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NEW HARBOR IN KANGERLUSSUAQ, WESTERN GREENLAND. FIELD INVESTIGATIONS AND UTILIZATION OF EXISTING MATERIALS

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ABSTRACT
The international airport of Greenland is located in Kangerlussuaq, making it an important connection point for tourists and transportation of goods. However, the existing harbor in Kangerlussuaq experiences major challenges in the form of extensive sedimentation of glaciofluvial sediments transported by rivers from the inland ice to the inner parts of the fjord. These sediment layers reduce the water depth and prevent container- and cruiseships to dock, imposing large additional maintenance costs, and inefficient operability. Through engineering geological field and lab investigations, a possible new harbor location around 10 km further out the fjord near Hancock Pynt, has been investigated. The onshore area was found to be highly suitable for a harbor support area, where a sub-base thickness of 1.8 m with gravel cover-layer was found adequate for the calculated design loads. Existing sediment deposits at the location are reusable as construction material and may reduce construction costs. Bathymetry investigations indicate however that measures must be taken to increase the water depth, and the offshore sediments were found not suitable as support for foundations.

INTRODUCTION
The town of Kangerlussuaq is located in central west Greenland, and holds the country's international airport. This makes the harbor an important connection point for goods and tourists. However, the current harbor in Kangerlussuaq experiences major challenges due to glacial sediment transportation from the Watson River, where the deposition of sediments makes it too shallow for large carriers or cruise ships to dock. To access the harbor, passengers and goods must be transported by smaller vessels from anchoring locations further out the fjord. This is expensive, inefficient and time consuming. The yearly cost of maintaining harbor operability is estimated to 1.0 mill. DKK. Thus, a new harbor further out the fjord would reduce maintenance costs and increase the harbor efficiency. The yearly savings of a new harbor is estimated to 3.5 mill. DKK (Hansen and Paulsen, 2014). The proposed new harbor location can be seen in Figure 1, and will be referred to as Hancock Pynt (HP). The selected location is based on seismic data collected in November 2012, where water depth and thickness of sediment layers in parts of the fjord were measured (Ploug et al., 2013).

Geological and climatic settings

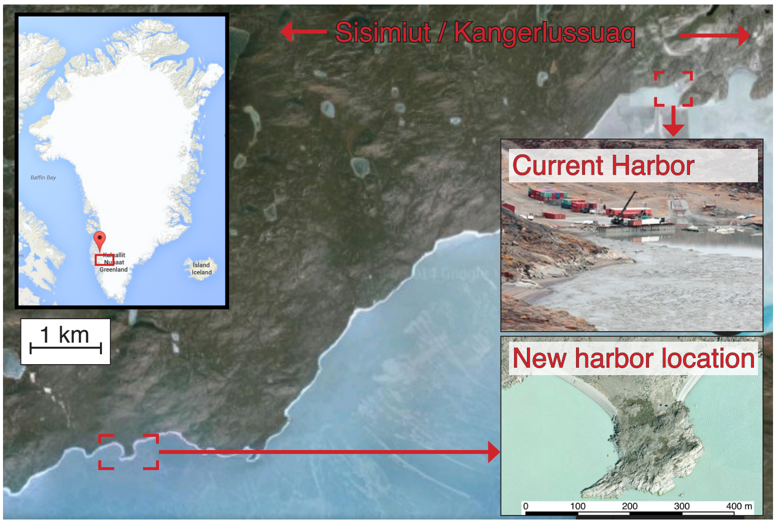

Figure 1. The current harbor in Kangerlussuaq and proposed new harbor location

The Kangerlussuaq fjord has been shaped by glaciers which during the last glaciation eroded the bedrock in the Kangerlussuaq area. This bedrock consists mainly of gneiss along with dark magmatic rocks (Henriksen, 2008). The inner fjord resembles a U-shaped basin filled with glaciomarine sediments (Nielsen et al., 2010). Kangerlussuaq is located in a climate of arctic tundra (Covey, 2002) with a mean annual temperature of $-5.7°C$ and a yearly precipitation of 149 mm measured in the years 1973-1999 (DMI, 2014). During the retreat of the Greenlandic inland ice in the late and post glacial time, marine glacial sediments have been deposited in the fjords on the west coast of Greenland (Foged and Clausen, 2003). Ground temperature measurements conducted in Kangerlussuaq during the period 1968-82 located permafrost at a depth of approximately 2.5 m below ground surface, and the permafrost thickness was estimated to be 127 ± 31 m (Vantatenhove and Olesen, 1994).

