# Appendix G.20
# Hatch Report – Pacific NorthWest LNG
# Construction Methodology for Marine Facilities

## Construction Methodology Description

# PNW LNG Construction Methodology for Marine Facilities (EA Submission)

### H345670-0000-12-124-0011

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**Appendix A MOF Dredging Methodology (EA Submission)**



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# 1. Introduction

PNW LNG has engaged Hatch to provide a description of plausible construction methodologies that could be used to build the marine infrastructure.

This report includes a description of spatial and temporal profiles of the required construction marine equipment so that overlaps with marine resources can be identified. The methodologies developed may be used to address information requests from the Environmental Assessment (EA) process.

The construction methodologies will primarily consider:

- Overall construction feasibility;
- Possible construction methods and techniques;
- Possible construction sequencing and schedule;
- Equipment and resource requirements; and
- Overall timing of construction activities by location.

## 1.1 Design Status

The design completed to date and the various design details submitted as part of the EA are only preliminary and not intended for construction. Although the marine facilities design has been advanced to a level sufficient for the purposes of planning and permitting, considerable engineering effort is still required to complete the design and finalize the design details to a stage that is ready to be constructed.

The final engineering design is subject to change pending the results of further geotechnical studies and other site investigations. The design of the marine foundations especially, is highly dependent on the results obtained from any further geotechnical and geophysical work. Thus as the final design is completed; various design details will change from what is shown on the current drawings. These design details can include, but are not limited to:

- Sizes and dimensions of structural beams and members;
- Span lengths;
- Diameter, wall thickness, length, arrangement and number of piles;
- Bracing configurations;
- Details of pile anchorage into rock;
- Dimensions of foundation elements such as pile caps and the bridge anchor block; and
- Materials used for construction such as steel versus concrete.



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## 1.2 Scope of Work

The project's marine infrastructure comprises several marine facilities which are within intertidal or sub-tidal waters. These marine facilities can be categorized into 4 main sites consisting of:

- LNG Jetty/Suspension Bridge (including LNG berth structures and the Suspension Bridge's SW Anchor Block and SW Tower foundations and associated scour protection armouring);
- Pioneer Dock;
- Materials Off-loading Facility (MOF); and
- Lelu Island Access Bridge.

Separate descriptions of the possible construction methodologies for each of these marine facilities will be provided. The construction methodologies will be developed with consideration of minimizing the potential for environmental disturbance.

The LNG Jetty has been divided into two main components consisting of the Suspension Bridge which spans over Flora Bank, and the Trestle and Berth Structures which make up the balance of the overall LNG Jetty. The Suspension Bridge is approximately 1,100 m long and is the closest jetty section to the shore. Since the Suspension Bridge spans over Flora Bank, it effectively avoids any form of marine foundations within Flora Bank. The Trestle section of the jetty and the Berth Structures are located in deeper waters and are comprised of pile-and-deck construction.

Because of the inherently different structures used for the two Jetty components, the construction methodologies will be divided between the Suspension Bridge and the Trestle/ Berth Structures. The Suspension Bridge component will be further divided between the bridge's superstructure and the bridge's marine foundations consisting of the Southwest Tower and Southwest Anchor Block. A comparison will be drawn between viable methods for constructing the Trestle and Berth Structure component.

## 1.3 Basis of Report

The basis of the construction methodology description for the Trestle and Berth Structures is the LNG Jetty configuration included with the Environmental Assessment drawing submission of November 14, 2014, Hatch Drawings # H345670-1000-12-040-0041 to 0050. It is our understanding that the various designs for the LNG Jetty from the three FEED contractors are very similar in nature and therefore the developed methodology would also be representative of the current LNG Jetty FEED configurations.

The basis of the construction methodology description for the Suspension Bridge marine foundations and superstructure is the concept drawings for Project: 14-204 Drawings #101 to #109, #120 to #124, and #130 to #131 as prepared by Infinity Engineering Group Ltd.

The basis for the construction methodology description for the Lelu Island Access Bridge, Pioneer Dock and the MOF is the FEED level design drawings.

## 1.4 Definitions

The following definitions describe basic marine terminology used in this report:

- **Pile:** A pile is a heavy column or post either driven or drilled into the ground or seabed to support the foundations of a structure. Piles are commonly made of either timber, concrete or steel. For the PNW LNG marine facilities, steel pipe piles fabricated from large diameter steel pipe sections welded together to form the final pile length are most practical.
- **Bent:** A bent is part of a bridge or trestle substructure and is comprised of a group of piles, typically installed in a single line, connected at their tops with a pilecap forming a single rigid frame that supports a vertical load. Bents are typically placed transverse to the length of the overall structure and directly support the beams and girders that support the deck of the bridge or trestle.
- **Cofferdam:** A cofferdam is a watertight enclosure from which water is pumped to expose the bottom of a body of water and permit construction.
- **Jetty:** A jetty is a pier structure built out into the sea or along the shore as part of a port. It typically includes a landing stage at which ships can dock or be moored. For the purposes of the PNW LNG project the LNG Jetty includes the Suspension Bridge, and the Trestle and Berth Structures.
- **Trestle:** A long bridge-like structure which typically spans from the shoreline out to berth structures located in deeper navigable waters. The trestle is supported on a series of bents spaced at regular intervals, using large beams or girders to span between the bent foundations. A trestle's deck typically includes space for a roadway, pipe supports, and pedestrian walkways.
- **Dolphin (Breasting dolphin) (Mooring dolphin):** A dolphin is a man-made marine structure that extends above the water level and is not connected to shore. Dolphins are usually installed to provide a fixed structure when it would be impractical to extend the shore to provide a dry access facility. Typical uses include extending a berth (a berthing dolphin) or providing a point to moor to (a mooring dolphin). The berthing dolphin is equipped with a fender which is used to "cushion" ship impacts whereas mooring dolphins are set a certain distance from the vessel and are equipped with hooks used to secure the vessel's mooring lines. For the PNW LNG facilities both berthing dolphins and mooring dolphins are required to securely hold a vessel in the berth. The dolphin structures typically consist of a number of piles set into the seabed and connected above the water level to provide a platform or fixing point. The piles are typically steel pipe piles connected by a reinforced concrete capping or a structural steel frame. Access to a dolphin is typically gained via a catwalk (pedestrian bridge).
- **Caisson:** "Caisson" is the French word for "box." A caisson is a huge box or tub made of steel-reinforced, waterproof concrete with a central core that is open from the top. For a wharf facility, the caisson comprises both the foundation and main superstructure of the wharf. For the Suspension Bridge, the caisson becomes the Anchor Block or the pilecap supporting the bridge tower.
- **Dock:** A structure extending alongshore or out from the shore into a body of water, to which boats may be moored.

- **Wharf:** A structure built typically along the shoreline of deeper navigable waters so that ships may be moored alongside to receive and discharge cargo and passengers.
- **Spud:** A spud is a retracTable leg or underwater column that is deployed from a dredge or marine derrick barge and is used for staying or maintaining the barge in position as construction proceeds.
- **Fender:** A fender is a rubber component that is attached to wharves, berthing dolphins, and other marine structures where vessels can make contact with the structure. The fender absorbs the impact energy of berthing (incoming) vessels and hence protects the structure and ship from damage. A fender is formed of rubber which absorbs berthing energy by virtue of the work required to deform it elastically by compression, bending or shear or by a combination of such effects. Because a fender is made of rubber it makes little to no noise as it is compressed or stressed by a berthing vessel.

# 2. LNG Jetty - Suspension Bridge Construction Methodologies

## 2.1 Bridge Marine Foundations

The bridge marine foundations are shown on concept drawings prepared by Infinity Engineering Group Ltd. (Infinity). Two major marine foundations are required for the proposed Suspension Bridge located at the SW Tower and SW Anchor Block. In addition to the concept drawings, the proposed construction methodologies as prepared by Infinity have been incorporated into the methodologies described below. These construction methodologies should be considered only as preliminary and may differ from the actual methods used during construction.

### *2.1.1 South West Anchor Block Construction*

#### *2.1.1.1 Conceptual Foundations Overview*

The preliminary design for the South West (SW) Anchor Block consists of a 45 m x 44 m x 21.3 m deep concrete cap supported by an 8 x 8 grid of 1800 mm diameter concrete filled pipe piles. The Anchor Block cap is hollow to accommodate a splay chamber where the bridge's cable strands are anchored to the cap.

The piles are battered at a 4:10 slope in the bridge longitudinal direction. The piles are approximately 65 m long, determined from borehole 34, and are driven through the overburden material down to bedrock. All of the piles are anchored into the bedrock with rock sockets with an embedment of 6.0 m. Out of the 64 total piles, there are 32 tension piles each anchored with 31-strand rock-anchors embedded 25 m into the bedrock. This foundation design is only preliminary and will change prior to the completion of a final design that is ready for construction.

The depth to rock at nearby boreholes B-33 and B-34 was determined as 70 m and 44 m, respectively, from the mudline. The bedrock is primarily overlain by poor quality material with only limited depths (~5 m) of more competent soil immediately above it. Further details are available in the *Draft Geotechnical Recommendations for Trestle and Berthing Area of Pacific Northwest LNG (PNWLNG) Project* by Fugro dated October 11, 2013.

#### *2.1.1.2 Construction Access*

Providing adequate access to the marine foundations is a primary consideration. This is especially so, considering the marine foundations for the bridge are fairly sizable, and will require not only a significant amount of working area for construction, but also a fairly significant portion of the overall bridge construction schedule.

Several potential options for providing the required amount of temporary construction access have been considered including methods such as:

- Self-elevating platforms (jack-up barges);
- Temporary working platforms supported on temporary piles;

- Temporary working platforms supported on the permanent piles;
- Marine-based methods using floating equipment; and
- Temporary/permanent Man-Made Islands.

A discussion of each option is provided below; challenges for each exist and their viability would need to be confirmed during the detailed design process. Once in place, each access option would need to be supplied, to varying degrees, with crew and materials by barges and vessels subject to tidal and other environmental restrictions such as wind, sea state and visibility.

2.1.1.2.1 Jack-Up Barge

A jack-up barge comes equipped with legs or "spuds" which can be lowered to sit on the seabed and used to lift the hull of the barge up above the sea level providing a large fixed working platform unaffected by tides and swells as shown in Figure 2-1. Jack-up barges are fairly common in marine construction and can be located at each foundation location.

The barges can be equipped with large cranes sufficient to support the anticipated pile driving, cleanout, and concrete foundation installation anticipated for the bridge marine foundations. The challenge at this site is the significant depth of weak materials above the bedrock. The bearing capacity of the seabed may be insufficient to support the concentrated loads from the spuds when the barge is elevated, causing the spuds of the barge to sink deep into the mud. This will make extending and retracting the spuds very difficult if not impossible. Also many jack-up barges do not have legs of sufficient length or size to be utilized at this location. Although there are rigs with legs up to and in excess of 80 m in length, none are currently believed to be within local contractor fleets.


**Figure 2-1: Elevated Jack-up Barge in Operation**

As given in Section 2.1.1.4 the overall schedule duration for construction of the SW Anchor Block foundation is 24 months. One or two Jack-up barges could be deployed to conduct construction operations on the foundation. With the exception of extreme sea states or weather conditions, it is anticipated that the jack-up barge(s) would stay in position for most of the foundation construction. The jack-up barges would need to be supplied by one or two supply barges carrying equipment or various construction materials. To resupply the work front and to accommodate the tidal windows, the supply barges would transit every day between an onshore staging area and the work site. For the 6 month period where concrete would be poured for the Anchor Block, at least three concrete supply barges would be required to work in rotation to make on average six deliveries per day in order to meet the quantities of concrete required. In practice the EPC contractor will attempt to maximize the concrete deliveries for any given work shift to provide a somewhat continuous concrete placement. The actual number of daily concrete supply trips may be as high as eight or more, depending on tidal restrictions. The EPC contractor may have to work double shifts during this 6 month period to accommodate tidal windows and meet the schedule.

2.1.1.2.2 Temporary Working Platform Supported on Temporary Piles

With this method, the temporary pile supported working platform provides a fixed structure upon which a crane and equipment can be placed to install the bridge foundations. The platform would be supported on a series of additional temporary piles and would be required to extend around the perimeter of the foundations in order to provide adequate access for constructing the foundations. An example of a temporary work platform supported on temporary piles is shown in Figure 2-2.

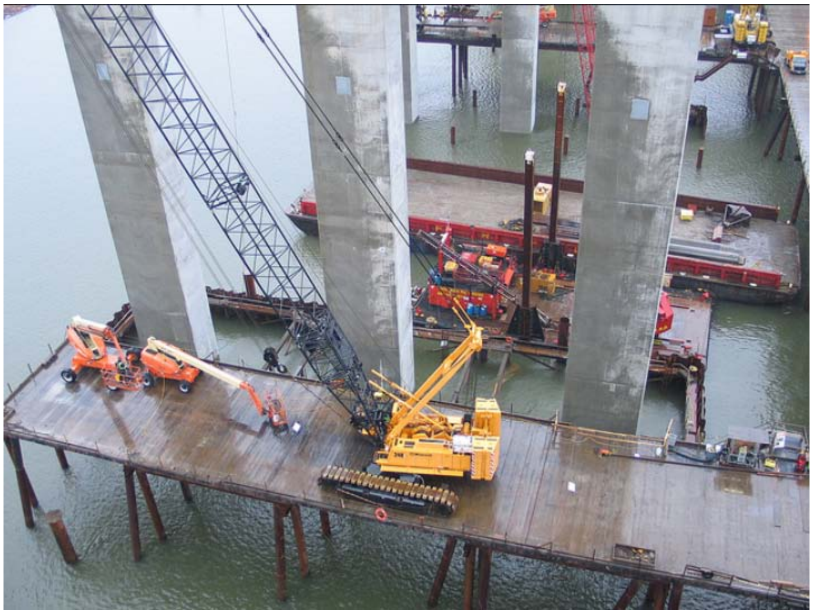

**Figure 2-2: Temporary Pile Supported Work Platform**

The platform(s) would allow the construction work to continue independent of the tides. The cranes and equipment would remain on the platform for the duration of the foundation construction. Similar to the Jack-up Barge method the crews on the working platform would need to be supplied by one or two supply barges carrying equipment or various construction materials. To resupply the work front and to accommodate the tidal windows, the supply barges would transit every day between an onshore staging area and the work site. For the 6 month period where concrete would be poured for the Anchor Block, at least three concrete supply barges would be required to make up to six deliveries per day.

For this option the working space would likely be kept to the absolute minimum in order to limit the amount of construction required for the temporary platform. The restricted space would inhibit the ability to store material and equipment which may lead to more frequent barge supply trips.

Completely removing the temporary piles installed for the platform after foundation construction would be challenging and potentially unrealistic. Consideration for leaving them in place or potentially cutting them off at the mudline would need to be evaluated.

2.1.1.2.3 Temporary Working Platform Supported on Permanent Piles

A temporary work platform supported on permanent foundation piles is similar in function to the previous method, however, the use of permanent bridge foundation piles for supporting the platform provides the advantage of not having to install and remove temporary piles. Prior to erection of the working platform, the first few initial permanent piles would need to be installed via marine-based methods using floating equipment (see Section 3.2). As a result, the installation of the initial permanent piles would be affected by tides and other marine constraints. Once enough permanent piles have been installed, the working platform can be erected and used to install the balance of the permanent piles and foundation works from a fixed platform. As rows of piles are completed, the working platform may be moved or indexed from one row of piles to the next until all the foundation piles are complete. Construction sequencing would require considerable coordination. Because the working platform is within the foundation footprint, the size of the working platform and hence, the amount of work space available would be limited. The construction supply logistics for this method would be very similar to the Jack- up Barge and Temporary Working Platform Supported on Temporary Piles methods in terms of number of barges and the frequency of supply trips.

2.1.1.2.4 Marine-Based Floating Equipment

For the floating marine-based construction method, the cranes, equipment and materials are all brought to the work front on floating flat deck barges. The primary piece of equipment required for marine-based construction is the marine derrick, which is essentially a flat deck barge that has a duty-cycle crawler crane mounted on it. A typical marine derrick is shown in Figure 2-3.

**Figure 2-3: Marine Derrick (Barge Mounted Crane)**

It is expected that there will be two to four barges positioned at each foundation location during construction; one or two marine derricks and one or two flat deck scows for material storage and acting as a work platform to perform miscellaneous tasks. Marine derricks and scows operated by local marine contractors vary in size and can be as large as 60 m long and 20 m wide. To resupply the work front and to accommodate the tidal windows, the marine derricks and scows would transit every day between an onshore staging area and the work site. Alternatively one or more supply barges could also be used for transporting materials from the mainland to the work front depending on construction requirements.

For the 6 month period where concrete would be poured for the Anchor Block, at least three concrete supply barges would be required to make up to six deliveries per day to meet the quantities of concrete required. To maintain somewhat continuous concrete placement, the EPC contractor may have to work double shifts during this 6 month period to accommodate tidal windows and meet the schedule.

The shallow water depths adjacent to the bridge's marine foundations necessitate any marine-based construction to be limited to tidal windows and weather delays. This would extend the construction period significantly as well as drive up construction costs, and therefore floating marine-based methods have a significant disadvantage compared to other options which are not tidal-dependent. This is especially so, when considering the sizable efforts required for constructing the bridge foundations. Additionally, the risk of equipment grounding and unintentionally disturbing sensitive habitat is of concern. Also if the marine derricks are spud barges, the use of spuds to anchor the barge into position will add to the habitat disturbance.

One advantage that marine-based-methods have over other methods is that additional crews and equipment can be readily deployed on multiple work fronts to mitigate delays and improve productivity. However, this advantage is somewhat lessened for the bridge foundations, since there is only enough room for a limited number of work faces before congestion occurs.

2.1.1.2.5 Temporary/Permanent Man-Made Islands

Due to the shallow water depths at the proposed bridge foundations, the construction of a man-made island at each foundation location was considered as a means to provide uninterrupted construction access. Following construction of the bridge, the islands could be removed, in case of temporary islands, or left in place permanently. The amount of material removed and the desired final elevation of the islands would be dependent on the intent of any offsetting measures such as the creation of additional inter-tidal habitat or the planting of additional eelgrass beds or riparian vegetation.

However, the islands would require placement and possibly reclamation, of a significant volume of material in the marine environment. The resulting footprint of the man-made islands on the marine floor would be considerable and at the SW Tower foundation would likely extend onto Flora Bank. Due to this large habitat disturbance footprint, the man-made island option was discounted and withdrawn from further consideration.

2.1.1.2.6 Construction Safety Zone and Navigation Protection

Regardless of the method selected to gain access for constructing the bridge foundations, there should not be an impediment to marine traffic. The work faces will occur in relatively small areas in open waters away from or adjacent to approach channels and will be surrounded by a cofferdam. A safety zone would be enforced around the cofferdam, and work site in general which would prohibit vessels from approaching closer than 50 m. Work site restrictions would be promulgated through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan. As a participant in the Port of Prince Rupert's Construction Coordination Committee for de-confliction with other users, PNW LNG's Marine Communications Plan will be used to alert mariners to project activities, hazards and safety measures such as enforced safety zones and marine route closures.

*2.1.1.3 Construction Process*

Once equipment access and a working area at the foundation locations have been provided either through the use of a Jack-up barge, temporary work platforms, etc., the construction of each marine foundation is presumed to generally follow a similar process. This consists of cofferdam installation, pile installation, concrete work on the Anchor Block or pilecap, concrete pedestal/tower base construction, cofferdam removal, and placement of scour protection. Each activity is described in more detail below.

1. Cofferdam installation is required around each of the marine foundations to allow dewatering and excavation to the depth of the concrete pile cap soffit to facilitate concrete works and construction. It is anticipated the cofferdams would be constructed of steel sheet piles driven to depth in the soft sediments and extending up beyond the high water level. Internal bracing within the cofferdam would be used to support the sheet

piles against external pressures. If the size of the foundations, particularly at the SW Anchor Block location, and the poor soil at the site preclude the use of a single sheet pile cell, utilizing a ring of self supporting sheet pile cells may be an option. After sheet pile installation, a steel template is installed inside the cofferdam to correctly position the piles. The bottom of the concrete foundations will be located to sit approximately at or slightly above the existing mudline to avoid the need for any overburden excavation from inside the cofferdam.

After the piles are installed, the steel template can be removed. Finally a concrete seal is constructed, via tremie pouring, at the bottom of the excavation to allow for dewatering. After the cofferdam is dewatered the piles are cut off at the bottom of the Anchor Block. Figure 2-4 shows a typical cofferdam after the concrete seal slab has been poured and the cofferdam has been dewatered.

