice can be lifted at its edge during a rising tide and this may induce an additional crack further offshore.

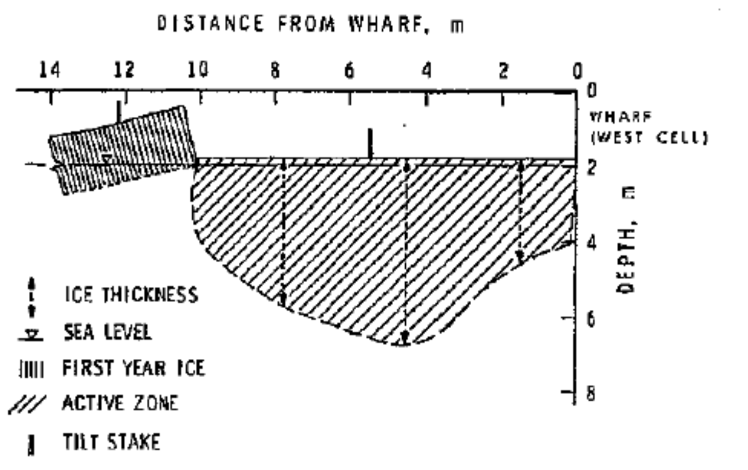


FIG. 9. Ice thickness profile along line W2, high tide (1100), March 28, 1980.

Figure 5.6: Geometry of ice at Nanisivik dock face by March 1980 (Frederking and M. Nakawo, 1984)

5.2 Effects of bustles and thickened active zone on ice loads

On isolated piles, the effective increase in pile diameter should be taken into account when estimating ice loads during the mobile freeze-up period when the ice grows to about 0.6m in thickness prior to it becoming landfast. However, if the piles are on the shore-side of an ice foot then a bustle will not form. We estimate the water depth to which an ice foot will extend to be approximately the tidal range – say about 2- 3m. This is very approximate and depending on the shore and causeway geometries it may be different. Only field observations can confirm. In preliminary design work, any isolated piles should account for an ice bustle up to about 0.5m thick by the end of freeze-up.

During break up, any ice bustles will have melted away before any interaction with the offshore ice floes which might be driven inshore.

On a linear dock face, any ice bustle will not increase the width for ice loading. Furthermore during the freeze-up period observations at Nanisivik suggest that the active hinge piece is minimal and about the same thickness as the ambient ice. In any case, even if it was thicker, the thinner ambient ice would preferentially fail on the outside of the thicker ice (and very likely in a rubbling load at a lower ice pressure than crushing). So, in summary the ice loads at the end of the freeze-up period which have been calculate in Section 3 should not be adjusted for tidal effects on ice geometry.

As already discussed, the presence of a layer of ice frozen to the dock face will actually be beneficial in distributing high local ice pressures over a larger area.

During mid-winter, the lateral ice pressures due to slow lateral ice movements will not be affected by the ice bustles or the thickened hinge piece. This is generally confirmed by investigations at the Nanisivik dock.

During break up, it is predicted that the ice bustle and thickened hinge piece will deteriorate in place and have limited ability to create higher loads on the dock face than those predicted in Section 3.

Vertical loads due to the presence of an ice bustle can be created by gravity and buoyancy forces as the water level changes due to tides and surges. These have been briefly examined for the configuration shown in Figure 5.7; some results are shown in Table 5.1. These loads are extremely small. The case of an ice sheet being caught under the edge of the bustle and failing in bending as the water level rises has also been examined; these results are shown in Table 5.2; they are also very small.

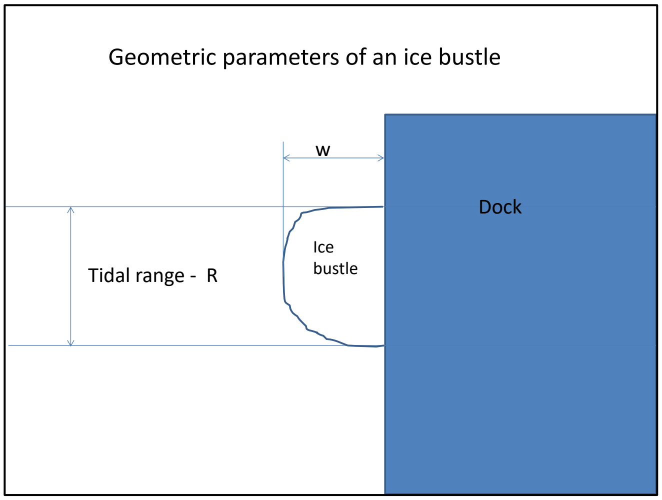


Figure 5.7: Parameters to estimate bustle vertical forces

Table 5.1: Loads due to gravity and buoyancy effects on typical ice bustles

	Bustle gravity/buoyancy loads				
R	w	ice density	Water density	Fd	Fu
m	m	kN/m3	kN/m3	MN/m	MN/m
2.4	1	9	10.3	0.0216	0.00312
2.4	1.5	9	10.3	0.0324	0.00468
3	0.5	9	10.3	0.0135	0.00195

Table 5.2: Loads due to ice sheet being caught under a bustle and failing in bending

Input Data in blue: Derived values in black and results in green		**winter 0.6m ice**	**Winter 2.05m but slow**
Flexural strength of ice	(kPa)	**500**	**300**
Specific weight of ice	(kN/m^3)	**8.98**	**8.98**
Specific weight of water	(kN/m^3)	**10.20**	**10.20**
Young's modulus	(kPa)	**5.00E+06**	**2.00E+06**
Waterline width (D)	(m)	**50**	**50**
Thickness for breaking (hb)	(m)	**0.6**	**2.05**
Characteristic length (Lc)	(m)	9.92	19.83
Ratio of structure width to characteristic length		5.04	2.52
First break length (R)	(m)	7.74	15.47
Results			
Vertical line load based on first break (MN/m)		**0.0033**	**0.023**

6.0 Closing Remarks

A summary of the key results relating to ice interaction for detailed design are provided in the Executive Summary and will not be repeated here.

The ice design criteria provided in this document are at the probability level required by ISO 19906 for this type of structure; that is the loads have a nominal annual probability level of 0.01 (or a 100 year return period). Strictly speaking to develop true "100 year " loads, a probabilistic methodology should be used; but deterministic approaches such as used in this work are allowed in ISO 19906. The approach is semi-probabilistic in that the main ingredient of ice thickness is based on statistical data. Unfortunately, no site-specific ice data has been collected, so Pond Inlet has been used as a proxy. Also, ice growth models have been used driven with local temperature data to refine the data used.

In general, when deterministic approaches are used with "100 year" ice thickness, the overall results tend to be conservative because other ingredients are often also chosen at maximum values. In this work, engineering judgement has been used to try to avoid this tendency. However, some conservatism is still likely included (as discussed in the report).

Because the loads and other derived parameters in this report are at the "100 year" level they should not be directly used to assess ice interaction during the one year of construction. A separate document is being issued on potential loads and risks on the uncompleted structure at various stages of construction.

The suggestions for safety class and load factors should be reviewed carefully and discussed with the Owner's engineer (or regulator). The Owner may decide to use higher load factors to minimize the risk of damage (and subsequent maintenance).

Several assumptions have been made about the ice behaviour especially in terms of thickness and mobility (and tidal actions). These assumptions are considered sound, based on available data and the experience of the ice team. The available data for this location are sparse. Therefore, it would be prudent to take advantage of the coming winter to conduct some "ground-truthing" and collection of additional targeted satellite imagery. Although this report does not address winter construction issues, there is also much value in using the coming winter to assess ice access issues and safety relating to construction plans when the ice is present (and access onto the ice may be part of those plans). The specifics of these observations will become clearer once the proposed construction plans have been firmed up.

A "design specification" which can be extracted as a summary of design ice loads for structural engineers is provided in Appendix A

7.0 Acknowledgements

In addition to the authors, several others have contributed to the ideas and the material presented in this report. The review of the prior ice criteria report, developed for preliminary design, by the BESIX/VanPile team gave vital feedback which has been taken into account in this recent work. In particular the careful reviews, ideas and suggestions by George Comfort (representing Hatch) have been invaluable.

8.0 References

Allyn and Croasdale (2017), Ore Dock – Baffinland Iron Mines: Mary River Expansion Stage 3. Ice Issues and development of ice design criteria. Short report for Vaile/Be six. March 7, 2017.

Allyn and Croasdale (2017), Ice Interaction Design Criteria Assessment : Ore Dock – Baffinland Iron Mines: Mary River Expansion Stage 3. Submitted to BESIX; Avenue des Communautés, 100 – 1200 Brussels – Belgium. Final Draft Version 4. November 17, 2017

Barker, A. and Croasdale, K. (2004), Numerical Modelling of Ice Interaction with Rubble Mound Berms in the Caspian Sea, in Proceedings of 17th International IAHR Symposium on Ice, Saint Petersburg, Russia.

BMT Fleet Technology Ltd. (2014), Mary River Mine dock Site Ice Engineering Study. For Hatch Engineering.

Croasdale, K. R. and Metge, M. (1991), Structure geometry and ice interaction. Proceedings of IUTAM-IAHR Symposium on Ice-Structure Interaction. St John's, Canada, 1991.

Croasdale, K.R., Cammaert, A.B. and Metge, M. (1994), A method for the calculation of sheet ice loads on sloping structures. Proceedings of the 12th International Symposium on Ice of IAHR. Trondheim, August 1994 Norway. (12 pages).

Croasdale, K. R. (2011), Platform shape and ice interaction: a review. *Proceedings, 21st International Conference on Port and Ocean Engineering under Arctic Conditions*, Montréal.

Croasdale, K. R. (2012), A simple model for first-year ridge loads on sloping structures. Proceedings of ICETECH, Banff, 2012.

CSA 471 -04. General requirements, design criteria, the environment, and loads. Canadian Standards Association, (CSA) 2004.

Frederking R. and N.K. Sinha (1977), Ice Action on Wharf at Strathcona Sound, POAC, 1977.

Frederking R. and M. Nakawo: Ice action at Nanisivik wharf, winter 1979-1980, Can J. Civ. Eng. Vol 11, 1984.

Frederking, R. (2012), Review of Standards for Ice Forces on Port Structures. 15th International Conference on Cold Regions Engineering, Quebec, Canada, August 19-22, 2012.

ISO 19906, 2010, Petroleum and Natural Gas Industries – Arctic Offshore Structures, International Standards Organization ISO 19906.

Jordan, I. J, F. S. Taylor and J. Wells (2009), Ice crushing, damaged layers, and pressure-area relationships, presented at 20th International Conference on Port and Ocean Engineering Under Arctic Conditions POAC 09 Proceedings Paper 09-128.

KRCA/CMO (2018), Milne Inlet Ice conditions: Volume 1: as relating to detailed design. For Besix VanPile Joint Venture by KRCA/CMO, November 30, 2018.

Lengkeek, H.J., Croasdale, K.R. and Metge. M. (2003), Design of Ice Protection Barrier in Caspian Sea, *22nd OMAE Conf.*, Cancun, June 2003.

Loset, S. and A. Marchenko (2009), Field studies and numerical simulations of ice bustles on vertical piles. Cold Regions Science and Technology, Aug 2009.

McKenna, R., Stuckey, P., Crocker, G., McGonigal, D., Croasdale, K., Verlaan, P. and Abuova, A. (2011), Ice Encroachment in the North Caspian Sea, in *Proceedings of POAC 2011*, Montréal, Canada.

Appendix A: Design Specification Summary – for designers

Appendix A: Design Specification Summary – for designers

A.1 Introduction

The intent of this Appendix is to give the vital information required for a structural engineer to design and perform checks on the dock in relation to ice loads.

The following loads and other parameters relating to ice interactions are unfactored. They are developed using the main parameter of ice thickness at the annual probability level of 0.01. In lieu of a full probabilistic analysis these represents the 100 year design parameters.

The International Standard ISO 19906 DIS (2018) is used for guidance.

Full details behind the summary of ice parameters in this Appendix are given in the main body of this report. If this Appendix has been separated from the report, the reference to the report is;

"Ice Interaction Design Criteria for Detailed Design: Ore Dock – Baffinland Iron Mines: Mary River Expansion Stage 3. V1 December 7, 2018. By KRCA/CMO.

A.2 Overview of ice conditions

At the high latitudes of this location, ice is present for about 9 months of the year with annual ice growth up to about 2m. The sheltered location at the head of Milne Inlet leads to the conclusion that level ice at various times through the winter represents the design ice feature. Although ridging may occur offshore, the limited ice movement will not allow significant ridges to act on the structure. In any case limited driving forces due to limited fetch, would limit the loads. Some minimal ice rubble may from at the dock and on the causeways and is accounted for. Tidal effects will create tidal cracks and create an ice bustle on the dock structures. Multi-year and glacial ice may be driven into Pond Inlet and the offshore end of Milne Inlet but is not likely to penetrate all the way to the dock.

Climate change may reduce the length of the ice season and annual ice growth thickness. However because of uncertainties in climate change, engineering practice requires designs to be based on the severest conditions. For ice this will be historical data, but for waves the potential for more open water may need to be assessed. Water level rise may increase the risk of over-topping by ice and waves.

The ice conditions through the winter can be divided into three periods. These are: (a) Freeze-up; (b) Mid-Winter; and (c) Spring Melt and Breakup. Ice interaction is different during each of these periods.

The key reference for ice conditions and ice parameters used in the ice loads and ice encroachment assessments in this document is *"Milne Inlet Ice conditions: Volume 1: as relating to detailed design" by KRCA/CMO, November 30, 2018.*

A.3 Ice loads by periods of the winter

A.3.1 Scenario 1: Freeze - up

The first period of interest is from the time of ice formation to the end freeze-up when the ice stabilizes and becomes landfast. This is defined as **Scenario 1** during which the ice is mobile and cold and is estimated to grow to 0.6m thick before it stabilizes into landfast ice. Movement of 0.6m thick ice can cause it to fail in crushing on the dock face

The global loads calculated as a function of loading width are shown in Table A.1

Note the widths chosen for calculation are based on the current design of the "combi-wall". spacing of the large cylindrical piles in the combi-wall system (3.369m and multiples thereof). The loads are based on the methodology in ISO 19906 and details of the calculation of these loads are given in the report.

Table A.1: "100 year" global ice loads calculated for Scenario 1 (Freeze-up)

No of piles	Loading width (m)	Ice Thickness (m)	Line load (MN/m)	Maximum per pile (MN)	Potential global load (MN)	Max. wind & current drag 2km fetch (MN)
sheet pile	1.540	0.6	1.07	1.65	1.65	6
1 pile tributary	3.369	0.6	0.94	3.18	3.18	6
2	6.738	0.6	0.85	2.85	5.69	6
3	10.107	0.6	0.79	2.67	8.00	6
4	13.476	0.6	0.76	2.55	10.19	6
5	16.845	0.6	0.73	2.46	12.29	6
6	20.214	0.6	0.71	2.39	14.33	6
7	23.583	0.6	0.69	2.33	16.31	6
8	26.952	0.6	0.68	2.28	18.24	6
9	30.321	0.6	0.66	2.24	20.14	6
10	33.690	0.6	0.65	2.20	22.01	6
11	37.059	0.6	0.64	2.17	23.84	6
56	190.494	0.6	0.50	1.68	94.31	6

It can be seen that the line load diminishes with loading width and that the potential global load could be as high as 94MN if the ice crushes over the full structure width. However, because this dock is at the end of a narrow inlet the driving forces on the ice are limited. In fact, even for a typical maximum sustained winds and currents the "static" driving force available is derived in this report as being about 6MN. However, this is not the design ice load because of another load case involving freely moving ice sheets that can be accelerated by the winds and currents.

If a large ice sheet collides with the dock at its maximum speed then the kinetic energy of the floe can impose larger forces than the static wind and current drag. These impact cases have also been analyzed for this initial freeze up period. Typical results are given in Table A2.

Table A2: loads due to impact of feely moving floes just prior to ice becoming landfast

ice thickness 0.6m		1500m floe	
wind speed (m/s)	20	15	10
floe speed (m/s)	0.45	0.337	0.225
force (MN)	25.4	22.6	18.3
loading width (m)	40	34.7	27.1
line load (MN/m)	0.64	0.65	0.68

The results in Table E2 indicate that the maximum global load is 25.4MN with a corresponding line load of 0.64MN.

How these line loads should be applied to the structure for structural design is described after the other two ice scenarios have been discussed.

A.3.2 Scenario 2: Ice is landfast

Scenario 2 is the period when the ice is landfast and will be essentially stationary (small slow movements only). The ice is cold and attains its maximum thickness of 2m. However, because of the relief of ice pressure due to creep, the loads will be lower than in other periods. The “100 year” load for this period is estimated at 0.3MN/m for all loading widths.

A.3.3 Scenario 3: Break - up

Scenario 3 is the break up period when the ice has deteriorated in strength prior to it becoming mobile again. It is also calculated to be thinner than the winter maximum and a value of 1.4m is used. The “100 year” line loads for this period are shown in Table A.3. (Line loads for Scenario 1 are also shown for comparison and it can be seen that they govern).

Table A.3: “100 year” global ice loads for Scenario 3 (Break up).

SCENARIO 3					
Break up	Warm weak ice		Strength Factor	0.32	
			Cr	0.896	
			n	-0.3	
			m	-0.16	
Crushing	ISO 19906				For comparison
			Arctic Warm	Arctic Warm	Scenario 1
width (m)	Thickness (m)	n	p (MPa)	Load (MN/m)	Load (MN/m)
3.26	1.7	-0.3	0.69	1.17	1.56
6.52	1.7	-0.3	0.62	1.05	1.39
9.78	1.7	-0.3	0.58	0.98	1.31
13.04	1.7	-0.3	0.55	0.94	1.25
16.30	1.7	-0.3	0.53	0.90	1.20
32.60	1.7	-0.3	0.48	0.81	1.08
183.00	1.7	-0.3	0.36	0.61	0.82
15.00	1.7	-0.3	0.54	0.92	1.22
30.00	1.7	-0.3	0.48	0.82	1.09

In many ice interaction scenarios, local ice pressures over small areas or widths can be higher than global ice pressures averaged over bigger widths or areas. The most severe local ice pressures in ISO have been developed based on high velocity ship impacts and on measurements on thick ice in confined situations; they are not considered suitable for this structure at this location.

In this case the ice is thin at freeze up (0.6m); and at break up, although thicker (1.4m), it is very deteriorated. The method for thin ice in ISO has been investigated and it gives a line load very close to the highest global line load of 1.07MN/m. For this reason, a separate set of criteria for local ice loads is not considered necessary. It is also noted that in the experience of the authors and advisors, local damage from ice to sheet piles at existing Arctic docks (such as Nanisvik) has not been experienced. The same is true for many sheet pile docks in the North Caspian Sea.

A.4 Design Case – with loading width limits

A.4.1 Loads perpendicular to dock

Scenario 1 gives the highest line loads and governs the ice design criteria. Because of limited driving forces and floe kinetic energy the loading widths for ice crushing (giving the highest line loads) are limited – as shown in Table A.4.

The limit of the 25MN (due to limited kinetic energy) from Table A2 has been applied. It is assumed that the dock design is not overly sensitive to actual total global load but that the line loads over various widths will control design. The maximum loading width of 40m can vary somewhat as will be seen when looking at corner and oblique loads.

Table A.4: Design line loads with the global load limit due to limited floe kinetic energy applied.

No of piles	Loading width (m)	Ice Thickness (m)	Line load (MN/m)	Maximum per pile (MN)	Potential global load (MN)	Adjusted for limited kinetic energy (MN)	Max. wind & current drag 2km fetch (MN)
sheet pile	1.540	0.6	1.07	1.65	1.65	1.65	6
1 circular pile	1.830	0.6	1.04	1.91	1.91	1.91	6
1 pile tributary	3.369	0.6	0.94	3.18	3.18	3.18	6
2	6.738	0.6	0.85	2.85	5.69	5.69	6
3	10.107	0.6	0.79	2.67	8.00	8.00	6
4	13.476	0.6	0.76	2.55	10.19	10.19	6
5	16.845	0.6	0.73	2.46	12.29	12.29	6
6	20.214	0.6	0.71	2.39	14.33	14.33	6
8	26.952	0.6	0.68	2.28	18.24	18.24	6
10	33.690	0.6	0.65	2.20	22.01	22.01	6
12	40.428	0.6	0.63	2.14	25.65	25.65	6
14	47.166	0.6	0.62	2.09	29.19	not possible	6
16	53.904	0.6	0.61	2.04	32.66	not possible	6
21	70.749	0.6	0.58	1.95	41.04	not possible	6
56	190.494	0.6	0.50	1.68	94.31	not possible	6

A.4.2 Corner loads

An ice sheet or floe can drift towards the dock at various angles of attack. A case which may require some special structural analysis is a corner loading. The type of scenario creating a corner loading is shown in Figure A.1.