Previous studies and paper objectives

Advanced laboratory tests conducted on sediment core samples collected from nearby locations in the fjord in 2013, revealed mainly cohesive sediments with low shear strength. The material was found to be unsuitable as foundation support (Orlander, 2014). Logistical challenges arise due to the remote location of the proposed new harbor. Thus utilization of existing materials can be economically beneficial. The main purpose of this paper is to get a good overview of the onshore sediment properties and also investigate possible reuse of existing material for the final design of the harbors support area. Bathymetry investigations and sampling of offshore sediments around HP will be used to study the correlation with results from seismic- (Ploug et al., 2013) and sediment investigations in the area (Orlander, 2014). Investigations related to the basement rock geology will be compared with a previous study by Sand et al. (2013). From this a placement of the harbor berth line and related measures necessary for construction will be discussed.

MATERIALS AND METHOD

The field investigations were conducted during 6 days in August 2014. The open source program QGIS has been used for GPS interpretation, area calculations, and editing.

Field investigations and sampling - Onshore

The onshore disturbed soil samples were collected by hand augers. Undisturbed intact samples were taken with B-tubes of brass. In situ undrained shear strength measurements of cohesive soil were conducted with a field vane, where the smallest vane was applied. All sampling was conducted according to the 'Manual for field tests' (DGI, 1999). It should be noted that the disturbed samples were collected by mixing a soil layer, and then putting a sample into airtight bags. A total of 49 samples containing from 0.1 to 0.7 kg of sediments were collected. The top layers were removed by a shovel due to dry and loose sediments. Drilling was stopped when assumed bedrock was hit.

The strength of the basement rock was tested in situ by the use of a Schmidt Hammer (SH), type N with a pressure of 2.207 Nm, according to ISRM (Aydin, 2008) and strength grading conducted according to Brown (1981). 20 SH measurements were made at each location. Uniform compression strength (UCS) was calculated using Katz's conversation (O. Katz and Roegiers, 2000). Cracks, strikes and dips were measured with a standard hand-held compass with an attached clinometer. Smaller rock samples were collected for laboratory investigations and classification purposes. All onshore locations were mapped with a hand-held GPS, and topography levelling was conducted with a small optical telescope and a 2 m long leveller.

Field investigations and sampling - Offshore

The offshore seabed sediments were collected with a Van Veen Grab. The boat-mounted GPS/Echo Sounder obtained the sample coordinates and depths. The tidal height when sampling was +0.5-1 m in the time interval 10:30-11.30 (local time) on the 6th of August 2014 (Mobilegraphics, 2014).

Laboratory experiments

All laboratory experiments were conducted in accordance with DGF-bulletin 15 (DGI, 2001). Tests on disturbed samples include water content (wc), grain size distribution, determination of plasticity limits (w_P), liquid limits (w_L), plasticity index (I_P) and Consitency index (I_C). Pore volume (e) and grade of packing of (I_D) were tested on the undisturbed intact samples. Organic content was measured through loss of ignition and color test. Grain size distributions were determined using Andreasens Pipette for $d < 63\mu m$, and by sieve tests for $d > 63\mu m$. It should be noted that the measurements for some of the offshore sediment tests gave incorrect results, with increasing passing percentage for the grain size, determined by pipette method. This error was most likely caused by flocculation of the sediment grains (N. Foged, nov. 2014. Pers. com). For determination of liquid limits, both Casagrande and Fall Cone apparatus were applied.

RESULTS

Onshore sediment investigation

The investigated sediment area on land illustrated in Figure 2 includes an area of about 3 hectares (ha). The total area has been divided into 3 sections, namely the northern section (N), south-western section (SW) and the south-eastern section (SE).

From Figure 2 it is seen that the depth to bed rock ranges from about 0.5 to 2 m. In general the top layer consist of 10 cm of peat and then a dry sand layer with varying content of gravel to about 1 m below ground surface. The sediments are classified as

Postglacial (PG). No shells or shell fragments were found which could indicate that the sediments are fresh water deposits (N. Foged, nov. 2014. Pers. com). A general content of fine sediment fractions ($< 2\,\mu m$) of $< 3\,\%$ were also measured.