**Figure 2-4: Cofferdam**

2. Pile installation commences after the cofferdam is excavated and the steel pile template has been installed.

   The proposed construction method for pile installation involves the following steps:

   - Set the pile into the sediments by gravity;
   - Vibrate the pile open-ended though the sediment until the tip reaches bedrock;
   - When the pile reaches bedrock, seat the pile into the bedrock with an impact hammer;
   - Extract sediments inside pile with a grab hammer or airlift methods;
   - Drill a hole in the bedrock with the drill inside the pile which is acting as a casing; and
   - For a rock doweled pile, advance pile into under-reamed hole and then grout pile; or alternatively for a rock socket pile, do not advance pile but insert rebar cage and pour tremie concrete into bottom of hole.

Pile driving and drilling including clean out and socketing, for the proposed 1800 mm diameter pipe piles is similar, only on a larger scale, to the trestle foundations as outlined in Sections 3.4 and 3.5. Any overburden material removed from inside the piles will be stockpiled on a barge and disposed of on land as discussed in Section 3.4.

One difference between the trestle piles and the Anchor Block piles is that the Anchor Block piles are filled with concrete over their full length. Thus, instead of stopping the concrete after the rock socket is complete, the concrete pour will continue until the pile is filled to the top. Since the pile driving operations are within the confines of the cofferdam, the effects to the surrounding marine life will be minimal. The piles will be approximately 65 m long and will require welded field splices. Field splices can either be done on a barge with the pile in a horizontal position or in-situ, depending on the size of crane used.

In addition to the rock sockets, rock anchors need to be installed for the 32 tension piles. The rock anchors consist of 31-strand tendons that are embedded 25 m into the bedrock and anchored within the Anchor Block. After drilling for the rock sockets, a smaller diameter drill bit is used to drill into the bedrock to the required depth. The strand anchors and an inner casing are lowered to the bottom of the drilled shaft. Concreting for the rock sockets is completed and the piles are filled with concrete. The rock anchors are completed by injecting grout from the bottom of the shaft upwards to a depth required to fully develop the anchor force. The strands are stressed from within the Anchor Block and the remaining length of the shaft is grouted to protect the anchors from corrosion.

3. Concrete foundation construction would progress similar to onshore construction after completing pile installation. The connection between the piles and pilecaps consists of a reinforced concrete plug inside each pile head with the reinforcement cage extending up into the mass concrete of the pilecap. Construction of the pilecaps and other concrete elements would consist of placing reinforcement, constructing formwork, and pouring concrete in a series of lifts separated by construction joints. Supply of the large volumes of concrete required for the foundations will need to be conducted via barge. As a result, concrete supply and hence concrete placement will be affected by tidal restrictions and may impose longer construction times. Considering the large volumes of concrete required for the bridge foundations, there are two possible methods for supplying concrete to the work face; one involves concrete produced by an onshore batch plant and delivered to the work face via concrete trucks on a barge, and the other involves using an actual floating batch plant located adjacent to the work site. For the first method, the barges will be loaded with a pump truck and several concrete trucks and will travel from the construction batch plant located on the mainland to the work front at approximately 1-hour intervals. Figure 2-5 shows a typical barge loaded with a pump truck and several concrete trucks. This method is generally slower and is restricted by a delivery window of about 2-hours which represents the time the concrete is allowed to sit in the trucks. As mentioned previously, considering the volume of concrete required, it is estimated that at

least six barge runs carrying at least four 10 $m^3$ concrete trucks and one concrete pump truck would be required to meet the needed concrete volumes and meet the schedule.

The floating batch plant method on the other hand is more efficient at supplying concrete to the work face, however it is more susceptible to tidal and weather related down times. The contractor will need to the weigh the pros and cons of either method or may choose to use a combination of both, depending on seasonal weather conditions.

**Figure 2-5: Barge Loaded with Concrete Pump Truck and Concrete Trucks**

Prior to casting the top portion of the Anchor Block, the suspension cable anchorage assemblies and splay saddles are installed and secured into the Anchor Block. These are large assemblies of prefabricated structural steel that will be transported to the foundation on barges and installed with the barge crane. Finishing works on the Anchor Block will be completed after the main suspension cables have been installed and adjusted.

4. Cofferdam removal can commence once the required foundation concreting works are above the high water level. Excavated material can be placed back within the cofferdam allowing for the removal of internal bracing layer by layer. The steel sheet piles can be removed with a vibratory hammer. Consideration for leaving the sheet piles in place below the top of the footing to help with scour prevention should be made.

5. Placement of scour protection around the perimeter of the Anchor Block would be conducted soon after removal of the cofferdam. Due to the shallow waters around the Anchor Block, the scour protection would be placed via marine-based methods using tidal windows and a flat deck barge equipped with an excavator or front end loader and a stockpile of riprap. The excavator or front end loader would place the scour protection to the appropriate thickness.

6. In lieu of using the traditional cofferdam approach as described above, an alternate method involving the use of a precast concrete caisson was also considered. This method involves fabricating a partial-depth precast concrete caisson either in a dry-dock or on a barge, towing the caisson to site, and then floating the caisson out to the Anchor Block location where it will be ballasted down to rest on the seabed floor and fixed in position with permanent steel pipe piles. The caisson forms the outer portion of the Anchor Block and will be part of the final structure. Once the caisson is in position, the rest of the foundation construction is completed in-situ. The piles are installed using the floor of the caisson as a template to correctly position each pile. Pile installation commences similar to the cofferdam option. Once the piles are installed, the caisson is dewatered and construction of the mass concrete Anchor Block can be completed within the confines of the caisson. This option limits the amount of work that would be performed by floating marine-based equipment using the cofferdam approach as well as eliminates the need for a temporary cofferdam and template. However, the required size of the caisson is relatively large and considering the weak soils that make up the seabed, stability and settlement issues of the caisson would need to be addressed. Due to the engineering and construction challenges associated with this method, it was dismissed from further consideration.

##### *2.1.1.4 Construction Schedule*

Construction of the SW Anchor Block will consume a considerable amount of the construction schedule and will be on the critical path. It is estimated to take approximately 24 months to construct the Anchor Block, with the following breakdown:

- 3 months to construct the cofferdam
- 10 months to install the piles and rock sockets
- 6 months to install the concrete Anchor Block mass concrete
- 3 months to install the rock anchors (work can overlap with constructing the mass concrete Anchor Block)
- 2 months to install the suspension cable anchorage assemblies and splay saddles

#### ***2.1.2 South West Tower Foundation Construction***

##### *2.1.2.1 Overview*

The preliminary foundation design for the South West (SW) tower consists of 4 rows of 7 piles each (total of 28). The piles in the outer periphery of the pile group are battered at a 1:10 slope. The piles are 1800 mm diameter steel pipes, filled with concrete with rock socketed ends. The rock bed at the SW tower location is approximately 100 m below Low Water Level. The approximate mud line and rock bed top level have been taken from Bore Hole BH – 33. The approximately 100 m long piles will be vibrated through the overburden material into the bedrock. A drilled rock socket of 6 m length will be provided. The pile is filled with concrete over its entire length and connected rigidly to the pile cap. The cap provided is 36.4 m ×

20.2 m × 4 m deep. A 13 m high concrete tower base extends from the pile cap to the underside of the superstructure. The tower base consists of two 4 m x 8.5 m columns connected with a 1 m thick infill wall for transverse rigidity. This foundation design is only preliminary and will change prior to the completion of a final design that is ready for construction.

#### *2.1.2.2 Construction Access*

Similar to the SW Anchor Block, there are several potential options for providing the required access to construct the tower foundation. The south edge of the footing cap however, is directly adjacent to Flora Bank and therefore there will be limited to no access on the south side of the foundation to avoid disturbance to Flora Bank.

#### *2.1.2.3 Construction Process*

The SW Tower foundation will be constructed in the same manner as the SW Anchor Block, by installing a cofferdam around the perimeter of the foundation, installing piles and rock sockets and finally concreting the pile cap. Scour protection would be placed shortly after removal of the cofferdam once the concrete foundation is complete. The equipment used will be the same as that for the Anchor Block and the number of daily barge transits will be similar. The volume of concrete required for the tower foundation however, is significantly less than the Anchor Block, so concreting will take less time and require less barge traffic. For the 4 month period where concrete would be poured for the SW Tower foundation, at least two concrete supply barges would be required to work in rotation to make, on average, one delivery per day in order to meet the quantities of concrete required. In practice the EPC contractor will attempt to maximize the concrete deliveries for any given work shift to provide a somewhat continuous concrete placement. The actual number of daily concrete supply trips may be as high as eight or more, depending on tidal restrictions. Concrete placement for the SW Tower should be coordinated to not overlap with concrete placement for the SW Anchor Block. This will allow for the least amount of concrete supply equipment required for the bridge foundations.

Similar to the Anchor Block, the concrete caisson method is a possible alternative to the use of cofferdams. However for the same issues and concerns surrounding the use of concrete caissons for the Anchor Block foundation, use of this method for the tower pilecap was not given much consideration past the initial concept development.

The concrete tower base will be constructed on top of the pile cap within the cofferdam using the same concreting methods as the pile cap. The pile cap will be used to anchor a tower crane that will be used to construct the steel portion of the tower. As a result, the cofferdam should be left in place at least until the tower crane base is installed.

*2.1.2.4 Construction Schedule*

Construction of the SW tower foundation will be on the critical path. It is estimated to take approximately 16 months to construct the foundation, with the following breakdown:

- 3 months to construct the cofferdam
- 9 months to install the piles and rock sockets
- 2 months to install the concrete pile cap
- 2 months to install the concrete tower base

## 2.2 Bridge Superstructure

### *2.2.1 South West Tower Construction*

*2.2.1.1 Overview*

The 128 m long portion of the towers above the concrete base are composed of structural steel legs connected with cross-bracing. The towers are 125 m high above deck level. The thick walled steel boxes are 2.2 m wide and vary in length from 5.5 m at the top to 7.0 m at the bottom. Each segment is approximately 5.0 m in height. The connections between the leg segments as well as the connections of cross members to the main tower legs are bolted splice connections detailed for repetition and ease of on-site installation. Saddles that carry the main suspension cables are positioned on top of the towers. Typically of cast steel, the saddles can also be custom fabricated using built up plates. They will be equipped with rollers to allow the main cables to shift/adjust under construction and normal loads.

*2.2.1.2 Construction Access*

Construction of the steel tower is done mainly from a tower crane and temporary work platforms supported off the permanent tower legs at required locations. The tower crane is supported on the permanent footing cap and is self-climbing (i.e. it increases in height as the tower height increases). The tower crane is braced to the permanent tower legs as its height increases. Figure 2-6 below shows a typical tower crane used for bridge tower construction.

**Figure 2-6: Tower Crane**

Temporary work platforms are installed to the tower segments before they are erected to provide access to the bolted connections. Work platforms are installed at the top of the tower legs to facilitate installation of the suspension cable saddles. The work platforms at the top of the towers typically remain in place for future maintenance and inspection of the cable saddles. Temporary stair scaffolding is typically provided the full height of the tower to provide access to the temporary work platforms.

##### *2.2.1.3 Construction Process*

Construction of the steel tower segments consists mainly of lifting prefabricated structural steel segments off the barge and onto previously installed segments using the tower crane. There are two tower legs for the SW tower braced to one another by structural steel bracing. Tower segments are spliced together with bolted connections. Bracing members are bolted to the tower leg segments as the tower erection progresses. Steel segments are typically trial assembled at the fabrication facility to ensure member fit-up and ease of installation on site. During trial assembly, adjacent segments are assembled using the final splice plates and about half the number of bolts required in the final connection. Corrections are made so that the segment fit-up and finally geometry is met. Finally, the pieces are marked, disassembled and shipped to site. Temporary working platforms will be pre-installed to the tower segments on site. This work will likely take place on the mainland to avoid having to install the platforms on a barge or high in the air.

##### *2.2.1.4 Construction Schedule*

The tower segments, with work platforms already installed, will be delivered using material supply barges with an anticipated delivery frequency of one barge per week. Construction of the SW Tower is estimated to take approximately 6 months.

### 2.2.2 Suspension Cable Construction

#### 2.2.2.1 Overview

The main suspension cables are fabricated from numerous small diameter (5 to 6 mm) high strength parallel wires that are galvanized. Cable clamps are provided every 10 m and serve two purposes. First the cable clamps are used to compact the parallel wires into a tight bundle of circular shape and minimum diameter. Second, the vertical hanger cables used to support the deck are connected to the cable clamps.

The vertical hanger cables that support the bridge deck from the main suspension cable consist of high strength wire ropes that connect to the cable clamp at the top and will utilize a stressable anchor socket at the lower connection to the superstructure. The stressable anchor sockets at the lower end will provide for adjustability to address fabrication and erection tolerances. The anchor sockets are connected to the superstructure steel girder by a steel pin with the adjustability coming from the threaded anchor socket itself.

#### 2.2.2.2 Construction Access

The majority of the construction for the main suspension cable happens from a hanging catwalk system suspended from the suspension cable itself. Hence, there is very little marine-based traffic required while constructing the main suspension cable. Barges will only be required for supplying materials for the catwalk system.

#### 2.2.2.3 Construction Process

The main suspension cable starts by stringing a small diameter pilot cable from the SW Anchor Block to the NE Anchorage. The pilot cable can be carried from one anchorage to the other by helicopter or by boat and then hoisted to the top of the towers using the tower cranes. Once the pilot cable is installed, it is then used to pull successively larger cables over the towers to the opposite anchor block. Through this means, multiple cables will be installed to allow for catwalks to be erected about 1.5 m below the proposed suspension cables over the full length of the bridge. The catwalks are an access walkway used by ironworkers to facilitate main suspension cable installation. After completion of the catwalks, the main suspension cables are installed by continuously pulling individual wires back and forth from anchorage to anchorage until the required number of wires is achieved (aerial spinning). The reels of wires are delivered to Lelu Island and will be fed to the spinning process from immediately behind the NE Anchorage. Figure 2-7 shows a suspension cable during the spinning process.

**Figure 2-7: Spinning of Main Suspension Cable**

The cable spinning is done using equipment supported by the cables themselves, thus no floating construction equipment is required during the cable installation process. After the requisite number of wires is installed, the bundles of wires are compacted into a circular pattern and the cable clamps are installed. The cable clamps serve to maintain the circular shape of the cable and for attaching the vertical hanger cable. A typical cable clamp is shown in Figure 2-8.

**Figure 2-8: Cable Clamp**

With the main suspension cable and cable clamps completed, cable cranes that ride on top of the suspension cable are installed. The cable craned will be used to install the vertical hanger ropes and the bridge deck segments.

After the superstructure has been constructed, finishing work on the main suspension cable is done. This includes final cable compaction, installation of cable wraps for corrosion protection and catwalk removal. As shown in Figure 2-9 this work is done from the catwalks and the newly installed superstructure, thus requiring no support from marine equipment.

**Figure 2-9: Final Cable Wrapping**

#### 2.2.2.4 *Construction Schedule*

It is expected that installation of the main suspension cables lasts approximately 8 months and must be completed before the superstructure segments can be installed. Cable finishing works is expected to last approximately 8 months and will begin after the superstructure is complete. During work on the main suspension cables there will be very little construction marine traffic, although crews will be continuously working above any marine traffic navigating below the suspension bridge.

### 2.2.3 *Deck Superstructure Construction*

#### 2.2.3.1 *Overview*

The suspended portion of the deck extends from the NE Tower to the SW Anchor Block (total length of 1490 m). Structurally the deck is composed of an orthotropic steel box girder supported on either side with the two suspension cable planes. The deck is suspended with four hanger cables spaced approximately 10 m apart along the longitudinal axis of the bridge. The orthotropic steel box is aerodynamically shaped for wind stability. Each segment is approximately 20 m long and is connected to adjacent segments by fully bolted splice connections. Each segment weighs approximately 200 metric tons.



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*2.2.3.2 Construction Access*

The steel box deck segments are lifted into position using a cable crane that travels along both suspension cables. The deck segments are delivered on a barge directly under their intended final position. Since the main span of the suspension bridge spans over the Flora Bank, all deck segments within the main span will need to be barged into position during high tides (water depths greater than 4 m). Once the segment is lifted off the barge (approximately 1 hour), the barge is no longer required and can be taken away from the Flora Bank. All other work for installing the deck segment can be done above water without any tidal restrictions.

A below-the-deck gantry will be used to facilitate installation of the bolted connections along the underside of the deck. This gantry will travel along the deck maintaining its position at the leading edge of deck construction.

*2.2.3.3 Construction Process*

The cable crane will hoist each segment from the barge into its final position where four hanger ropes will be attached. The segment is bolted to the previously installed segment and the cable crane is released. The cable crane walks itself up the suspension cable and sets itself into position for the next segment and the process repeats until all segments are installed. Figure 2-10 shows installation of a deck segment of similar shape as that proposed for this bridge.

**Figure 2-10: Deck Segment Installation**

Prior to installation, the steel box segments will be stored on the staging area on the mainland from the time they arrive on site from the fabricator until the time they are ready for installation.

*2.2.3.4 Construction Schedule*

It is expected that construction of the suspended superstructure will last approximately 9 months. With an estimated 75 deck segments, it is anticipated that each deck segment will take approximately three to four days to install, with only one of these days involving marine traffic to transport the segment to its final position. Only those areas which are within a safety radius of the deck section being lifted will be restricted from traffic underneath the bridge. The balance of the passage underneath the bridge should remain open to marine traffic throughout the duration of the suspended superstructure construction. The erection of deck sections will be conducted at high tide to mitigate the possibility of grounding the supply barge onto Flora Bank. The erection of deck sections will also need to be timed to occur during minimal wind conditions.

There is an estimated 8 months of additional finishing work to take place on the suspension cables and bridge deck once the suspended superstructure is complete. However, all of this work will be within the confines of the new bridge and will pose minimal interruption to normal marine traffic.

# 3. LNG Jetty - Trestle & Berth Construction Methodologies

## 3.1 Overview

The construction methodologies described in this section are suitable for pile-and-deck structures as required for the LNG Jetty Trestle and Berth Structures component. Pile-and-deck structures are commonly used for marine facilities in British Columbia. For this type of structure, individual support piles are either driven into overburden sediments, if the overburden sediments are deep enough, or drilled into bedrock. The piles provide support to a pile cap and deck superstructure.

The trestle structural configurations as given in the Environmental Assessment submission and the various FEED contractor bid submissions are all pile-and-deck type structures with some variations with respect to materials and member sizes. In general, the various designs for the trestle can be described as a deck superstructure supported on steel pipe pile bents installed at 36 m spacing. The pipe piles vary from 914 mm to 1219 mm in diameter and are either seated on the bedrock or socketed into the underlying bedrock. Each bent has at least 4 piles, including vertical and battered piles, and has a steel box beam or precast concrete pile cap which spans perpendicular to the trestle alignment. The pile caps in turn support steel box beams or precast concrete girders which span between the bents and form the main trestle superstructure and vehicle roadway. The pipe racks consist of steel truss assemblies spanning between the bents and are installed adjacent to the roadway. A typical LNG trestle structure is shown in Figure 3-1.

**Figure 3-1: Typical LNG Trestle Structure under Construction**

The two principal methods proposed for constructing the LNG Jetty Trestle and Berth Structures are:

- The **Marine-Based Method**, which uses floating equipment; and
- The **Cantilever Method**, which uses a mobile construction platform that progresses above the water supported by the very trestle foundations that it installs.

Although either method may be used to construct the entire Trestle, as discussed in the following sections, the floating Marine-Based Method is more effective in deeper water, whereas the Cantilever Method is more efficient and quicker in shallow waters, since marine-based equipment will be restricted to tidal windows and hence longer construction times.

## 3.2 Floating Marine-Based Method

Where the water depths are sufficient enough to not limit construction to tidal windows, marine derricks and floating equipment may be used to install the Trestle and Berth Structures. There is less likelihood of barges grounding in deeper waters where the seabed is sub-tidal. Also sub-tidal habitat is less sensitive to disturbance. In shallow waters, the work would be limited to tidal windows and the risk of equipment grounding and disturbing the sensitive inter-tidal habitat is greater. Also the cost of construction will be higher and take longer due to the constant interruption of work to accommodate tidal windows.

For this construction method, the cranes, equipment and materials are all brought to the work site on floating flat deck barges. In general, the methods and techniques used to install and drill piles are similar to the Cantilever Method with the exception that the crane is on a floating platform and not a fixed one. As such, the construction activities are more susceptible to delays due to sea state and environmental conditions. In addition, temporary piles and falsework may be required to support templates for accurately installing the permanent piles.

Placement of scour protection around the perimeter of each pile for either the Trestle or the Berth Structures would be conducted soon after installation of the piles and before the installation of the main deck elements to allow easier access for floating equipment. For structures located in shallow waters, the scour protection would be placed via marine-based methods using tidal windows and a flat deck barge equipped with an excavator or front end loader and a stockpile of riprap. The excavator or front end loader would place the scour protection to the appropriate thickness around each pile. For structures in deeper waters, in lieu of an excavator, a marine derrick fitted with a clamshell grab may be used for more accurate placement of the scour protection.