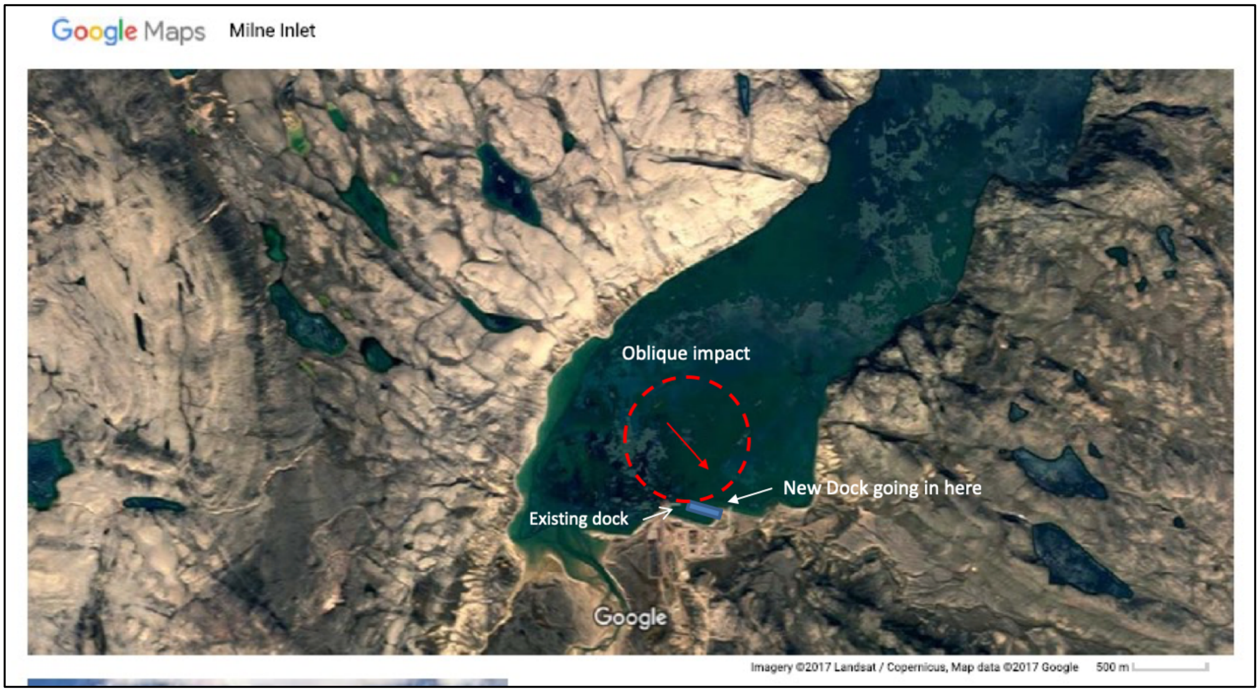


Figure A.1: Oblique impact and corner loading

As penetration of the ice floe by the structure proceeds, the width of loading increases and involves more circular piles. The geometry for calculation is shown in Figure A.2. To get the maximum width of loading an energy calculation is performed. The base-case floe is a maximum of 1500m size drifting at 0.3m/s; the energy calculation indicates a maximum width of loading when the floe stops of 11m. The loads associated with such an interaction are shown in Table A.5.

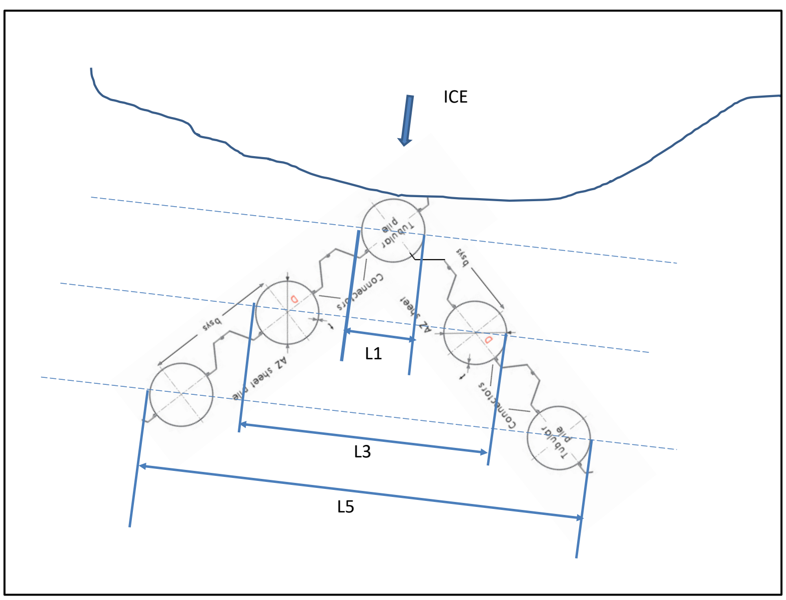


Figure A.2: Geometry of corner loading and loading widths

Table A.5: Corner loading

Corner loading	45 degrees	ISO 19906			Cr			
					2.8	With exposure factor		
				m	-0.16	0.80	Max.	
No of	Loading	Ice		Arctic cold	Arctic cold	Effect of exposure	Per pile Case A	Global load
piles	width (m)	Thickness (m)	n	p (MPa)	Line Load (MN/m)	Line load (MN/m)	(MN)	(MN)
corner pile	1.830	0.6	-0.38	2.84	1.71	1.37	2.50	2.50
3	6.600	0.6	-0.38	2.32	1.39	1.11	2.45	7.34
5	11.360	0.6	-0.38	2.12	1.27	1.02	2.32	11.58

A.4.3 Oblique loads

Oblique loads on the front face can be handled in a similar manner. In this case, the effective width of interaction can be determined from the principles of geometry shown in Figure A.3. The loading widths are those involved in generating the ice force and determining the line loads. The length of the quay (wq) which is loaded is then determined from the angle of attack a using the expression;

$$wq = wl/sina \qquad \text{(A.1)}$$

Based on engineering judgement, considering the shape of the inlet and dock location, it is recommended that the minimum angle of attack considered should be 40 degrees. Table A.6 shows how the length of loaded dock relates to the loading width for the energy dissipation calculation and for the critical cases given earlier in Table A.2. For design, also note that the loading on the piles and dock face is in the direction of ice movement.

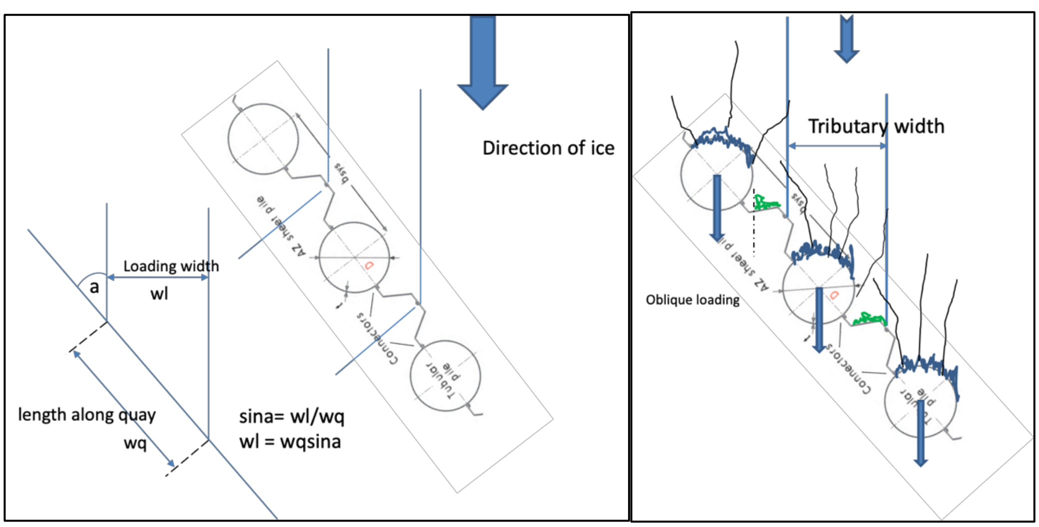


Figure A.3: Oblique loading

Table A.6: Oblique loading – floe impact situation

ice thickness 0.6m		1500m floe	
wind speed (m/s)	20	15	10
floe speed (m/s)	0.45	0.337	0.225
force (MN)	25.4	22.6	18.3
loading width (m)	40	34.7	27.1
line load (MN/m)	0.64	0.65	0.68
Ange of attack (deg)	40	40	40
loaded width on dock (m)	62.2	54.0	42.2

A.4.5 Global load variability across width

The preceding tables have shown how the line loads can be higher over smaller widths. In conducting structural analyses this should be recognized; however, in using a higher load over a smaller width, the average over the width being analyzed will need to be maintained at the values given. An example is given below (note the widths are chosen by a whole number of piles in the combi-wall which are at 3.37m spacing).

Total width being analyzed is say 9x3.37 = 30.3m

For the controlling Scenario 1, the average line load over 30.3m = 0.66MN/m (see Table A.1)

At a width of 3.37m (tributary for one pile) the line load is 0.94MN/m, therefore, the line load over the remainder of the 30.3m is = (30.3*0.66 – 3.37*0.94)/ (30.3-3.37) = 0.625MN/m.

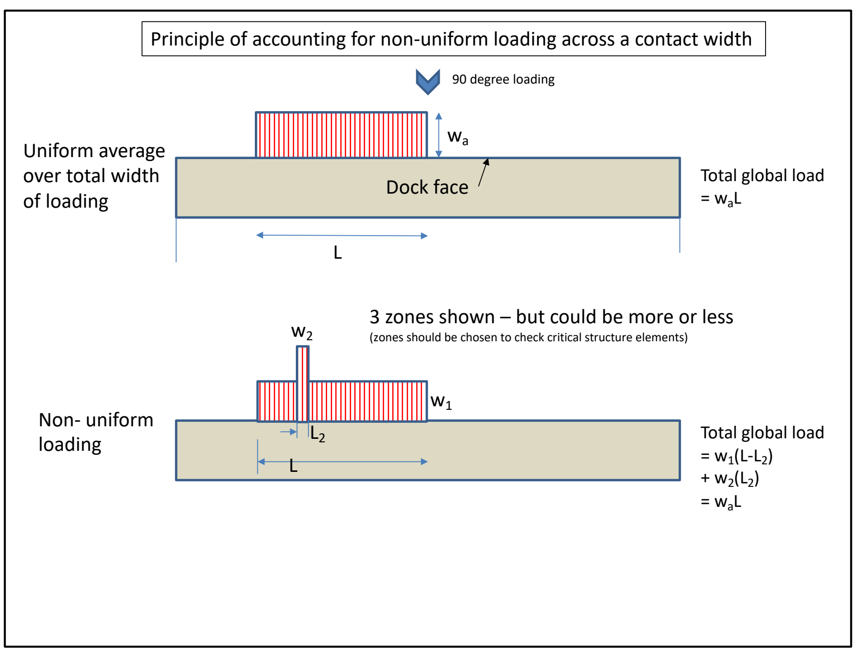


Figure A.4: Non-uniform loading – principles of calculation

A.4.6 Load paths

The ice will act across the selected width to be analyzed in the manner shown in Figure A.5 Crushing will occur across the whole width. There may be some process whereby the circular piles which are leading create fractures and spalls in the ice which lead to lower line loads on the sheet piles, but this cannot be taken for granted.

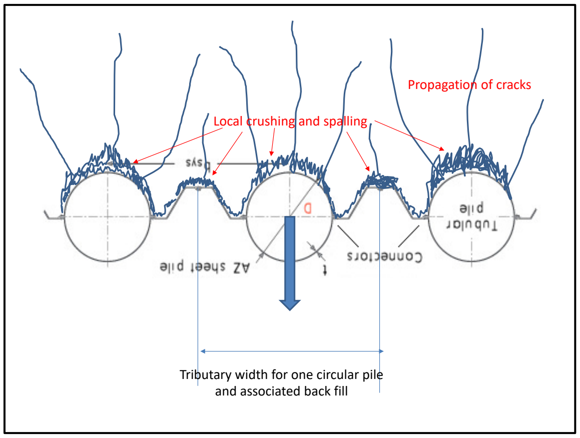


Figure A.5: the nature of ice action on the combi-wall

Critical widths and analysis of structural integrity will depend on load paths from the ice line into the major resisting components of the structure. This cannot be determined "a priori" because it depends on relative stiffnesses of the various load paths. But the load paths may be bounded. The issues are shown in Figure A.6

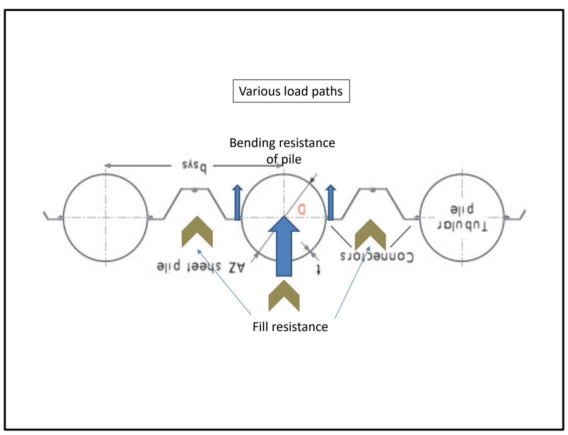


Figure A.6: Issues relating to load paths

The main issue is how much the backfill can carry in shear through the fill versus the bending capacity of the piles. This may depend on whether the back fill is frozen. It is beyond the ice engineering aspects of developing the ice load criteria to answer this.

A key issue is whether the ice loads on then sheet piles have to be reacted by their connection to the circular piles or whether the backfill can absorb the load (or a proportion of the load). For ice loading, it has to be assumed that the sheet piles see the same line loading as the rest of the combi-wall. Relative stiffness does not affect the ice loading (as there is a continuous failure of the ice – creating more like a hydrostatic pressure) but it may affect the load paths. It is up to then structural engineer to figure that out. One bound is that all the ice loading on the sheet piles is transmitted sideways to the circular piles.

A.4.7 Ice load application levels

The ice loads can be assumed to act at the mid-point of the ice thickness over the full range of expected water levels. It can also be assumed that the top of the ice surface is at the water level.

A.4.8 Pressure ridge building at dock

As discussed in the main report (Section 4.6), one scenario checked in relation to ice encroachment on the dock face was the rare situation of a pressure ridge building at the dock face. This is illustrated in Figure A.7.

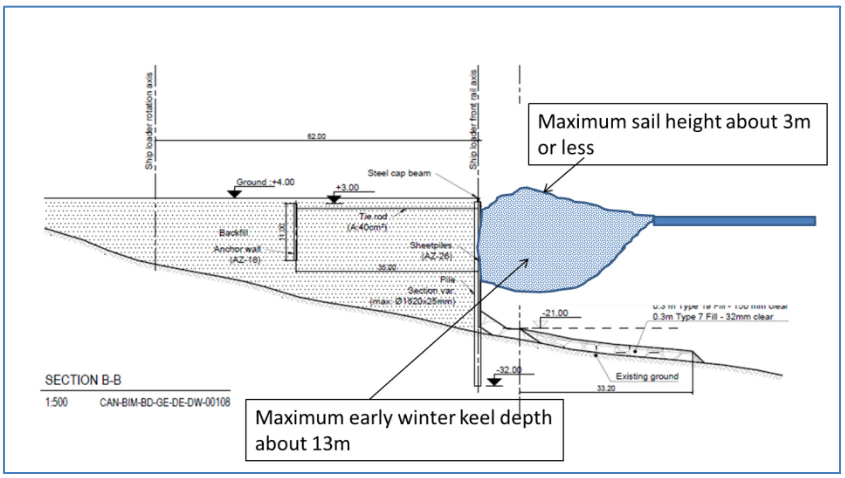


Figure A.7: Scenario of ridge building at the dock

The extreme driving forces to create such a ridge are limited to about 9MN (to be conservative in estimating encroachment this was increased from the value of 6MN estimated for Table A1). Although ridge building forces are less than crushing forces it is calculated that this force can only create a ridge in ice up to about 24cm thick. Such a ridge would have a keel limited in depth to about 8m. If such a ridge is being created at the dock, there will be a force at the water line of 9MN. In addition, the keel can impose a pressure as a granular material acting on a vertical face. In this case it is inverted compared to a granular material in air. The effective buoyant force is the "gravity force" and a friction angle of 35 degrees can be taken for the ice rubble. The pressure will be highest at the top and zero at the bottom. These loads are quite small. For an 8m keel they are estimated at the top to be about 0.5*tan30*0.74*8^2 = 13kPa. So, the average is 6.75kPa. For an 8m keel, the line load is about 0.05MN/m . Note that this is not added to the 9MN. – but about equal to it. The process is one of the line load initially being applied at the water line; as the ridge builds the point of application of the line load moves down to be at the centre of load application of the keel based on these hydrostatic forces. This is approximately 1/3 of the keel depth from the water line (2.6m).

A.4.8 Ice bustles and ice foot due to tidal effects

A.4.8.1 Overview

Once formed, an ice sheet is subject to water level changes such as tides and surges. Far offshore the ice sheet will simply rise and fall with water level with no stresses induced into it.

Where the ice sheet is in contact with a fixed structure and/or the shoreline, stresses can be induced into the sheet forming cracks. At a gently sloping shoreline, the ice becomes frozen to the shore. In this situation, as depicted in Figure A.8 there is an "ice foot" usually grounded extending from shore a certain distance to the tidal cracks. There can be two cracks with a short length of ice hinged between. At high tide the water floods over the ice foot and thickens it so that its elevation is about the same as the top of the floating ice at high tide. At low tide there is a difference equal to about the tidal range.

Because a crack is maintained, if ice pressure builds up, this is also where a shore ridge (or grounded ice rubble can occur) especially early in the winter.

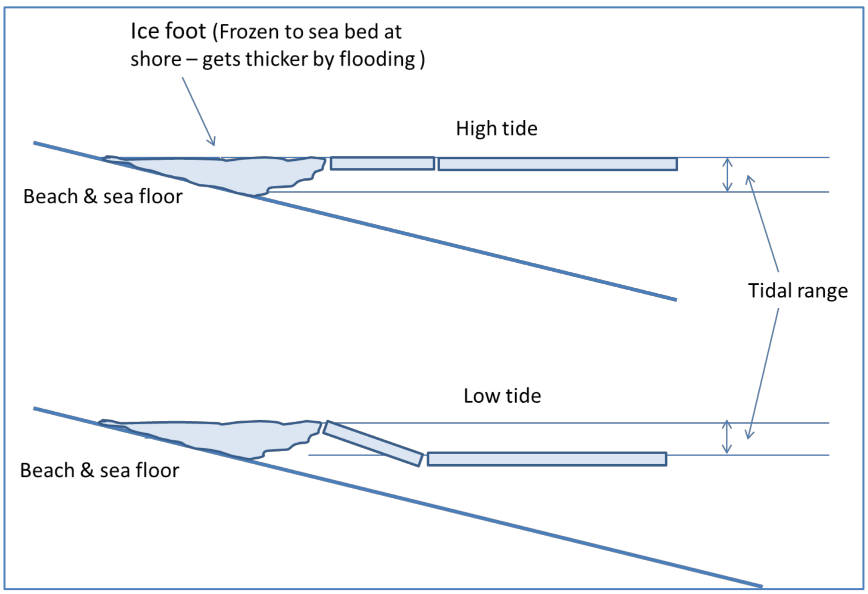


Figure A.8: Ice foot at shore line (or on sloping rock slope)

In the case of a single vertical faced pile, the initial crack is often right at the pile and is in the form of a shear crack. At each tide the ice sheet goes up and down and the water in the crack between the ice and the pile freezes to the pile - gradually increasing its width with an "ice bustle". On a vertical dock face ice can also form a solid ice bustle (but linear). This situation is shown in Figure A.9

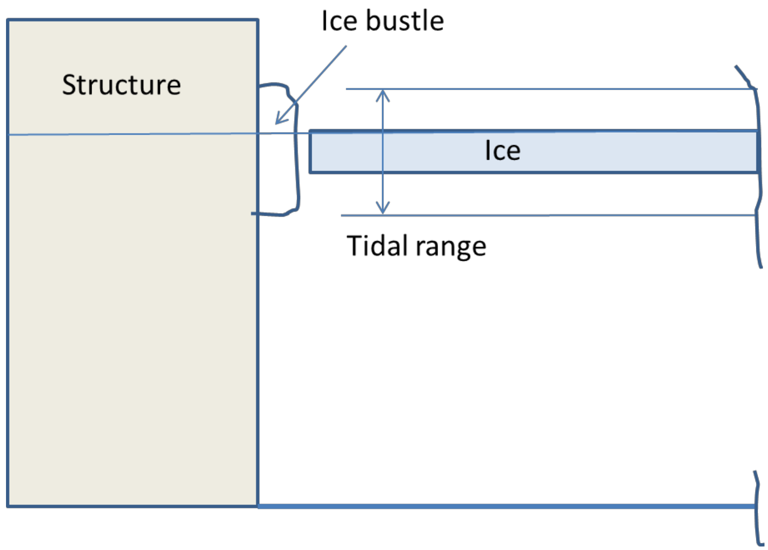


Figure A.9: Linear ice bustle on dock face

At the Nanisivik dock it was reported by Frederking and Sinha (1977) that the ice bustle was 1.2m thick at the end of November. They also identified an active zone beyond the ice frozen to the structure similar to the hinged ice piece shown in Figure 5.1. At the dock this is shown conceptually in Figure A.10.