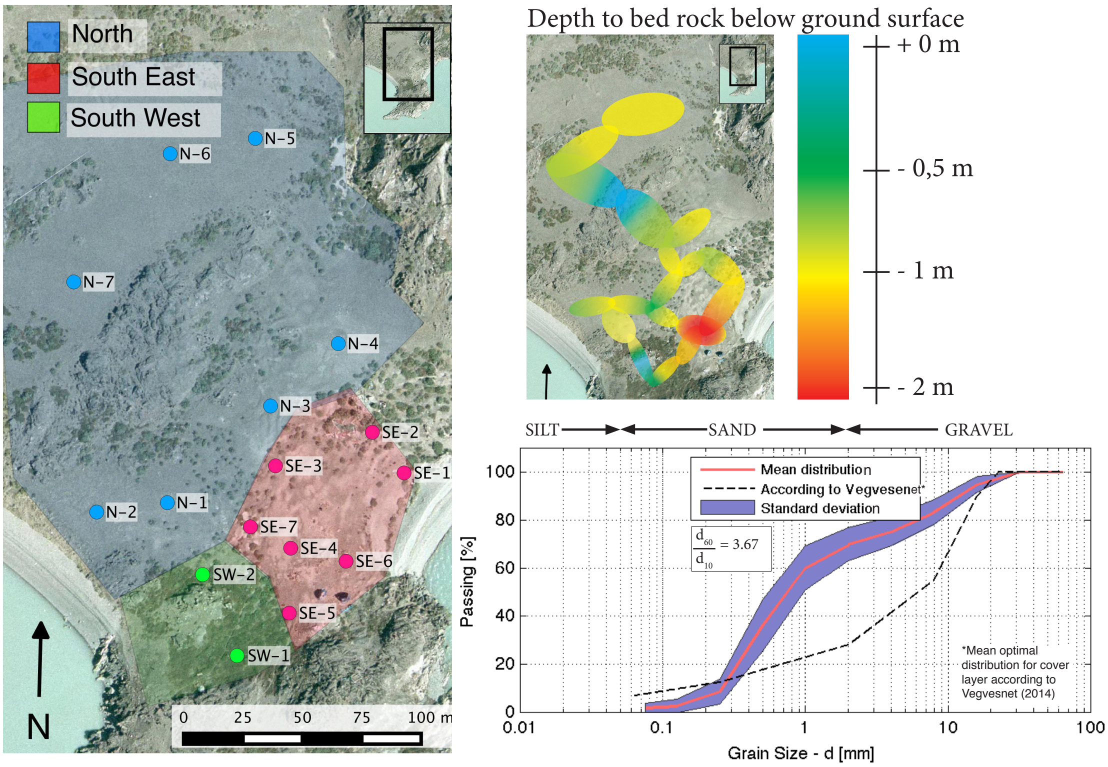

Figure 2. Location of boreholes, depth measurements and mean grain size distribution.

The Northern section includes a centered area consisting of bedrock outcrop. In general the area was found to be very dry with a thin layer of peat covering multiple sand layers with a varied content of gravel as seen in Figure 3. Loss of ignition was measured to $< 1\%$ for all layers in borehole N-6 and the color test indicated little humus, hence the sand material in this area possess little organic material. The **South-Western section** consists of a low vegetation area with heather and bushes, also partly covered by bedrock outcrop. A grey silty sand layer of 10-30 cm thickness was located above assumed bedrock, as seen in Figure 3. The **South-Eastern section** is by far the most diverse area consisting of large amounts of silt and clay located at borehole SE-4, SE-6 and SE-7 as seen in Figure 3. A natural pore volume of $e_{nat} = 0.65$ and a grade of packing $I_D = 0.43$ was measured for the sand layer in borehole SE-6 at a depth of 88 cm. A stony beach separates the area from the fjord to the east, consisting mainly of thin flat shaped stones in the range 20-60 mm in diameter and with a distinct smooth surface. This characteristic stony layer stretches at the most about 10 m into the SE area from the beach with a depth of 55 cm measured at borehole SE-1. A silt/clay outcropping area was located around borehole SE-7, holding a width of 3-4 m and length of around 18 m with a north-west strike. Undrained shear resistances $c_v = 120, 260, 256\,kPa$ and remoulded shear resistances $c_{vr} = 20, 22, 60\,kPa$, were measured 6, 31 and 51 cm below ground surface respectively in borehole SE-7.