### *3.2.1 Construction Equipment*

The primary piece of equipment required for marine-based construction is the marine derrick. A marine derrick is essentially a flat deck barge that has a duty-cycle crawler crane mounted on it. In order to drive piles and drill into rock the crane must be able to accommodate various driving and drilling rig equipment. Both vibratory and impact hammers will be required for the initial setting of the piles. The drill rig will typically be a diesel powered drill unit capable of accommodating various types of drill bits.

The size of crane is dictated by the capacity requirements of the heaviest pile and by the length of crane boom required for properly handling and pitching the longest pile. Because the floating marine equipment can be brought relatively close to the work face, the crane will have a smaller lift radius than other construction methods and therefore the capacity requirements will be less onerous. Local marine contractors have duty-cycle cranes with capacities ranging up to 350 tons which, depending on boom length, can easily pick loads ranging up to 60 to 100 tons based on a 15 m to 10 m lift radius.

For the heaviest or longest piles, in-situ field splicing is an alternative, if cranes with the necessary capacity or boom length are not readily available. Although field splicing is not uncommon, it should be limited due to the time and expense required to make field welds over open seas as well as associated quality issues. Field splicing may also be difficult to execute, if the piles sink into the weak seabed material under their own weight.

#### *3.2.2 Marine-Based Construction Schedule and Equipment Profile*

With marine-based construction, the basic sequence for installation of the pile foundations and the trestle superstructure is similar to the Cantilever Method. For the most part the actual installation techniques will be the same except the equipment is based on floating platforms instead of a fixed platform. Although mobilization and demobilization will be quicker with marine-based methods, the possible need for erecting false work and temporary templates for installing the pile bent foundations may reduce overall productivity. For those piles that need to be socketed into the rock, the critical activity will be the drilling operation which may take slightly longer with floating equipment than it would with the Cantilever Method. Also for those pile foundations that are not socketed, the productivity of the marine-based crane operations may be reduced due to weather and sea state delays. However, unlike the Cantilever Method, overall productivity can be easily improved by just adding additional crews and equipment. Multiple crews and equipment can advance on multiple work fronts. The productivity rate will be directly proportional to the number of additional crews and equipment deployed.

The overall estimated schedule duration for the construction of the trestle and berth structures is 14 months and 18 months respectively. Since neither of these structures is on the critical path, it is possible to construct them either simultaneously or sequentially. As a possible mitigation measure, constructing the two components sequentially may help minimize construction equipment requirements and reduce possible cumulative noise effects from multiple work sites.

For the LNG Jetty Berth Structures it is expected that there will be one or two marine derricks and one or two flat deck scows mobilized to conduct the work. For the Trestle it is expected that one marine derrick and one flat deck scow would be mobilized. Since these construction sites are in open water, there is space for additional crews and equipment to be mobilized as required to improve productivity. Although the construction vessels are not subject to tidal restrictions in the deep waters of the LNG Berth Structures and the outer Trestle, it is unlikely the EPC contractor will allow the equipment to stay in open water overnight. Thus the marine derricks and scows will likely be moved between the construction site and a safe anchorage or staging area daily. The anchorage or staging area could be located in Port Edward. However to mitigate possible marine traffic congestion in Porpoise Channel, the working vessels could transit to a location within the inner harbour of Prince Rupert.

Since the marine derricks and scows would be transferred to and from a staging area every day, they could be used to resupply the work front without the need for additional supply barges. Alternatively one or more supply barges could also be used for transporting materials from the mainland to the work front depending on construction requirements.

In general the construction of the Trestle and Berth Structures via marine-based methods should not be an impediment to marine traffic. The work faces will occur in relatively small areas in open waters away from or adjacent to approach channels. A safety zone would be enforced around the work site which may prohibit vessels from approaching closer than 50 m. Work site restrictions would be promulgated through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan.

## 3.3 Cantilever Construction Method

The Cantilever Construction Method, also known as the “cantitravel”, “over-the-top” or “cantilevered bridge” method, is a common method of constructing pile-and-deck trestles which has been used successfully around the world. This method uses a span by span approach for building the trestle using a mobile work platform that is supported above tidal waters. The pile guides and templates are “cantilevered” out from the main work platform to facilitate a work front that is located a full span away from the last finished bent. This method produces a minimum amount of disturbance to the existing ground or seabed, while still allowing work to proceed from the elevated construction platform which is supported on the same pile foundations used to support the final structure.

To accommodate the Suspension Bridge construction, the working platform would be required to progress from the LNG berths towards the SW Anchor Block to avoid interference between work fronts. In this situation the first few Trestle bent foundations at the LNG berths would need to be installed using marine-based methods. Once these initial bents were erected, the working platform and crane for the Cantilever Method would be assembled and erected on these bents. The working platform would then complete the rest of the trestle advancing towards the bridge’s SW Anchor Block.

When cantilever construction is used, the construction methodology must be taken into account during the design phase to make the construction process as efficient as possible. The structural design is typically optimized to suit the construction methodology in terms of the selected member sizes, spans, materials, connections, detailing, etc.

As mentioned above, the working platform, also known as a “cantitraveler” or “cantilevered bridge” is a primary piece of equipment required for cantilever construction. A photo of a typical cantitraveler platform is shown in Figure 3-2. A cantitraveler or cantilevered bridge is basically a customized mobile working platform which provides:

- A support platform for a crawler crane and associated equipment such as vibratory hammers, impact hammers, and drilling equipment;
- Integral guides and templates used to locate, drive and install foundation piles; and
- A platform which facilitates the installation of superstructure elements and the finishing of connections between foundations and superstructure elements.

For cantilevered construction, the exact sequence of construction operations will vary from project to project depending on various factors such as trestle layout and overall construction methodology. Regardless of the actual construction sequence however, a main benefit for this type of construction is the repeatability of the construction cycle for each pile foundation.

In general terms the construction cycle for any given pile foundation starts with the installation of the piles. The installation of the piles is followed by the erection and assembly of the transverse pilecap which essentially completes the pile bent foundation. Depending on the construction process, once a pile bent is completed, trestle deck elements may be erected or temporary bracing installed to provide longitudinal stability to the bent. Upon completion of the cycle, the platform is advanced forward one span to begin a new cycle and initiate construction on the next pile bent. Possible construction sequences and techniques for pile installation and drilling which may be used for the PNW LNG trestle are described in more detail in the following sections.

**Figure 3-2: Cantilever Construction with Cantitraveler Platform**

*Source: www.prumologistica.com.br/en/Pages/default.aspx*

### *3.3.1 Construction Equipment*

Selection and design of the required construction equipment is integral with both the trestle design and the chosen construction methodology. The design of the trestle itself will typically be developed with consideration of the intended construction equipment, sequencing and overall process. Likewise the design of the construction platform will also be developed to facilitate an efficient construction process. Both design of the trestle and the construction platform will be based on the optimization of various interrelated parameters including trestle span length, construction platform dimensions, crane size, weights of components, structural interferences, structural connections, etc.

The two main pieces of equipment for cantilevered construction is the construction platform and the crane. The construction platform can come in various shapes and sizes, but there are two basic types which are most commonly used consisting of:

- Cantitraveler platform; and
- Cantilevered bridge.

Both types of construction platforms and the crane requirements are described below.

#### *3.3.1.1 Cantitraveler Platform*

A cantitraveler platform as shown in Figure 3-2 is made up of a main carriage frame that supports a work deck for the crawler crane, equipment, and personnel. The main frame of the cantitraveler is mounted on steel wheel bogies which ride on rails attached to the top flanges of heavy rail girders.

The rail girders are separate components designed to bear on the trestle pilecaps and take the large construction loads from the cantitraveler and transfer them directly to the pile foundations. The rail girders are necessary to support the heavy construction loads, since it would be uneconomical to design the deck elements of the final trestle structure to support such loads. The rail girders are typically individual simple span girders which can be picked up by the cantitraveler crane and leapfrogged from the back span to the fore span allowing the cantitraveler to index forward as each pile bent foundation is completed. Because the main carriage of the cantitraveler is usually positioned over a single span it also allows the crane to install deck elements in the back span behind the cantitraveler or in some cases the fore span in front of the cantitraveler once the next pile bent is complete. A cable system comprised of winches, pulleys and sheaves that are restrained in the trestle superstructure is used to advance the cantitraveler.

The guides or templates used for positioning, setting and installing the piles are typically located at the ends of truss frames cantilevered out from the side of the main carriage frame. The pile guides or templates are customized to suit the trestle layout and can be adjusted or dismantled as required.

#### *3.3.1.2 Cantilevered Bridge*

Another type of platform which may be used in cantilevered construction is the cantilevered bridge as shown in Figure 3-3. A cantilevered bridge is comprised of a steel through-truss which extends over multiple spans of the trestle. The bridge has a platform on its upper chords for supporting a crawler crane and has a rail mounted gantry crane running between the main trusses. The functionality of the cantilevered bridge is similar to that of the cantitraveler in that the bridge supports a crawler crane for installing the foundation piles, has built-in piling templates for positioning the piles, and also facilitates the installation of trestle deck components. A cable system with winches, pulleys and sheaves is employed to advance the cantilevered bridge from span to span, similar to the cantitraveler system.

Unlike the cantitraveler system however, the cantilevered bridge does not use wheels and rail girders to facilitate movement. Instead, the cantilevered bridge bears directly on the foundation pilecaps and is typically slid forward on guides and bearing pads attached to the pilecaps.

Another primary difference between the two platform types is the method by which trestle deck elements are erected and installed. Since the cantilevered bridge covers multiple spans and the top platform and bracing prevents the crawling crane from gaining access to the trestle deck below, the bridge is equipped with an internal gantry crane that can be used to erect and install trestle deck elements in the spans covered by the bridge.

**Figure 3-3: Cantilever Construction with Cantilevered Bridge**

*Source: Coastal Engineer for Maritime Applications*

#### *3.3.1.3 Crane*

The required crane size is directly related to the trestle span and weight of the structural members required to be installed by the crane. As the spans increase, the crane reach increases and a larger crane is required to lift the same weight at the extended reach. Also, as the span increases, in general, so will the pile size and the size of hammers required for the installation. The crane size quickly limits out and sets the maximum span length.

The Trestle section of the LNG Jetty is located further from the shore well past the outer edge of Flora Bank. In this area the bedrock is much deeper than it is near Lelu Island and it is estimated that pile lengths may be as long as 80 m to 100 m or greater with corresponding weights of 60 to 75 tonnes or more. To lift a 60-tonne pile to accommodate a 36 m trestle span would require a 600-tonne capacity crane. This is a very large crawler crane size and may be impractical for the Cantilever Method. However, it is also likely not possible to pitch and drive such long piles since the required size of crane boom would be excessively high. Therefore it may be necessary to splice the piles in-situ, which will not only reduce the length of piles required to be handled and pitched but will also bring down the required lifting capacity for the crane.

Another method of keeping the crane size to a minimum is to reduce the size of the trestle spans and hence the required lifting radius of the crane, or alternatively use temporary spud piles to allow the crane to get close to the work front.

The maximum size of crawler crane that local marine contractors have in their fleets is 350 tonnes. Although the trestle designs currently call for 36 m spans, as discussed above, it is likely that the trestle span will need to be reduced or temporary spud piles used to accommodate the Cantilever Method. Past projects in BC have been limited to spans between 12 and 16 m using locally available equipment. Clearly, with the scope of this project, the crane equipment will be optimized for the installation method and we expect a span length of 25 to 30 m can be reached.

### *3.3.2 Construction Sequencing*

The basic construction cycle for building a trestle span, as shown in Figure 3-4, consists of the following general construction steps:

1. Installation of piles;
2. Installation of pile cap;
3. Installation of temporary longitudinal bracing;
4. Transfer of rail girders;
5. Movement of cantitraveler platform; and
6. Installation of longitudinal girders and deck elements.

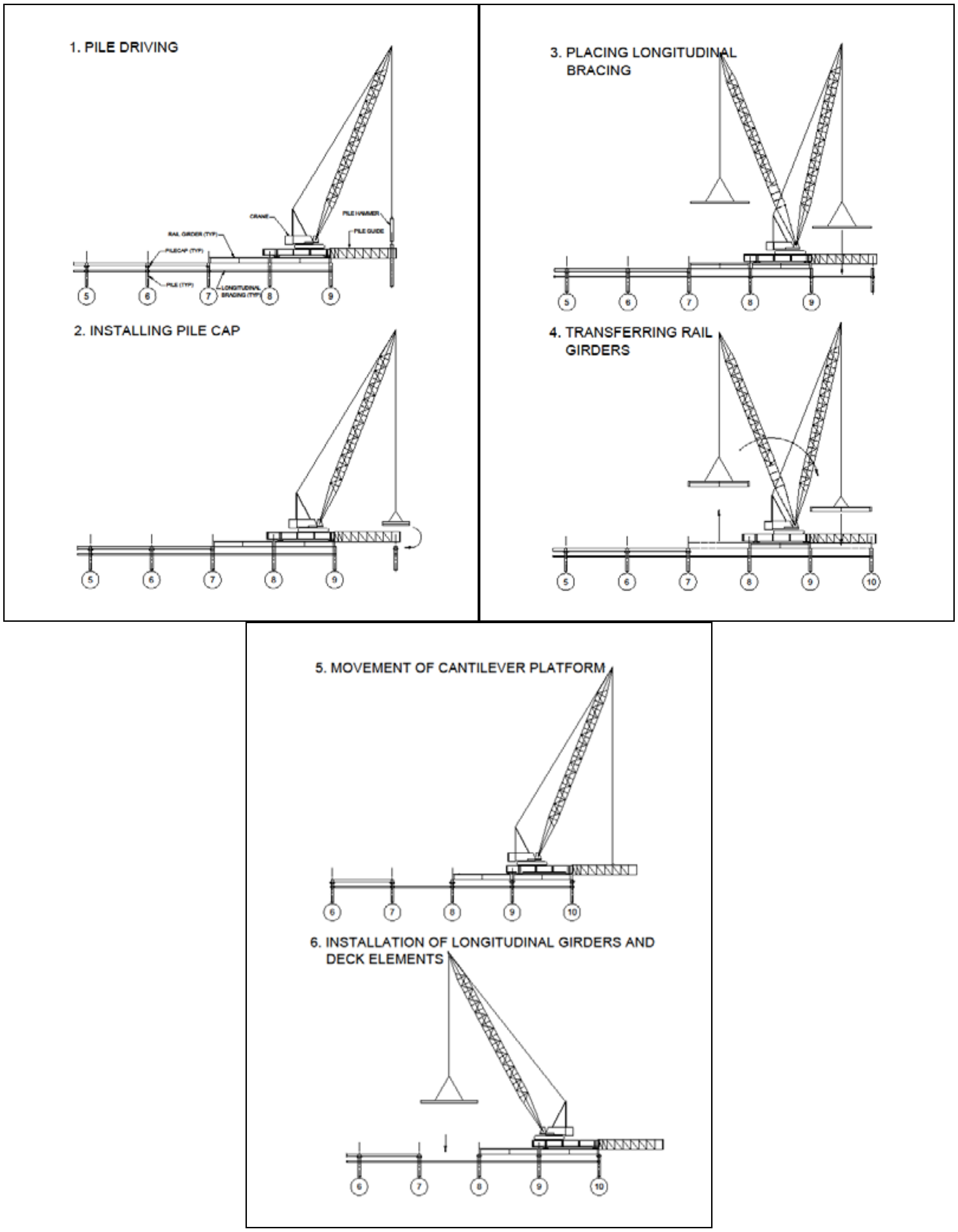


**Figure 3-4: Cantitraveler Construction Sequence**

Depending on the design and geotechnical conditions, the pipe piles may be either driven into the seabed overburden material, or driven through the overburden material and then drilled into the underlying bedrock to achieve the required bottom fixity. Various techniques for driving and drilling piles are described in more detail below in Sections 3.4 and 3.5.



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After all the piles for a particular bent have been installed and cut-off to the proper elevation, the pilecap is erected and attached to the pile heads thus completing the pile bent foundation. The connection between the pile heads and the pile cap varies depending on the type of materials used in the design:

1. For a steel pilecap, a basic field weld would be used to directly connect the pilecap to the steel pipe piles.
2. If a cast-in-place concrete pilecap is used, reinforced concrete plugs would be cast into the pile heads with the reinforcement extending up into the pilecap to provide the connection. Formwork would be set up above the water, supported on the pipe piles, reinforcement for the pile cap placed, and finally the pile cap would be poured. The formwork would be removed after several days of curing time.
3. If a precast concrete pile cap is used, concrete infill plugs would be cast in the pile heads similar to the cast-in-place construction. The infill plug reinforcement cages would extend up through holes in the precast units to provide the connection to the pilecap. The pilecap is complete when infill concrete is poured into the precast unit to monolithically join the elements together. The precast option does not require formwork like a regular cast-in-place pilecap would. But, regardless of which concrete option is used, those elements that have some amount of cast-in-place concrete will require at least 7 days curing before loads may be placed on the structure.

Upon completion of a pile bent, the crane on the cantitraveler will install bracing to stabilize the bent in the longitudinal direction. Once the new pile bent is stabilized, the rail girders are advanced to the next position and the cantitraveler is moved to the next span to initiate the next construction cycle.

Throughout the above process, there are several options for installing the trestle superstructure and deck elements. One option is to install the trestle girders or deck elements with the crane on the cantitraveler before it is moved into the next position. The trestle girders may either be installed in the fore span in front of the cantitraveler or the back span depending on the selected construction methodology. Another option is to allow the cantitraveler to be moved forward and immediately begin on the next bent foundation while a second smaller crane follows behind on the finished trestle deck and provides a second work front by installing the trestle superstructure and deck elements on the back spans behind the cantitraveler.

As shown in Figure 3-5 the basic construction sequence described above for a cantitraveler platform is essentially the same for a cantilevered bridge with some minor exceptions as follows:

1. Installation of piles;
2. Installation of pile cap;



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3. Pull back cantilevered bridge and install longitudinal trestle girders in the new span;
4. Place guides and bearing pads on new pile foundation;
5. Move cantilevered bridge forward onto new pile foundation and install deck elements in span behind cantilevered bridge; and
6. Index the bridge forward to begin new cycle.

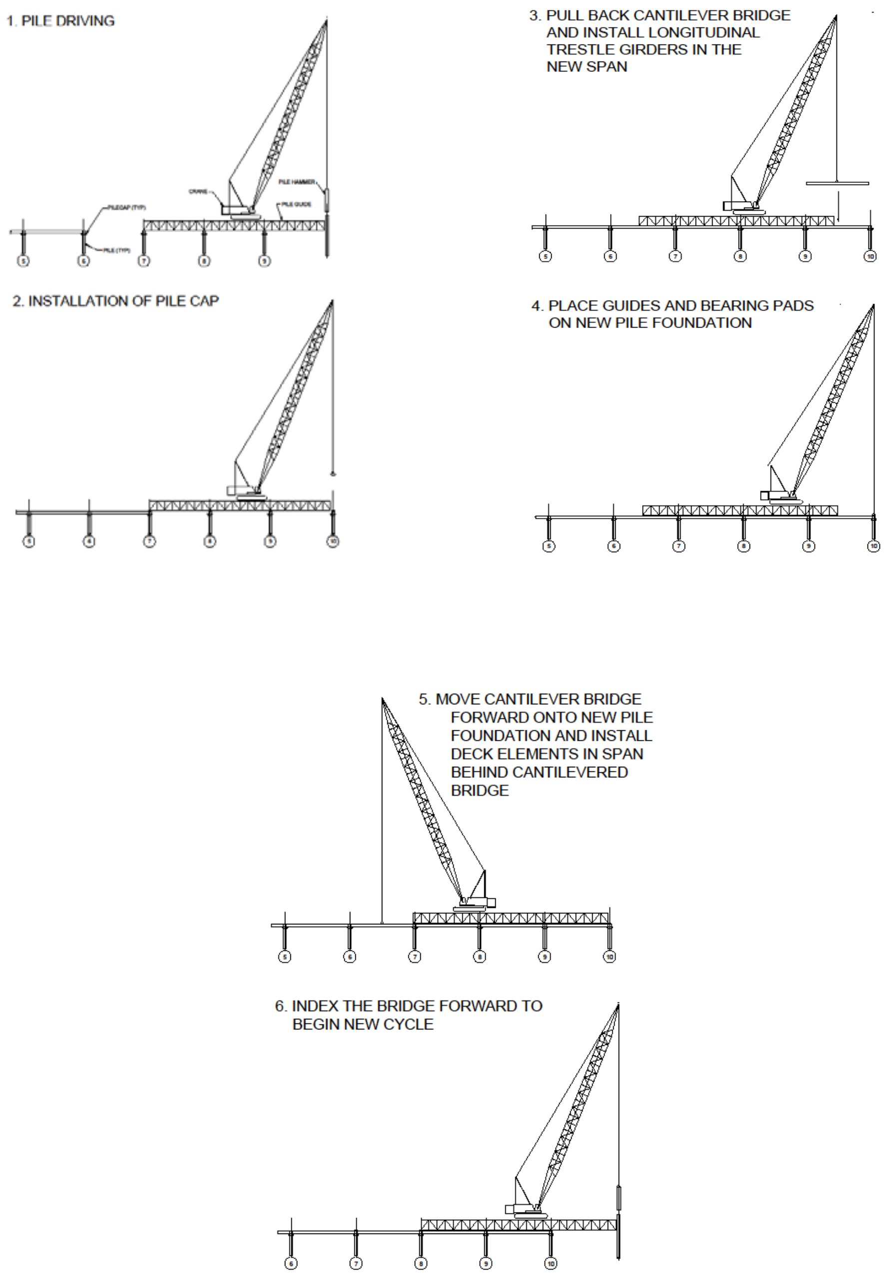


**Figure 3-5: Cantilevered Bridge Construction Sequence**



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Similar to the Marine-Based Method, placement of scour protection around the perimeter of each pile would be conducted soon after pile installation. The scour protection could be placed via marine-based methods using tidal windows and a flat deck barge equipped with an excavator or front end loader and a stockpile of riprap. This work could be conducted simultaneously as the cantilever platform advances along the trestle length. Alternatively the cantilever platform crane could be used to install the scour protection as each bent is completed prior to the erection of deck elements.