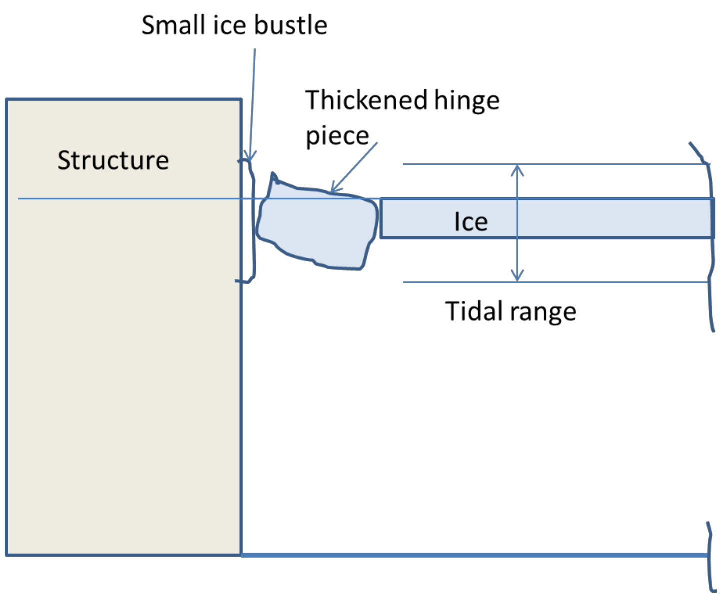


Figure A.10: Thickened hinge piece reported by Frederking at the Nanisivik Dock

The hinge piece can be thicker than the offshore ice because of surface flooding and refreezing due to the tidal cycles. At Nanisivik, the geometry of the thickened hinge piece is shown in Figure A.10. It was about 5m thick at its maximum on March 28 – although only about 2m thick at the dock face. Also shown in Figure A.10 is that sometimes the adjacent ice can be lifted at its edge during a rising tide and this may induce an additional crack further offshore.

A 4.8.2 Effects of bustles and thickened active zone on ice loads

On isolated piles, the effective increase in pile diameter should be taken into account when estimating ice loads during the mobile freeze up period when the ice grows to about 0.6m in thickness prior to it becoming landfast. However if the piles are on the shore-side of an ice foot then a bustle will not form. We estimate the water depth to which an ice foot will extend to be approximately the tidal range – say about 2- 3m. This is very approximate and depending on the shore and causeway geometries it may be different. Only field observations can confirm. In preliminary design work, any isolated piles should account for an ice bustle up to about 0.5m thick by the end of freeze up.

During break up, any ice bustles will have melted away before any interaction with the offshore ice floes which might be driven inshore.

On a linear dock face, any ice bustle will not increase the width for ice loading. Furthermore during the freeze up period observations at Nanisivik suggest that active hinge piece is minimal and about the same thickness as the ambient ice. In any case, even if it was thicker, the thinner ambient ice would preferentially fail on the outside of the thicker ice (and very likely in a rubbling load at a lower ice pressure than crushing). So, in summary the ice loads at the end of the freeze up period need not be adjusted for tidal effects on ice geometry.

The presence of a layer of ice frozen to the dock face will actually be beneficial in distributing high local ice pressures over a larger area.

During mid-winter, the lateral ice pressures due to slow lateral ice movements will not be affected by the ice bustles or the thickened hinge piece. This is generally confirmed by investigations at the Nanisivik dock.

During break up, it is predicted that the ice bustle and thickened hinge piece will deteriorate in place have limited ability to create higher loads on the dock face than those predicted d=for Scenario 3.

Vertical loads due to the presence of an ice bustle can be created by gravity and buoyancy forces as the water level changes due to tides and surges. These have been briefly examined for the configuration shown in Figure A.11; some results are shown in Table A.7. These loads are extremely small. The case of an ice sheet being caught under the edge of the bustle and failing in bending as the water level rises has also been examined; these results are shown in Table A.8; they are also very small.

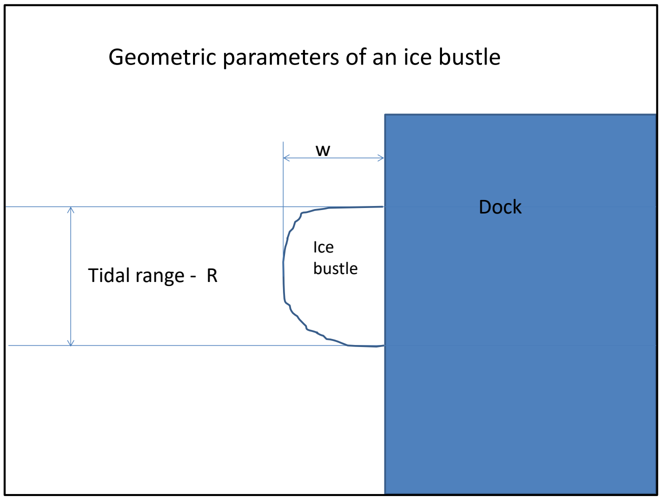


Figure A.11: Parameters to estimate bustle vertical forces

Table A.7: Loads due to gravity and buoyancy effects on typical ice bustles

	Bustle gravity/buoyancy loads				
R	w	ice density	Water density	Fd	Fu
m	m	kN/m3	kN/m3	MN/m	MN/m
2.4	1	9	10.3	0.0216	0.00312
2.4	1.5	9	10.3	0.0324	0.00468
3	0.5	9	10.3	0.0135	0.00195

Table A.8: Loads due to ice sheet being caught under a bustle and failing in bending

Input Data in blue: Derived values in black and results in green		winter 0.6m ice	Winter 2.05m but slow
Flexural strength of ice	(kPa)	**500**	**300**
Specific weight of ice	(kN/m^3)	**8.98**	**8.98**
Specific weight of water	(kN/m^3)	**10.20**	**10.20**
Young's modulus	(kPa)	**5.00E+06**	**2.00E+06**
Waterline width (D)	(m)	**50**	**50**
Thickness for breaking (hb)	(m)	**0.6**	**2.05**
Characteristic length (Lc)	(m)	9.92	19.83
Ratio of structure width to characteristic length		5.04	2.52
First break length (R)	(m)	7.74	15.47
Results			
Vertical line load based on first break (MN/m)		**0.0033**	**0.023**

A.5 ice loads on the exposed causeways

As an overall comment, most of the rock slopes (especially the causeway to shore) will not be exposed to significant ice push because of the sheltering effects of the outer quay and the shoreline itself. Furthermore, apart from the potential for armour stones to be pushed around, past experience suggests that global failures of rock berms is unlikely because of their high resistance to lateral ice loads. Therefore, the scenarios which follow are unlikely to be critical.

The potential critical failure modes of a rock berm are shown in Figure A.12. This figure is taken from early Caspian Sea work and some of the comments on the figure are not strictly relevant to this design. The message is that global sliding and decapitation resistance need to be higher than the ice loads; but some local edge failures may be acceptable (but should also be avoided).

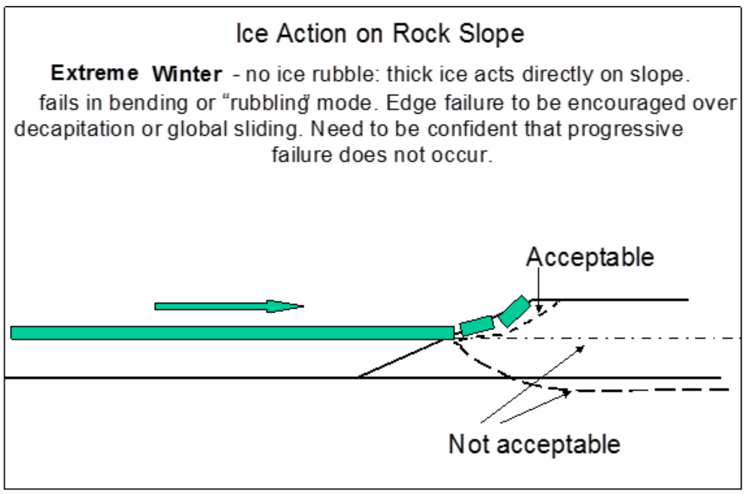


Figure A.12: General potential failure modes of a rock berm (Croasdale, 2003)

In reality, the interaction mode depicted of thick winter ice acting directly on the slope is unlikely once we use the same periods of ice loading as previously used for ice loads on the quay. These now follow. The approach for global loads will be to define some bounding interaction cases for which the rock slopes and causeway can be checked for overall stability and potential slope failures.

A.5.1 Scenario 1: Early winter with mobile ice up to 0.6m thick

The first bounding case will be to assume the 0.6m thick ice acts directly on the rock slope and fails in bending with some ride up and rubble creation. This is shown conceptually in Figure A.13.

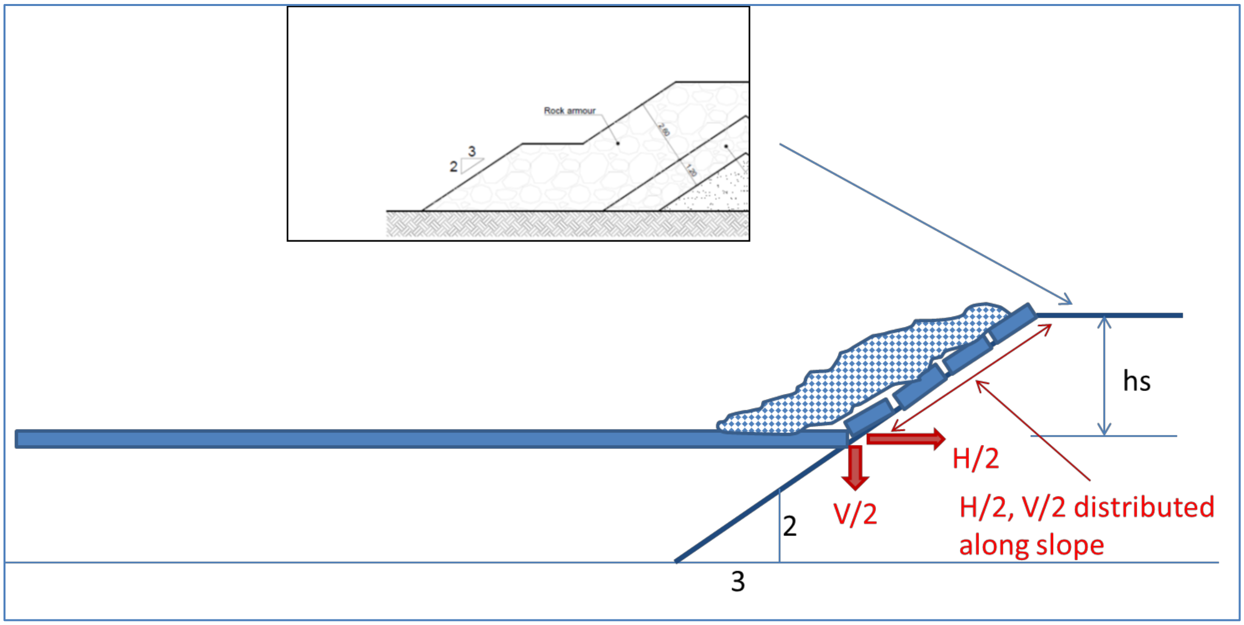


Figure A.13: Interaction of level ice on a rock slope – idealized to a uniform slope with ice failing in bending with ride up and rubble formation

The ice loads can be calculated using the method for ice action on a sloping face (ISO 19906 and Croasdale et al, 1994, 2016). In this case the slope angle is 34^{o} and an ice thickness of 0.6m will be used. Table A.9 gives the other detailed inputs and outputs (from a KRCA spreadsheet). The calculated value of H is 0.16 MN/m. It should be noted that if the rubble angle of repose is chosen to equal the slope angle it implies a single layer of ice ride up. If as in many situations ice rubble after riding up the slope tumbles back on the ice riding up, then the total global load is higher because ice has to be pushed through the ice rubble. In many cases the rubble is not a linear slope but can be flat near the top with a steeper slope lower down. The angle of repose in the spreadsheet is for an idealized linear slope and is chosen to give a plausible volume of ice rubble on the slope.

Table A.9: Early winter level ice (0.6m) acting on rock slope and failing in bending; slope height is 4m

Input Data in blue: Derived values in black and results in green		rock slope base case - winter 0.6m ice
Flexural strength of ice	(kPa)	500
Specific weight of ice	(kN/m^3)	8.98
Specific weight of water	(kN/m^3)	10.20
Young's modulus	(kPa)	5.00E+06
Poisson's ratio		0.3
Slope Angle	(deg)	34
Rubble angle of repose	(deg)	24.0
Rubble friction angle	(deg)	0
Waterline width (D)	(m)	30
Slope height (hs)	(m)	4
Width for HT (Dt)	(m)	30.00
Width for centre ride up	(m)	30.00
Ice-slope friction		0.5
Ice-ice friction		0.25
Thickness for ride up	(m)	0.6
Thickness for breaking (hb)	(m)	0.6
Non simultaneous factor for HB		1
Non - sim factor for breaking in ride up limit		1
Angle difference for ride up load	(deg)	5
Rubble porosity		0.15
Cohesion of rubble	(kPa)	5
m - fraction of h to be added to slope ride up (abnormal)		0.1
Characteristic length (Lc)	(m)	9.92
Ratio of structure width to characteristic length		3.02
First break length (R)	(m)	7.74
wB	(m)	54.32
Limit avg ride up height + hb/2	(m)	3.00
Second break length (R2)	(m)	3.4
Third break length	(m)	1.5
Block length ratio to thickness (2nd break)	(n)	5.6
y (second break) - has to be less than h	(m)	0.77
Results		
Predicted Horizontal Load (MN)		6.37
Predicted Horizontal Load less HT (MN)		4.82
Horizontal line load (MN/m)		0.16
Vertical line load (MN/m)		0.09

A.5.2 Scenario 2: Mid-winter with stable ice up to 2.05m thick.

In this period the ice will likely be frozen to the rock slope and there will low loading caused only by thermal and tidal jacking. It is likely that the vertical motion due to tides will occur a few m from the slope at a tidal crack. The loading should be assumed the same as for global loads on the dock of 0.3MN/m.

A.5.3 Scenario 3: Break -up

In most years the ice will melt in place with little ice action. However, it is possible (as with global loads on the dock face) that large offshore floes released during the break up period could be blown inshore and interact with some parts of the rock slopes. Therefore, the same loading situations are assumed in as Scenario 1 except that the ice is thicker but considerably weaker.

For ice acting directly on the rock slopes and failing in bending, the same spreadsheet is used and results added for this period with a thickness of 1.4m but weaker ice (50kPa flexural strength – from the Timco & Johnson, 2002). The results are shown in Table A.10. The loads are higher with values of 0.30MN/m for H and 0.17MN/m for V.

Table A.10: Break up case added for level ice acting directly on a rock slope

Input Data in blue: **Derived values in black** **and results in green**		**rock slope base case - winter 0.6m ice**	**break up 1.4m weak ice**
Flexural strength of ice	(kPa)	**500**	**50**
Specific weight of ice	(kN/m^3)	**8.98**	**8.98**
Specific weight of water	(kN/m^3)	**10.20**	**10.20**
Young's modulus	(kPa)	**5.00E+06**	**5.00E+05**
Poisson's ratio		**0.3**	**0.3**
Slope Angle	(deg)	**34**	**34**
Rubble angle of repose	(deg)	**24.0**	**24.0**
Rubble friction angle	(deg)	**0**	**0**
Waterline width (D)	(m)	**30**	**30**
Slope height (hs)	(m)	**3**	**3**
Width for HT (Dt)	(m)	**30.00**	**30.00**
Width for centre ride up	(m)	**30.00**	**30.00**
Ice-slope friction		**0.5**	**0.5**
Ice-ice friction		**0.25**	**0.25**
Thickness for ride up	(m)	**0.6**	**1.4**
Thickness for breaking (hb)	(m)	**0.6**	**1.4**
Non simultaneous factor for HB		**1**	**1**
Non - sim factor for breaking in ride up limit		**1**	**1**
Angle difference for ride up load	(deg)	**5**	**5**
Rubble porosity		**0.15**	**0.15**
Cohesion of rubble	(kPa)	**5**	**5**
m - fraction of h to be added to slope ride up (abnormal)		**0.1**	**0.1**
Characteristic length (Lc)	(m)	9.92	10.53
Ratio of structure width to characteristic length		3.02	2.85
First break length (R)	(m)	7.74	8.22
wB	(m)	54.32	55.82
Limit avg ride up height + hb/2	(m)	**3.00**	**3.00**
Second break length (R2)	(m)	3.4	3.6
Third break length	(m)	1.5	1.5
Block length ratio to thickness (2nd break) (n)		5.6	2.5
y (second break) - has to be less than h	(m)	0.77	0.37
Results			
Predicted Horizontal Load (MN)		6.37	12.77
Predicted Horizontal Load less HT (MN)		4.82	8.92
Horizontal line load (MN/m)		0.16	0.30
Vertical line load (MN/m)		0.09	0.17

A.5.4 Summary of loads on the exposed causeways

A summary of these loads is given in Table A.11. These are on a nominal width of 30m. It is assumed that stability calculations will be done on a linear basis. So limits to actual widths are not imposed. However, when looking at limits to ice encroachment it is noted that limited kinetic energy will likely limit the widths of ice that can be driven to the top of the causeways to about 30m.

Table A.11: Line loads on the exposed rock slopes (30m width)

Scenario	Bending failure and ride up	
	Horizontal MN/m	Vertical MN/m
Scenario 1: Freeze-up (0.6m strong ice)	0.16	0.09
Scenario 2: Break-up (1.4m weak ice)	0.30	0.07

A.6 Ice encroachment

It is predicted that even using very conservative assumptions there will be no ice encroachment onto the dock surface (because of the deep water and limited ice motion possible against the dock).

Ice encroachment is more likely and may occur on the exposed causeways or rock berms facing offshore. But this process is also subject to the constraints relating to driving forces and limited kinetic energy; and whether large floes can push past the dock face. The analysis in this report indicates that the smaller floes that can miss the dock face can have sufficient energy to fail and ride up about 8m. This situation is shown in Figure A14 when the water level is at MWL. If the incident occurred at HAT water level then some encroachment can be expected as depicted in Figure A15.

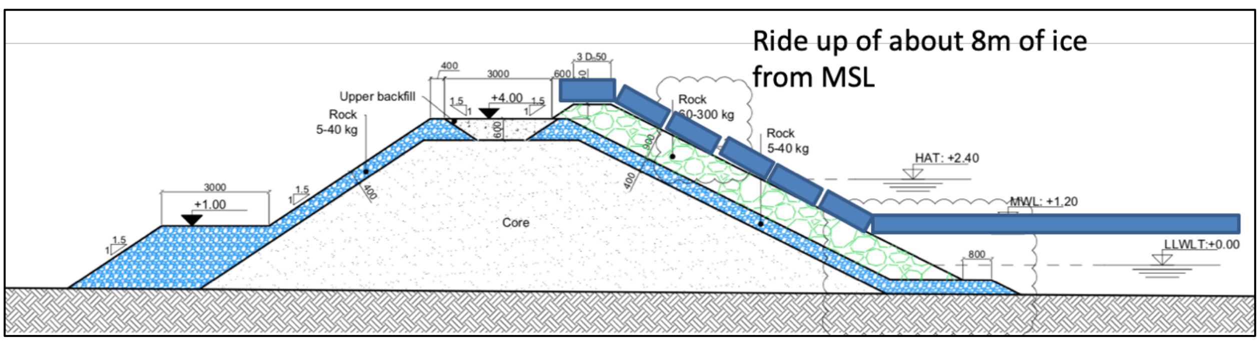


Figure A14: Maximum calculated ice ride up based on kinetic energy of interacting floe (over a 30m width) referenced to MSL

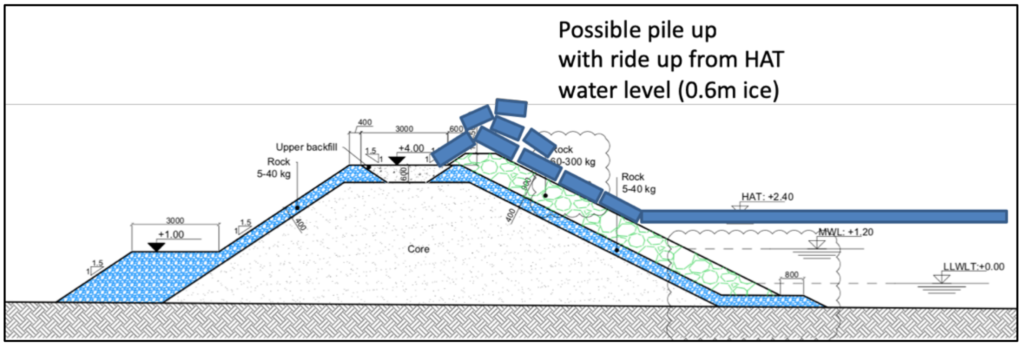


Figure A.15: Encroachment possible at high water level on exposed causeway slopes

The consequences of this kind of encroachment are not considered high as the dock is not being used in the winter and sensitive equipment does not appear to be located where encroachment may occur. Note that the mooring piles could be subject to encroachment of rubble ice, but the loads will be due to loose ice blocks and not decisive. There is a slight risk of encroachment in a similar type scenario at break up, but again the floes would have to bypass the main dock face and act only on the causeways. It is mentioned because there may be operations underway at break up with equipment and personnel on the causeways. If ice floes are still in the area, it will be prudent to have some kind of alert system and operational procedures in place.