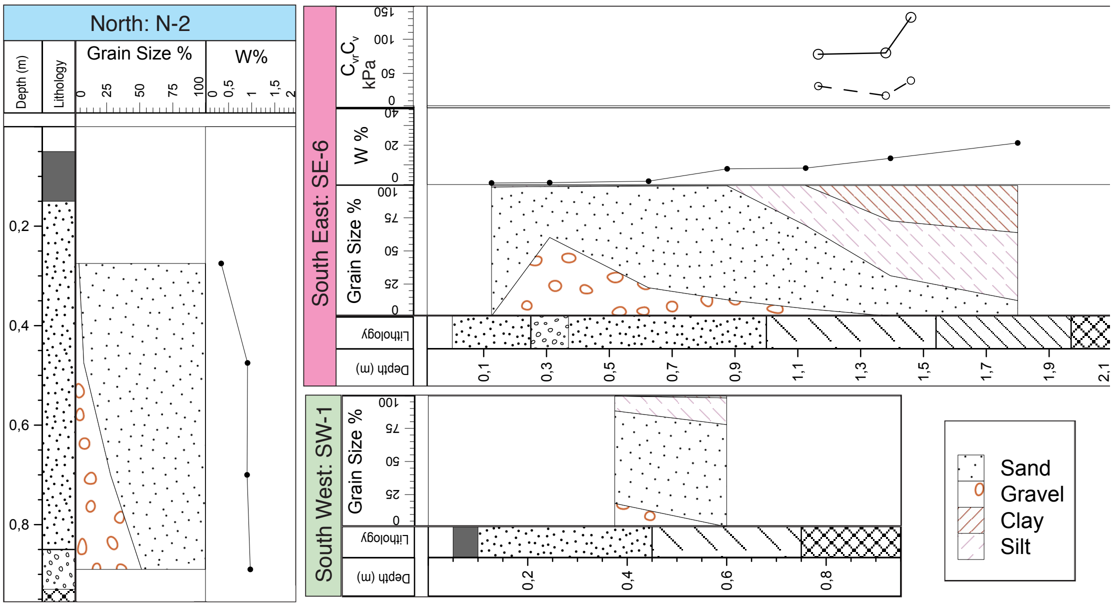

Figure 3. Boring profiles, one from each section.

Basement rock investigations

The basement rock area at HP consists mainly of light colored grey gneissic rock. Foliations consist of bands with alternating paler layers and are dominated by feldspar and quartz minerals as well as darker layers of minerals like hornblende and biotite.The gneiss surface is generally slightly weathered. Some distinct layers of black amphibolite (widths 5-100 cm) with strike direction east and layers of paler coarse-grained granite also with strike direction east, are present in the area. The amphibolite are comprised of plagioclase, feldspar, hornblende and some reddish grenates, while the granite consists of coarse-grained quartz and feldspar (in some parts classified as pegmatite). The amphibolite and granite pegmatite layers are often more weathered than the surrounding gneiss. Several boudins consisting of amphibolite, parallel to the surrounding gneissic foliations, were located at the southern part of the investigated area.

Larger and smaller cracks in the basement rock area at HP are illustrated in Figure 4 on the following page. Green lines indicate smaller cracks with measured openings ranging from 0.2 to 10 cm while red lines indicate larger cracks with measured openings up to 100 cm, often with large variations in size along the length of the crack. Most of the mapped cracks continued into the fjord and since further investigations offshore were not conducted, the lines are just illustrations of the cracks strike direction from shore. Table 1 lists the Schmidt Hammer (SH) results, where 20 measurements were taken for each test, with the standard deviation ($R-\sigma_s$). In total 13 tests were conducted.

Table 1. Measured rebound (R) SH-values and uniform compression strength (UCS).

Rock type	R-value	R-σ_s	UCS [MPa]	Grade	Term	n
Gneiss	48 - 70	2.87-8.17	55-242	R4-R5	S-VS	10
Amphibolite	57 - 63	2.34-5.26	98-154	R5	VS	2
Granite	61	7.11	127	R5	VS	1

Term: S - Strong, VS - Very strong / n - number of tests

Offshore investigation

Depth measurements

The depth measurements are illustrated with colored dots around HP in Figure 4. The depths increase rapidly, and ranges from 5-20 m in a distance of 5-100 m from shore. The near shore water depth was found to be greater on the east side.