### *3.3.3 Cantilever Construction Schedule and Equipment Profile*

Mobilization and start-up includes assembly of the construction platform and crane over open waters. The initial pile bents will be installed using marine-based methods and then the cantilever work platform can be erected from barges using marine derricks so that the balance of the trestle can be completed with the Cantilever Construction Method.

The assembly of the construction platform and the initial start-up of the trestle construction may take a few weeks to a month depending on the geotechnical conditions and the number of initial trestle bents that need to be constructed.

Once the start-up process is completed, typical cantilever construction can achieve an average productivity rate of approximately 2 days for one complete span (including the foundation and superstructure). This productively rate however, is typical for projects where the spans are less than what is proposed for the PNW LNG trestle and where the piles are only driven into the overburden soils.

For the PNW LNG project, the spans may be up to 36 m and the local geotechnical conditions require the piles to be driven into the overburden soils and drilled into the underlying bedrock. Of the various construction activities that make up a typical production cycle, the pile drilling operations will be the most time consuming, especially if the piles are required to be socketed to significant embedment depths. It is estimated that pile drilling and socketing may take up to 3 to 4 days per pile, and since the piles must be drilled sequentially, a typical span with a 4 pile bent may require 14 to 18 days to complete. For pile bents with additional piles, add 3 to 4 days per pile to complete the given span. If the piles are not required to be socketed into the rock, such as for the spans in deeper water, the productivity rate will improve.

Once the trestle is complete, disassembly of the heavy construction crane and travelling platform will be conducted using floating marine derricks which will lift the crane and platform components onto flat deck barges for demobilization. The disassembly and demobilization process is estimated to take several weeks.

The overall estimated schedule duration for the construction of the trestle structures is 14 months. However since the Trestle is not a critical path item, there is enough schedule float to allow for longer Trestle construction times without delaying the overall project.

Although marine derricks and scows would not be required for the Cantilever Construction Method other than the initial mobilization, delivery of piles and other construction materials will be required using supply barges. In shallow waters the supply barge movements would be dependent on tidal windows while for those sections of the Trestle in deeper waters they will not. Although the supply barges are not subject to tidal restrictions in the deeper waters around the outer trestle, it is unlikely the EPC contractor will allow the equipment to stay in open water overnight. Thus regardless of the water depths, the supply barges will be moved between the construction site and a safe anchorage or staging area daily. The anchorage or staging area could be located in Port Edward. However to mitigate possible marine traffic congestion in Porpoise Channel, the supply barges could transit to a location within the inner harbour of Prince Rupert.

In general the construction of the trestle via Cantilever Construction methods should not be an impediment to marine traffic. The work face will occur in a relatively small area in open waters and only one round-trip transit per day would be expected from the supply barge. A safety zone would be enforced around the work site which may prohibit vessels from approaching closer than 50 m. Work site restrictions would be promulgated through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan.

## 3.4 Pile Driving Techniques

The pile installation techniques described below are applicable to either Cantilever or Marine Based Methods.

The steel pipe piles proposed for the trestle foundations will be supplied in approximately 12 m (40 ft) long sections and will require welding to be assembled and spliced into their final length. Because the seabed overburden material at the project site is predominantly comprised of weak marine clay, it is anticipated the steel pipe piles will sink some distance into the overburden material under their own weight, essentially “setting” themselves into the seabed. This can make in-situ field splicing of the pile sections difficult. As a result, the piles will likely need to be preassembled and spliced lengths that can be practically handled prior to being transported to site.

An efficient way of transporting long piles to site is on flat deck barges, which can be kept at the project site and used to stockpile the piles. When it comes time to install a pile, the barge can be brought into position during the tidal windows, and then the pile can be picked and pitched by a crane. For relatively long piles, the rigging may need to be adjusted in order the pick and pitch the pile without exceeding the bending strength of the pile. Once the pick is complete the barge can be moved away to a secure anchorage until it is required again. Stockpiling the piles on a flat deck barge is also advantageous in that it avoids any requirements for a pile lay-down area on Lelu Island.

Once a pile is positioned by a crane and set into the seabed, it needs to be driven through the overburden material until it reaches bedrock. A pile may be driven either closed-ended or open-ended depending on the design requirements and geotechnical conditions. However if the pile is required to be socketed into the bedrock, it will need to be driven open-ended, and cleaned out prior to rock drilling.

For the driving operation, either an impact or vibratory hammer may be used. Although impact hammers have the ability the drive piles through harder material, they are much noisier than vibratory hammers and do cause greater underwater pressure disturbances. Fortunately it is anticipated that a vibratory hammer will be adequate enough to advance the pile through the weak overburden material until the pile tip is seated close to bedrock.

For the last few meters above the bedrock however, an impact hammer will likely be required to advance the pile through any harder till layers until the pile tip actually reaches bedrock. Once the bedrock is reached, the impact hammer will drive the piles approximately 1 m into the bedrock to properly seat the pile and/or “seal” it for socketing. Although an impact hammer will be noisier than a vibratory hammer, it will only be required for a short duration to drive the pile the remaining depth and tap it into the bedrock.

Measures such as the use of bubble curtains may be employed to mitigate underwater overpressures from the pile driving operations and keep underwater acoustics within allowable limits. Typically the overpressure limit for pile driving is set at approximately 30 kPa, but may vary at the regulator’s discretion.

If the pile is designed to be only seated on the bedrock, then the pile installation is complete. However, if the pile is designed to be socketed into the bedrock, the overburden material left inside the pipe will need to be cleaned out prior to the drilling operation. A grab hammer will typically be used to clean out the pile, however if the material is weak and loose enough it may be possible to flush or airlift it out of the pile. The removed material is stockpiled on a barge for disposal. Filter cloth and other materials may be used on the barge to reduce turbidly caused by any remaining water runoff from the stockpile.

## 3.5 Pile Drilling Techniques

The pile drilling techniques described below are applicable to either Cantilever or Marine Based Methods.

Driving piles into rock is not feasible, unless the rock is very soft and weak. Therefore for most types of rock, drilling equipment and techniques must be employed to fix piles into the rock. Depending on the design requirements and the degree of moment capacity and fixity required at the bottom of the pile, piles may be installed into bedrock either as rock doweled piles or rock socketed piles.

For a rock doweled pile, the pile itself is inserted down the entire length of a hole drilled into the bedrock. The pile can be fully grouted in the hole or advanced with a friction fit, thereby providing fixity at the bottom of the pile as well as development of the pile's full moment capacity.

A rock socket is similar to a rock dowel, but in lieu of inserting the steel pile all the way into the drilled hole, the steel pipe pile is only embedded a certain distance into the rock and the balance of the hole is filled with a cast-in-place reinforced concrete core. The concrete core also extends a certain distance up into the hollow of the pile above the rock to facilitate load transfer. The reinforced concrete core acts like an extension of the steel pile into the rock and thereby provides anchorage and a degree of fixity at the bottom of the pile. The reinforced concrete core however, will not provide the full moment capacity of the steel pipe pile therefore a rock socketed pile is not as strong as a rock doweled pile.

For installing a pile into rock, initially the pipe pile must be driven a certain distance into the bedrock in order to "seal" the hole and prevent overburden material from sloughing in as the hole is advanced past the tip of the pipe pile. Various drill bits and techniques may be employed depending on the type of rock being drilled and whether it is a rock socketed pile or rock doweled foundation. Some typical drill bits which may be employed include down-the-hole hammers, churn drills, core barrels, rock augers, tri cone bits and other rotary bits. The contractor will use the most effective bit given the local rock conditions. Drill bits are typically attached to a rotating kellybar operated from a diesel-powered drill unit mounted on a frame or the leads of a crane.

The installation of a rock socketed pile, involves inserting the drill bit into the pile from the top and lowering it to the bottom. The drill unit rotates the bit, grinding the rock away at the bottom of the hole and advancing it with a diameter slightly smaller than the inside diameter of the pile. The hole is drilled to the desired embedment depth while the drill tailings are simultaneously airlifted to the surface through the kellybar. Once the hole is complete and cleaned, a reinforcement cage and tremie concrete can be placed, completing the socket.

For a rock doweled pile, the hole must be drilled with a diameter large enough to allow the pile itself to be inserted. However, since the drill bit is generally smaller than the pile's inside diameter (so that it may be inserted into the pile) special techniques must be used to drill an oversized hole.

One method is to use a down-the-hole hammer with undercutting capability. Although the overall hammer bit is small enough to fit inside the pile, once the tip of the bit is advanced beyond the pile end, under reaming cutter wings extend out from the bit, drilling a hole slightly larger in diameter than the pile.

Another method involves the attachment of a sacrificial cutting ring to the pile itself. The drill bit is used to drill a slightly undersized hole similar to a rock socket. The cutting ring on the end of pile is advanced by rotating the pile itself, typically in the opposite direction of the drill bit, and reaming out the hole thereby enlarging it and allowing the pile to be embedded. For this method the drill unit must have the capability to rotate the pile.

A third method for installing a rock dowel involves the use of a large diameter pipe casing. Using the same techniques as for piles, the oversized casing is driven down, sealed into the rock and cleaned out. Using standard rock socket drill techniques, a hole is drilled in the rock that is smaller than the casing but larger than the pile. The pile is inserted into the casing from above and grouted into the hole. The casing is then removed and positioned for the next pile. This technique has some disadvantages compared to the previously mentioned methods in that it requires multiple handling of large long pipes, including pile insertion and casing removal. This may require a crane with a very large boom. Another disadvantage is the greater volume of material required to be cleaned out of the casing as compared to other methods where just the pile is installed. Also, when the casing is removed there may be additional disturbance and soil sloughing around the pile. This is avoided with the other methods.

For all of the above drilling techniques, the tailings from the drill process are typically extracted by airlift systems. The removed material can be run through a cyclone to extract water before it is stockpiled on a barge for disposal. Filter cloth and other materials may be used on the barge to reduce turbidly caused by any remaining water runoff from the stockpile.

## 3.6 Comparison of Cantilever and Marine-Based Construction Methods

Construction using the Cantilever Method has several advantages over marine-based methods including:

- The avoidance of the use of floating equipment, if the construction front can be supplied entirely via the trestle. If not, then only minimal use of floating equipment will be required to supply the construction front with larger items such as piles.
- Easy workface access for crews and supplies from behind the cantilever platform via the already finished portions of the trestle structures.
- The cantilever platform is essentially a fixed structure which is less sensitive to environmental conditions and can provide a substantial reduction in delays due to weather conditions, tidal variations, wind, currents, and waves.
- The fixed nature of the cantilever platform allows for higher quality work and accurate placement of piles and other structural elements.
- Small environmental footprint. Area of disturbance limited to footprint of piles.
- Dredging is not required as part of the construction process for foundation installation.

Disadvantages for the Cantilever Method include:

- To accommodate Suspension Bridge construction, the cantitraveler platform and crane will need to be erected in open waters in order to construct the Trestle. This operation will be highly dependent on sea state and weather conditions.

- Increased construction risk due to the single work face. If a problem occurs, it must be solved before the work can advance.
- Increased schedule risk due to the single workface. If construction falls behind schedule there is little opportunity to increase productivity.
- For a given lift weight, larger crane sizes are generally required for the Cantilever Method due to the larger lift radius. To reduce the lifting radius and keep the crane to a reasonable size, the trestle span lengths may need to be reduced or temporary pile supports used to keep minimize the lift radius while maintaining the Trestle design span length.

Construction using the Marine-Based Method has the following advantages over the Cantilever Method:

- Reduced construction risk since multiple work faces may be progressed simultaneously. Several crews and derricks may advance multiple work faces simultaneously. If there is a problem at one work face, progress can still be made at other work faces while the problem is resolved;
- Reduced schedule risk due to the ability to add work faces. If construction falls behind schedule there is opportunity to add additional crews and marine derricks at other work faces to increase overall productivity; and
- The design trestle span length of 36 m can be constructed and does not need to be reduced.
- Dredging is not required as part of the construction process for foundation installation.

Disadvantages for the Marine-Based Method include:

- Increased schedule risk, since Marine-Based methods are subject to tidal windows if the water depth is limited;
- Increased schedule risk, since Marine-Based methods are more vulnerable to delays due to weather and sea state; and
- Larger environmental footprint due to greater potential in shallow water for barges grounding and disturbing the seabed habitat.

# 4. Lelu Island Access Bridge Construction Methodologies

## 4.1 Construction Methodology

The Access Bridge to Lelu Island crosses over the Lelu Island Slough and is comprised of pile and deck construction, as based on current FEED designs. The piled foundation bents include a row of steel pipe piles with a pilecap and are spaced up to 45 m apart. Large bridge girders span between the bents and support the deck of the bridge. The slough is very shallow and comprises soft mud substrates.

Due to the shallow water within Lelu Island Slough, the most viable construction method will be the Cantilever Construction Method. As described previously this method will involve the use of a construction platform that is supported on the same pile foundations that will support the completed bridge structure. This method will have the least impact on the slough, since no construction equipment will touch the inter-tidal habitat. All construction activity will take place from above, supported from the pile foundations. The disturbance footprint will be limited to only the piles that are required to support construction and the bridge structures.

The maximum span that can be readily accommodated with the Cantilevered Construction Method and locally available cranes is typically in the range of 20 to 25 m. Since the spans between the permanent bridge foundations are almost double, the need for temporary intermediate pile bents between the permanent foundations will likely be required to facilitate construction. Either one or two intermediate temporary pile bents may be needed between each permanent bent depending on the size and allowable lifting radius of the selected crane. The size of crane and the construction span lengths can be optimized based on the lifting requirements of the piles and construction equipment. Regardless of the final construction spans, the installation of temporary piles will only increase the disturbance footprint by a minimal amount. Although it is plausible the temporary piles will remain in-situ for the duration of the bridge construction, it is more likely the temporary piles will be extracted as soon as the cantitraveler platform has advanced to the next span. Therefore the temporary piles may only be in place for one to two weeks before being extracted.

One advantage of using the Cantilevered Construction Method is the possible synergies with construction of the LNG Jetty Trestle. If the design and construction of the Access Bridge and LNG Jetty Trestle are coordinated, it may be possible to use the same working platform for both structures and achieve construction cost savings.

Because of the difficult marine access to this site, the work face will be supplied exclusively from land. A staging area will likely be required along Skeena Drive near the site for the stockpiling of piles and other construction materials. It is anticipated the cantilever work platform would be launched from the mainland and work towards Lelu Island. The bridge superstructure and deck would be completed as the work platform advances and would be used by trucks to supply the work face with equipment and materials. The bridge abutments on either side of the slough will be constructed using standard road building equipment and should not affect the marine habitat.

Other construction methods such as Marine Based methods are not considered practical for constructing the Access Bridge. Since the water is so shallow, it will be difficult to get large floating barges with relatively deep draughts into the slough, even at high tides. The probability of the floating construction equipment grounding in the slough is very high. Even if the equipment could get close enough to the work face at high tides, tidal restrictions would be prohibitive in terms of an allowable work window.

The shallow waters also preclude the use of jack-up barges which would initially require enough water depth to be able to approach the work site before they could spud down and elevate the barge. Even if a jack-up barge could enter the slough at high tides, anecdotal evidence suggests there would be a problem with the spuds sinking into the mud bottom of the slough. The spuds of the jack-up barge would also likely cause much more habitat disturbance than the temporary piles of the Cantilevered Construction Method.

## 4.2 Construction Schedule and Equipment Profile

The overall estimated construction schedule for the Lelu Island Access Bridge is 14 months. The use of the Cantilevered Construction Method will be less disruptive to navigation than other methods as it precludes the use of floating equipment which may block the slough. As a result there may be opportunity to keep the slough open to navigation as much as possible and allow small boats and recreational vessels to transit the slough’s central channel during high tides. Since the bridge will be constructed span by span across the channel, during construction activities which occur on either side of the central channel, navigation may be allowed to continue without interfering with construction activities. As long as a certain safety distance is maintained from the work front (approximately 50 m) vessels could freely navigate without restriction.

It is likely however, that at certain points during construction, navigation through the slough will be prohibited for short durations. For example, navigation through the slough may be prohibited for two to three weeks while the bridge span over the central channel is being constructed. During navigation closures, marine traffic will be required to navigate around Lelu Island.

# 5. Pioneer Dock Construction Methodologies

## 5.1 Construction Methodology

Although the overall configuration of the Pioneer Dock varies amongst the different FEED designs, the construction activities required to install these facilities are essentially the same regardless of which design is selected. The Pioneer dock may be comprised of various berth configurations suited for different purposes depending on the contractor requirements. The Pioneer Dock may include:

- An aggregate loading facility; and/or
- A Roll-on/Roll-off (Ro-Ro) facility.

The aggregate loading facility simply consists of a set of multiple pile-supported berthing/mooring dolphins serving as an aggregate barge berth and a landing area on shore for receiving the barge ramp. The aggregate barge will be berthed with tug assistance against the dolphins and then indexed forward until the vessel's ramp can be lowered onto the ramp landing area to allow rock trucks to access the barge.

The Pioneer Dock may also include a Ro-Ro facility comprised of multiple pile-supported berthing/mooring dolphins, and a floating dock. The floating dock may consist of a concrete pontoon secured by guide piles which keep the pontoon in position as it floats with the tides. The ferry or Ro-Ro vessel is berthed against the dolphins with tug assistance and then indexed forward until the vessel's ramp engages the main floating pontoon. An intermediate pontoon provides an articulation in the vehicle ramps between the main pontoon and the onshore abutment which allows Ro-Ro vessels to be loaded and unloaded under a large tidal range.

A variation for the Ro-Ro facility uses a spud barge as the floating dock in lieu of a concrete pontoon. Similar to the pontoon's guide piles, the spuds keep the barge in position and allow it to float freely with the tides. A pivoting vehicle ramp spans from a shore abutment to the spud barge deck providing vehicle access to and from the vessels. This variation of the design is advantageous in that it does not require separate berthing/mooring dolphins; since the incoming barges and ferries simply berth and moor against the floating spud barge.

Despite the differences amongst the various FEED designs, for each facility the construction activities can be divided into a marine scope and an upland scope. The marine scope activities potentially include the installation of piles, installation of floating pontoons and marine lifting of vehicle ramps. The upland scope activities include fill work and onshore ramp abutment installation.

The predominant marine construction activity for the Pioneer Dock will be the installation of the piles for the berthing dolphins and for the floating pontoons. Standard marine-based methods and equipment as described previously would be used to install the piles.

The floating pontoons for the Ro-Ro facility would be manufactured offsite in dry-dock and floated to site just prior to installation. Alternatively the pontoons could be manufactured on a barge and then brought to site and skidded off the barge just prior to installation. A small work tug or standard marine derrick would be used to either float or lift the pontoons into their final position. Likewise the vehicle ramps would be brought to site on a barge and lifted into position with a marine derrick.

The concrete abutments for the vehicle ramps would be installed with land-based equipment and should not affect the marine habitat. For the aggregate loading facility, standard road building equipment would be used to build the barge ramp landing area on the foreshore. Excavators would be used to dress the perimeter of the ramp landing area with riprap to prevent scour and erosion.

## 5.2 Construction Schedule and Equipment Profile

The overall estimated construction schedule for the Pioneer Dock is 8 months. The marine equipment spread expected for the Pioneer Dock installation would be a marine crane derrick for conducting pile installation and marine lifts, as well as a flat deck scow for stockpiling piles, supplies and equipment. A small work tug would also be used every other shift to relocate the barges from one work face location to another. Because of the shallow waters in which the work takes place, the marine derrick may likely be a spud barge which will lower its spuds to stabilize the crane platform and keep it in position. Based on local marine contactor fleets the maximum size of the construction barges will probably be approximately 60 m x 20 m which should allow the barges to be kept within in the cove and not stick our or block traffic in Porpoise Channel. A safety zone would be enforced around the work site which may prohibit vessels from approaching closer than 50 m. Work site restrictions would be promulgated through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan.