QIA 83 ATTACHMENT 1: TABLE 1 UNDERWATER NOISE SCENARIOS MODELLED IN TSD #24, APPENDIX B

Table 1 Underwater Noise Scenarios modelled in TSD #24, Appendix B

Scenario	Operational Phase	Operation(s) and location abbreviation	Month used for basis of model	Daily duration(s)	Other details
1	2A	Vibratory pile driving at ND	August	960	Sheet piles (35 m length × 0.5 m width x 12.7 mm thick) driven by the ICE 28D vibratory driver.
2		Mooring PP at ED	October	86400	
3		Berthing PP at ED with 1 tug		12960	
4		Mooring PP at R1		86400	
5		Transiting PP at MP1		NA	Transit speed = 5 kn
6		Transiting PP with 2 tug escorts at MP1		NA	Transit speed = 5 kn
7		Transiting PP at KO		NA	Transit speed = 9 kn
8		Transiting PP at MI		NA	
9		Transiting PP at ES		NA	
10		Transiting PP at PI		NA	
11		Aggregate scenario including: • Vibratory pile driving at ND • Berthing PP at ED with 1 tug • Transiting PP with 2 tug escorts at MP1 • Mooring PP at A1 • Mooring PP at R1 • Transiting PP at MI	August	As described above per activity	

QIA 85 ATTACHMENT 1: TABLES 1-8

Table 1 Site name abbreviations used in Tables 2 - 8

Site name	Abbreviation
Pond Inlet	PI
Eclipse Sound	ES
Milne Inlet	MI
Koluktoo Bay	KB
Ragged Island, anchor point 1	R1
Ragged Island, anchor point 2	R2
Ragged Island, anchor point 3	R3
Ragged Island, anchor point 4	R4
Ragged Island, anchor point 5	R5
Milne Port, anchor point 1	A1
Milne Port, anchor point 2	A2
Existing dock (Post-Panamax capable)	ED
New dock (Cape size capable)	ND
Milne Port, transiting 1	MP1
Milne Port, transiting 2	MP2

Table 2. Comparison of mooring scenarios at Milne Port: Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 2: Mooring PP at ED			Mooring CS at ND			Scenario 12: Mooring PP at ED and CS at ND			Scenario 17: Mooring CS at A2		
	R_{max} (km)	$R_{95\%}$ (km)	*Area* (*km²*)	R_{max} (km)	$R_{95\%}$ (km)	*Area* (*km²*)	R_{max} (km)	$R_{95\%}$ (km)	*Area* (*km²*)	R_{max} (km)	$R_{95\%}$ (km)	*Area* (*km²*)
120	2.27	1.70	3.15	2.55	1.96	3.60	3.64	2.27	6.49	1.58	1.11	3.38
130	0.28	0.24	0.14	0.35	0.28	0.15				0.22	0.21	0.15
140	0.06	0.05	0.01	0.05	0.05	0.01				0.03	0.03	0.00
150	0.02	0.02	<0.002	0.02	0.02	<0.002				0.02	0.02	<0.002
160	0.02	0.02	<0.002	0.01	0.01	<0.002				<0.02	<0.02	<0.002
170	–	–	–	–	–	–				–	–	–
180	–	–	–	–	–	–				–	–	–
190	–	–	–	–	–	–				–	–	–
200	–	–	–	–	–	–				–	–	–

Table 3. Comparison of berthing scenarios: Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 3: Berthing PP at ED			Berthing CS at ND			Scenario 13: Berthing PP at ED and CS at ND		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	11.16	9.80	26.08	11.19	9.74	26.53	11.33	9.90	27.12
130	10.79	7.52	18.37	10.71	8.07	19.31			
140	2.76	1.95	5.46	2.78	1.94	6.11			
150	0.51	0.40	0.35	0.47	0.39	0.34			
160	0.11	0.10	0.02	0.11	0.10	0.02			
170	0.04	0.04	<0.002	0.04	0.04	<0.002			
180	0.04	0.04	<0.002	0.04	0.04	<0.002			
190	0.04	0.04	<0.002	0.04	0.04	<0.002			
200	–	–	–	–	–	–			

Table 4. Comparison of mooring scenarios at Ragged Island: Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 4: Mooring PP at R1			Mooring CS at R4		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	1.42	0.95	2.44	1.07	0.65	1.38
130	0.20	0.18	0.11	0.11	0.11	0.04
140	0.04	0.03	0.004	0.04	0.04	0.004
150	0.02	0.02	<0.002	0.02	0.02	<0.002
160	0.02	0.02	<0.002	0.01	0.01	<0.002
170	–	–	–	–	–	–
180	–	–	–	–	–	–
190	–	–	–	–	–	–
200	–	–	–	–	–	–

Table 5. Comparison of transiting scenarios (5 kn speed, no escorts): Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 5: Transiting PP at MP1			Transiting CS at MP2			Scenario 15: Transiting PP at MP1 and CS at MP2		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	1.70	1.30	3.31	3.69	2.37	7.87	4.42	3.07	10.36
130	0.28	0.27	0.21	0.50	0.41	0.47			
140	0.04	0.04	0.01	0.07	0.07	0.02			
150	0.02	0.02	<0.002	0.01	0.01	<0.002			
160	0.02	0.02	<0.002	0.01	0.01	<0.002			
170	0.01	0.01	<0.002	–	–	–			
180	–	–	–	–	–	–			
190	–	–	–	–	–	–			
200	–	–	–	–	–	–			

Table 6. Comparison of transiting scenarios (5 kn speed, with escorts): Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 6: Transiting PP at MP1 (with escorts)			Transiting CS at MP2 (with escorts)			Scenario 16: Transiting PP at MP1 and CS at MP2 (with escorts)		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	1.98	1.68	5.30	3.66	2.60	8.86	4.40	3.52	13.02
130	0.36	0.33	0.29	0.50	0.45	0.58	–	–	–
140	–	–	–	–	–	–	–	–	–
150	–	–	–	–	–	–	–	–	–
160	–	–	–	–	–	–	–	–	–
170	–	–	–	–	–	–	–	–	–
180	–	–	–	–	–	–	–	–	–
190	–	–	–	–	–	–	–	–	–
200	–	–	–	–	–	–	–	–	–

Table 7. Comparison of transiting scenarios (9 kn speed) at KO and MI: Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 7: Transiting PP at KO			Scenario 8: Transiting PP at MI			Scenario 18: Transiting CS at KO			Scenario 19: Transiting CS at MI		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	10.14	6.58	67.38	14.80	10.33	141.57	12.34	9.65	109.92	23.50	13.68	242.29
130	3.41	2.32	12.64	3.27	1.94	7.81	6.21	3.86	33.18	7.97	4.24	28.09
140	0.54	0.52	0.90	0.27	0.26	0.22	0.72	0.69	1.58	0.83	0.74	0.55
150	0.09	0.09	0.03	0.09	0.09	0.02	0.13	0.12	0.05	0.13	0.12	0.05
160	0.03	0.03	0.003	0.03	0.03	0.003	0.04	0.04	0.005	0.04	0.04	0.005
170	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002
180	0.01	0.01	<0.002	0.02	0.02	<0.002	0.01	0.01	<0.002	0.02	0.02	<0.002
190	–	–	–	–	–	–	–	–	–	–	–	–
200	–	–	–	–	–	–	–	–	–	–	–	–

Table 8. Comparison of transiting scenarios (9 kn speed) at ES and PI: Threshold distances to broadband SPL (10 Hz–25 kHz). A dash indicates that threshold was not reached.

SPL (dB re 1 µPa)	Scenario 9: Transiting PP at ES			Scenario 10: Transiting PP at PI			Scenario 20: Transiting CS at ES			Scenario 21: Transiting CS at PI		
	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*	R_{max} (km)	$R_{95\%}$ (km)	*Area (km²)*
120	19.24	11.20	212.43	9.82	5.93	69.31	29.49	18.22	563.18	19.79	10.06	185.44
130	2.22	1.79	8.15	2.86	1.52	4.67	6.84	3.46	30.31	4.92	3.40	17.66
140	0.28	0.26	0.23	0.26	0.26	0.22	0.78	0.75	1.37	0.39	0.38	0.46
150	0.09	0.09	0.03	0.09	0.09	0.02	0.13	0.12	0.05	0.13	0.12	0.05
160	0.03	0.03	0.003	0.03	0.03	0.003	0.04	0.04	0.005	0.04	0.04	0.005
170	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002
180	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002	0.02	0.02	<0.002
190	–	–	–	–	–	–	–	–	–	–	–	–
200	–	–	–	–	–	–	–	–	–	–	–	–

QIA 95 ATTACHMENT 1: ADAPTIVE MANAGEMENT APPROACH

Baffinland

ᑎᑎᖅᑲᖅ ᑐᑭᓯᐅᒪᔾᔪᑎᒃᓴᖅ

ᐅᑲᖓ: ᓯᐅᒋ ᐅᐃᓕᐊᒻᓴᓐ ᐸᑐᐊᕆ, ᐅᖃᐅᔾᔨᒋᐊᖅᑎᒻᒪᕆᒃ ᐊᖓᔪᖅᑳᒧᑦ, ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ

ᐅᕙᓐᓄᑦ: ᒪᐃᑯᓐ ᓗᐊᑦ-Hᐅᐃᐅᓪ, ᐊᐅᓚᑦᑎᔨ ᑎᒥᐅᔫᔅ ᒪᑭᒪᐅᑎᖃᕈᓐᓇᕐᓂᖓᓄᑦ, ᓄᓘᔭᓕᕆᔨᒃᑯᑦ

ᐱᑦᔪᑎᖃᑦᑐᖅ: *ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ, ᐱᓗᐊᖏᓂᖕᓴᐅᑎᑦᑎᓂᖕ ᐊᒻᒪᓗ ᐃᓕᑦᑑᔾᔨᓂᕐᒧᑦ ᑲᒪᒋᔭᖃᓐᓂᕐᒧᑦ ᐊᑦᖏᖕᑐᒃᑯᑦ ᑕᑯᓈᖕᑕᐅᓂᖓ*

<u>ᐋᖅᑭᒋᐊᒃᑲᓂᖅᑕᑦᑕᕈᓐᓇᖅᑐᒥᒃ ᐊᐅᓚᑦᑎᓂᖅ</u>

ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᐱᓕᕆᓇᓯᐊᖅᓯᒪᒻᒪᕆᒻᒪᑕ 2018-ᒥ ᑐᓯᐅᒪᓂᖕᓴᐅᓕᕐᑯᓪᓗᑕ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᒃᑯᓐᓂᑦ ᐊᒻᒪᓗ ᓄᓇᓖᑦ ᑭᒡᒐᖅᑐᐃᔨᖕᑕ ᐃᓱᒫᓘᑎᒋᔭᖕᒋᓐᓂᒃ ᑐᕌᖓᔪᑦ ᐅᓪᓗᒥᐅᔪᖅ ᐱᓕᕆᓂᐅᔪᒧᑦ ᐊᒻᒪᓗ ᓇᓗᓇᐃᖅᑕᐅᓗᐊᖅᓯᒪᔪᑦ ᐃᓱᒫᓘᑎᖕᒋᑦ ᑕᒪᑐᒨᖓ ᐊᖕᒋᓪᓕᕙᓪᓕᐊᑐᐃᓐᓇᑎᓪᓗᒍ ᐱᓕᕆᐊᖑᔪᑦ ᐊᒻᒪᓗ ᐅᒥᐊᕐᔪᐊᑦ ᑎᑭᖃᑦᑕᖅᑐᑦ ᐅᓄᖅᑑᓂᖕᒋᓐᓄᑦ ᐃᑲᔫᑎᔪᑦ ᖄᓇᐅᔭᓕᐅᕋᓯᐊᕐᓂᒃᑯᑦ ᑲᔪᓯᓐᓇᑐᒃᑯᑦ ᐱᕈᖅᐸᓪᓕᐊᕐᑯᓪᓗᒍ ᐱᓕᕆᐊᖑᔪᖅ.

ᐱᑕᖃᐸᕆᖅᑐᑦ ᐱᔪᓐᓇᐅᑎᑦ ᐱᔭᕆᐊᖃᑦᑎᑎᒻᒪᑕ ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᐃᑲᔪᖅᑕᐅᖃᑦᑕᕐᓗᑎᒃ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᖕᒋᓐᓂᑦ, ᓄᓇᓕᖕᒐ ᒥᑦᑎᒪᑕᓕᒃ ᐊᒻᒪᓗ ᐊᓯᖕᒋᓐᓂᒃ ᒪᓕᒐᓕᖃᑎᑦᑎᔨᐅᔪᓂᒃ ᐱᓕᕆᓂᖃᑎᓪᓗᒋᑦ ᖃᐅᔨᓴᕐᓂᖕᒋᓐᓄᑦ ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᓇᑐᓇᐃᖅᑎᕆᓇᓯᐊᖅᑐᒃ ᓇᓚᐅᑦᑖᖅᑕᐅᓯᒪᔪᖅᑯᑎᖕᒋᓐᓂᒃ ᑭᖑᓪᓕᖅᐹᒥᒃ ᐊᕙᐅᑎᐅᑉ ᐊᒃᑐᖅᑕᐅᓂᕆᔭᖕᒋᓐᓄᑦ ᑎᑎᕋᖅᓯᒪᔪᑦ ᐱᓕᕆᓂᐅᔪᒧᑦ. ᐊᒻᒪᑦᑕᐅᖅ, ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᑐᓴᕋᓯᐊᖕᒋᓐᓇᖅᑕᖅᑐᑦ ᑖᒃᑯᓈᓐᖓᓕᒃᑖᖅ ᓄᓇᓖᓐᓂᑦ ᐊᒻᒪᓗ ᐊᓯᖕᒋᓐᓂᒃ ᐃᓱᒫᓘᑎᖃᑦᑐᓂᒃ ᐊᐅᓚᓂᕆᔭᐅᔪᖅ ᐸᕐᓇᒃᓯᒪᑦᑎᐊᕐᑯᓪᓗᒍ. ᑐᓴᖅᑕᐅᔪᑦ ᑖᒃᑯᓈᖓᓄᑦ ᐃᓚᒋᔭᐅᔪᓂᒃ ᐱᑕᖃᑦᑎᑕᐅᖃᑦᑕᖅᑐᑦ ᐋᖅᑭᒃᑎᕆᓪᓗᑎᒃ ᐊᕐᕌᒍᕐᒋᑦᑐᓂᒃ ᐊᑐᖅᑕᐅᒍᓐᓇᖅᑐᓂᒃ ᒫᕿᒃᓕᖓᔪᒃᑯᑦ ᑐᓴᖅᑎᑦᑎᓇᓯᐊᕐᓂᕐᒧᑦ (ᑕᑯᓗᒍ ᑎᑎᕋᖅᓯᒪᔪᖅ 1). ᐱᓕᕆᓇᓯᐊᕐᓂᐅᖃᑦᑕᕋᓗᐊᖅᑎᓪᓗᒍ, ᐃᓕᑕᕆᔭᐅᖃᑦᑕᖅᐳᖅ ᐃᓱᒫᓘᑎᑦᑲᐃᓐᓇᐅᔭᕐᓂᖓᓂᒃ ᓄᓇᓕᓐᓂ ᐃᓚᒋᔭᐅᔪᓂᑦ ᐊᒻᒪᓗ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᖕᒋᑦ ᒥᒃᓵᓄᑦ ᐱᓕᕆᐊᖑᔪᔅ ᐊᒻᒪᓗ ᓇᓗᓇᖅᑑᓂᖓᓂᒃ ᐱᓕᕆᐊᖑᕙᑦᑐᓂᒃ ᖃᐅᔨᓴᕐᓂᒃᑯᑦ. ᑕᒪᓐᓇ ᐱᑦᔪᑎᒋᑦᑐᒍ ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᐋᖅᑭᒃᓯᓯᒪᕗᑦ ᐊᑐᕐᓂᐊᓕᕐᓗᑎᒃ ᐋᖅᑭᒋᐊᒃᑲᓂᖅᑕᑦᑕᖅᑐᒥᒃ ᐊᐅᓚᑦᑎᕙᓪᓕᐊᓂᕐᒥᒃ ᐊᒻᒪᓗ ᐊᓯᒃᑲᓐᓂᖕᒋᓐᓂᒃ ᑲᒪᒋᔭᕆᐊᖅᑕᑦᑎᒪᓂᒃ (ᑎᑎᕋᖅᓯᒪᔪᖅ 1). ᐋᖅᑭᒋᐊᒃᑲᓂᖅᑕᑦᑕᕈᓐᓇᖅᑐᒥᒃ ᐊᐅᓚᑦᑎᓂᖅ ᐱᕕᖅᑕᑎᑎᒻᒪᑦ ᐱᓕᕆᐊᖑᔪᓂᒃ ᖃᐅᔨᓴᕐᓂᕐᒥᒃ ᐃᒻᒥᒃᑰᖓᓪᓗᑎᒃ ᐊᒻᒪᓗ ᐊᕗᑎᖕᒋᑦᑑᓐᓄᑦ ᖁᓕᓃᓐᓄᑦ ᐱᓕᕆᐊᖑᓪᓗᓂ.

ᓯᕗᓪᓕᖅᐹᖅ, ᐱᓕᕆᓂᒃᑯᑦ ᐊᒃᑐᐃᓂᕐᒥᒃ ᖃᐅᔨᓴᕐᓂᖅ ᐱᓕᕆᐊᖑᕙᑦᑐᖅ ᐊᒻᒪᓗ ᖃᐅᔨᓴᖅᑕᐅᓪᓗᓂ ᓄᓘᔭᓕᕆᔨᒃᑯᑦ ᓴᓇᔪᓕᕆᔨᖕᒋᓐᓄᑦ ᖃᔨᒪᓪᓕᕆᑦᑐᓄᑦ ᐊᒻᒪᓗ ᖃᐅᔨᒪᔨᑕᓄᑦ. ᖃᐅᔨᓴᕈᑎᕕᓃᑦ ᑐᓂᔭᐅᖃᑦᑕᖅᑐᑦ ᐊᒻᒪᓗ ᖃᐅᔨᓴᖅᑕᐅᓪᓗᑎᒃ ᓯᓚᑖᓃᑦᑐᓄᑦ ᕿᒥᕐᕈᔨᓄᑦ ᐊᒻᒪᓗ ᓴᓇᔪᓕᕆᓂᖕᒧᑦ ᖃᐅᔨᒪᔾᔪᒪᕆᓐᓄᑦ, ᕿᑭᖅᑕᓂᒃ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᖕᒋᑦ ᐊᒻᒪᓗ ᓄᓇᓕᖕᓂ ᓄᓇᖃᑦᑎᐅᔪᓄᑦ.