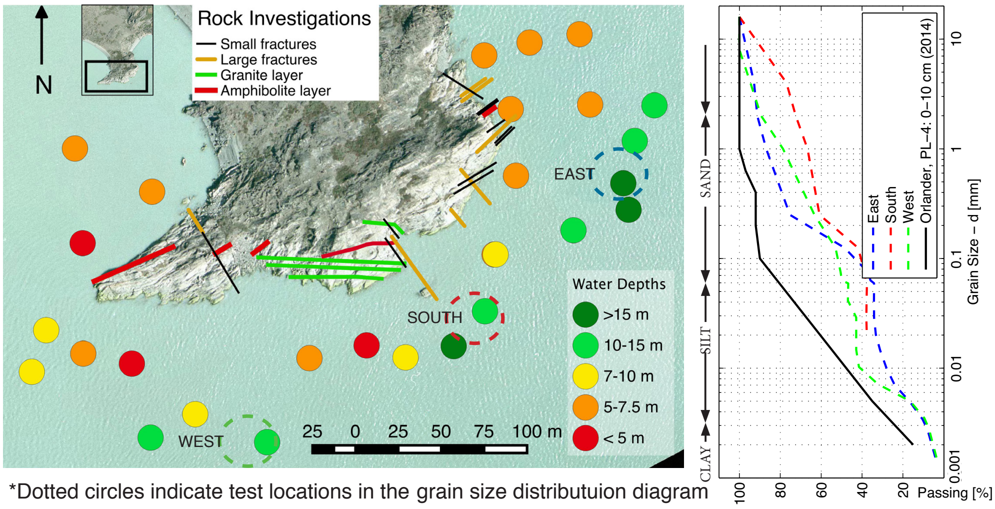

Figure 4. Sampling and depth measurements and basement rock cracks and layers.

Sediments

Table 2 summarize the offshore results for the areas indicated in Figure 4 with dashed colored circles. For the test conducted on the sample from area south, an increased passing percentage due to flocculation as described in section , starting at 0.01 mm was observed, thus only measurements until the start of the flocculation are included.

Table 2. Results from the areas as seen in Figure 4. * Values from Fall Cone.

Area	wc [%]	w_P* [%]	w_L [%]	I_P [%]	I_C [-]	$> 63\mu m$ [%]	$> 0.2\mu m$ [%]	$< 0.2\mu m$ [%]
East	44	24.3	33.5	9.2	-1.2	62	29	9
South	24	16.55	21.9	5.3	-0.55	62	38	[-]
West	27	16.0	22.8	6.7	-0.75	54	37	9

RESULT DISCUSSIONS AND DESIGN CONSIDERATIONS

Onshore sediment area

The borings revealed no indication of permafrost and it is assumed that rock was preventing the auger from penetrating deeper into ground. The bottom of the boreholes were checked with a steel bar where possible. The borings were stopped when the auger spun around without moving further down. A 2 m core of rock is required to conclude that bedrock has been located (T. Ingeman-Nielsen, nov. 2014. Pers. com). This is not possible with hand augers. However since the depths in the investigated area are fairly consistent, and bedrock outcrops several places are prominent, the assumption that borehole depths are down to bedrock are highly plausible. A more thorough investigation in the deepest area around borehole SE-4 and SE-6 are recommended to completely exclude the possibility of permafrost. However, the assumed absence of permafrost and the general shallow bedrock depth strongly favors the construction of a new harbor support area at HP.

The high content of fine sediments located in section SW and SE can result in low bearing capacity and increased deformations (Berntsen, 2013). The great proportion of silt also makes the material highly susceptible to frost heave. This can lead to damage to pavements and buildings either from heaving of the frozen ground or thawing causing collapse of the ground (Wallace, 1987). Therefore, the cohesive material in both the SW and SE section are proposed excavated.

Basement rock area

All measured basement rocks are found to be of strong - very strong grading as seen in Table 1. The range of R-values for the gneiss indicates some differences in the rock strength. The Schmidt Hammer (SH) only gives the rebound value of the surface, thus reduced results can possibly be caused by locally weathered basement rock surface (Aydin, 2008). The results are seen to be within the common physical properties of gneiss and granite (Waltham, 2009) and are also found to be strongly coherent with the results from Sand et al. (2013). Their point load tests also gave higher UCS results than the calculated UCS from R-measurements. This further supports that the investigated basement rock area consists of high strength rocks strongly suitable for constructional purposes. However since the study by Sand et al. (2013) only includes 7 tests, and because the SH measurements are associated with some uncertainties, core samples and proper laboratory measurements should be conducted to acquire more accurate results from HP. Also, special attention to fractures and weathered amphibolite and granite pegmatite layers as seen in Figure 4 should be given.

Design of support area

Behind the berth line and basement rock area, a support area is to be constructed with entrance, parking, office building and a container storage area. An initial area of 1 ha with possibilities of expanding to 2 ha is desired by the municipality (H. Holt, nov. 2014. Pers. com). The areal estimations from section shows that this is possible. As part of lowering the project expenses the area is planned covered with gravel (Hansen and Paulsen, 2014). The area of about 1 ha illustrated in Figure 5 is considered. The presented design follows the Heavy Duty Pavement manual (Knapton, 2008) and the road constructing handbook from the Norwegian Directorate of Public Roads (Vegdirektoratet, 2014).

Design loads

A reach stacker (container truck) of type C4230 TL/5 is proposed to operate the area, and corresponding static and dynamic loads have been calculated using SEWL (Single Equivalent Wheel Load). A design period of 20 years with a total of 260.000 passes of the reach stacker have been assumed (250 passes every week). Four times every year a shipment of maximum 120 containers is expected to arrive the harbor (Hansen and Paulsen, 2014). The allocated container storage area is designed to withstand loadings from containers stacked in blocks, 5 in height. Corresponding static loads affecting the support area have been calculated. Design loads of 566 kN and 914 kN from SEWL and containers respectively were found.