# 6. MOF Construction Methodologies

There are various design configurations for the MOF and as such the construction methodology will be tailored to suit the type of marine structure selected by the EPC contractor. Unlike the other marine facilities however, the MOF also has a requirement to dredge rock and sediment within the MOF cove and its approaches to provide a water depth that can accommodate the vessels expected to offload at the facility. Both the dredging and construction methodologies for the MOF are discussed in the following sections.

## 6.1 Dredging Methodology

Dredging between Lelu Island and Ridley Island (Porpoise Channel) is planned to provide adequate depth for the MOF berths. Approximately 790,000 $m^3$ of material will need to be removed for the MOF of which approximately 590,000 $m^3$ is expected to be rock (phyllite/schist). Further description of the MOF dredging methodology is detailed in Appendix A.

The methodology used in this study was based mainly on the geotechnical information provided by PNW LNG, tidal variations and Hatch’s experience in rock drilling/blasting (D&B) and dredging operations. This is a preliminary assessment and final methodology should be selected by the contractor. Dredgeate and rock material not used for onshore purposes will be disposed at sea in Brown Passage as indicated in the EA submission.

Due to the important tidal variation in the area of study, the dredging area will be divided into two regions with distinct D&B and dredging methodologies:

- Area below MSL to be performed with marine-based equipment;
- Area above MSL to be performed by land-based equipment.

Most of the dredging area is located outside a natural navigation channel and is not expected to disturb navigation activities in the area.

The site conditions, geotechnical information, dredging volume, disposal options and productivity will determine the most suitable dredging equipment. Alternative equipment options are also detailed in Appendix A.

### *6.1.1 Marine Sediment Dredging*

An estimated 200,000 $m^3$ of marine sediment will have to be dredged prior to the commencement of the drilling blasting activities. All sediment will be disposed at sea in Brown’s Passage.

The most common equipment for dredging marine sediments is a Trailing Suction Hopper Dredger (TSHD). The TSHD is a self propelled vessel that accumulates the dredged material into its hopper and disposes this material through doors located at the bottom of the vessel’s hull. TSHDs come in a wide range of hopper volumes. As the hopper volumes increase so too does the vessel’s draft. Therefore it is suggested that a small TSHD with a capacity up to

5,000 $m^3$ be used so that a wider dredging area can be accessed. No auxiliary equipment is necessary for a TSHD operation other than a crew vessel.

Since 90% of the sediment volume is located below MSL it is expected that the TSHD will be able to tackle most of the marine sediment dredging. A 5,000 $m^3$ TSHD's monthly dredge production rate is estimated to be approximately 150,000 $m^3$. This means that approximately 2 months would be required to dredge 90% of the marine sediment.

### 6.1.2 *Drilling and Blasting*

Drilling and blasting is used to fragment the rock layers with explosives. Drilled holes are filled with explosives and laid out in a grid pattern. A common pattern for blasting operations consists of drilling holes of about 130 mm in diameter in a 3 m x 3 m square grid. The blasting material most widely used in environmental sensitive areas, as a substitute to dynamite, is water-based gel explosives.

#### 6.1.2.1 *Marine-Based Equipment*

The most common equipment for drilling in offshore areas is a jack-up barge (also known as a self elevating platform) equipped with multiple drilling towers.

The jack-up barge consists of a floating pontoon with movable spuds, capable of raising its hull above water surface. An example of jack-up barge is shown in Figure 6-1. Its approximate dimensions are 40 m long x 30 m wide x 5 m moulded depth.

The main advantage of using a jack-up barge is its ability to withstand heavy seas, currents and the tidal range to maintain stability for drilling activities and positioning. The main disadvantage is the jack-up barge is not self propelled and is dependent on tugs for transportation and positioning.

The jack-up barge can be positioned in most of the drilling areas, up to an elevation of +5.40 m CD. This is the upper access limit-based on an estimated jack-up barge draft, when floating, of about 2 m and barge deployment occurring when tides reach their highest levels. Even though the time span for highest tides is short in a semi diurnal tide region, the jack-up barge could be positioned fairly quickly with the help of tugs and anchors positioned on shore.

**Figure 6-1: Typical Jack-up Barge**

#### *6.1.2.2 Land Based Equipment*

The upland area will be drilled with mobile drilling rigs and will need road access and level terrain for stability. Drilling rigs can be deployed on wheels or on tracks as shown in Figure 6-2. Therefore, it will be necessary to fill some upland areas to create level access to the rigs. Shallow areas between +5.40 m CD (draft limited depth) and HHWL (+7.40 m CD) will also have to be filled in order to allow access for land-based drilling equipment. No auxiliary equipment is required for the drilling rigs. Fill and cut to provide level access of the land-based drill rigs will not be assessed in the study.

**Figure 6-2: Typical Drilling Rig on Tracks**

### *6.1.3 Rock Dredging*

#### *6.1.3.1 Marine Based Equipment*

The most suiTable equipment for recovering blasted rock is the marine backhoe dredger (BHD), as shown in Figure 6-3. BHD is a stationary water-based excavator mounted on a dedicated dredging pontoon that has a rotating table. The position and stability is maintained by three spud poles, where two spuds are fixed to the front side of the pontoon near the excavator crane and one tilting spud at the aft side. The equipment can move with the tilting spud and direction can be adjusted with the two-part articulated arm. In the dredging position the front spuds are firmly planted in the seabed. The dredger then raises itself partially out of the water to further anchor the spuds. With the pontoon slightly lifted out of the water, part of the weight of the dredger is now transferred via the spuds to the seabed, resulting in an increase in anchoring.

Because of the hydraulic action of its stick and bucket cylinders, BHDs exert positive forces pushing the bucket into the material to be excavated.



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**Figure 6-3: Typical Backhoe Dredger**

BHDs are becoming the dredging equipment of choice as dredging operations and projects have expanded. This equipment can dig at greater depths and have greater installed power, therefore can be utilized cost-effectively in heterogeneous soils containing a mixture of clay, sand, cobbles and boulders. Because BHDs can generate reasonable cutting force, they are also suiTable for digging heavy clays, soft stones, blasted rock and soil containing fractured rocks or rubble.

BHDs are equipped with accurate positioning systems and can operate with precise underwater profiles. The bucket position is followed by the operator on a computer screen in real time and the seabed elevation is updated automatically in the dredging software once the bucket removes material from the seabed.

The suggested minimum requirement for the BHD is specified as 3,000 kW power capacity, 10 $m^3$ bucket volume and approximate pontoon dimensions being 60 m long x 18 m wide x 4 m moulded depth, which falls into the class of mega BHD.

The main advantage of using a BHD is its ability to dig into hard soils containing boulders or debris due to its hydraulic power and can reach water depths up to 20 m. The main disadvantage of BHD is that it mainly relies on the pontoon's buoyancy to support its weight as the pontoon spuds do not have the capacity to carry it. The BHD has to then work with enough under keel clearance in order to allow safe water draft for dredging operations. The semi diurnal tidal variation at the MOF site will restrict the area of influence for this equipment and force it to move a great deal to cover the intertidal area. Another disadvantage of BHDs is downtime due to waves and cross-currents. Since the MOF dredging area is protected it is not expected that climate conditions will cause equipment standby.

Dredging is expected to start only after blasting is complete and surveyed. The BHD bucket dumps the rock material into a self propelled split barge moored alongside the dredger. The suggested minimum requirement for the split barge volume capacity is 700 $m^3$ with bow thrusters.

Auxiliary equipment involved in marine dredging includes:

- 1 multicat barge (25 m long x 12.7 m wide x 2.3 m moulded depth) with 40 t crane capacity;
- Self-propelled split barge;
- 1 survey and crew launch boat; and
- 1 operational launch boat.

It is suggested that rock disposal site be positioned on intertidal area with access to land-based equipment once water level drops. According to client communication the dredged rock will be used for civil works. Therefore, the split barge would have to dump the rock disposal material as close as possible to shore at high tide.

Dredging is expected to start only after blasting is complete and surveyed. Due to safety reasons no vessels are allowed to operate near the blasting area.

Production rate for the marine-based BHD is estimated to be 37,500 $m^3$/month considering an operation rate of 350 hours/month and a production rate of 85.8 $m^3$/hour with a 10 $m^3$ bucket. Higher productivities may be achieved with a larger bucket size.

#### 6.1.3.2 *Land Based Equipment*

The most suitable equipment to dredge the upland areas as well as the intertidal areas that are inaccessible by a marine-based BHD, is an elevated land-based backhoe excavator mounted on an under-carriage with spread tracks as shown in Figure 6-4.

This versatile equipment can operate at water depths up to 5 m and dig into hard soils containing boulders or debris due to its hydraulic power. The main objective of the land-based backhoe is to move the blasted and disposed rock towards the upland and make it available for civil works and earth moving equipment (not part of the scope of this study).

**Figure 6-4: Typical Elevated Backhoe**

The elevated backhoe will gradually drag and move the blast rock material upland. The estimated production rate for land-based backhoe is 60,000 $m^3$/month considering an operation rate of 350 hours/month and production rate of 200 $m^3$/hour with a 10 $m^3$ bucket.

Since the total volume to be dredged by land-based equipment is approximately 118.000 $m^3$, the estimated schedule for one land-based backhoe is about 2 months.

Land-based dredging can be performed simultaneously with the marine-based dredging.

#### *6.1.4 Phasing and Schedule*

Marine sediment dredging equipment will have to be mobilized prior to drilling and blasting activities and rock dredging. It is recommended that the faster dredging method be selected in order to avoid delays in the schedule.

Drilling and blasting activities will be performed from the same equipment, either a jack-up barge with drilling equipment, or alternative equipment such as a specialized drill vessel. Upland drilling and blasting activities can proceed simultaneously with marine drilling and blasting activities. Equipment production will have to be assessed in more detail in order to determine the phasing of the works for the MOF Dredging.

To speed up production, dredging with a BHD can be supplemented with an elevated land-based excavator which has the capability to reach the seabed as deep as Elev. -8.0 m CD. The land-based excavator would dredge the foreshore and shallow waters, while the BHD would dredge the deeper waters down to Elev. -12 m CD.

As per client information, the in-water blasting window for Porpoise Channel is a 2.5-month period from November 30 to February 15, subject to site-specific changes based on species use and discussions with DFO. Blasting should be performed only during low tide. However, the proposed schedule for MOF blasting and dredging activities is 7 months. In order to achieve the blasting schedule, drilling operations should be mobilized well in advance of the actual blasting window to prepare the blasting boreholes. Also, onshore blasting is not subject to the same restrictions as in-water blasting and can be conducted outside the 2.5-month period.

Table 6-1 summarizes the proposed equipment presented above.

**Table 6-1: Summary Equipment Table - MOF Dredging Methodology**

| Sediment Dredging | | Rock Drilling/Blasting | | Rock Dredging | |
|---|---|---|---|---|---|
| Marine | Land | Marine | Land | Marine | Land |
| TSDH | Elevated BHD | Jack-up drill and blast barge | Drill rigs on tracks for blast | Marine Backhoe Dredger (BHD) | Elevated BHD |

## 6.2 Construction Methodology

The exact construction methodology for the MOF will be dependent on the type of marine structure selected by the EPC contractor. Based on the currently proposed FEED designs, the MOF is comprised of two wharf structures which, depending on the design concept, may be constructed using Concrete Caissons, Steel Sheet Pile Bulkheads, or Pile-and-Deck type structures. Despite the wide variety of possible structural configurations, the required marine construction equipment will generally be similar to the other marine facilities.

Similar to the Pioneer Dock, the construction activities for the MOF can be divided into a marine scope and an upland scope. Both the marine and upland scope activities will vary depending on the selected MOF configuration.

If a Pile and Deck wharf structure is selected for the MOF, the construction methodology would be the same as for the other facilities requiring marine piles. Standard marine-based methods and equipment as described previously would be used to install the piles. Pile vibratory and drilling techniques, as described for the LNG Jetty Trestle, would be used to mitigate noise concerns during pile installation. Once the piles have been installed, the deck superstructure can be installed from the land, or from the waterside, or both, depending on the framing layout and the materials used. Typically a retaining wall is required to prevent scour or undermining of the landside abutment of the wharf. The landside abutment or interface would be constructed using land-based equipment and should not affect the marine habitat. If a revetment slope under the wharf structure is used for scour protection in lieu of a retaining wall, the riprap protection would be placed after pile installation and before the deck was constructed.

For Steel Sheet Pile Bulkhead structures, a marine derrick and flat deck scow would be required similar to the Pile and Deck design. The marine derrick would use a vibratory hammer to install the sheet piles into the overburden sediments, thereby limiting noise levels. The flat deck scow would be used to stockpile the sheet piles and equipment. The marine equipment is used primarily to install the steel sheet piles and once the bulkhead is complete, the balance of the work can be conducted by land-based equipment. When the bulkhead wall is near completion, the space between the bulkhead wall and shoreline would be filled and compacted in a series of lifts using land-based equipment. Since the activities occur behind the wall, there is less probability of generating turbidly in the water column. Anchor systems for the bulkhead wall, including walers, connections, tie-rods and concrete deadman anchors, would be installed in between lifts at the appropriate elevations. The tops of the steel sheet piles would be cut-off level and to the proper elevation. Once the backfill behind the bulkhead wall was complete a concrete cope beam and decking would be formed and installed from the land side. Finally, the marine derrick would use a clamshell grab to place scour protection in front of the bulkhead in the berth pocket to prevent undermining of the bulkhead wall.

For the Concrete Caisson concept, again the basic marine construction equipment deployed would be a marine derrick and flat deck scow. In addition, a hopper barge may be used for the initial placement of the engineered fill for the caisson mattress pad. The installation of concrete caissons will require more dredging and rock blasting than the other concepts since enough space must be provided for the caissons and mattress pad upon which the caissons sit. After initial placement of the engineered fill, the marine derrick will use a special spreader bar to distribute and screed the engineered fill to produce a mattress pad with a flat level bearing surface. The concrete caissons would be fabricated in dry-dock and towed to site just prior to installation. Tugs and the marine derrick, which would likely be a spud barge, would be used for initial placement of the caissons and for ballasting the caissons to seat them on the mattresses. Once the infill panels are installed between the caissons and the caissons are fully ballasted into position, the balance of the backfilling behind the caissons and completion of the deck and cope beam on top of the caissons can be completed with land-based equipment. Finally, the marine derrick will use a clamshell grab to place scour protection in front of the caissons in the berth pocket to prevent undermining of the mattress pad.

## 6.3 Construction Schedule and Equipment Profile

The overall estimated construction schedule for the MOF construction is 12 months. Regardless of the configuration of the MOF wharf structures, the basic marine construction equipment deployed would be a spud barge type marine derrick and a flat deck scow. The activities performed by this equipment will be slightly different depending on the structure type being built. Work tugs would also be deployed to move the marine equipment from one work face to another. For the concrete caisson design a hopper barge may also be deployed for a brief period for placing engineered fill for the mattress pad.

The contractor may elect to mobilize two crews to work on two work faces simultaneously. This essentially means that two sets of marine derricks and scows would be required. These construction barges would be located in the cove and would be staged so as to not block Porpoise Channel. Therefore the construction equipment should not be an impediment to marine traffic in Porpoise Channel. A safety zone would be enforced around the work site which may prohibit vessels from approaching closer than 50 m. Or the safety zone may be set up around the entrance to the MOF cove to correspond with a silt curtain deployment. In either case, work site restrictions would be promulgated through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan.

# 7. Conclusions

## 7.1 Progression of Marine Facilities Design

Although the marine facilities design has been advanced to a level sufficient for the purposes of planning and permitting, the design will be subject to change prior to the finalization of the design details. The current design details which have been submitted as part of the EA are only preliminary, and considerable engineering effort is still required to complete the designs to a level that is ready for construction. The designs will be received as they currently stand by the EPC contractor and will be modified to suit that contractor's particular construction methodology as it fits into the framework of the EA and the project's overall Construction Management Plan. Factors which will influence the evolution of the design include:

- Changes resulting from further geotechnical and geophysical investigations;
- Changes resulting from further site and metocean investigations;
- Changes to suit EPC contractor's equipment, construction preferences, and schedules; and
- Changes to suit regulated environmental restrictions and mandated mitigations.

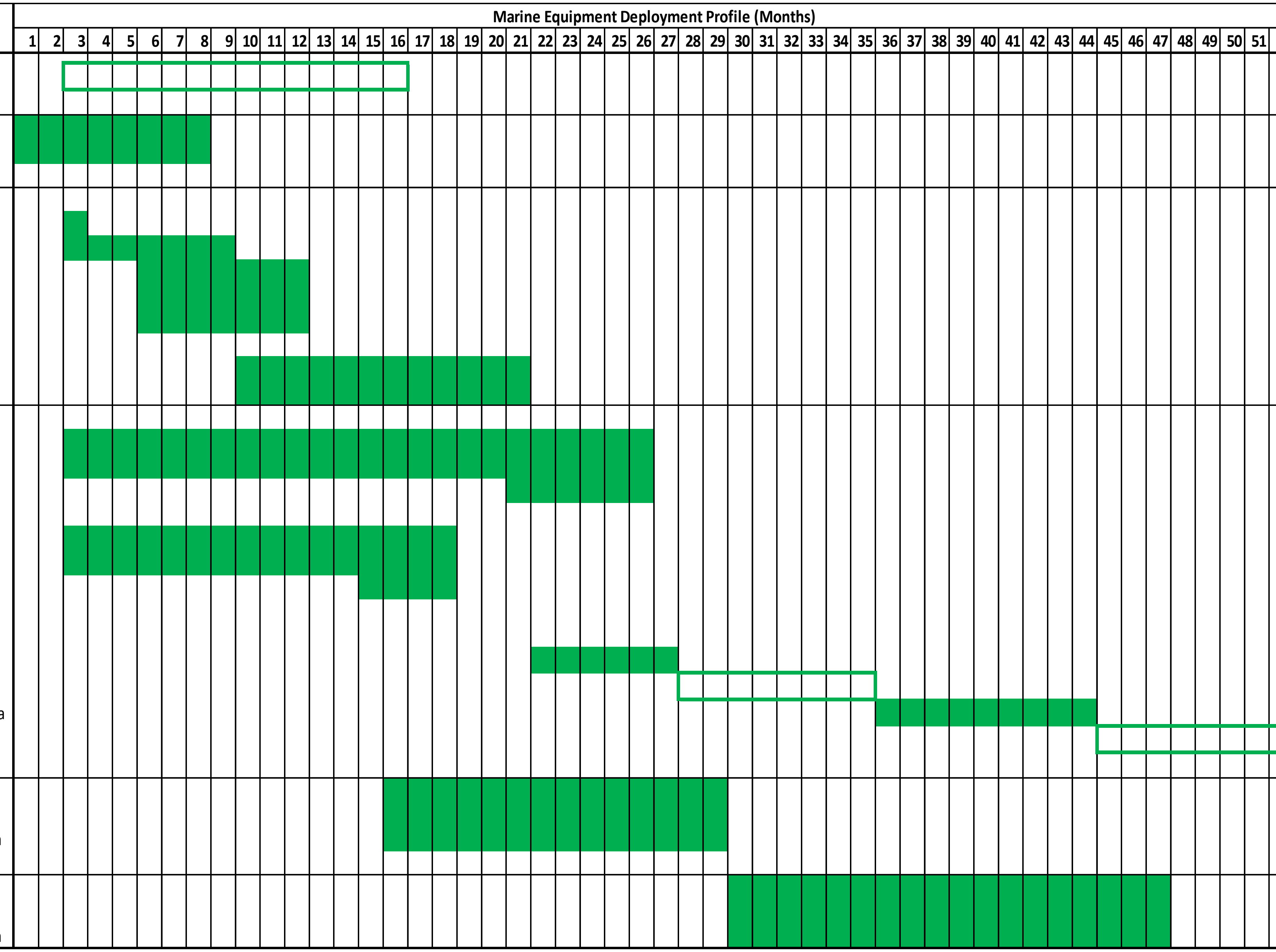
## 7.2 Construction Schedule and Equipment Profile

A summary of the estimated construction schedules and equipment profiles discussed above for each marine construction site is given in Figure 7-1. For the sake of conservatism, marine-based construction methods have been assumed for all applicable construction sites, since this represents the largest floating equipment profile that may have an effect on marine traffic. The minimum required equipment is identified for each construction site as well as the estimated frequency of floating equipment transits. Considering the estimated overall construction durations for each marine work site, a temporal profile is given for each piece of marine-based equipment. This allows a quick determination of the estimated construction fleet in operation at any given time.

As can be seen by Figure 7-1 at its peak, the construction fleet may consist of up to 4 to 7 marine derricks and 4 to 7 flat deck scows operating simultaneously over 3 different sites. Also if the MOF dredging program lags, a jack-up barge working beside a marine backhoe dredger, and split hopper barge in the MOF cove may also be an ongoing operation during the peak time. The total construction fleet mobilized may include anywhere from 14 to 18 working barges and perhaps a dozen or so supply barges transiting from the various construction sites to staging and anchoring areas. This fleet will be supported by several crew boats, utility tugs and other small craft vessels.