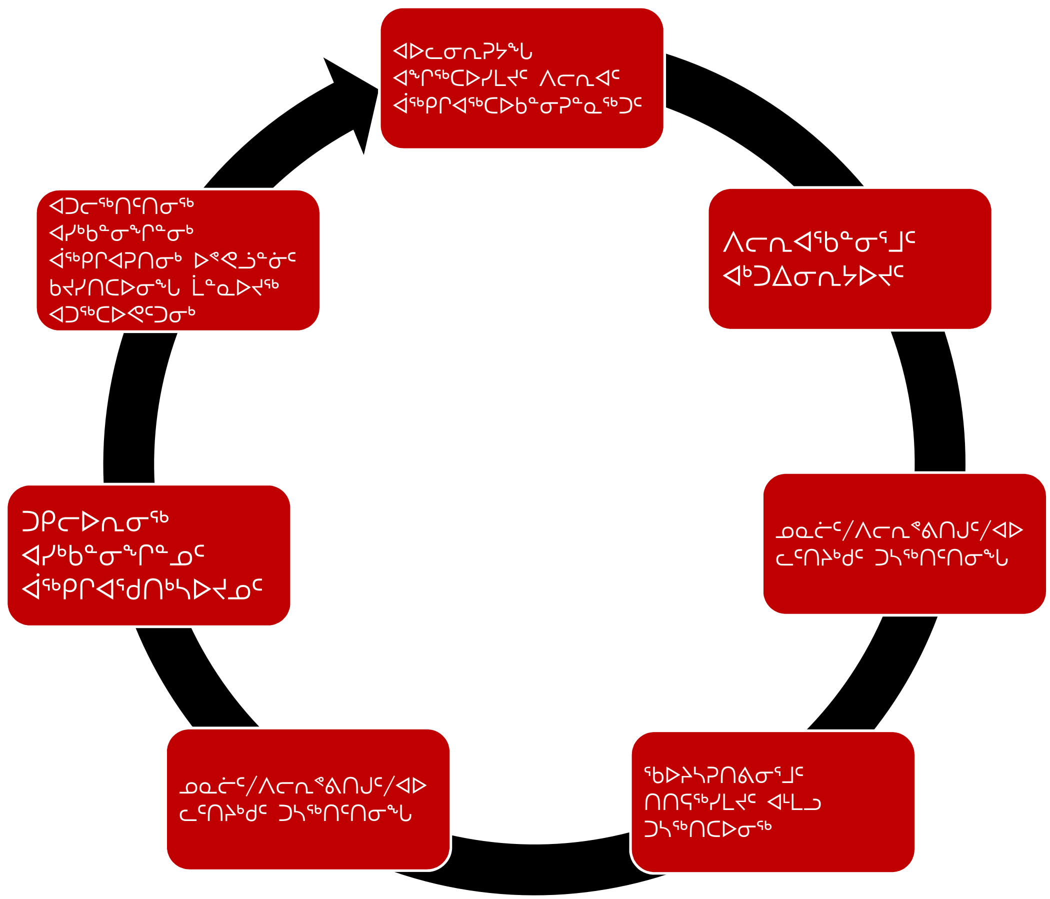
Baffinland

ᑎᑎᕋᖅᓯᒪᔪᑦ 1 – ᐱᓕᕆᐊᕐᒥᒃ ᕿᒥᕐᕈᕚᓪᓕᐊᓂᖅ

ᑐᓴᕐᓴᓂᒃ ᑐᓴᖅᑕᐅᑎᑦᑎᖅᑳᑕ ᕐᓂᖅ	ᓇᑐᓇᐃᖅᓯᔾᔪᑎᑦ	ᐱᕙᒃᓴᖅᑫᕙᖓ	ᑲᑐᔾᔭᖅᑎᒌᒃᑐᐊᑕᐃᑦ							
			ᓄᓇᕗᑦᒥ ᐊᕙᑎᓕᕆᓂᕐᒧᑦ ᑲᑎᒪᔩᑦ	ᓄᓇᕗᑦᒥ ᐃᒪᓕᕆᔨᒃᑯᑦ ᑲᑎᒪᔨᖏᑦ	QIA	ᒪᓕᒐᑎᒍᑦ ᑲᑐᔾᔭᖅᑎᒌᑦ	ᐱᓕᕆᔨᐅᔪᑦ (ᓄᓇᕗᑦ ᒐᕙᒪᒃᑯᑦ, ᐃᒪᕐᒥᐅᑕᓕᕆᔨᒃᑯᑦ, ᒥᕐᖑᐃᓯᖅᕕᓕᕆᔨᒃᑯᑦ, ECCC, QIA, ᒥᑦᑎᒪᑕᓕᒃᒥ ᐊᖑᓇᓱᒃᑐᓕᕿᔨᒃᑯᑦ ᑲᑎᒪᔨᖏᑦ)	ᓴᓇᔪᓕᕆᓂᕐᒧᑦ ᖃᐅᔨᒪᒻᒪᕆᑦᑐᑦ	ᐊᖑᓇᓱᒃᑐ ᓕᕆᔨᒃᑯᑦ	Hᐋᒻᓚᑦᖑᔪᑦ
ᐊᕐᕌᒍᑕᒫᑦ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ ᐅᓂᒃᑳᑦ - ᓯᕗᓪᓕᖅᐹᖅᓯᐅᑎᑦ	ᓯᕗᓪᓕᖅᐹᖅᓯᐅᑦ ᐅᓂᒃᑳᖅ – ᖃᐅᔨᓴᕈᑎᕙᓃᑦ ᓄᓇᒦᑦᑐᓄᑦ ᓂᕐᔪᑎᓄᑦ ᐊᒻᒪᓗ ᐃᒫᓂ ᖃᐅᔨᓴᕋᓱᐊᕐᓃᑦ	ᓈᕝᕗᐊᓕ 15			✓	✓	✓	✓	✓	
ᐊᕐᕌᒍᑕᒫᑦ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ ᐅᓂᒃᑳᑦ - ᑭᖑᓪᓕᖅᐹᖓ	ᑭᖑᓪᓕᖅᐹᖅ ᐅᓂᒃᑳᖅ – ᐊᕐᕌᒍᑕᒫᑦ ᖃᐅᔨᓴᕈᑎᕙᓃᑦ ᓄᓇᒦᑦᑐᓄᑦ ᓂᕐᔪᑎᓄᑦ ᐊᒻᒪᓗ ᐃᒫᓂ ᖃᐅᔨᓴᕈᑎᕙᓃᑦ. ᐃᓚᓕᐅᔾᔨᔪᑦ ᐅᖃᐅᓯᐅᓯᒪᔪᓂᒃ ᐱᓕᕆᔨᐅᔪᓄᑦ ᓯᕗᓪᓕᖅᐹᖅᓯᐅᑎ ᐅᓂᒃᑳᒧᑦ	ᒫᑦᓯ 31	✓	✓	✓	✓	✓	✓	✓	✓
NIRB/NWB ᐊᕐᕌᒍᑕᒫᖅᓯᐅᑦ ᐅᓂᒃᑳᑦ	ᐅᓂᒃᑳᓖᑦ ᐊᕐᕌᒍᑕᒫᑦ ᐊᐅᓚᓂᕆᔭᐅᔪᒧᑦ ᐱᓕᕆᐊᕐᓂᒃ, ᖃᐅᔨᓴᕐᓂᐅᔪᓂᒃ ᐊᒻᒪᓗ ᒪᓕᑉᐸᑦᑕᖏᓐᓂᒃ ᒪᓕᒐᑎᒍᑦ ᐱᔪᓐᓇᐅᑏᑦ	ᒫᑦᓯ 31	✓	✓	✓	✓	✓	✓	✓	✓
ᐅᖃᐅᓯᒃᓴᓪᓗᐊᑕᓄᑦ ᑲᑎᒪᓃᑦ – ᐅᓯᐊᕐᔪᐊᓕᕆᓂᖅ, ᐊᓯᖏᓪᓗ.	ᑲᑎᒪᖅᑲᑎᒌᑦᑐᑎᒃ ᑲᑎᒪᓃᑦ ᑖᒃᑯᐊᓗ MHTO, Hᐋᒻᓚᒃᑯᑦ ᐊᒻᒪᓗ QIA ᑭᒡᒐᑐᖅᑎᖏᑦ ᐅᖃᐅᓯᖅᑲᑦᑐᑎᒃ ᐃᓱᒫᓘᑎᓪᓗᐊᑕᓂᒃ ᐊᒻᒪᓗ/ᐅᕝᕙᓘᓐᓃᑦ ᐃᓚᖓᓄᑦ ᐱᓕᕆᐊᖑᔾᔪᑉ ᐊᐅᓚᓂᕆᔭᖓᓄᑦ	ᐃᓱᐊ ᐊᒻᒪᓗ ᐱᕆᐊᕐᓂᖓ ᐅᒥᐊᖅᔪᐊᖅᑕᖃᓐᓇᐅᑉ, ᐱᑕᖃᕆᐊᖃᑦᑎᓪᓗᒋᑦ			✓			✓	✓	

ᑐᓴᕐᓴᓂᒃ ᑐᓴᖅᑕᐅᑎᑦᑎᖃᑦᑕ ᕐᓂᖅ	ᓇᓗᓇᐃᖅᓯᔾᔪᑎᑦ	ᐱᕕᒃᓴᖃᕖᓐᖓ	ᑲᑐᔾᔨᖃᑎᒌᓪᓗᐊᑕᐃᑦ							
			ᓄᓇᕗᑦᒥ ᐊᕙᑎᓕᕆᓂᕐᒧᑦ ᑲᑎᒪᔩᑦ	ᓄᓇᕗᑦᒥ ᐃᒪᓕᕆᔨᒃᑯᑦ ᑲᑎᒪᔨᖏᑦ	QIA	ᒪᓕᒐᑎᒍᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ	ᐱᓕᕆᔨᐅᔪᑦ (ᓄᓇᕗᑦ ᒐᕙᒪᒃᑯᑦ, ᐃᒪᕐᒥᐅᑕᓕᕆᔨᒃᑯᑦ, ᒥᕐᖑᐃᓯᖅᕿᓕᕆᔨᒃᑯᑦ, ECCC, QIA, ᒥᑦᑎᒪᑕᓕᒃᒥ ᐊᖑᓇᓱᒃᑐᓕᕿᔨᒃᑯᑦ ᑲᑎᒪᔨᖏᑦ)	ᓴᓇᔪᓕᕆᓂᕐᒧᑦ ᖃᐅᔨᒪᒻᒪᕆᑦᑐᑦ	ᐊᖑᓇᓱᒃᑐ ᓕᕆᔨᒃᑯᑦ	Hᐋᓪᓚᑦᖑᔪᑦ
ᐱᓕᕆᐊᕐᒧᑦ ᐅᓪᓗᒥᒨᖓᔪᓂᒃ ᑐᓴᖅᑎᑦᑎᓂᕐᒧᑦ ᑲᑎᒪᓃᑦ	ᐊᕐᕌᒍᑕᒫᖅᓯᐅᑎᒥᒃ ᑲᑎᒪᕐᑲᑎᕐᑲᐸᑦᑐᑦ ᐃᓄᓐᓂᒃ ᐊᒻᒪᓗ QIA ᑮᓇᖅᑐᐃᔾᔨᓐᓂᑦ ᐅᓪᓗᒥᒨᖓᔪᓂᒃ ᑐᓴᖅᑎᑕᐅᓪᓗᑎᒃ ᐱᓕᕆᖅᑕᐅᔪᒪᔪᑦ ᑕᒪᑐᒧᖓ ᑲᔪᓯᔪᒧᑦ ᐊᐅᓚᓂᕆᔭᐅᔪᓄᑦ ᐅᕙᓅᓐᓂᑦ ᐊᓯᔾᔨᖅᑕᐅᔪᒪᔪᓄᑦ ᐱᓕᕆᐊᖑᔪᑎᒍᑦ	ᐊᕐᕌᒍᑕᒫᑦ, ᑲᑎᒪᑦᐊᕐᑲᕋᐃᒻᒪ ᑕ	✓	✓	✓	✓	✓	✓	✓	✓
ᐱᓕᕆᕝᕕᐅᔪᒧᑦ ᐳᓚᕋᓐᓃᑦ	ᐳᓛᕆᐊᕐᓂᖅ ᓄᓘᔭᓄᑦ ᐊᒻᒪᓗ ᕿᓐᖑᐊᓄᑦ ᑕᑯᔭᖅᑐᖅᑐᑎᒃ ᐊᐅᓚᓂᕆᔭᐅᔪᓂᒃ ᐊᒻᒪᓗ ᐅᖃᐅᓯᖃᑦᑐᑎᒃ ᐃᓱᒫᓗᑕᐅᔪᓂᒃ ᓄᓇᒦᑦᑐᑎᒃ	ᐳᓛᕆᐊᖅᑐᖃᕆ ᐊᖃᐸᑦ ᐅᕙᓅᓐᓂᑦ ᑐᒃᓯᕌᖑᓯᒪᐸᑦ						✓	✓	✓
ᖃᐅᔨᓴᕐᓃᑦ /ᕿᓇᐅᔭᐃᑦ ᖃᐅᔨᓴᖅᑕᐅᓂᖏᑦ	ᒪᓕᒐᑎᒎᖓᔪᑦ ᖃᐅᔨᓴᕐᓃᑦ ᕿᓇᐅᔭᓂᒃ ᐅᕙᓅᓐᓃᑦ ᕿᒥᕐᕈᓂᐅᔪᑦ ᒪᓕᒃᑎᐊᕋᓗᐊᕐᒪᖔᑕ ᐊᑐᖅᑕᐅᔪᓂᒃ ᐱᔪᓐᓇᐅᑎᓂᒃ ᐊᒻᒪᓗ ᐊᖏᖅᑕᐅᔭᕆᐊᖃᑦᑐᓂᒃ	ᐊᐅᔭᒃᑯᓖᖅ (ᔪᓚᐃᒥᑦ ᓯᑎᐱᕆᒧᑦ)	✓	✓	✓	✓				
ᐃᓚᐅᖃᑕᐅᓂᖅ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ	ᑳᑐᓛᒃᑕᐅᓯᒪᓂᒃᑯᑦ ᐃᖅᑲᓇᐃᔮᓴᐃᑦ ᐅᕙᓅᓐᓂᑦ ᐱᓕᕆᖃᑎᒌᓂᒃᑯᑦ ᐊᕙᑎᓕᕆᓂᕐᒧᑦ ᖃᐅᔨᓴᕐᓂᖅ	ᐊᐅᔭᒃᑯᑦ ᓄᓇᒥ (ᔪᓚᐃᒥᑦ ᐊᒃᑑᕙᒧᑦ)				✓		✓	✓	
ᑐᓴᖅᑕᐅᔪᑦ ᑕᒪᑐᒧᖓ ᐊᓯᒃᑲᓂᖓᓄᑦ ᐋᖅᑭᒋᐊᖅᓯᔾᔪᑎᐅᔪᓄᑦ	ᑐᓂᓯᓂᖅ ᓴᓇᔪᓕᕆᓂᕐᒧᑦ ᐅᖃᐅᓯᐅᔪᓂᒃ ᐊᒻᒪᓗ ᑭᐅᔾᔪᑎᓂᒃ ᐅᕙᓅᓐᓂᑦ ᑲᑎᒪᕐᑲᑎᕐᑲᓪᓗᑎᒃ ᐅᖃᐅᓯᖃᕐᓗᑎᒃ ᐋᖅᑭᒋᐊᖅᑕᐅᔪᒪᔪᓂᒃ ᐅᕙᓅᓐᓂᑦ ᐃᓚᒋᐊᕈᑎᓂᒃ ᒫᓇᐅᔪᖅ ᐋᖅᑭᒋᐊᖅᑎᕆᔾᔪᑎᓄᑦ	ᐱᔭᐅᔭᕆᐊᖃᕐᐸᑦ	✓	✓	✓	✓	✓	✓	✓	✓

Baffinland



<u>ᑎᑎᕋᖅᓯᒪᔪᖅ 1 – ᓄᓘᔾᓕᕆᔾᑯᑦ ᐋᖅᑭᒋᐊᒃᑲᓐᓂᖅᑲᑦᑕᕈᓐᓇᖅᑐᒥᒃ ᐊᐅᓚᑦᑎᔾᔪᑎᖓ</u>

ᑭᐅᔾᔪᑎᑦ ᓄᓇᓖᑦ ᐃᓱᒫᓘᑎᖏᓐᓄᑦ

ᐅᖃᐅᓯᕆᔭᐅᓚᐅᕐᒥᑎᓪᓗᒍ, ᖃᐅᔨᓴᕈᑎᕕᓃᑦ ᓇᓗᓇᐃᖅᓯᓯᒪᔪᑦ ᑖᓐᓇ ᐱᓕᕆᓂᐅᔪᖅ ᐱᓗᐊᖅᑐᒥᒃ ᐅᕙᓘᓐᓃᑦ ᓱᕋᑦᑎᕆᔪᒥᒃ ᐊᒃᑐᐃᓂᖃᖕᒋᑦᑐᖅ ᐊᕙᑎᒥᒃ, ᖃᐅᔨᓵᖃᑦᑕᕐᓂᐅᓯᒪᔪᑦ ᕿᖑᓕᕇᑦ ᓄᓇᓕᓐᓂᒥᐅᑕᐅᔪᓄᑦ ᐊᒻᒪᓗ ᑖᒃᑯᐊ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᓇᓗᓇᐃᖅᓯᓯᒪᔪᑦ ᓇᓗᓇᕐᓂᕆᔾᔭᖏᓐᓄᑦ ᖃᐅᔨᓴᕈᑏᑦ ᐊᒻᒪᓗ ᐃᓱᒫᓘᑎᖃᑦᑐᑎᒃ ᑕᒪᑐᒧᖓ ᐱᓕᕆᐊᕐᖑᔪᒧᑦ. ᑕᐃᒪᐃᒻᒪᑦ, ᐊᕐᕌᒍᓐᓂ 2017 ᐊᒻᒪᓗ 2018, ᓄᓘᔭᓕᕆᔩᒃᑯᑦ ᐅᖃᓪᓚᖃᑎᖃᑦᑐᑎᒃ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᖏᓐᓂᑦ ᐊᒻᒪᓗ ᒥᑦᑎᒪᑕᓕᒃ ᐊᖑᓇᓱᒃᑐᓕᕆᔩᒃᑯᖏᓐᓂᑦ ᓇᓗᓇᐃᖅᓯᓚᐅᖅᑐᑦ ᐊᓯᒃᑲᓐᓂᐊᓂᒃ ᐋᖅᑭᒋᐊᖅᓯᔾᔪᑎᓭᓂᒃ ᐱᖏᐊᖅᑎᑦᑎᔪᖃᕆᐊᓕᒃ ᐊᒻᒪᓗ ᐊᑐᓕᖅᑎᑕᐅᓗᑎᒃ, ᐅᕝᕕᑐᖅᑎᐊᓂᕐᒧᑦ ᐱᓕᕆᐊᖃᕋᓱᐊᕐᓂᖅ ᐊᑐᖅᑕᐅᑎᐊᕐᓂᐊᕐᒪᑦ ᐱᓕᕆᓂᐅᔫᒃᑯᑦ.

2018–ᒥ, ᑖᒃᑯᐊᑦᑕᐅᖅ ᓄᓘᔭᓕᕆᔩᒃᑯᑦ ᑲᒪᒋᔭᖃᓗᐊᓐᖑᐊᓚᐅᕐᒥᔪᑦ ᐊᓯᒃᑲᓐᓂᕐᖏᓐᓂᒃ ᐋᖅᑭᒋᐊᖅᑎᕆᔾᔪᑎᒃᓴᓂᒃ ᐊᒻᒪᓗ ᖃᐅᔨᓴᕐᓂᕐᒥᒃ ᑭᐅᔾᔪᑎᐅᑦᑎᐊᖅᑐᑦ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᒃᑯᑦ ᐊᒻᒪᓗ ᒥᑦᑎᒪᑕᓕᒃᒥᐅᑦ ᐃᓱᒫᓘᑎᒋᔭᖏᓐᓂᒃ ᑕᒪᑐᒧᖓ ᐅᒥᐊᕐᔪᐊᖃᖃᑦᑕᓂᕐᒧᑦ, ᑕᕆᐅᕐᒥᐅᑕᐃᑦ ᓂᕐᔪᑎᓂᒃ ᐊᒻᒪᓗ ᐳᔪᖃᖃᑦᑕᕐᓂᖓᓄᑦ. ᐊᑕᖏᔫᒃᑯᑦ ᑕᐅᑐᒃᑕᐅᓂᖓ ᐊᑐᓕᖅᑎᑎᔾᔪᑎᕕᓃᑦ ᐋᖅᑭᒋᐊᖅᑲᑦᑕᖃᓐᓂᕈᓇᓂᕐᒧᑦ ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᓄᓘᔭᓕᕆᔩᒃᑯᓐᓂᑦ ᐱᓕᕆᐊᖑᓯᒪᔪᖅ ᐱᖃᓯᐅᑎᓯᒪᑦᓲᖅ ᑖᕙᓂ ᑎᑎᕋᖅᓯᒪᔪᖅ 2-ᒥ.