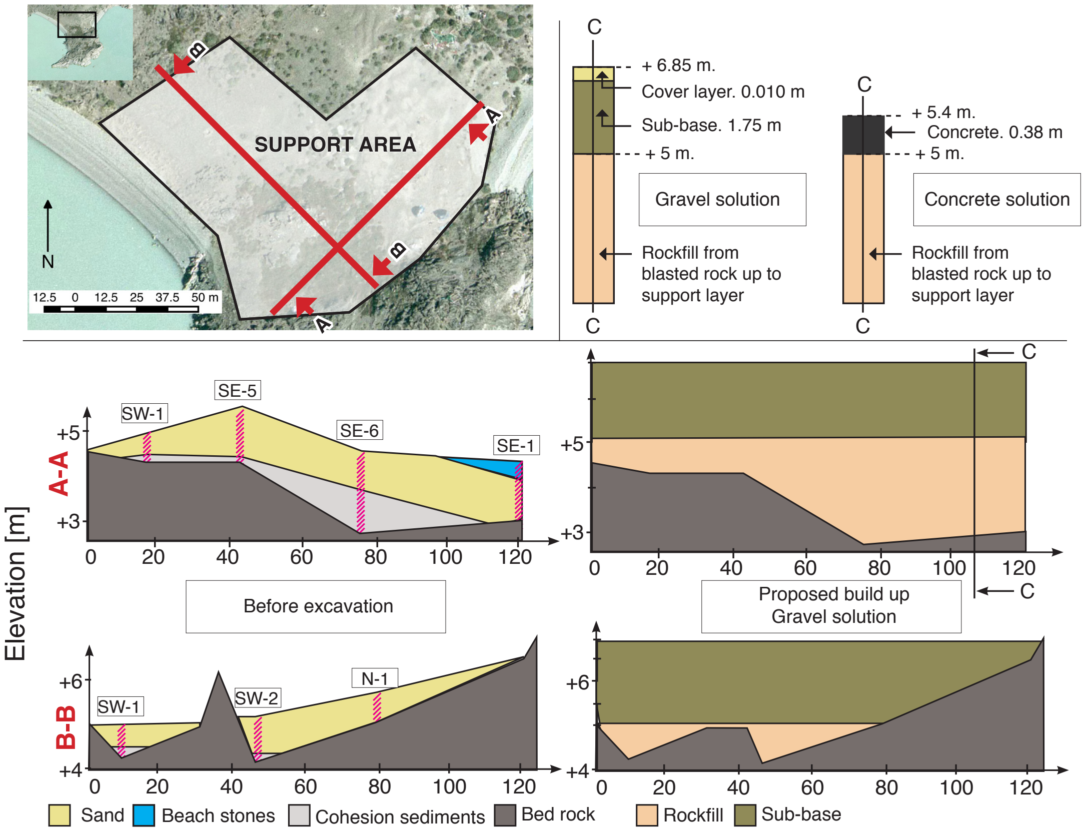

Figure 5. Overview of support area and cross sections illustrating suggested designs.

Existing materials

By excavating all of the fine sediments, the sub-base or rockfill can be placed directly on bedrock. This will significantly minimize the frost and deformation related issues. Roughly estimated a total of 2200 m^3 of fine sediments must be removed from section SE and SW. Apart from the cohesive sediments, the materials in the investigated area mainly consist of sand with some gravel. Rough calculations estimate this to occupy about 24000 m^3. Naturally it would be of great economical benefit for the project if some of these materials could be reused in the support area design or in other parts of the harbor construction.

Figure 2 illustrates the averaged grain size distribution for all investigated sand and gravel layers in the area. It is evident that the grain size distribution for the existing material do not fully match the required distribution for a gravel road cover layer. Gravel from demolition of basement rock related to the harbor construction may be added to better fit the distribution. However as the existing material is seen to be poorly sorted ($C_u = 3.67$), it should be suited as cover layer also without major adjustments.

Related to the harbor construction, significant amounts of concrete will most likely be required, which contains 60-70 % sand and gravel (Gjerp et al., 2009). If this can be mixed on site utilizing the existing natural material as aggregates, it may reduce the project expenses. A well graded material is a requirement for concrete aggregates. The optimal grain size distribution would follow a more straight line than seen in Figure 2 (Gjerp et al., 2009). However the material can easily be processed on site if required. The low content of humus (<1%) is favorable both for utilization as concrete aggregates and as gravel cover layer. Some further investigations regarding durability and

grain shape as well as tests related to chemical and physical requirements should be conducted.
The existing sand and gravel material is found to contain highly interesting properties and it is therefore recommended that all of the sand and gravel in the area are preserved for later utilization in the harbor construction.