The fleet will be distributed over a wide area of several kilometres from Lelu Island Slough to the LNG Berth locations out in open water. Much of the construction activities will occur in relatively confined or contained work sites. With a few exceptions, these work sites will be mainly located in coves and inlets away from regular marine traffic channels or in areas along the perimeter of Flora bank which are naturally probative to marine through-traffic.

| Construction Site | Marine Equipment | Duration | Round Trip Transits |
|---|---|---|---|
| **Lelu Island Access Bridge** | None | 14 mo | None |
| **Pioneer Dock** | 1 Derrick | 8 mo | 1 per day from staging area |
| | 1 Scow | 8 mo | 1 per day from staging area |
| **MOF** | | | |
| Dredging Overburden | Suction Hopper Dredge | 1 mo | 20 to 40 to Ocean Disposal |
| Blasting Rock* | 1 Jack-up Barges | 7 mo | none |
| Dredging Blast Tailings | 2 Marine Backhoe Dredger | 7 mo | none |
| | 1 Split Hopper Barge | 7 mo | 1 per day to disposal area |
| | 1 Multicat Barge | 7 mo | 1 per day from staging area |
| Wharf Structures | 1-2 Derricks | 12 mo | 1 per day from staging area |
| | 1-2 Scows | 12 mo | 1 per day from staging area |
| **Suspension Bridge** | | | |
| **SW Anchor Block Foundation** | 1-2 Derricks | 24 mo | 1 per day from staging area |
| | 1-2 Scows | 24 mo | 1 per day from staging area |
| | 3 Concrete Supply barges | 6 mo | 6 per day from batch plant |
| **SW Tower Foundation** | 1-2 Derricks | 16 mo | 1 per day from staging area |
| | 1-2 Scows | 16 mo | 1 per day from staging area |
| | 2 Concrete Supply barges | 4 mo | 1 per day from batch plant |
| **Superstructure** | | | |
| SW Tower | Supply barge | 6 mo | 1 per week |
| Suspension Cable | None | 8 mo | |
| Deck Superstructure | Supply barge | 9 mo | 2 per week from staging area |
| Finishing Work Cables/Deck | None | 8 mo | |
| **Trestle** | 1 Derrick | 14 mo | 1 per day from staging area |
| | 1 Scow | 14 mo | 1 per day from staging area |
| | 1 Supply Barge | 14 mo | 2 per week from staging area |
| **LNG Berths** | 2 Derricks | 18 mo | 1 per day from staging area |
| | 2 Scows | 18 mo | 1 per day from staging area |
| | 2 Supply Barges | 18 mo | 2 per week from staging area |




* Blasting operations must take place in blasting window between November 30 and February 15, plus or minus any negotiated window extensions that might be obained.

**Figure 7-1: Marine Facility Construction Schedule and Equipment Profile**

Daily transits are expected for much of the working vessels as they are deployed and recalled to accommodate tidal restrictions, weather conditions, material resupply, and work shift requirements. Staging areas and anchorages can be located to minimize disruptions to marine traffic. During construction activities much of the fleet will remain stationary while construction operations are being conducted and will only transit for the reasons discussed above.

In general the construction of the marine facilities should not be an impediment to marine traffic. Mitigations include enforcing a safety zone around each work site and promulgating work site restrictions through a variety of channels, such as Notices to Mariners, in accordance with the Marine Communication Plan. As a participant in the Port of Prince Rupert's Construction Coordination Committee for de-confliction with other users, PNW LNG's Marine Communications Plan will be used to alert mariners to project activities, hazards and safety measures such as enforced safety zones and marine route closures.

# 8. References

BC Marine and Pile Driving Contractors Association. 2006. Best Management Practices for Pile Driving and Related Operations.

Draft Geotechnical Recommendations for Trestle and Berthing Area of Pacific Northwest LNG (PNWLNG) Project: October 11, 2013, prepared by Fugro.

Coastal Engineering for Maritime Applications. 2014, PIANC World Congress San Francisco USA, Madsen, L.P.

PNW LNG (Pacific Northwest LNG Limited Partnership). 2014a. Environmental Assessment Certificate Application. [Online] http://a100.gov.bc.ca/appsdata/epic/html/deploy/epic_project_home_396.html, accessed on September 12, 2014.

PNW LNG (Pacific Northwest LNG Limited Partnership). 2014b. Draft Table of Commitments (updated as of July 14, 2014), prepared by Stantec.

# Appendix A
# MOF Dredging Methodology
# (EA Submission)

Internal Memo

H345670

November 25, 2014

To: Chuck Rosner

From: Rene Berenger/Roslin Arbuckle

cc: Otavio Sayao

Checked: Otavio Sayao

## Pacific Northwest LNG
## Lelu Island LNG Terminal

# MOF Dredging Methodology (EA Submission)

## 1. Introduction

As part of the Pacific Northwest LNG project, dredging between Lelu Island and Ridley Island (Porpoise Channel) is planned to provide adequate depth for the MOF berths. Hatch conducted a preliminary review of potential methods to complete this dredging work. The final methodology will be selected by the contractor.

Figure 1-1 and Figure 1-2 show the conceptual plan and cross section of the proposed dredging works in the MOF area. The MOF area will be extended for the proposed berths and deepened to -12 m CD (shown in red). The shoreline of the MOF area will be a vertical bulkhead wall. This design is preliminary and subject to change.

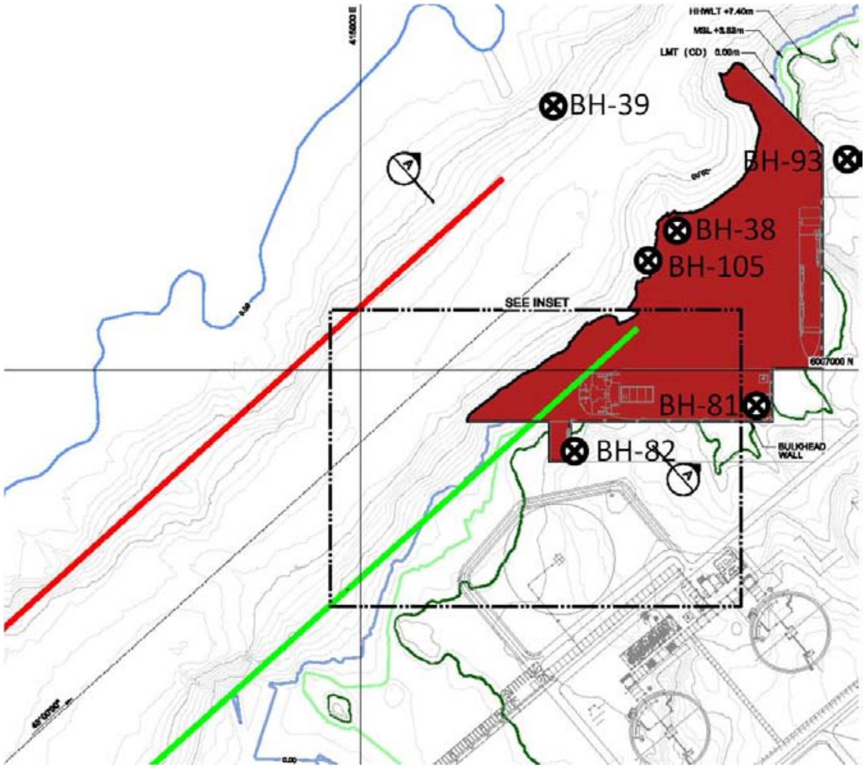



**Figure 1-1: Plan View of the Proposed Dredging Works**

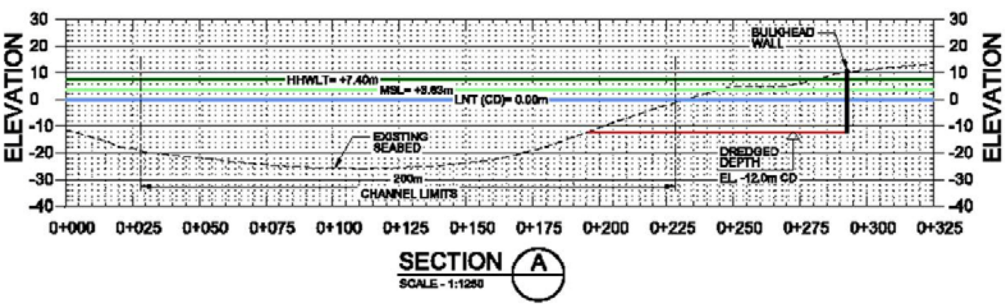



**Figure 1-2: Cross Section of Proposed Dredging Works**

Most of the dredge area is composed of rock material that will have to be drilled and blasted prior to dredging. The overall volume estimation informed by PNW LNG (meeting on November 4, 2014) is 790,000 $m^3$ where 200,000 $m^3$ corresponds to marine sediments and 590,000 $m^3$ corresponds to rock material.

Since sediment dredging will be executed prior to rock drilling and blasting (D&B), and rock D&B activities will be executed before rock dredging, a distinguished methodology is proposed for each one of these activities.

The methodologies adopted in this study were based primarily on the geotechnical information provided by PNW LNG, tidal variations and Hatch's experience in rock D&B and dredging operations.

In Figure 1-3 the blue line represents the LLWL elevation (0.00 m CD), the light green line represents the MSL elevation (+3.83 m CD) and the dark green line represents the HHWL (+7.40 m CD).

The significant tidal variation in the MOF site will force the use of different equipment for wet and dry land areas. Figure 1-3 depicts the working areas limited by tide levels as detailed in Section 4.1.

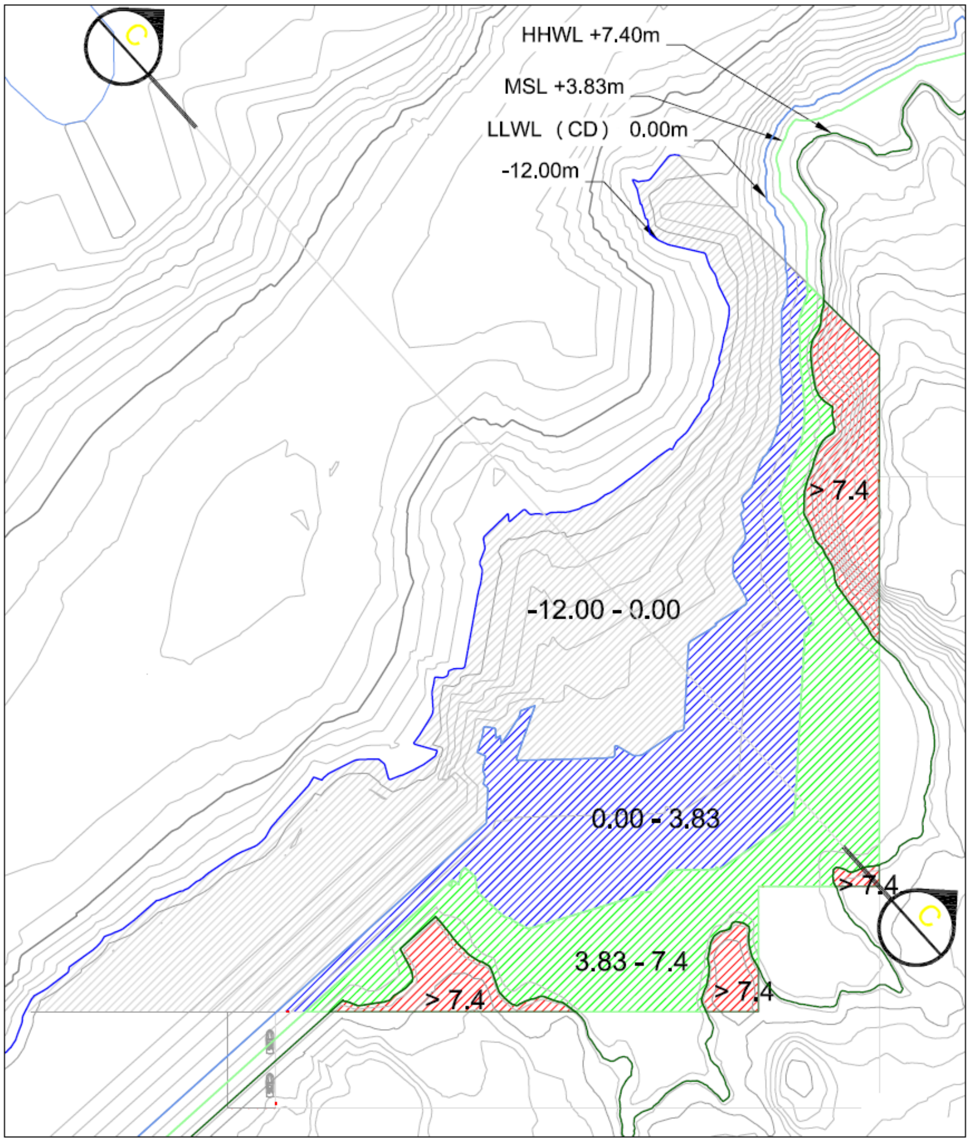



**Figure 1-3: Plan View of the Working Areas**

## 1.1 Drilling and Blasting Activities

The methodology approach for D&B activities will be examined vertically considering the maximum possible drill length of 20 m (personal communications with D&B company, Maxam, October 28, 2014). The maximum vertical dredging height for the project area is between dredge depth (-12.00 m CD) and HHWL (+7.40 m CD) with corresponds to a vertical distance of 19.40 m.

Figure 1-4 below depicts the vertical approach to D&B activities and Table 1-1 presents the associated quantities to each working area.

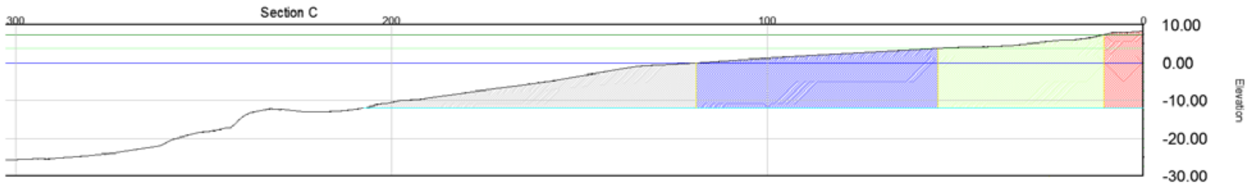



**Figure 1-4: Cross Section Vertical Approach**

**Table 1-1: Calculated Drilling Rock Volumes Vertical Approach**

| Working Areas | Volume Percentage | Volume ($m^3$) | Surficial Area ($m^2$) | Equivalent Depth (m) |
|---|---|---|---|---|
| Above HHWL (+7.4m CD) | 24% | 143,540 | 7,240 | 19 |
| Upper Intertidal: HHWL to MSL (+7.4 to +3.83 m CD) | 36% | 210,770 | 13,730 | 15 |
| Lower Intertidal: MSL to LLWL (+3.83 to 0.00 m CD) | 22% | 131,560 | 14,650 | 9 |
| Below LLWL (0.00 m CD) | 18% | 104,130 | 22,415 | 5 |
| Total | 100% | 590,000 | 58,035 | - |

## 1.2 Dredging Activities

The methodology approach for dredging activities will be examined horizontally.

Figure 1-5 below depicts the horizontal approach to dredging activities. Table 1-2 and Table 1-3 present the associated quantities to each working area for marine sediments and rock volumes respectively.

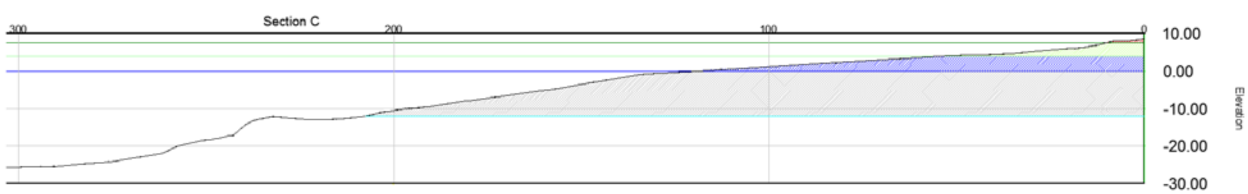



**Figure 1-5: Cross Section Horizontal Approach**

**Table 1-2: Calculated Marine Sediment Dredge Volumes Horizontal Approach**

| Working Areas | Volume | Volume ($m^3$) | Surficial | Equivalent |
|---|---|---|---|---|



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| | Percentage | | Area (m²) | Depth (m) |
|---|---|---|---|---|
| Above HHWL (+7.4m CD) | 5% | 9,920 | 7,240 | TBD |
| Upper Intertidal: HHWL to MSL (+7.4 to +3.83 m CD) | 5% | 10,611 | 13,730 | 3.57 |
| Lower Intertidal: MSL to LLWL (+3.83 to 0.00 m CD) | 14% | 27,733 | 14,650 | 3.83 |
| Below LLWL (0.00 m CD) | 76% | 151,736 | 22,415 | 12.00 |
| **Total** | **100%** | **200,000** | **58,035** | **-** |

As detailed further in Section 2.2.3, the sediment layer generally ranges from 0.6 to 2 m thick and is underlain by rock material. It is expected that the sediment layer is thicker in deeper areas and thinner in shallow areas.

**Table 1-3: Calculated Rock Dredge Volumes Horizontal Approach**

| Working Areas | Volume Percentage | Volume ($m^3$) | Surficial Area (m²) | Equivalent Depth (m) |
|---|---|---|---|---|
| Above HHWL (+7.4m CD) | 5% | 29,264 | 7,240 | TBD |
| Upper Intertidal: HHWL to MSL (+7.4 to +3.83 m CD) | 5% | 31,304 | 13,730 | 3.57 |
| Lower Intertidal: MSL to LLWL (+3.83 to 0.00 m CD) | 14% | 81,812 | 14,650 | 3.83 |
| Below LLWL (0.00 m CD) | 76% | 447,621 | 22,415 | 12.00 |
| **Total** | **100%** | **590,000** | **58,035** | **-** |

Rock volumes for the working area above HHWL comprehends elevations up to +24.00 m CD in the northern extreme of MOF area and +12.00 m CD in the southern extreme of MOF area. Removal of rock volumes apart from the area depicted in Figure 1-3 is not part of the scope of this study.

# 2. Geotechnical Review

## 2.1 Seismicity

Lelu Island is at about Latitude 54.209 and Longitude -130.299.

The seismic hazard was assessed in accordance with the 2010 NBCC. Interpolated firm ground, seismic hazard values were obtained from the seismic hazard database of Natural Resource Canada. The peak ground acceleration for a 1:2,475 year earthquake is 0.179 g.

## 2.2 Subsurface Conditions

An assessment of the subsurface conditions around the proposed dredge areas have been carried out. The following sections summarize and interpret the relevant information from the available references.

### *2.2.1 Data Sources and Limitations*

Much of the relevant information related to the seabed conditions are taken from survey work performed for the turning basin area. Unfortunately the limits of this survey work do not fully extend into the planned dredge area. The information covers roughly the deeper portions of the dredge area. In addition, there is no report that accompanies the survey work describing the sample collection methods.

The penetration depth of investigation of all Electro-Magnetic (EM) systems is dependent on the EM frequency, coil orientation, system elevation above surface, data errors, and the electrical resistivity of the subsurface. Generally, at the highest frequency, depths of investigation are just a few meters. At the lowest frequency, 400 Hz, the depth of investigation may be on the order of 80 m. A discussion of the depth of investigation should be included in the survey report and may explain the small discrepancies we have observed.

The following extracts from the reference information relate to the foundation conditions.

### *2.2.2 FPI-7028-TURNBASIN-001-Chart1B, Chart4B*

These charts show the bathymetry, slope interpretation and surficial features of the turning basins. It can be interpreted that generally the steeper topography is associated with exposed rock and that shallower topography has marine sediments cover the area.