Baffinland

<u>ᑎᑎᕋᖅᓯᒪᔪᖅ 2 –ᐃᒪᕐᒥᐅᑕᓂᒃ ᐊᒻᒪᓗ ᐳᔪᕐᓗᕐᒧᑦ ᐊᒃᑐᖅᑕᐅᓂᑦ, ᖃᐅᔨᓴᕐᓂᖅ ᐊᒻᒪᓗ ᐱᓗᐊᖑᑎᑦᑎᓂᖅᓵᐅᓇᓱᐊᕐᓂᖅ</u>

<table>
<tr><th>ᐱᕐᔪᑎᐅᔪᖅ</th><th>ᐊᒃᑐᐃᓂᖃᕈᓐᓇᕐᓂᖓ</th><th>ᓯᕗᓂᐊᒍᑦ-2018 ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎ ᐱᓗᐊᖑᑎᑦᑎᓂᖅᓵᐅᓇᓱᐊᕐᓂᕐᒧᑦ ᐊᒻᒪᓗ ᐱᓕᕆᐊᒃᓴᕆᔭᐅᔪᓄᑦ</th><th>ᖃᐅᔨᓴᕐᓂᖅ</th><th>ᓇᐃᓈᖅᓯᒪᔪᑦ ᖃᐅᔨᓴᕈᑎᕕᓃᑦ ᐊᒻᒪᓗ ᓄᓇᓖᑦ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒋᑦ ᑐᓴᖅᑎᑎᔪᑎᖏᑦ</th><th>ᐊᓯᒃᑲᓂᖏᑦ ᐱᓗᐊᖑᑎᑦᑎᓂᖅᓵᐅᓇᓱᐊᕐᓂᕐᒧᑦ ᐊᑐᖅᑕᐅᓇᓱᐊᖅᑐᑦ</th><th>ᐱᕐᔪᑎᖃᑦᑐᑦ ᐱᓕᕆᐊᕐᒍᓂᐊᖅᓯᒪᔪᓄᑦ ᐅᑯᑎᒍᓇ NIRB–ᑯᑦ ᐃᓱᒪᓴᖅᓯᐅᕈᑎᖃᕋᓱᐊᕐᓂᖅᑲᓄᑦ ᐊᒻᒪᓗ ᑭᒃᑯᐃᓇᐃᑦ ᑎᑎᕋᖅᓯᒪᔪᑯᑎᖏᑦ</th></tr>
<tr><td rowspan="5">ᑕᕝᐅᕐᒥᐅᑕᐃᑦ ᓂᕐᔪᑎᑦ</td><td>ᓂᐹᕿᔪᓄᑦ ᐸᕝᕕᓴᐃᓂᖅ</td><td rowspan="4">ᑎᑎᕋᖅᑕᐅᓯᒪᔪᑦ ᑖᒃᑯᓂ ᐅᒥᐊᕐᔪᐊᓚᕆᓂᕐᒧᑦ ᐊᒻᒪᓗ ᐃᒪᕐᒥᐅᑕᓂᒃ ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎᒥ (2016):

1. ᑕᒪᐅᓇᖕᓴᐃᓐᓇᖅ ᐃᖏᕐᕋᕙᓪᓗᑎᒃ ᐊᒻᒪᓗ ᓱᒃᑲᓂᖃᓪᓗᑎᒃ ᐊᔪᓐᓇᖏᑉᐸᑦ
2. ᐱᑕᖃᓗᐊᕌᓗᑦᑎᖏᓪᓗᒋᑦ ᐅᒥᐊᕐᔪᐊᑦ ᓄᖅᑲᖕᒫᕈᐃᓐᓇᖅᑐᑦ ᑐᓚᒃᑕᖅᕕᒻᒥ
3. ᐅᒥᐊᕐᔪᐊᒥ ᐊᒻᒪᓗ ᐃᒪᕐᒥᐅᑕᓂᒃ ᖃᐅᔨᓴᖅᑏᑦ ᐃᑭᒪᕕᒋᔭᕆᐊᖃᑦᑕᕆᐊᓕᓐᓂᑎᑕᐅᓗᑎᒃ ᖃᐅᔨᓴᕐᓂᐊᕐᒪᑕ ᓂᕐᔪᑏᑦ ᐃᒪᕐᒥᐅᑕᐃᑦ ᖃᓄᐃᓕᐅᖃᑦᑕᕐᓂᖏᓐᓂᒃ</td><td>o ᓴᖅᑭᑉᐸᓪᓚᐊᑦᑕᐅᑎᒋᔪᑦ ᑐᓴᕆᐊᕈᑎᑦ ᖃᐅᔨᓴᕈᑎ – ᓇᒦᓪᓚᑦᑖᕐᓂᖓᓂᒃ ᓄᓇᕐᔪᐊᑉ ᖃᐅᔨᓴᒃᑲᐃᔾᔪᑏᑦ
o ᐃᓗᕕᓕᒻᒥ ᐱᓕᕆᓂᖅ
o ᐅᒥᐊᕐᔪᐊᕐᒥ ᖃᐅᔨᓴᕐᓂᖅ
o ᑲᖏᖅᓗᐊᕐᒥ ᑑᒑᓕᓐᓂᒃ ᓂᕕᖓᑖᓕᖅᓯᐃᓂᖅ /ᖃᖓᑕᓲᒃᑯᑦ ᖃᐅᔨᓴᕐᓂᖅ</td><td rowspan="4">ᑕᕝᐅᕐᒥᐅᑕᐃᑦ ᓂᕐᔪᑏᑦ ᐸᕝᕕᓴᒃᑕᐅᑲᐃᓐᓇᖃᑦᑕᕐᓂᖏᑦ ᐅᒥᐊᕐᔪᐊᓄᑦ ᐊᒻᒪᓗ ᓄᓇᓕᓐᓂᖓᒍᖅᑐᓂᑦ
ᐅᓄᕐᓂᖏᑦ (ᓇᓂᖃᑦᑕᕐᓂᖏᑦ ᐊᒻᒪᓗ ᐅᓄᖅᑑᓂᖏᓐᓄᑦ) ᐊᒃᑐᖅᑕᐅᓯᒪᖏᑦᑐᑦ ᐱᓕᕆᔪᓄᑦ ᑕᕝᐅᕐᒥᐅᑕᐃᑦ ᓂᕐᔪᑏᑦ ᐃᑉᐱᒍᓯᓲᖑᖏᑦᑐᑦ ᐅᒥᐊᕐᔪᐊᓂᒃ
ᐅᒥᐊᕐᔪᐊᒧᑦ ᐊᐳᖅᑕᐅᔪᖃᓚᐅᖅᓯᒪᖏᑦᑐᖅ ᐅᓪᓗᒥᒧᑦ ᓄᓇᓚᓐᓂᒥᐅᑕᐃᑦ ᐅᖃᖅᓯᒪᔪᑦ ᐃᓱᒪᓘᑎᓯᐅᖏᓐᓇᖅᑐᒥᒃ ᖃᓄᖅ ᓂᐹᕿᓂᖓ ᐅᒥᐊᕐᔪᐊᑦ ᐊᒃᑐᐃᓂᖃᕈᓐᓇᕐᒪᖔᑕ ᐃᒪᕐᒥᐅᑕᓂᒃ ᐅᑯᐋᓐᓃᑦ ᕿᒪᑎᑕᐅᓚᕐᒪᖔᑕ ᐱᓕᕆᕕᐅᔪᒥᑦ</td><td rowspan="5">1. ᓱᒃᑲᐃᓂᖅᓴᐅᖃᑦᑕᓚᕐᓗᑎᒃ ᐅᒥᐊᕐᔪᐊᑦ ᕿᓐᖑᐊᓂ
2. ᓱᒃᑲᓚᔭᕆᐊᖃᓐᓂᖓ ᑭᓪᓚᖃᑦᑎᑕᐅᓗᓂ ᐃᓗᓐᓇᖏᓐᓄᑦ ᐱᓕᕆᔪᑦ ᐅᒥᐊᕐᔪᐊᖏᓐᓄᑦ
3. ᖃᐅᔨᒪᔭᐅᑎᓪᓗᒍ ᓱᑲᓂᖅᓯᕆᐊᖃᑦᑕᖓᓚ ᑭᓪᓚᖓ ᐅᒥᐊᕐᔪᐊᑦ ᑳᐱᑕᖏᓐᓄᑦ ᑕᒪᒃᑯᑎᒎᓇ ᒪᓚᒃᑲᔾᔨᐅᔪᑦ ᐅᒥᐊᕐᔪᐊᓚᕆᔾᔪᓄᑦ (ᐊᑕᐅᓯᑐᐃᓐᓇᐅᖏᑦᑐᒧᑦ ᐅᓯᒃᑕᐸᑦᑐᓄᑦ)
4. ᕿᒪᐃᑦᑕᐃᓚᓗᓂ ᐅᒥᐊᕐᔪᐊᓄᑦ ᐊᖁᑎᐅᖃᑦᑕᖅᑐᒥᒃ
5. ᖃᐅᔨᓴᕐᓗᓂ ᒪᓕᓐᓂᖏᓐᓂᒃ ᓱᒃᑲᓂᕆᔭᕆᐊᖃᑦᑕᖏᓐᓂᒃ ᐊᒻᒪᓗ ᐊᓯᐊᒎᖅᑕᐃᓚᓂᖏᓐᓄᑦ ᐅᒥᐊᕐᔪᐊᑦ ᐊᖅᑯᑎᖅᓯᕆᐊᖃᑦᑕᖓᒍᑦ ᐅᑯᑎᒎᓇ ᓄᓇᓚᓐᓂᓪᓗᓂ ᖃᐅᔨᓴᕐᓂᖅ (ᐋᖅᑭᒃᑕᐅᓂᖓ ᓴᖅᑭᑉᐸᓪᓚᐊᑦᑕᐅᑎᒋᔪᑦ ᑐᓴᕆᐊᕈᑎᑦ ᖃᐅᔨᓴᕈᑎᐅᑉ ᐊᒻᒪᓗ ᓇᒦᓪᓛᑦᑖᕐᓂᖓᓄᑦ ᓄᓇᕐᔪᐊᖅ ᖃᐅᔨᒃᑲᐃᔾᔪᑎᒃᑯᑦ
6. ᐅᓄᖅᑎᓗᐊᖏᓪᓗᒋᑦ ᐅᒥᐊᕐᒧᐊᑦ ᐃᖏᕐᕋᖃᑦᑕᖅᑐᑦ ᑕᒪᐅᓇ ᐊᑐᖅᑕᐅᖃᑦᑕᖅᑐᑎᒍᑦ
7. ᐅᐸᑦᑕᐃᓚᓗᒋᑦ ᐃᓱᒪᒋᔭᐅᑎᐊᕐᕆᐊᖃᖅᑐᑦ ᐱᑕᖃᑦᑐᑦ ᐱᒻᒪᕆᐅᔪᓂᒃ ᓇᔪᖃᕝᕕᐅᔪᑦ ᐊᒻᒪᓗ/ᐅᕙᓘᓐᓂᑦ ᐃᕐᓂᐅᖅᕕᐅᖃᑦᑕᖅᕖᑦ
8. ᐱᓕᕆᖃᑎᖃᑦᑐᓂ ᒥᑦᑎᒪᑕᓚᐅᑉ ᐊᖑᓇᓱᒃᑐᓚᕆᐊᒃᑯᖅᑎᓂᑦ ᐋᖅᑭᒃᓯᓗᑎᒃ ᑭᓪᓕᖅᓵᓂᐊᖅᑐᓂᒃ ᓄᖅᑲᖕᒫᖅᕕᐅᕙᑦᑐᓄᑦ ᐊᒃᑐᐃᓂᖃᕐᑯᖅᑎᑐᑦ ᐱᒪᓂᐅᔪᓂᒃ ᐊᖑᓇᓱᒃᕕᐅᖃᑦᑕᖅᑐᓂᒃ ᑕᒫᓂ ᑲᖏᖅᑐᒥ
9. ᐅᖅᓱᐊᓗᖃᑦᑎᑦᑎᓂᖅ ᐃᒪᓐᓇᒧᑦ $300,000 ᐊᕋᒍᑕᒫᑦ ᐊᖑᓇᓱᒍᓐᓇᖅᑯᓗᑎᑦ (ᐊᕋᒍᓄᑦ ᖁᓕᓄᑦ)
10. ᑮᓇᐅᔭᖃᑎᑎᓂᖅ $200,000 ᐊᕋᒍᑕᒫᑦ ᐃᑲᔪᕐᓗᑎᑦ ᓄᓇᔫᓕᓐᓂ ᖃᐅᔨᒪᓇᓱᓐᓂᖅ ᐊᒻᒪᓗ ᖃᐅᔨᓴᕐᓂᖅ (ᐊᕋᒎᓄᑦ ᖁᓕᓄᑦ)
11. ᐅᒥᐊᕐᔪᐊᖃᑦᑎᑦᑎᓂᖅ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ ᐃᑲᔪᕐᓗᑎᑦ ᓄᓇᓖᑦ ᐱᓕᕆᐊᕐᕆᓇᓱᐊᖅᑕᖏᑦ (ᓲᕐᓗ ᖃᐅᔨᒪᓇᓱᓐᓂᖅ/ᖃᐅᔨᓴᕐᓂᖅ ᐊᓯᖏᓪᓗ)</td><td rowspan="5">ᑭᐅᔾᔪᑎᑦ ᑭᖑᓪᓕᖅᐹᒧᑦ ᑎᑎᕋᖅᓯᒪᔪᑯᑦ ᑐᓂᔭᐅᓯᒪᔪᑦ ᓄᓘᔪᓂ ᐋᖅᑭᐊᖅᑎᖕᓂᕐᒧᑦ ᑐᒃᓯᕋᐅᑎᑦ –ᐅᓄᖅᓯᕆᐊᖅᑎᖕᓂᖅ ᐅᔾᔨᖅᑕᖅᑕᐅᔪᓂᒃ, ᐅᖅᓱᐊᓗᒃᑲᕝᕕᓂᒃ, ᐊᒻᒪᓗ ᕿᓐᖑᐊᓂ ᐋᖅᑭᒃᑕᐅᓯᒪᒍᓐᓇᖅᑐᓄᑦ (ᓄᓘᔭᓕᕆᔾᔪᑯᑦ–2018a):
ᑭᐅᔾᔪᑎᑦ ᐃᒪᕐᒥᐅᑕᓕᕆᔾᔪᑯᓐᓄᑦ ᐅᖃᐅᓯᖅ ᓇᒪ 1
ᑭᐅᔾᔪᑎ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᖏᓐᓄᑦ ᐅᖃᐅᓰᑦ ᓇᒪ 3
ᑭᐅᔾᔪᑎ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᖏᓐᓄᑦ ᐅᖃᐅᓰᑦ ᓇᒪ 4
ᑭᐅᔾᔪᑎ ᐅᑯᓄᖓ WWF ᓇᒪ 6
ᐃᓚᒍᑕᖅ A ᓄᓇᓕᓐᓂ ᑎᑎᕋᖅᕕᐅᔪᖅ "ᖃᐅᔨᒃᑲᓂᓚᐅᕆᑦ ᓄᓘᔭᓕᕆᔾᔪᑯᑦ ᐅᒥᐊᕐᔪᐊᖅᑕᐅᖃᑦᑕᕐᓂᖓᓄᑦ ᐊᒻᒪᓗ ᑕᕝᐅᕐᒥᐅᑕᓂᒃ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ" "

ᓄᓇᕗᒻᒥ ᐊᕙᑎᓕᕆᓂᕐᒧᑦ ᑲᑎᒪᔩᑦ ᐊᑐᖅᑕᐅᓇᓱᐊᖅᑐᒧᑦ ᐊᑐᐊᖅᓯᔾᔪᑎᑦ – ᐅᔭᕋᖅᑕᖅᑕᐅᔩᑉ ᐃᓚᒋᐊᖅᑕᐅᓂᖓᓄᑦ ᑐᒃᓯᕋᐅᑎ – ᐊᓯᒃᑲᓂᖏᑦ ᑐᑭᓯᐅᒪᔾᔪᑎᑦ ᐱᔩᒪᓂᐅᔪᑦ (ᓄᓘᔭᓕᕆᔾᔪᑯᑦ 2018b)
ᑭᐅᔾᔪᑎ ᐅᖃᐅᓯᕐᒧᑦ ᓇᒪ. 7 a) ᐊᒻᒪᓗ c) –

BIM / QIA ᑐᑭᒋᐊᑕᐅᓯᒪᔪᑦ ᐊᒻᒪᓗ ᐱᓕᕆᐊᕐᕆᓴᐅᔩᒫᓂᐊᖅᑐᑦ (ᓄᓘᔭᓕᕆᔾᔪᑯᑦ 2018c)
ᑭᐅᔾᔪᑎ ᐃᓱᒫᓘᑎᒧᑦ ᓇᒪ 4
ᑭᐅᔾᔪᑎ ᐃᓱᒫᓘᑎᒧᑦ ᓇᒪ. 5
ᑭᐅᔾᔪᑎ ᐃᓱᒫᓘᑎᒧᑦ ᓇᒪ 10</td></tr>
<tr><td>ᐊᓯᔾᔨᕐᓂᖓ ᐆᒪᔪᑦ ᓇᓃᓐᓂᕆᔭᖏᑕ ᑕᒫᓂ ᐃᒫᓂ</td><td>o ᓴᖅᑭᑉᐸᓪᓚᐊᑦᑕᐅᑎᒋᔪᑦ ᑐᓴᕆᐊᕈᑎᑦ ᖃᐅᔨᓴᕈᑎ – ᓇᒦᓪᓚᑦᑖᕐᓂᖓᓂᒃ ᓄᓇᕐᔪᐊᑉ ᖃᐅᔨᓴᒃᑲᐃᔾᔪᑏᑦ
o ᐃᓗᕕᓕᒻᒥ ᐱᓕᕆᓂᖅ
o ᐅᒥᐊᕐᔪᐊᕐᒥ ᖃᐅᔨᓴᕐᓂᖅ
o</td></tr>
<tr><td>ᐊᓯᔾᔨᕐᓂᖓ ᐅᓄᖅᑐᓂᖏᑕ ᑕᒫᓂ ᐃᒫᓂ</td><td>o ᓴᖅᑭᑉᐸᓪᓚᐊᑦᑕᐅᑎᒋᔪᑦ ᑐᓴᕆᐊᕈᑎᑦ ᖃᐅᔨᓴᕈᑎ – ᓇᒦᓪᓚᑦᑖᕐᓂᖓᓂᒃ ᓄᓇᕐᔪᐊᑉ ᖃᐅᔨᓴᒃᑲᐃᔾᔪᑏᑦ
o ᐃᓗᕕᓕᒻᒥ ᐱᓕᕆᓂᖅ
o ᐅᒥᐊᕐᔪᐊᕐᒥ ᖃᐅᔨᓴᕐᓂᖅ
o</td></tr>
<tr><td>ᐃᖏᕐᕋᓂᖓᓯᖓᑦ ᐊᓯᔾᔨᕐᓂᖏᑦ</td><td>o AIS ᖃᐅᔨᓴᕈᑎ – ᓇᒦᓪᓚᑦᑖᕐᓂᖓᓂᒃ ᓄᓇᕐᔪᐊᑉ ᖃᐅᔨᓴᒃᑲᐃᔾᔪᑏᑦ
o ᐃᓗᕕᓕᒻᒥ ᐱᓕᕆᓂᖅ
o ᐅᒥᐊᕐᔪᐊᕐᒥ ᖃᐅᔨᓴᕐᓂᖅ
o ᑲᖏᖅᓗᐊᕐᔪᒻᒥ ᑑᒑᓕᓐᓂᒃ ᓂᕕᖓᑖᓕᖅᓯᐃᓂᖅ /ᖃᖓᑕᓲᒃᑯᑦ ᖃᐅᔨᓴᕐᓂᖅ</td></tr>
<tr><td>ᐱᑕᖃᓐᓂᓴᖓ ᐃᒪᕐᒥᐅᑕᓂᒃ ᓂᕐᔪᑎᓂᒃ ᐊᖑᓇᓱᒃᑕᓂᕐᒧᑦ</td><td>ᑎᑎᕋᖅᑕᐅᓯᒪᔪᖅ ᑖᒃᑯᓂ ᐅᒥᐊᕐᔪᐊᓚᕆᓂᕐᒧᑦ ᐊᒻᒪᓗ ᐃᒪᕐᒥᐅᑕᓂᒃ ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎᒥ (2016):