Design materials
The support area must be constructed such that it withstands the applied loads and climatic challenges such as frost heave throughout the lifespan of the structure. The foundation materials must have a certain quality and thickness to avoid failure mechanisms. A study by Balstrup & Foegt (1979) in Greenland, found that compacted rockfill of on-site blasted rock, gave a well-graded and frost secure base layer. The nearby mountain ridge to the north-west of the support area is planned demolished related to the new road connection (Hansen and Paulsen, 2014), thus considerable amounts of rockfill will be available. To ensure satisfying foundation capacity, the whole support area should be excavated and replaced with rockfill up to the bottom contour at +5 meters above mean water level, as seen in Figure 5. Two design proposal are presented, one with gravel cover and crushed rock as sub-base and one with concrete. The two design proposals are illustrated in Figure 5. The rockfill is assumed to have a California Bearing Ratio (CBR) of 80 % (HighwayAgency, 2009). With an assumed packing grade $I_D = 0.4$ (Balstrup and Foegt, 1979) for all of the filling material, around 20250 m^3 and 7400 m^3 demolished rock are required for the gravel and concrete solution respectively. For the gravel solution, the crushed rock sub-base material is assumed to be either of Type 1 or 2 as defined in Clauses 803 and 804 in 'Spesification for Highway Works' (HighwayAgency, 2009) The sub-base thickness must be 1560 mm to withstand the SEWL design loads, and 1740 mm to withstand the container loads. The existing sand and gravel material can finally be placed as a cover layer, with a thickness of 5-8 cm (Vegdirektoratet, 2014). Strict requirements regarding grain size distribution, rock strength and compaction of layers for the crushed rock sub-base must be followed (HighwayAgency, 2009). For the concrete solution a C25/30 type concrete following British standard have been chosen (BS8500, 2006). Here a concrete thickness of 338 mm for the SEWL and 377 mm for the static container load is found to be sufficient. A combination of crushed rock as sub-base and concrete top layer may also be a possible design, hence the required concrete thickness can be reduced. However this is not further discussed in this article.

Offshore sediments and bathymetry
Sediment considerations
From Figure 4 it is evident that the sampled material outside HP consists mostly of sand and silt and a clay content of about 10 %. It possess slightly plastic properties. There is an indication of a slightly finer material in the material sampled from west versus east. The sediments are seen to be more coarse than Orlander's results from nearby locations in 2014. This is positive since Orlander concluded that the fine materials gave very little strength and large deformation indexes. However, the negative consistency index for HP as seen in Table 2 indicates similarly to Orlander, a weak material with low bearing capacity and is therefore not suitable to support foundations. The material should be removed and the harbor foundations placed directly on bedrock. To minimize the future risk of failure, the harbor berth line should be placed as close to the basement rock at

HP as possible. The seismic results from 2013 roughly estimates the thickness of the sediments to vary from 2 to 8 m on the south-east part of HP (Ploug et al., 2013). However, these are measurements taken further from land, thus closer investigations related to the thickness of sediments and location of bedrock outside HP must be conducted.

Bathymetry considerations

A minimum water depth of 10 m during lowest tide is required for the new harbor (Hansen and Paulsen, 2014). It is therefore evident from Figure 4 that one of the most crucial aspects concerning the harbor construction is the current water depth. Sufficient depth is reached around 50 m from shore. The displayed depth measurements have not been adjusted for tidal effects, thus the actual depths are even lower than the ones indicated. During the field investigation period, strong tidal currents were observed. To best handle these forces a berth line parallel to the fjord to the south of HP and/or a berth line at the more sheltered eastern side would be preferable. The bathymetry measurements indicate that a significant amount of sediments and/or bedrock must be removed in order to achieve the required depth for both cases. However further investigations regarding tidal currents, wind, waves and ice forces must be conducted before a final design of the harbor can be proposed.

CONCLUSION

Engineering geological field investigations at Hancock Pynt reveal that the onshore sediment area is highly attractive for a future harbor support area. Some fine sediments must be excavated, but the shallow bedrock depth and easily accessible rockfill from the nearby planned road construction, makes this a highly feasible part of the harbor project. Possibilities of future support area expansion are also highly present. Laboratory results on sampled material reveal that the sand and gravel deposits may be reused as cover material for the support area and/or as aggregates for concrete. This may lead to reduced project expenses. A crushed rock sub-base thickness of 1740 mm is required to withstand the design loads and support the gravel cover. As an alternative design, a 377 mm thick concrete layer can be used.