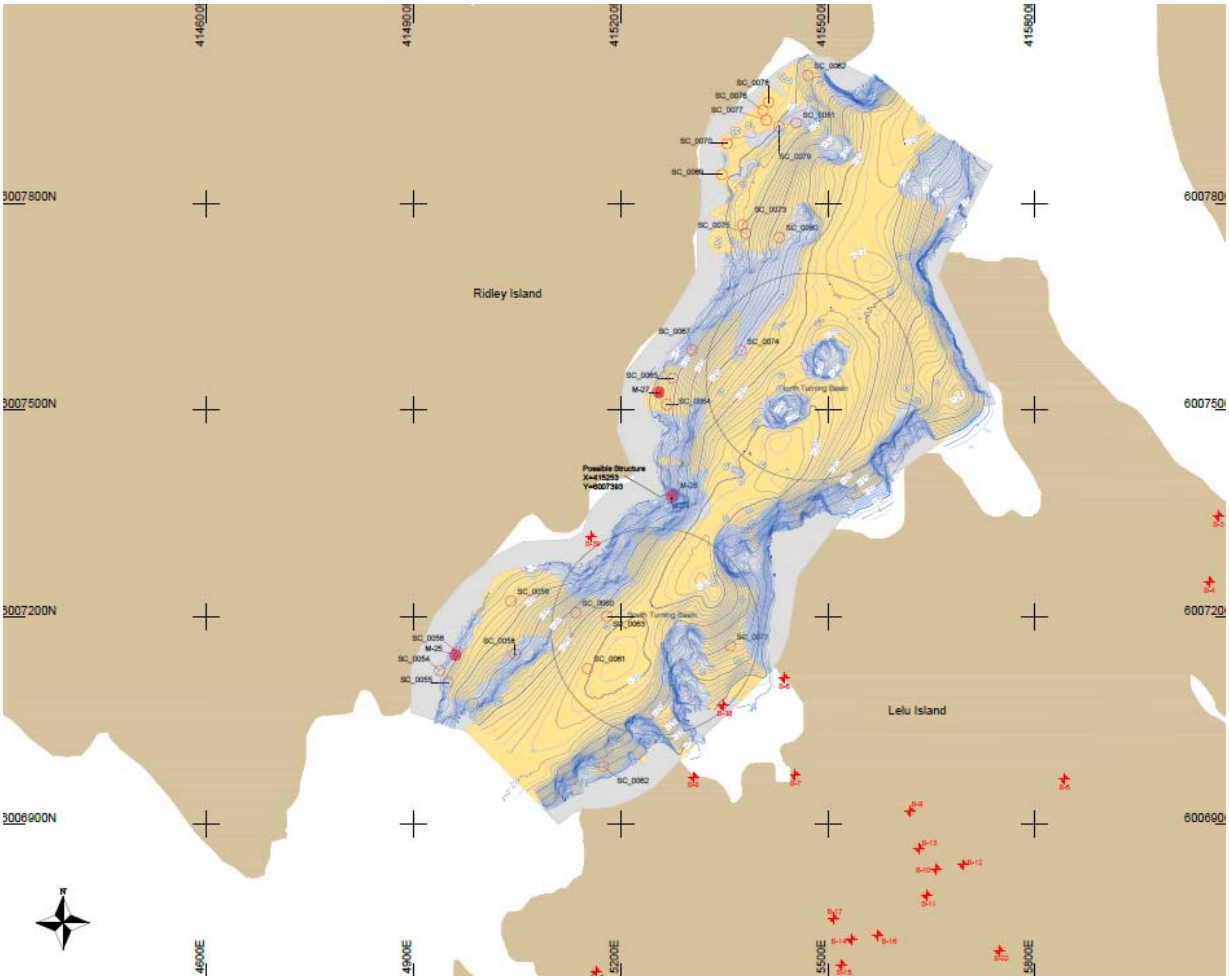


**Figure 2-1: Plan View of Bathymetry and Surficial Features (FPI-7028-TURNBASIN-001-Chart1B)**

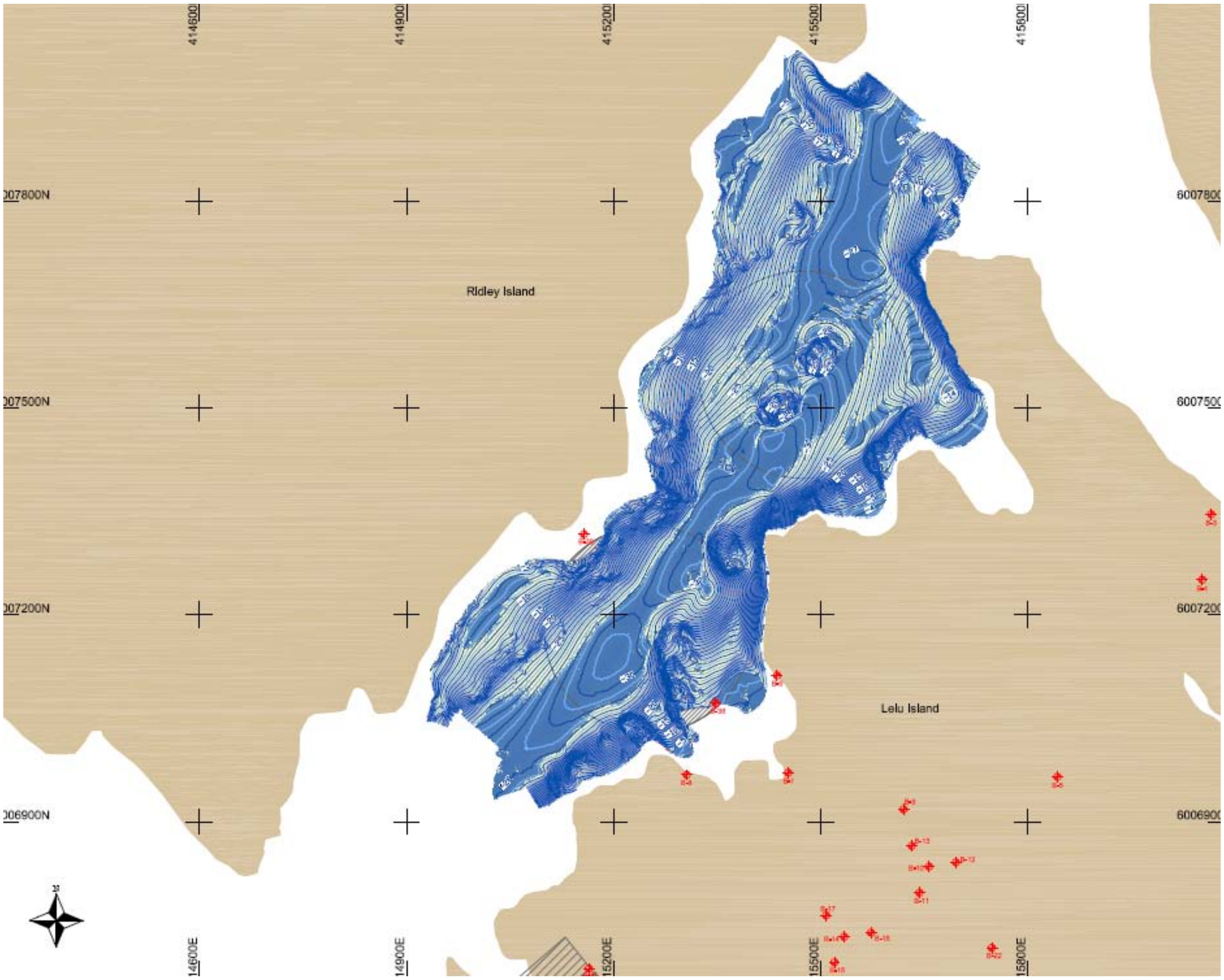


**Figure 2-2: Plan View of the Bathymetry and Slope Interpretation (FPI-7028-TURNBASIN-004-Chart4B)**

### *2.2.3 Boreholes*

There are three boreholes that were drilled within the geophysical survey area near the channel adjacent the dredge area. These boreholes include BH-38, BH-39 and BH-105. All of these holes fall within areas designated on Chart 1B as Bedrock.

There are three other boreholes that were drilled outside of the geophysical survey area but within the proposed dredge area near the planned MOF wharf locations, namely BH-81, BH-82 and BH-93. All of these holes fall outside the survey limits of Chart 1B. However, BH-81 is located within the planned MOF area excavation and provides direct subsurface information.

The approximate locations of the boreholes are shown in Figure 1-1.

#### *2.2.3.1 Borehole BH-38*

Borehole BH-38 generally consists of about 2 m of marine sediment underlain by phyllite. The marine sediment is described as very soft sandy clay. The phyllite is generally described as fresh and hard with an RQD between 45 and 100 (Average RQD =75 ) and 37 to 85 degree dipping joints observed. There is a thin layer (less than 200mm) of highly-weathered rock.

The presence of marine sediment indicates that there is some discrepancy with the Charts. Some of the error could be due to the resolution of the measurements or the method of data collection which may not “see” thinner layers of sediment.

#### 2.2.3.2 *Borehole BH-39*

Borehole BH-39 generally consists of about 0.5 m of highly weathered phyllite underlain by moderately strong phyllite becoming fresh and strong at about 5 m depth.

The absence of marine sediment at this borehole location aligns well with the Charts, however highlights the inconsistency in the accuracy of the charts.

#### 2.2.3.3 *Borehole BH-105*

Borehole BH-105 generally consists of about 2 m of marine sediment underlain by 9 m thick schist (2 to 11 m depth) and greater than 29 m thick phyllite (11 to >40 m depth). The marine sediment is described as very soft sandy clay. The Schist is generally described as slightly to intensely weathered and moderately hard to soft with RQD between 0 and 55. The phyllite is generally described as moderately weathered and hard to very hard with RQD between 55 and 100 (Average RQD = 67 ) and fractures dipping at 35 degrees.

The presents of marine sediment indicates that there is some discrepancy with the Charts. Some of the error could be due to the resolution of the measurements or the method of data collection which may not “see” thinner layers of sediment.

#### 2.2.3.4 *Borehole BH-81*

Borehole BH-81 generally consists of about 1.8 m of marine sediment underlain by 4.2 m thick phyllite (1.8 to 6 m depth) and greater than 10 m thick mica schist (6 to >16 m depth). The marine sediment is described as very soft peat and silty sand. The phyllite is generally described as fresh to slightly weathered and moderately hard to hard with RQD between 10 and 65. The mica schist is generally described as fresh and hard with RQD between 10 and 65 (Average RQD = 40).

#### 2.2.3.5 *Borehole BH-82*

Borehole BH-82 generally consists of about 0.6 m of marine sediment underlain by greater than 14.6 m thick phyllite (0.6 to >15 m depth). The marine sediment is described as very soft peat and lean clay. The phyllite is generally described as moderate to slightly weathered and hard to very hard with RQD between 10 and 100 (Average RQD = 70) and 40 to 80 degree dipping joints observed.

#### 2.2.3.6 *Borehole BH-93*

Borehole BH-93 generally consists of about 1.5 m of marine sediment underlain by greater than 19.5 m thick phyllite (1.5 to >21 m depth). The marine sediment is described as very soft peat (about 1.2 m thick) and silty sand with gravel. The phyllite is generally described as intensely to moderately weathered and hard to very hard with RQD between 0 and 100 (Average RQD = 65) and 18 to 70 degree dipping joints observed.

### 2.2.4 Interpretation Summary

#### 2.2.4.1 Dredge Area (Channel Side)

It is anticipated that the subsurface conditions near the planned dredge bench will be similar to the conditions in BH-38 and BH-105.

In general it would be anticipated that:

- the anisotropic behaviour and the influence of the rock mass structure associated with phyllite/schist is minor and assumed homogenous.
- the 37 to 85 degree bedding/foliation in phyllite is unlikey to daylight in the 2H:1V or 3H:1V dredge bench excavation slopes. Therefore, failure of the rock slope has not been considered further.
- the marine clay will have a minimum undrained shear strength of 10 kPa and increase in strength with depth at a rate of 1 kPa/m.

#### 2.2.4.2 Dredge Area (Wharf Side)

It is anticipated that the subsurface conditions near the planned MOF excavation and wharf installation area will be similar to the conditions in BH-81, 82 and 93.

In general it would be anticipated that:

- the anisotropic behaviour and the influence of the rock mass structure associated with phyllite/mica schist is minor and assumed homogenous.
- the 37 to 85 degree bedding/foliation in phyllite/mica schist could potentially be daylighting in the vertical excavation, which could result in potential tensile failure along the foliation/bedding planes.
- the marine clay will have a minimum undrained shear strength of 10 kPa and increase in strength with depth at a rate of 1 kPa/m.

Table 2-1 summarizes the material properties adopted for further assessment. These numbers represent ballpark figures and should not be used in the design.

**Table 2-1: Summary of Slope with Input Parameters**

| Material | Cohesion (kPa) | Friction (Deg) | Unit Weight ($kN/m^3$) |
|---|---|---|---|
| Marine Clay (Long Term) | Function(depth) ~ 0 kPa | 28 | 18 |
| Marine Clay (during excavation) | Function(depth) ~ 15 kPa | 0 | 18 |
| Rock (Phyllite) | Function(Overburden) >10 kPa | 50 | 26 |

### 2.2.5 Seismic Performance

A stability or liquefaction assessment has not been carried out. It is unlikely that liquefaction will be an issue. Stability under a major seismic event should be reviewed.

## 2.3 MOF Area Material Quantities

The total Estimated total MOF area volume is about: 790,000 $m^3$. It is estimated that the subsurface conditions near the MOF area will consist of the following:

- Estimated volume of marine sediments: 200,000 $m^3$; and
- Estimated volume of rock: 590,000 $m^3$.

# 3. Site Conditions

## 3.1 Bathymetry and Topography

- Hydrographic survey data was collected by McElhanney Consulting Services Ltd (Contracted by KBR LLC). The survey was conducted in August 2012. Bathymetric data was also collected from the Canadian Hydrographic Service (CHS) surveys including nautical charts CHS 3958 Prince Rupert Harbour (1:20,000), CHS data sheets, CHS multi beam data, and CHS 500 m grid data.
- Topography for Lelu Island is from 1 m LiDAR contours produced by McElhanney Consulting Services (2012).

## 3.2 Tides and Water Levels

Tide levels near Lelu Island are measured from local Hydrographic Tide Table at Port Edward, BC, as published in the Canadian Tide and Current Tables, Volume 7 (Table 3-1). The tidal range in this region is significant, with a variation in water elevation over 7 m.

**Table 3-1: Hydrographic Tide and Chart Datum at Port Edward, BC**

| Tide Level | Elevation (m) (Chart Datum) |
|---|---|
| Higher High Water Level (Large Tide) | 7.4 |
| Higher High Water Level (Mean Tide) | 6.1 |
| Mean Sea Level | 3.8 |
| Lower Low Water Level (Mean Tide) | 1.3 |
| Lowest Normal Tide (Chart Datum) | 0.0 |
| Lower Low Water Level (Large Tide) | 0.0 |

## 3.3 Currents

The current characteristics were extracted from Hatch's hydrodynamic simulation from January 18, 2014 to February 22, 2014 using CMS-Flow model (2D depth averaged current velocities).

Tidal currents are presented in Table 3-2. Figure 3-1 shows the selected point at the proposed dredge site where currents were evaluated. The MOF dredge area is sheltered from the higher currents seen in Porpoise Channel.

**Table 3-2: Current Magnitude at the MOF Dredged Area**

| Current Magnitude | MOF |
|---|---|
| Maximum Current | 0.82 m/s |
| Mean Current | 0.27 m/s |

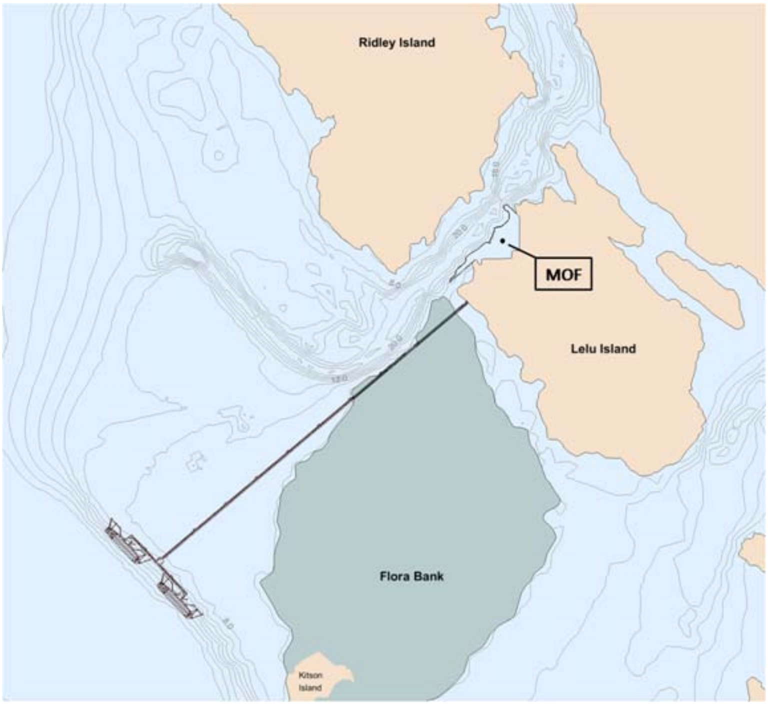


**Figure 3-1: Current Model Output Locations**

### 3.4 Wind

Figure 3-2 displays the wind magnitudes and directions of the edited Holland Rock data. The winds are edited for a closer match with the PNW LNG buoy (Ref: H345670-0000-12-124-0005, Rev B). The winds are predominately from the southeast with a small component from the northwest. The PNW LNG buoy is located at 54.198°N, 130.340°W.

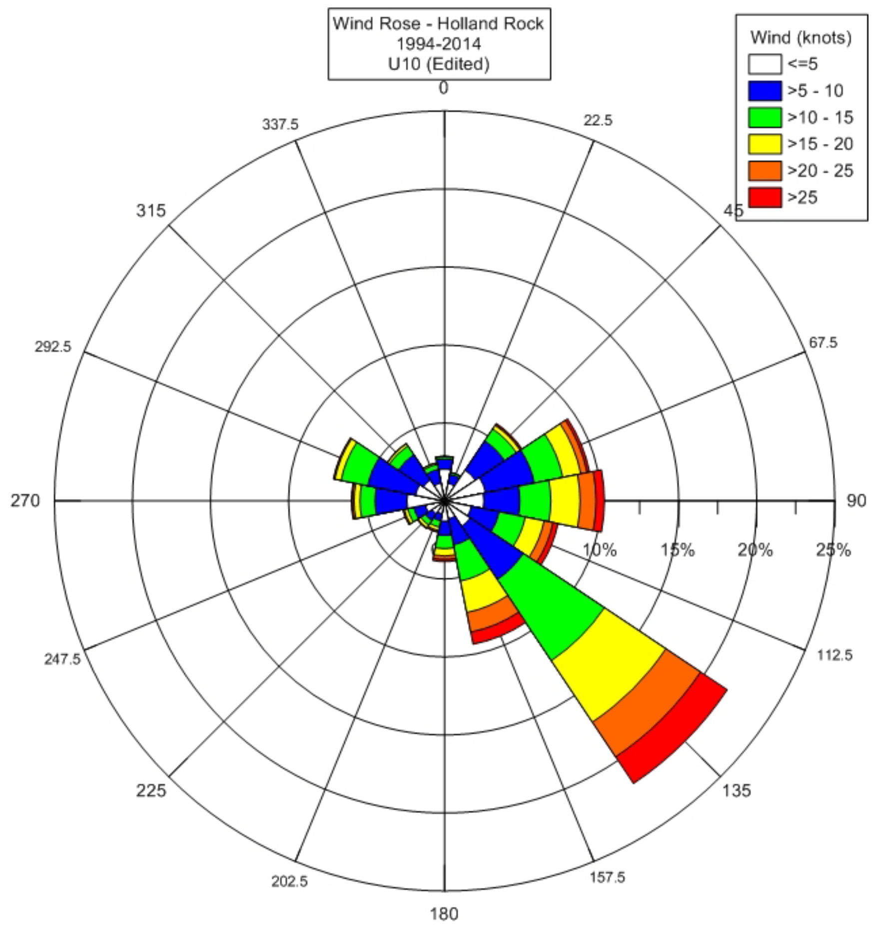


**Figure 3-2: Wind Rose of the 20-year Holland Rock Data (Edited)**

### 3.5 Waves

Hatch has not conducted wave modelling in the area of the MOF. A wave study conducted by Moffat & Nichol (KBR Document No. MA-ATA-G40-0003, 2013) found that waves at the MOF rarely exceed 0.1 m. Wave periods exceeded 6 s and 13 s approximately 50% and 10% of the time, respectively.

## 4. Dredging Methodology

The methodology used in this study was based mainly on the geotechnical information provided by PNW LNG, tidal variations and Hatch's experience in rock drilling/blasting (D&B) and dredging operations. This is a preliminary assessment and final methodology should be selected by the contractor.

Table 4-1 summarizes the suggested and alternative dredging equipment presented in the following chapters.

**Table 4-1: Equipment Summary Table – MOF Dredging Methodology**

| Equipment | Sediment Dredging | | Rock Drilling/Blasting | | Rock Dredging | |
|---|---|---|---|---|---|---|
| | Marine | Land | Marine | Land | Marine | Land |
| Suggested | TSDH | Elevated BHD | Jack-up drill and blast barge | Drill rigs on tracks for blasting | Marine Backhoe Dredger (BHD) | Elevated BHD |
| Alternative | DOP pumps | - | Self propelled drill and blast vessel | Excavator with hammer (no blasting) | - | - |

## 4.1 Operational Considerations

Due to the important tidal variation in the area of study, the dredging area will be divided into two regions with distinct D&B and dredging methodologies:

- Area below MSL to be performed with marine-based equipment;
- Area above MSL to be performed by land-based equipment;

It is important to remember that water level variation during spring tides can reach rates of approximately 1 meter per hour, and during neap tide approximately 0.5 meter per hour.