1. ᑕᒪᐅᓇᖕᓴᐃᓐᓇᖅ ᐃᖏᕐᕋᕙᓪᓗᑎᒃ ᐊᒻᒪᓗ ᓱᒃᑲᓂᖃᓪᓗᑎᒃ ᐊᔪᓐᓇᖏᑉᐸᑦ
2. ᐱᑕᖃᓗᐊᕌᓗᑦᑎᖏᓪᓗᒋᑦ ᐅᒥᐊᕐᔪᐊᑦ ᓄᖅᑲᖕᒫᕈᐃᓐᓇᖅᑐᑦ ᑐᓚᒃᑕᖅᕕᒻᒥ
3. ᐅᒥᐊᕐᔪᐊᒥ ᐊᒻᒪᓗ ᐃᒪᕐᒥᐅᑕᓂᒃ ᖃᐅᔨᓴᖅᑏᑦ ᐃᑭᒪᕕᒋᔭᕆᐊᖃᑦᑕᕆᐊᓕᓐᓂᑎᑕᐅᓗᑎᒃ ᖃᐅᔨᓴᕐᓂᐊᕐᒪᑕ ᓂᕐᔪᑏᑦ ᐃᒪᕐᒥᐅᑕᐃᑦ ᖃᓄᐃᓕᐅᖃᑦᑕᕐᓂᖏᓐᓂᒃ

ᑎᑎᕋᖅᑕᐅᓯᒪᔪᖅ ᐃᓄᐃᑦ ᐊᒃᑐᖅᑕᐅᓂᖏᓐᓇᖅᑕᖅᐸᓐᓂᖓ ᐃᑲᔫᑎᒥ (2013):
4. ᓂᕐᔪᑎᓄᑦ ᐃᑲᔪᖅᑕᐅᔾᔪᑎᓄᑦ ᑮᓇᐅᔭᑦ</td><td>o ᐃᓗᕕᓕᒃᒥ ᐅᒥᐊᕐᔪᐊᒦᑦᑐᓂ ᐱᓕᕆᓂᖅ
o ᑲᖏᖅᓗᐊᕐᔪᒻᒥ ᑑᒑᓕᓐᓂᒃ ᓂᕕᖓᑖᓕᖅᓯᐃᓂᖅ
o ᐅᒥᐊᕐᔪᐊᕐᒥ ᖃᐅᔨᓴᕐᓂᖅ
o</td><td>ᐅᓄᕐᓂᖏᑦ (ᓇᓂᖃᑦᑕᕐᓂᖏᑦ ᐊᒻᒪᓗ ᐅᓄᖅᑑᓂᖏᓐᓄᑦ) ᐊᒃᑐᖅᑕᐅᓯᒪᖏᑦᑐᑦ ᐱᓕᕆᔪᓄᑦ

ᐅᓪᓗᒥᒧᑦ ᐅᒥᐊᕐᔪᐊᑦ ᐊᐳᖅᓯᔪᖃᓚᐅᖅᓯᒪᖏᑦᑐᑦ ᖃᐅᔨᒪᔭᐅᔪᒃᑯᓪᓚ

ᓄᓇᓖᑦ ᐊᖑᓇᓱᒃᑎᖏᑦ ᐅᖃᖅᓯᒪᔪᑦ ᒥᑭᓪᓚᕚᓪᓚᐊᒋᐊᖏᓐᓂᒃ ᓂᕐᔪᑏᑦ ᑕᒫᓂᖃᑦᑕᖅᑐᑦ ᐊᖑᓇᓱᒃᑕᐅᓪᓗᑎᒃ
ᓄᓇᓖᑦ ᐊᖑᓇᓱᒃᑎᖏᑦ ᐅᖃᖅᓯᒪᔪᑦ ᐸᕝᕕᓴᐃᖃᑦᑕᖏᓐᓄᑦ ᐊᖑᓇᓱᒃᕕᑐᖃᐅᖃᑦᑕᖅᑐᓂᒃ ᐊᒻᒪᓗ ᐱᓇᓱᒃᕕᐅᖃᑦᑕᖅᑐᓂᒃ ᐅᒥᐊᕐᔪᐊᖃᖃᑦᑕᓂᖓᓄᑦ</td></tr>
</table>

Baffinland

ᐱᑦᓯᑎᐅᔪᖅ	ᐊᒃᑐᐃᓂᖃᕐᓂᖓ	ᓯᕗᓂᐊᒍᑦ-2018 ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎ ᐱᔪᓐᓇᕆᑦᑎᓂᖅᓴᐅᓇᓱᐊᕐᓂᕐᒧᑦ ᐊᒻᒪᓗ ᐱᓕᕆᐊᒃᓴᖅᓴᐅᔪᓂᑦ	ᖃᐅᔨᓴᕐᓂᖅ	ᓇᐃᓈᖅᓯᒪᔪᑦ ᖃᐅᔨᓴᕆᑎᕕᓃᑦ ᐊᒻᒪᓗ ᓄᓇᕐᓂᑦ ᐊᒻᒪᓗ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᑐᓴᖅᑎᑦᑎᔪᑎᖏᑦ	ᐊᓯᒃᑲᓂᖏᑦ ᐱᓗᐊᖏᑎᑦᑎᓂᖅᓴᐅᓇᓱᐊᕐᓂᕐᒧᑦ ᐊᑐᖅᑕᐅᓇᓱᐊᖅᑐᑦ	ᐱᑦᓯᑎᖃᑦᑐᑦ ᐱᓕᕆᐊᕐᒍᓂᐊᖅᓯᒪᔪᓄᑦ ᐅᑯᑎᒎᓇ NIRB−ᑯᑦ ᐃᓱᒪᓴᖅᓯᐅᕈᑎᖃᕐᓯᐊᕐᓂᖏᓐᓄᑦ ᐊᒻᒪᓗ ᑭᒃᑐᐃᓇᐃᑦ ᑎᑎᖅᓯᒪᔪᖁᑎᖏᑦ
ᑕᕝᕙᐅᑉ ᐊᕙᑎᖓ	ᓇᓗᓇᐃᖅᓯᔾᔨᑎ ᐃᒫᓂ ᑕᒫᓂᕐᒥᐅᑕᑐᖅᑳᕐᓂᖅᑐᑦ ᐆᒪᔪᐃᑦ	ᑎᑎᕋᖅᑕᐅᓯᒪᔪᑦ ᒪᓕᒡᒍᕆᑦ ᑖᕝᕙ ᐅᒥᐊᕐᔪᐊᓕᕆᓂᕐᒧᑦ ᐊᒻᒪᓗ ᐃᒪᕐᒥᐅᑕᓂᒃ ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎᒥ (2016): 1. ᖃᐅᔨᒪᓇᓱᑦᑎᐊᕐᓗᓂ ᑕᕝᕙᐅᖑᓂᖓ ᓱᕋᕝᓯᒪᖕᒋᒃᑲᓗᐊᕐᒪᖔᑦ	o ᑕᕝᕙᐅᖓᓂᒃ ᖃᐅᔨᓴᕐᓂᖅ ᐃᒪᕐᒥ ᐃᑦᑕᖅᑯᐊᑕᐅᖃᑦᑕᖅᑐᒥᒃ o ᐃᒫᓂ ᑕᒫᓂᕐᒥᐅᑕᑐᖅᑳᕐᓂᖅᑐᑦ ᐆᒪᔪᐃᑦ ᖃᐅᔨᓴᖅᑕᐅᓂᖅᑐᑦ	ᑕᒪᕐᒥᒃ ᐅᒥᐊᕐᔪᐊᓚᒥᑦ ᖃᐅᔨᒃᑲᐃᓯᒪᕗᑦ ᒪᓕᒃᑎᐊᓂᕐᒥᓂᒃ ᐃᑦᑕᖅᑯᑎᖃᖅᑕᖅᑕᓂᕐᒧᑦ ᐃᒥᕐᒥᒃ ᑕᐅᖅᔫᓂᒃᑯᑦ ᑕᒪᒃᑯᑎᒎᓇ ᑕᕝᕙᐅᖓᒥᒃ ᖃᐅᔨᓴᕐᓂᒃᑯᑦ ᐃᑦᑕᖅᑯᓯᐅᔾᔭᐅᓯᒪᔪᑦ ᔫᖅᑭᑎᒍᑦ ᐃᒫᓂ ᑖᒫᓂᕐᒥᐅᑕᑐᖅᑳᕐᓂᖅᑐᓂᒃ ᐆᒪᔪᕋᓛᑦ ᖃᐅᔨᔪᖅᑲᖅᓯᒪᖅᑐᖅ ᐱᓕᕆᐊᑎᒍᑦ ᖃᐅᔨᒪᓇᓯᐊᕐᓂᒃᑯᑦ ᐃᑲᔪᖅᑕᐅᑎᐊᕐᓂᒃᑯᑦ ᑐᓴᖅᑎᑦᑎᓂᖅ ᐊᖅᑭᓪᓕᕝᓪᓕᖅᑎᑕᐅᓂᖓᓄᑦ ᐃᒫᓂ ᑖᒫᓂᕐᒥᐅᑕᑐᖅᑳᕐᓂᖅᑐᓂᒃ ᐆᒪᔪᕋᓛᓂᒃ ᖃᐅᔨᓴᕐᓂᖅ ᐱᖃᓯᐅᔾᔨᓗᓂ ᐃᒥᓪᓚᑦ ᓄᓇᓕᒥᐅᑕᐃᑦ ᐅᖃᖅᓯᒪᔪᑦ ᐃᓱᒫᓗᑎᒥᓂᒃ ᑖᓐᓇ AIS ᐃᓚᓕᐅᔾᔭᐅᕙᓪᓕᐊᒪᓚ ᐱᓕᕆᕕᐅᔪᓄᑦ ᐅᒥᐊᕐᔪᐊᑕᖃᖃᑦᑕᓕᕐᓂᖓᓄᑦ	1. ᐃᓚᒋᐊᕐᓗᒍ ᐊᐅᓚᑕᐅᓂᖓᓄᑦ/ᐱᐅᓂᖓᓄᑦ ᐊᑐᐊᖅᓯᔾᔪᑎᑦ ᑕᕝᕙᒥᒃ ᖃᐅᔨᓴᕐᓂᕐᒧᑦ 2. ᐊᑐᕐᓂᖅ ᐅᖓᓯᑦᑐᒥ ᐊᐅᓚᑦᑎᔾᔪᑎᐅᔪᓐᓇᖅᑐᒃ ᐃᒪᐅᑉ ᐊᑖᓂ ᐊᐅᓚᓲᓚᐅᕈᑎᒃᑯᑦ ᖃᐅᔨᓴᕐᓂᐅᔪᓄᑦ ᐊᒻᒪᓗ ᐆᒪᔪᕋᓛᑦ ᓄᐊᑕᐅᖃᑦᑕᖅᑐᑦ ᖃᐅᔨᓴᕐᓂᖃᑦᑎᓪᓗᑎᑦ ᐅᒥᐊᕐᔪᐊᓄᑦ ᐅᒧᔭᐅᔪᓄᑦ	ᑭᐅᔾᔪᑎ ᑭᖑᓪᓕᖅᐹᒥ ᑎᑎᖅᑕᐅᓯᒪᔪᒧᑦ ᑐᒃᓯᕋᐅᑎᑦ ᓄᓘᔭᒥ ᐊᖅᑭᖅᑎᐊᒃᑲᓂᖅᑕᑦᑎᕙᓂᕐᒧᑦ ᑐᒃᓯᕋᐅᑏᑦ – ᐅᔭᕋᖅᑕᖅᑕᐅᔫᑉ ᐃᓚᒋᐊᖅᑕᐅᑲᓂᕐᓂᖓ, ᐅᖅᓱᖅᑲᐅᑏᑦ, ᕿᓐᖑᐊᓂ ᐋᖅᑭᒃᑕᐅᓯᒪᔪᒍᓐᓇᖅᑐᓄᑦ (ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018a) **ᑭᐅᔾᔪᑎ ᕿᑭᑦᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᖏᓐᓄᑦ ᐅᖃᐅᓯᖅ ᓇᒻᒪ 2 -** ᓄᓖᓴᓕᕆᔨᒃᑯᑦ ᑭᐅᔾᔪᑎᖏᑦ ᕿᒥᕐᕈᔭᑦ ᐅᖃᐅᓯᖏᓐᓄᑦ 2017 NIRB−ᑯᑦ ᐅᓂᒃᑳᖓᓂ (ᓄᓖᓴᓕᕆᔨᒃᑯᑦ, 2018d) **ᑭᐅᔾᔪᑎᖓ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᖏᓐᓄᑦ ᓇᒻᒪ 37** **ᑭᐅᔾᔪᑎᖓ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᑦᔨᖃᑎᒌᖏᓐᓄᑦ ᐱᖃᓯᐅᑎᓯᒪᔪᖅ 2 ᐅᖃᐅᓯᖅ**
ᐳᔪᖅ	ᐱᕈᖅᑐᖃᓂᖓᓂᒃ ᐊᒻᒪᓗ ᐊᑦᔨᒌᖕᒋᑦᑑᓂᖕᒋᓐᓂᒃ	ᑎᑎᕋᖅᑕᐅᓯᒪᔪᑦ ᑖᕝᕙ ᓄᓇᒥ ᐆᒪᔪᐃᑦ ᐊᕙᑎᖓᓂᒃ ᖃᐅᔨᓴᕐᓂᖅ ᐊᒻᒪᓗ ᐊᐅᓚᑦᑎᓂᕐᒧᑦ ᐸᕐᓇᐅᑎᑦ (2018): 1. ᐱᓕᕆᓂᐅᔪᑦ ᐸᕐᓇᑕᐅᓯᒪᓂᐊᖅᑐᑦ ᐊᒻᒪᓗ ᐱᓕᕆᐊᕐᒍᖅᑕᑦᓱᑎᒃ ᒥᑭᓪᓕᕚᓪᓕᑎᑦᑎᓇᓱᐊᕐᓗᑎᒃ ᐱᓕᕆᓂᒃᑯᑦ ᐊᑐᖅᑕᐅᖅᑲᖅᓯᒪᔪᓂᒃ 2. ᓇᐅᒃᑯᑦ ᐊᒻᒪᓗ ᖃᖓᒃᑯᑦ ᓈᒻᒪᑉᐸᑦ, ᐳᔪᖃᓗᐊᕝᔭᐃᒃᑯᑎᒥᒃ ᐊᑐᖅᑐᖃᐃᓐᓇᕆᐊᓕᒃ ᐊᑐᕆᐊᓂ, ᐱᓗᐊᖅᑐᒥᒃ ᐊᑐᖅᑕᐅᓗᐊᓐᖑᐊᖅᑲᖅᑐᓂ ᐊᐳᑎᐅᑎᓐᓇᒍ 3. ᐋᖅᑭᒃᓯᓗᑎᒃ ᑕᓗᒃᓴᓂᒃ ᓯᖁᓪᓗᐃᔾᔨᑎᓄᑦ	o ᐳᔪᑖᖅᑲᑦᑕᓂᕐᒧᑦ ᖃᐅᔨᓴᕐᓂᖅ o ᐱᕈᖅᑐᖃᓐᓂᖓᓂᒃ ᖃᐅᔨᓴᕐᓂᖅ o ᐱᕈᖅᑐᐃᓂᒃ ᐊᒻᒪᓗ ᐃᑦᔪᖓᓂᒃ ᓴᕙᕋᔭᒃᓴᓂᒃ ᖃᐅᔨᓴᕐᓂᖅ	ᐳᔪᕈᖅᑕᕐᓂᖓ ᐊᖏᓂᖅᓴᐅᕗᖅ ᐃᓱᒪᒋᔭᐅᓚᐅᖅᑐᒥᒃ ᑖᕝᕙ ᑭᖑᓪᓕᖅᐹᒥ ᐊᕙᑎᒥᒃ ᐊᒃᑐᐃᓂᖃᕐᓂᕐᒧᑦ ᑎᑎᖅᓯᒪᔪᑦ ᖁᑦᑎᓂᖅᓴᐅᓂᖓ ᐳᔪᖃᑦᑕᕐᓂᐅᑉ ᐊᒃᑐᐃᓂᖃᖅᑎᑦᑐᖅ ᐱᕈᖅᑐᐃᑦ ᖃᓄᐃᓐᓂᖅᑲᓐᓂᖏᓐᓂᒃ ᐊᒻᒪᓗ ᓴᕙᕋᔭᒃᓴᖃᕐᓂᖏᑦ ᖃᐅᔨᓴᐅᓯᒪᔪᑦ ᐃᑦᔪᒥ ᐊᒻᒪᓗ ᐱᕈᖅᑐᓂ ᐅᖓᑖᓅᖓᖅᑐᖅ ᐊᑐᐊᖅᓯᔾᔪᑎᑎᒍᑦ ᐱᕈᖅᑐᐃᑦ ᐅᓄᕐᓂᖏᑦ ᐊᒻᒪᓗ ᐊᑦᔨᒌᖕᒋᓐᓂᖏᑦ ᓱᕋᖅᓯᒪᖕᒋᑦᑐᑦ ᐱᕈᖅᑕᕐᓂᖃᔭᕆᑎᒍᑦ ᐃᓱᒫᓗᑎᔭᐅᖕᒋᓐᓇᖅᑐᖅ ᓄᓇᖅᑎᑎᐅᔪᓄᑦ ᑕᑯᒃᓴᐅᓂᖓ ᐳᔫᑉ ᐊᒻᒪᓗ ᓄᓇᒨᖅᑕᑦᑕᓂᕆᔭᖓ ᐃᓱᒫᓗᑎᐅᖕᒋᓐᓇᖅᑐᖅ ᓄᓇᖅᑎᑎᐅᔪᓄᑦ ᓴᕙᕋᔭᒃᓴᑖᕈᓐᓇᖅᓯᒪᖕᒋᓐᓂᖏᑦ ᓂᕿᑎᑦ ᐊᖑᓇᓱᒃᑕᐅᕙᑦᑐᑦ ᐊᒻᒪᓗ ᐆᒪᔪᓄᐊᖅᑕᑦᑕᓂᖕᒋᑦ ᐃᓚᒃᓴᔭᐃᑦ	1. 6−ᖑᔪᑦ ᐊᓯᒃᑲᓂᖕᒋᑦ ᐳᔪᒧᑦ ᖃᐅᔨᓴᕈᑏᑦ ᐋᖅᑭᓱᖅᑕᐅᓂᐊᕐᒥᔪᑦ ᐊᐅᔭᒍᖅᑲᐅᓕᖅᐸᑦ/ᐅᑭᐊᒃᓵᒥ 2018 2. ᐳᔪᖅᑖᖅᑲᑦᑕᓂᕐᒧᑦ ᖃᐅᔨᓴᕈᑏᑦ ᐃᓕᔭᐅᓂᐊᖅᑐᑦ 1 km ᐊᖏᓯᓐᓂᖅᑲᓪᓗᑎᒃ, ᐱᖓᓱᓂ ᐃᓂᒃᓴᕆᔭᐅᔪᓂᒃ (ᑲᓇᓐᓇᖅ/ᐱᖓᓇᖅ ᐊᑐᕆᐊᒥ) ᐊᑕᐅᓯᖅ ᐃᓕᔭᐅᓯᒪᓂᐊᖅᑐᖅ ᖃᓂᒋᔭᖓᓂ km 25 ᐊᑕᐅᓯᖅ ᑕᒫᓂ km 56 ᐊᑕᐅᓯᖅ ᑕᒫᓂ km 75–80 3. ᑲᔪᓯᔭᐅᓗᓂ ᐃᓚᒋᐊᖅᓯᒪᔪᖅ ᐲᖅᓴᐅᑎ ᐊᒻᒪᓗ ᐃᒥᕐᒧᑦ ᐳᔪᕈᔪᔭᐃᒃᑯᑎ 4. ᐊᓯᒃᑲᓂᖕᒋᓐᓂᒃ 25-ᓂᒃ ᐅᓪᓗᓂᒃ ᓯᖁᓪᓗᐃᔾᔨᑎᓄᑦ ᐱᓗᑎᒃ	ᑭᐅᔾᔪᑎ ᑭᖑᓪᓕᖅᐹᓄᑦ ᑎᑎᖅᓯᒪᔪᒃᑯᑦ ᑐᒃᓯᕋᐅᑏᑦ ᓄᓘᔭᓂ ᐋᖅᑭᒃᑲᓂᖅᑕᑦᑎᕙᓂᕐᒧᑦ ᑐᒃᓯᕋᐅᑏᑦ – ᐅᔭᕋᖅᑕᖅᑕᐅᔫᑉ ᐃᓚᒋᐊᖅᑕᐅᓂᖓ, ᐅᖅᓱᐊᓗᖅᑲᐅᑎ, ᐊᒻᒪᓗ ᕿᓐᖑᐊᓂ ᐋᖅᑭᒃᑕᐅᓯᒪᒍᓐᓇᖅᑐᓄᑦ (ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018a) **ᑭᐅᔾᔪᑎ ECCC ᐅᖃᐅᓯᖓᓄᑦ ᓇᒻᒪ 2** **ᑭᐅᔾᔪᑎ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᐅᖃᐅᓯᖓᓄᑦ 6** BIM / ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᑐᑭᑖᕈᑎᖏᑦ ᐊᒻᒪᓗ ᐱᓕᕆᐊᒃᓴᖅᓴᖏᑦ QIA (ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018c) **ᑭᐅᔾᔪᑎ ᐃᓱᒪᒍᑎᒧᑦ ᓇᒻᒪ 11** **ᑭᐅᔾᔪᑎ ᐃᓱᒪᒍᑎᒧᑦ ᓇᒻᒪ 12** **ᑭᐅᔾᔪᑎ ᐃᓱᒪᒍᑎᒧᑦ ᓇᒻᒪ 13**

ᖃᐅᔨᒪᔾᔪᑏᑦ:

ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018a. ᓄᓖᓴᑦ ᐋᖅᑭᐊᒃᑲᓂᖅᑕᖅᑕᐅᓂᖅᑐᓄᑦ ᑐᒃᓯᕋᐅᑎ – ᐅᔭᕋᖅᑕᖅᑕᐅᔫᑉ ᐃᓚᒋᐊᖅᑕᐅᓂᖓ, ᐅᖅᓱᐊᓗᖅᑲᐅᑎ ᐊᒻᒪᓗ ᕿᓐᖑᐊᓂ ᑐᓚᒃᑕᖅᕕᖓᓂ ᐋᖅᑭᐊᕈᑎᒃᓴᓂᒃ ᐋᖅᑭᐊᒃᑲᓂᖅᑕᖅᑕᐅᓂᖅᑐᓄᑦ ᑐᒃᓯᕋᐅᑎ ('ᐅᔭᕋᖅᑕᖅᑕᐅᔫᑉ ᐃᓚᒋᐊᖅᑕᐅᓂᖓᓄᑦ ᑐᒃᓯᕋᐅᑦ') (ᐃᐳᕈ 23, 2018) – ᑭᐅᔾᔪᑎ ᐅᖃᐅᓯᕐᓄᑦ. ᐋᒌᓯ 9 2018.

ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018b. ᓄᓇᕗᒻᒥ ᐊᕙᑎᓕᕆᓂᕐᒧᑦ ᑲᑎᒪᔨᖏᑦ ᐱᓕᕆᓇᓱᐊᕐᓂᕐᒧᑦ ᐊᑐᐊᖅᓯᔾᔪᑎᖏᑦ – ᐅᔭᕋᖅᑕᖅᑕᐅᔫᑉ ᐃᓚᒋᐊᖅᑕᐅᓂᖓᓄᑦ ᑐᒃᓯᕋᐅᑎ – ᐊᓯᒃᑲᓂᖕᒋᓐᓂᒃ ᑐᕈᓯᐅᒪᔾᔪᑎᓂᒃ ᐱᔭᕆᐊᖃᕐᓂᖅ ᔩᓐ 20 2018. NIRB−ᑯᑦ ᑭᒃᑐᐃᓇᕐᓄᑦ ᑎᑎᖅᑲᖅ ID: 318283

ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018c. ᓄᓖᓴᓕᕆᔨᒃᑯᑦ/ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᑐᑭᑖᕈᑎᖓ ᐊᒻᒪᓗ ᐱᓕᕆᐊᒃᓴᖅᓴᖏᑦ Excel ᓇᓗᓇᐃᖅᓯᔾᔪᑎᖓ (180802-SR17-QIA-Baffinland2018IssueResolution-ENG). ᐊᖏᕈᑕᐅᓚᐅᖅᑐᖅ ᐋᒌᓯ 2, 2018 ᐅᖃᐅᓯᐅᓚᐅᖅᑐᖅ ᕿᑭᖅᑕᓂ ᐃᓄᐃᑦ ᑲᑐᔾᔨᖃᑎᒌᑦ ᐋᒌᓯ 3-ᒥ ᑎᑎᖅᑲᖓᓄᑦ ᑖᒃᑯᓄᖓ NIRB−ᑯᓐᓄᑦ..

ᓄᓖᓴᓕᕆᔨᒃᑯᑦ 2018d. ᓄᓖᓴᓕᕆᔨᒃᑯᑦ ᑭᐅᔾᔪᑎᖓ ᕿᒥᕐᕈᐃᔪᑦ ᐅᖃᐅᓯᖃᓯᒪᔭᖏᓐᓄᑦ ᑖᒃᓯᒎᖓ 2017 NIRB−ᑯᑦ ᐊᕐᕌᒍᑕᒫᖅᓯᐅᑎᖓᑕ ᐅᓂᒃᑳᖓᓄᑦ. ᔪᓚᐃ 12, 2018.

MEMORANDUM

Baffinland Environmental Monitoring, Mitigations and Adaptive Management Overview

---

Approach to Adaptive Management

Baffinland Iron Mines Corporation (Baffinland) has invested significant efforts in 2018 to further enhance our understanding of the Qikiqtani Inuit Association (QIA) and local community member's concerns related to the current Project and their specific concerns with increasing production and shipping rates to support economically sustainable growth of the Project.

Existing permits requires that Baffinland receives input from QIA, the community (namely Pond Inlet) and other regulators on the results of annual monitoring conducted by Baffinland to confirm the predictions of the Final Environmental Impact Statement for the Project. In addition, Baffinland solicits feedback directly from the communities and other concerned parties to inform operational planning. Feedback from these stakeholders is provided by establishing several different avenues for two-way information sharing (see Table 1). Despite the above efforts, it is acknowledged that there remains some ongoing concerns amongst community members and QIA about the Project and uncertainty in the results in the monitoring programs. To address this Baffinland has implemented a process to develop adaptive management measures and additional mitigations (Figure 1). The adaptive management process allows for Projects-effects monitoring to be assessed at two levels.

First, Project-effects monitoring is conducted and assessed by Baffinland's technical experts and consultants. The results of the monitoring programs are then shared with and assessed by external reviewers and technical specialists, the QIA and local community members.

Table 1 – Project Review Process

Information Sharing Activity	Description	Annual Schedule	Key Organization							
			Nunavut Impact Review Board (NIRB)	Nunavut Water Board (NWB)	QIA	Regulatory Authorities	Working Groups (Government of Nunavut, Department of Fisheries and Oceans, Parks Canda, Environment and Climate Change Canada, Qikiqtani Inuit Association, Mittimatalik Hunter and Trappers Organization)	Technical Experts	MHTOs	Hamlets
Annual Monitoring Program Reports - Draft	Draft report – Results from annual monitoring program for terrestrial and marine monitoring efforts	February 15			✓	✓	✓	✓	✓	
Annual Monitoring Program Reports - Final	Final report – Results from annual monitoring program for terrestrial and marine monitoring efforts. Incorporates feedback received from Working Groups on the draft report	March 31	✓	✓	✓	✓	✓	✓	✓	✓
NIRB/NWB Annual Reports	Summarizes annual operational activities, monitoring programs and compliance with regulatory permits	March 31	✓	✓	✓	✓	✓	✓	✓	✓

Information Sharing Activity	Description	Annual Schedule	Key Organization							
			Nunavut Impact Review Board (NIRB)	Nunavut Water Board (NWB)	QIA	Regulatory Authorities	Working Groups (Government of Nunavut, Department of Fisheries and Oceans, Parks Canda, Environment and Climate Change Canada, Qikiqtani Inuit Association, Mittimatalik Hunter and Trappers Organization)	Technical Experts	MHTOs	Hamlets
Topic Specific Meetings – Shipping etc.	Face-to-face meetings held with MHTO, Hamlets and QIA Representatives to discuss specific issues and concerns and/or key components of Project operations	End and beginning of shipping season, As needed			✓			✓	✓	
General Project Update Meetings	Annual meetings held with Inuit and QIA representatives to update interested parties on ongoing operations or any proposed changes to the Project	Annually, As needed	✓	✓	✓	✓	✓	✓	✓	✓
Site Visits	Visit to the Mary River and Milne Port site to see live operations and discuss issues on the ground	As needed or requested						✓	✓	✓

Information Sharing Activity	Description	Annual Schedule	Key Organization							
			Nunavut Impact Review Board (NIRB)	Nunavut Water Board (NWB)	QIA	Regulatory Authorities	Working Groups (Government of Nunavut, Department of Fisheries and Oceans, Parks Canda, Environment and Climate Change Canada, Qikiqtani Inuit Association, Mittimatalik Hunter and Trappers Organization)	Technical Experts	MHTOs	Hamlets
Inspections/ Audits	Regulatory audits or inspections to ensure compliance with existing permits and approvals	Typically during summer (July to September)	✓	✓	✓	✓				
Participation in Monitoring Programs	Contract employment opportunity or joint-collaboration on environmental monitoring programs	Summer Field Season (July to October)				✓		✓	✓	
Input into additional mitigation measures	Submission of technical comments and responses or face-to-face meetings to discuss proposed revisions or additions to existing mitigation measures	As needed	✓	✓	✓	✓	✓	✓	✓	✓

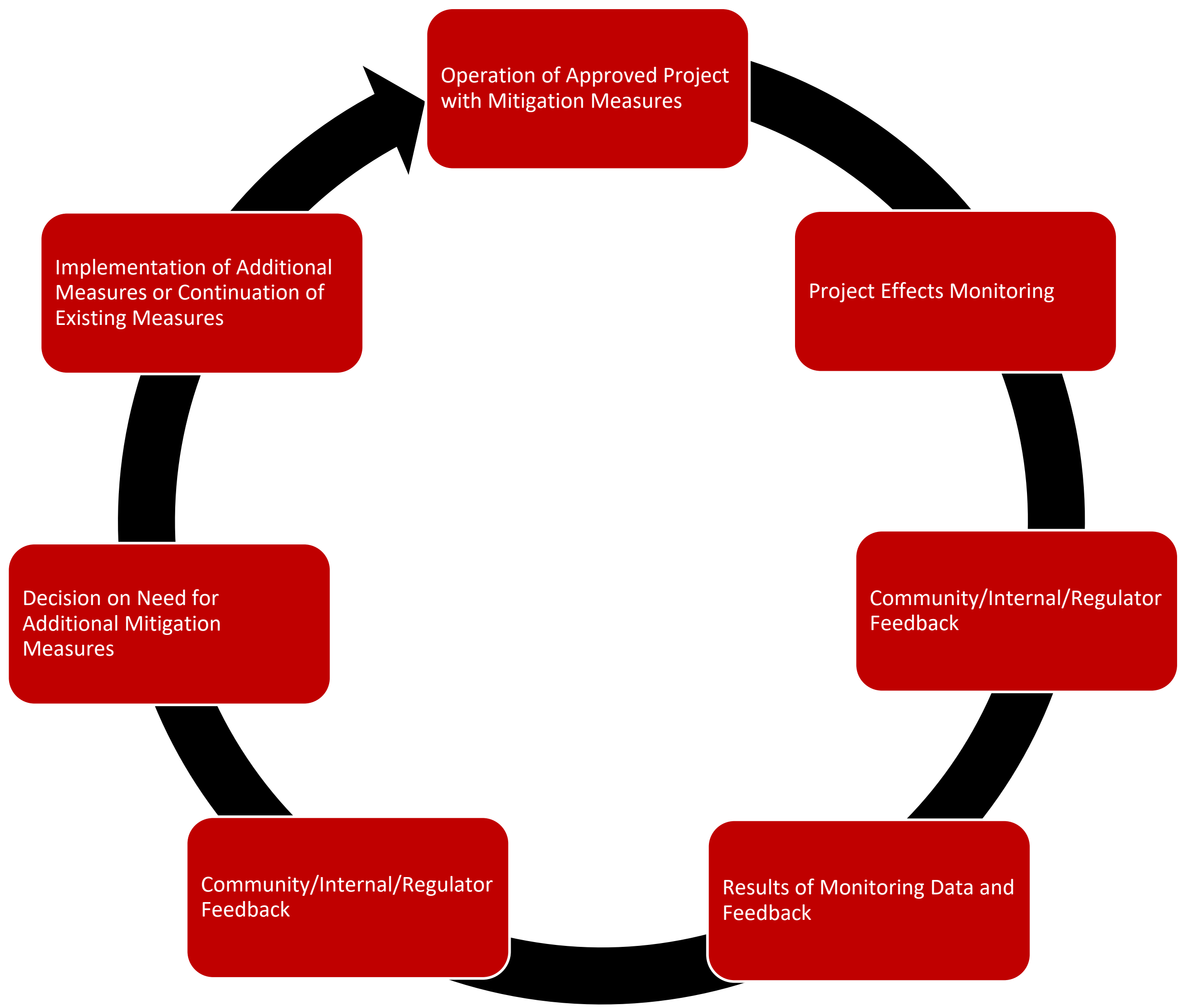


Figure 1 – Baffinland's Adaptive Management Process

Response to Community Concerns

As previously mentioned, while the monitoring results have indicated that the Project is not having a significant or harmful effect on the environment, subsequent assessments conducted by community members and the QIA have identified a lack of certainty in the results of the monitoring programs and concerns related to Project operations. Therefore, throughout 2017 and 2018, Baffinland in consultation with the QIA and Mittamatalik Hunter and Trappers Organization (MHTO / Pond Inlet Hunter and Trappers Organization) determined that additional mitigation measures should be developed and implemented, which ensure a precautionary approach is being applied in Project operations.

In 2018, prominent attention was given by Baffinland to develop additional mitigations and monitoring that meaningfully responds to QIA and Pond Inlet community member's concerns related to shipping operations, marine mammals and dust fall. An overview of the results of the implementation of the adaptive management process executed by Baffinland is presented in Table 2.

Table 2 – Marine and Dust Effects, Monitoring and Mitigation

Topic	Potential Effect	Pre-2018 Management Plan Mitigations and Commitments	Monitoring Program	Summary of Monitoring Results and Community and QIA Feedback	Additional Mitigation Measures	Reference to commitments made through the NIRB reconsideration process and public record
Marine Mammals	Acoustic Disturbance	As listed in the Shipping and Marine Wildlife Management Plan (2016): 1. Maintain constant course and speed when possible 2. Reduce vessel idling time at dock 3. Shipboard and Marine Wildlife Observers to be on select vessels to monitor interactions with marine mammals	o Automated Information System – Exact Earth Notifications o Bruce Head Vessel-Based Project o Ship- Based Observer Program o Tremblay Sound Narwhal Tagging Program /Aerial Survey	Marine mammals experience temporary and localized disturbance as a result of Project shipping Relative population levels (distribution and abundance) are not being affected by the Project Marine mammals tend to avoid vessels No ship-strikes have been recorded to date Community members have expressed an ongoing concern for how noise from vessels could be affecting marine mammals or driving them out of the Project area	1. Reduce Vessel Speed to 9 knots within Milne Inlet 2. Apply speed limit to all Project vessels 3. Communicate speed limits to vessel captains through Standing Instructions to Masters (not only ore carriers) 4. Avoid deviation from nominal shipping route 5. Monitor adherence to speed limit and deviation from nominal shipping route through Community-Based Monitoring (installation of Automated Information System at MHTO office) and Exact Earth Notification Alert System 6. Minimize multi-vessel transits within the corridor 7. Avoid sensitive areas that contain critical habitat and/or traditional calving ground 8. Work with the MHTO to establish parameters for drifting areas to avoid interaction with important hunting areas in the Inlet 9. Provision of fuel in the amount of $300,000 annually to enable hunting practices (up to 10 years) 10. Provision of $200,000 annually to support community based monitoring and research (up to 10 years) 11. Provision of research vessel to support community initiatives (i.e. monitoring/research etc.)	Response to final written submissions on the Mary River Modification Application – Production Increase, Fuel Storage, and Milne Port Accommodations (Baffinland 2018a): **Response to DFO Comment No. 1** **Response to QIA Comment No. 3** **Response to QIA Comment No. 4** **Response to WWF No. 6** **Appendix A Community Fact Sheet "Learn more about Baffinland's shipping and marine monitoring programs"** Nunavut Impact Review Board Process Guidance – Production Increase Proposal – Additional Information Requirement (Baffinland 2018b) **Response to Comment No. 7 a) and c) –** BIM / QIA Resolutions and Commitments (Baffinland 2018c) **Response to Concern No. 4** **Response to Concern No. 5** **Response to Concern No. 10**
	Change in animal distribution in the region		o Automated Information System – Exact Earth Notifications o Bruce Head Vessel-Based Project o Ship-Based Observer Program			
	Change in abundance in the region		o Automated Information System – Exact Earth Notifications o Bruce Head Vessel- Based Project o Ship-Based Observer Program			
	Alteration of migration patterns		o Automated Information System – Exact Earth Notifications o Bruce Head Vessel-Based Project o Tremblay Sound Narwhal Tagging Program /Aerial Survey			
	Availability of marine mammals for harvesting	As listed in the Shipping and Marine Wildlife Management Plan (2016): 1. Maintain constant course and speed when possible 2. Reduce vessel idling time at dock 3. Shipboard and Marine Wildlife Observers to be on select vessels to monitor interactions with marine mammals	o Bruce Head Vessel- Based Project o Tremblay Sound Narwhal Tagging Program o Ship-Based Observer Program	Relative population levels (distribution and abundance) are not being affected by the Project No ship-strikes have been recorded to date Local hunters have reported decreased numbers of marine mammals available for harvesting		

Topic	Potential Effect	Pre-2018 Management Plan Mitigations and Commitments	Monitoring Program	Summary of Monitoring Results and Community and QIA Feedback	Additional Mitigation Measures	Reference to commitments made through the NIRB reconsideration process and public record
		As listed in the Inuit Impact Benefit Agreement (2013): 4. Wildlife Compensation Fund		Local hunters have reported disturbances to traditional hunting areas and practices as a result of shipping operations		
Marine Environment	Introduction of Aquatic Invasive Species	As listed in the Shipping and Marine Wildlife Management Plan (2016): 1. Ensure salinity of ballast water is within standards	o Salinity Testing of Ballast Water o Aquatic Invasive Species Monitoring	All vessels have demonstrated compliance with ballast water exchange via salinity testing of ballast water tanks No Aquatic Invasive Species have been identified through Project monitoring Positive feedback for expanding Aquatic Invasive monitoring to include Ragged Island Community members have reported a concern that Aquatic Invasive are being introduced to the Project area as a result of shipping operations	1. Increased Quality Control and Quality Assurance procedures for salinity testing 2. Use of Remote Operative Vehicle system for underwater video surveys and specimen collection for assessing ship hull fouling	Response to final written submissions on the Mary River Modification Application – Production Increase, Fuel Storage, and Milne Port Accommodations (Baffinland 2018a) **Response to QIA Comment No. 2 -** Baffinland Responses to Reviewer Comments on 2017 NIRB Report (Baffinland, 2018d) **Response to QIA Comment Number 37** **Response to QIA Attachment 2 Comment 5**
Dust	Vegetation Abundance and Diversity Vegetation Health (Metals Accumulation)	As listed in the Terrestrial Environment Monitoring and Management Plan (2018): 1. Project activities will be planned and conducted to minimize the Project footprint 2. Where and when appropriate, dust suppressants may be used on the roads, particularly on heavy-use sections during snow-free months 3. Install shrouds on crushers	o Dust Fall Deposition Monitoring Program o Vegetation Abundance Monitoring o Vegetation and Soil Base Metals Monitoring	Dust fall deposition is greater (higher volume) than levels predicted in the Final Environmental Impact Statement Higher level of dust fall is not affecting vegetation health and level of metals detected in soil and vegetation are not exceeding regulatory guidelines Vegetation abundance and diversity is consistent with baseline levels Ongoing concern from community members about visual effects of dust fall and geographic extent of deposition Ongoing concern from community members about potential for uptake of metals by mammals harvested by hunters and bioaccumulation	1. Six (6) additional dust fall samplers will be installed in late summer/fall of 2018 2. Dust fall monitors will be placed at the 1 km distance, in three paired locations (east/west of the road) one pair will be located near km 25 one pair around km 56 one pair around km 75–80 3. Continue increased calcium chloride and water dust suppression 4. Procured 50 additional covers for crushers	Response to final written submissions on the Mary River Modification Application – Production Increase, Fuel Storage, and Milne Port Accommodations (Baffinland 2018a) **Response to ECCC Comment No. 2** **Response to QIA Comment No. 6** BIM / QIA Resolutions and Commitments (Baffinland 2018c) **Response to Concern No. 11** **Response to Concern No. 12** **Response to Concern No. 13**

References:

Baffinland 2018a. Mary River Modification Application – Production Increase, Fuel Storage and Milne Port Accommodations Modification Proposal ('Production Increase Proposal') (April 23, 2018) – Response to Comments. August 9 2018.

Baffinland 2018b. Nunavut Impact Review Board Process Guidance – Production Increase Proposal – Additional Information Requirement. June 20 2018. NIRB Public Registry ID: 318283

Baffinland 2018c. Baffinland/Qikiqtani Inuit Association Resolution and Commitments Excel Spreadsheet (180802-SR17-QIA-Baffinland2018IssueResolution-ENG). Agreed Upon August 2, 2018 as referenced in QIA's August 3rd letter to NIRB.

Baffinland 2018d. Baffinland Response to Reviewer Comments on the 2017 NIRB Annual Report. July 12, 2018.