Investigations of offshore sediments revealed fine grained, slightly plastic material with low bearing capacity. To minimize the possibilities of soil failure and structure deformations, the berth line should be placed as close to the existing basement rock as possible. Bathymetry investigations indicate however that to achieve the required depth, offshore sediments and/or basement rock must be excavated. Further investigations to determine the depth of sediments and locating bedrock must be conducted before a final design is determined.

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References

Aydin, A., 2008. Isrm suggested method for determination of the schmidt hammerrebound hardness: Revised version. Tech. rep., Department of Geology and Geological Engineering, The University of Mississippi,.

Balstrup, T., Foegt, N., 1979. Godthåp airport runway placed on dam of compacted rockfill. Tech. rep., Dannish Geotechnical Institute.

Berntsen, G., 2013. Drenering. NCC Roads.

Brown, E., 1981. Rock characterization, testing and monitoring - isrm suggested mmethod. Pergamon, Oxford, 171–183.

BS8500, 2006. Bs 8500: Part 1: 2006 concrete - complementary british standard to bs en 206-1. part 1: Method of specifying and guidance for the specifier.

Covey, C., 2002. Earth's climate: past and future - by william f. ruddiman. w.h. freeman and company, 2001. isbn 0-7167-3741-8 (pbk.), 465 pp.+ xxi. Global and Planetary Change 30 (3-4), 329.

DGI, 1999. Bulletin 14: Felthåndbogen. Dannish Geotchnical Society.

DGI, 2001. Bulletin 15: Laboratoriehåndbogen. Dannish Geotchnical Society.

DMI, 2014. Klimanormaler for grønland.
URL `http://www.dmi.dk/groenland/arkiver/klimanormaler/`

Foged, N. N., Clausen, H., 2003. Engineering geology and geotechnical engineering in west greenland. permafrost related features in sisimiut. Proceedings of the 14th Conference on Engineering Geology.

Gjerp, P., Opsahl, M., Smeplass, S., 2009. Grunnleggende betongteknologi, 2nd Edition. Vol. 4.

Hansen, J., Paulsen, H. H., 2014. Havnerapport kangerlussuaq. Tech. rep., Rambøll.

Henriksen, N., 2008. Geological history of Greenland: four billion years of earth evolution. GEUS.

HighwayAgency, November 2009. Manual of contract documents for highway works. Volume 1 Specifications for highway works. Series 800. Road Pavements.
URL `http://www.standardsforhighways.co.uk/`

Knapton, J., December 2008. Heavy Duty Pavements - the structural design of heavy duty pavements for ports and other industries. The precast concrete paving and kerb association, 4th Edition.
URL `www.paving.org.uk`

Mobilegeographics, 2014. Url: [http://tides.mobilegeographics.com], last checked: 09.09.14.

Nielsen, M. H., Erbs-Hansen, D. R., Knudsen, K. L., 2010. Water masses in kangerlussuaq, a large fjord in west greenland: the processes of formation and the associated foraminiferal fauna. POLAR RESEARCH 29 (2), 159–175.

O. Katz, Z. R., Roegiers, J., 2000. Evaluation of mechanical rock properties using a schmidt hamer. International Journal of Rock Mechanics.

Orlander, T., 2014. Geotechnical properties of glaciomarine sediments for harbour construction in kangerlussuaq fjord. Tech. rep., Department of Civil Engineering, Technical University of Denmark, Kgs. Lyngby, Denmark.

Ploug, J., Ingeman-Nielsen, T., Nielsen, M. H., 2013. Seismiske undersøgelser for havneprojekt i kangerlussuaq. Tech. rep., Center for Arktisk Teknologi Institut for byggeri og anlæg 2013.

Sand, A. H., Skajaa, J. M., Christensen, M. T., 2013. Undersøgelse af fjeldkvalitet i forbindelse med havneprosjekt i kangerlussuaq. Tech. rep., Technical University of Denmark.

Vantatenhove, F., Olesen, O., 1994. Ground temperature and related permafrost characteristics in west greenland. PERMAFROST AND PERIGLACIAL PROCESSES 5 (4), 199–215.

Vegdirektoratet, 2014. Vegbygging. Håndbok N200. Statens vegvesen.

Wallace, M., 1987. How to prevent frost heave. Concrete Construction 32 (4), 369, 371–372.

Waltham, T., 2009. Foundations of engineering geology. Taylor and Francis.