Most of the dredging area is located outside a natural navigation channel and is not expected to disturb navigation activities in the area.

During blasting activities the traffic in Porpoise Channel might have to be stopped.

## 4.2 Suggested Equipment

The site conditions and geotechnical information described above aid the choice of the most suitable dredging equipment. Dredging volume, disposal options and productivity will also play an important role in the determination of most suitable dredging equipment. As the majority of the material is rock, D&B will be used to break up the rock material before removal (dredging).

### *4.2.1 Marine Sediment Dredging*

The estimated 200,000 $m^3$ of marine sediment will have to be dredged prior to the commencement of the drilling blasting activities. All sediment will be disposed at sea in Brown's Passage.

The most common equipment for dredging marine sediments is a Trailing Suction Hopper Dredger (TSHD). The TSHD is a self propelled vessel that accumulates the dredged material into its hopper and disposes this material through doors located at the hull of the vessel. TSHDs are presented in a wide range of hopper volumes, however, the bigger the hopper capacity, the larger the vessel draft. Therefore, it is suggested that a small TSHD with a capacity up to 5,000 $m^3$ is used to access a wider area of dredging. No auxiliary equipment is necessary for a TSHD operation other than a crew vessel.

Since 90% of the sediment volume is located below MSL it is expected that the TSHD will be able to tackle most part of the marine sediment dredging. A 5,000 $m^3$ TSHD's monthly production is estimated to dredge approximately 150,000 $m^3$ per month what would result in approximately 2 months to dredge 90% of the marine sediment.

The abovementioned production rate was calculated based on the following information:

- 4,000 $m^3$ effective hopper volume;
- 0.6 hopper volume reduction factor;
- 6.5 hours operational cycle;
- 455 working hours per month; and
- 168,000 $m^3$ per month.

The remaining 10% of the sediment volume located above MSL is to be dredged by elevated excavators mounted with buckets or DOP pump. The dredged material should be transported to disposal site by self-propelled split barges.

Considering an elevated excavator with a production rate around 60,000 $m^3$ per month (detailed in Section 4.2.3.2), the marine sediment volume located above MSL would be dredged in approximately 1 month.

Marine sediment dredging above and below MSL could be performed simultaneously.

#### *4.2.1.1 Sediment Disposal*

The dredged marine sediment will be disposed at Environmental Canada's (EC) designated disposal site Brown Passage located approximately 33 km (18 nmi; nautical miles) from the MOF site (Figure 4-1).

It is estimated, based on the abovementioned sediment volumes below MSL to be dredged by a TSHD, that the number of trips to Brown Passage will range between 30 to 50. The remainder volume above MSL would require an additional 10 to 20 trips by a self-propelled split barge.

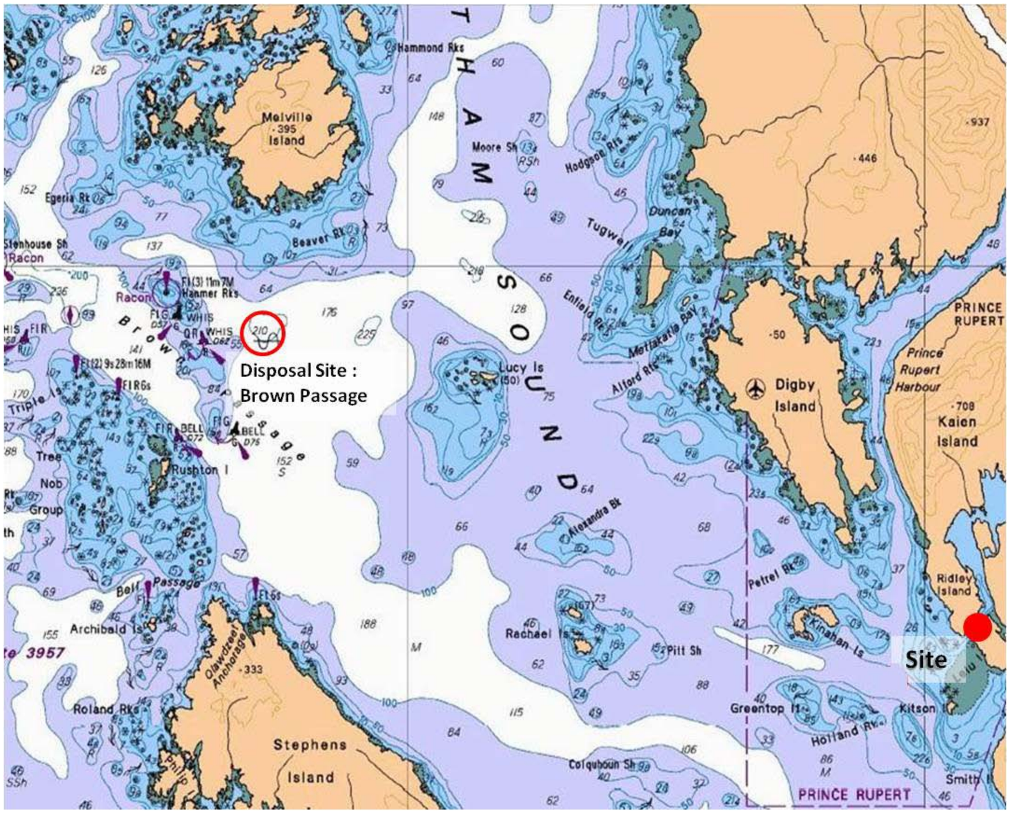


**Figure 4-1: Brown Passage (CHS 3800)**

Table 4-2 summarizes the equipment production and preliminary schedule for marine sediment dredging.

### 4.2.2 *Drilling and Blasting*

Drilling and blasting (D&B) is used to fragment the rock layers with explosives. Drilled holes are filled with explosives and laid out in a grid pattern.

A common grid for blasting operations consists of 3 m x 3 m squared grid with drilling holes of about 130 mm in diameter (personal communication, Maxam, October 28, 2014).

#### 4.2.2.1 *Marine-Based Equipment*

The most common equipment for drilling in offshore areas is a jack-up barge (also known as a self elevating platform) equipped with multiple drilling towers.

The jack-up barge consists of a floating pontoon with movable spuds, capable of raising its hull above the water surface. An example of a jack-up barge is shown in Figure 4-2. Its approximate dimensions are 40 m long x 30 m wide x 4 m moulded depth.

The main advantage of using a jack-up barge is its ability to withstand heavy seas, currents and the tidal range to maintain stability for drilling activities and positioning. The main disadvantage the jack-up barge that it’s not self propelled and is dependent on tugs for transportation and positioning.

The jack-up barge can be positioned in most of the project areas, up to an elevation of +5.40 m CD. This is the upper access limit based on an estimated jack-up barge draft, when floating, of approximately 2 m and barge deployment occurring when tides reach their highest levels. Even though the time span for highest tides is short in a semi diurnal tide region, the jack-up barge could be positioned fairly quickly with the help of tugs and anchors positioned on shore.

Auxiliary equipment for self elevated jack-up barge consists of tugs for positioning and a crew vessel.

**Figure 4-2: Typical Jack-Up barge (www.cmp.uk.com)**

#### 4.2.2.2 *Land-Based Equipment*

The upland area will be drilled with mobile drilling rigs and will need road access and levelled terrain for stability. Drilling rigs can be deployed on wheels or on tracks (Figure 4-3). Therefore, it will be necessary to fill some upland areas to create levelled access to the rigs.

Shallow areas between +5.40 m CD (jack-up draft limited depth) and HHWL (+7.40 m CD) will also have to be filled in order to allow access for land-based drilling equipment. No auxiliary equipment is required for the drilling rigs.

Fill and cut to provide level access of the land-based drill rigs will not be assessed in the study.

**Figure 4-3: Typical Drilling Rig on Tracks (www.minerex.com)**

#### 4.2.2.3 *Blasting*

The blasting material most widely used in environmentally sensitive areas, as a substitute to dynamite, is water-based gel explosives. The main features and benefits are:

- Safe to use;
- User friendly;
- High energy Ammonium Nitrate in suspension pumpable non explosive;
- Suited to hard rock mining, tough breaking conditions;
- Completely waterproof after two components mixed and chemically reacted;
- Dual density system provides low and high energy product out of single unit;
- Store as an oxidiser only.

The following characteristics apply to water gel explosives:

- High energy content (3.00 MJ/kg);
- Low water content (~10%). Better use of explosive energy as less water (inert) needs to be heated to the explosion temperature (~2,500 K);
- Uses high energy water soluble fuels;
- 100% Water resistant. It is cross linked in the borehole which renders it resistant to dynamic ground water in boreholes and stemming contamination;
- Totally pumpable. Makes possible a consistent displacement of the borehole water interface by loading from the bottom up;
- Uniform performance irrespective of water presence in the boreholes;

- Universal bulk explosive for wet/dry conditions. Makes it possible to drill expanded patterns consistently, irrespective of weather conditions.

Figure 4-4 depicts the appearance of water gel explosives.

**Figure 4-4: Water Gel Explosive Before and After Sensitizing (www.maxam.net)**

Dredge spoil depends on blasting setup and soil conditions. Less fractured rock will better transfer the explosives energy. The top layer of weathered rock tends to be more fractured and will therefore dissipate more of the blasting energy.

The blasting setup can adapt to the type of dredging equipment such as BHD bucket size once the equipment is chosen. It is expected that rock spoil will be used for civil works.

An average rock volume blasted per detonation is approximately 15,000 $m^3$ (personal communications with D&B company, Maxam, October 28, 2014).

The least risk work window for the Lower Skeena region, including Porpoise Channel, is from November 30, 2014 to February 15, 2015 as stated by DFO. This window is subject to site-specific changes based on species use and discussions with the Department of Fisheries and Ocean (DFO). All in water blasting has to be completed during this least risk work window. Blasting should also only be conducted during low tide. Due to noise restrictions from Port Edward blasting cannot be conducted at night.

Drilling operations should be mobilized well in advance of the least risk work window to prepare the blasting boreholes.

In order to complete the detonation with marine-based equipment in the available blasting window, a blasting rate of 15,000 $m^3$ per 4 days should be envisioned (112,500 $m^3$ per month).

The abovementioned blasting rate was calculated based on the following information:

- ≈250,000 $m^3$ rock volume below MSL;
- 15,000 $m^3$ per detonation;
- 18 detonations;
- 75 days blasting window; and
- 4 days per detonation.

As discussed in the section above upland D&B operations will rely on road access and earth fill. Upland drilling and blasting activities can be executed in parallel to marine drilling and blasting activities. It is expected that upland D&B schedule will be shorter than marine D&B due to simpler logistics involved with equipment mobilization and operation.

D&B equipment production rates should be investigated in more detail in order to assess the most suitable equipment for the available project schedule.

Table 4-3 summarizes the equipment production and preliminary schedule for D&B activities.

### *4.2.3 Rock Dredging*

#### *4.2.3.1 Marine=Based Equipment*

The most suitable equipment for recovering blasted rock in the MOF area is a marine backhoe dredger (BHD) as shown in Figure 4-5. A BHD is a stationary water-based excavator mounted on a dedicated dredging pontoon that has a rotating table. The position and stability is maintained by three spud poles, where two spuds are fixed to the front side of the pontoon near the excavator crane and one tilting spud at the aft side. The equipment can move with the tilting spud and direction can be adjusted with the two-part articulated arm. In the dredging position the front spuds are firmly planted in the seabed. The dredger then raises itself partially out of the water to further anchor the spuds. With the pontoon slightly lifted out of the water, part of the weight of the dredger is now transferred via the spuds to the seabed, resulting in an increase in anchoring.

Because of the hydraulic action of its arm and bucket cylinders, BHD's exert positive forces pushing the bucket into the material to be excavated.

**Figure 4-5: Typical Backhoe Dredger**

BHDs are becoming the dredging equipment of choice as dredging operations and projects have expanded. This equipment can dig at greater depths and have greater installed power, therefore can be utilized cost-effectively in heterogeneous soils containing a mixture of clay, sand, cobbles and boulders. Because BHDs can generate reasonable cutting force, they are also suitable for digging heavy clays, soft stones, blasted rock and soil containing fractured rocks or rubble.

BHDs are equipped with accurate positioning systems and can operate with precise underwater profiles. The bucket position is followed by the operator on a computer screen in real time and the seabed elevation is updated automatically in the dredging software once the bucket removes material from the seabed.

The suggested minimum requirement for the BHD is specified as 3,000 kW power capacity, 10 $m^3$ bucket volume and approximate pontoon dimensions being 60 m long x 18 m wide x 4 m moulded depth, which falls into the class of a mega BHD.

The advantage of using a mega BHD is its ability to dig into hard soils containing boulders or debris due to its hydraulic power and can reach water depths up to 20 m. The main disadvantage of BHD is that it largely relies on the pontoon's buoyancy to support its weight as the pontoon spuds do not have the capacity to carry it. The BHD has to then work with enough under keel clearance in order to allow safe water draft for dredging operations. The semi diurnal tidal variation at the MOF site will restrict the area of influence for this equipment and force it to move a great deal to cover the intertidal area. Another disadvantage of BHD is downtime due to waves and cross-currents. Since the MOF dredging area is protected, it is not expected that site climate conditions will cause equipment stand-by.

Dredging is expected to start only after blasting is complete and surveyed. The BHD bucket dumps the rock material into a self propelled split barge moored alongside the dredger. The suggested minimum requirement for the split barge volume capacity is 700 $m^3$ with bow thrusters.

Auxiliary equipment involved into marine dredging includes:

- 1 multicat barge (25 m long x 12.7 m wide x 2.3 m moulded depth) with 40 t crane capacity;
- Self propelled split barge;
- 1 survey and crew launch boat; and
- 1 operational launch boat.

It is suggested that the rock disposal site be positioned in an intertidal area with access to land-based equipment once water level drops. According to client communication, the rock dredged material will be used for Lelu Island civil works. Therefore, the split barge will have to dump the rock disposal material as close as possible to shore at high tide.

Production rate for the marine-based BHD is estimated to be 37,500 $m^3$/month considering an operation rate of 350 hours/month and a production rate of 85.8 $m^3$/hour with a 10 $m^3$ bucket. Higher productivities may be achieved with a larger bucket size.

The abovementioned production rate was calculated based on the following information:

- Back hoe bucket volume: 10 $m^3$;
- Cycle: 30 buckets per hour;
- Theoretical production: 165 $m^3$/h;
- Effective production: 107.25 $m^3$/h;
- Effective production hours: 350 hours per month; and
- Monthly production: 37,537 $m^3$/month.

With the calculated monthly production, one marine BHD would need over 12 months to dredge the rock volume below MSL.

#### 4.2.3.2 *Land-Based Equipment*

The most suitable equipment to dredge the upland areas as well as the intertidal areas that are inaccessible by a marine-based BHD, is an elevated land-based backhoe excavator mounted on an under-carriage with spread tracks as shown in Figure 4-6.

This versatile equipment can operate at water depths up to 5 m and dig into hard soils containing boulders or debris due to its hydraulic power. The main objective of the land-based backhoe is to move the blasted and disposed rock towards the upland and make it available for civil works and earth moving equipment (not part of the scope of this study).

**Figure 4-6: Typical Elevated Backhoe (www.jandenul.com)**

The elevated backhoe will gradually drag and move the blast rock material upland. The estimated production rate for land-based backhoe is 60,000 $m^3$/month considering an operation rate of 350 hours/month and production rate of 200 $m^3$/hour with a 10 $m^3$ bucket.

Considering the total volume to be dredged by land-based equipment is approximately 118,000 $m^3$, the estimated schedule for one land-based backhoe is about 2 months.

Land-based dredging can be performed simultaneously with the marine-based dredging.

##### *4.2.3.3 Combined Marine and Land Rock Dredging Production*

Dredging with suggested marine-based equipment (BHD) presents several advantages despite the estimation to deliver slow production rates. The elevated excavator allotted for dredging above MSL can also dredge rock volumes up to approximately -8.00 m CD since this equipment can work up to 5 m underwater and has an arm reach up to 8 m. The marine BHD will finalize the rock dredging excavating the volume from -8.00 m CD up to final dredge level -12.00 m CD from deep water limits through berth vertical wall limit.

The updated volume breakdown for rock dredging considering that the elevated excavator will dredge from -8.00 m CD upwards and the marine BHD from -8.00 m CD downwards will be:

- Below -8.00 m CD for marine BHD: total surface area x 4 m = 54,000 $m^2$ x 4 m = 216,000 $m^3$ (≈40% of 590,000 $m^3$)
- Above -8.00 m CD for elevated excavator: 374,000 $m^3$ (≈60% of 590,000 $m^3$)

The revised schedule for the abovementioned volume breakdown is 7 months for the marine BHD and 7 months for the elevated excavator.

Figure 4-7 to Figure 4-9 depict the phasing for rock dredging to be performed by an elevated excavator and a marine BHD.

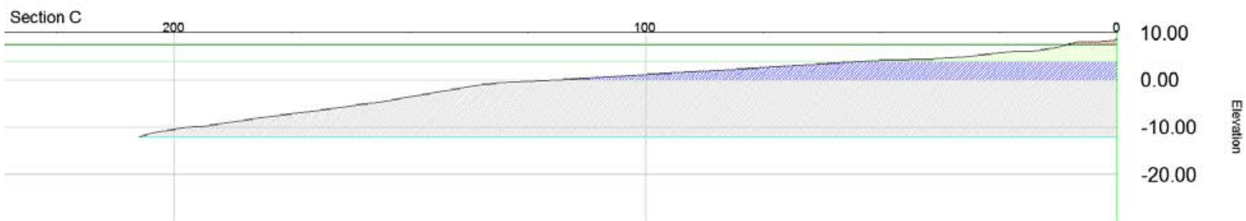


**Figure 4-7: Rock dredging Phase 0**

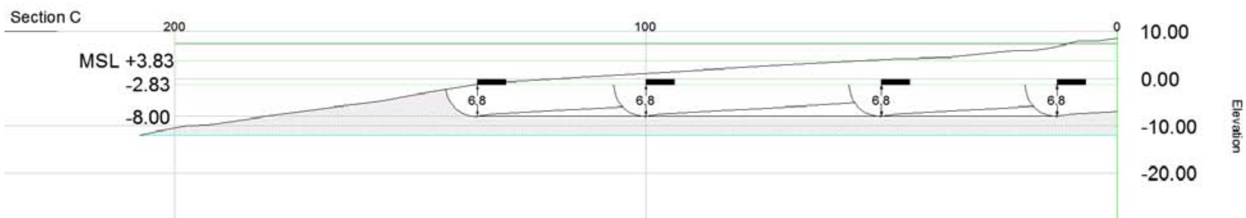


**Figure 4-8: Rock dredging Phase 1 – Land-Based Elevated Excavator**

Figure 4-8 depicts the position of the base of the elevated excavator 5 m below MSL (+3.8 m CD). The digging reach of the arm is represented by the arc and the maximum dredge level possible for the elevated excavator is around -8.0 m CD.

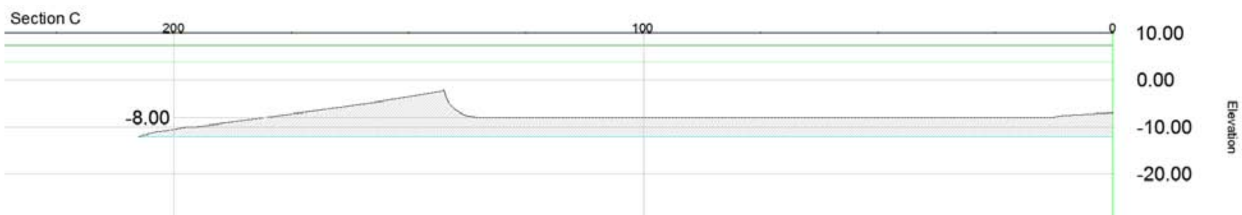


**Figure 4-9: Rock dredging Phase 2 – Marine BHD Excavator**

Figure 4-9 depicts the remaining section area to be dredged by the marine BHD.

Table 4-4 summarizes the equipment production and preliminary schedule for rock dredging.

## 4.3 Alternative Equipment

### *4.3.1 Marine Sediment Dredging*

Dragflow pumps (also known as DOP pumps) equipped with GPS positioning system would accurately remove the marine sediment above the rock profile.

A DOP pump can be deployed with the help of an excavator to dredge the marine sediments up to the excavator operational depth/reach (from shallow areas up to -6.00 m CD) or from a jack-up barge for deeper areas.

A DOP pump can be mounted on a variety of equipment such as land-based cranes, or excavators (for operational depths from shallow areas up to -6.00 m CD); and marine-based barges, excavators, tugs, etc. The production rate of a DOP pump will highly depend on the ability to move/drag the host equipment is capable of.

An average monthly production for a 12” DOP pump is of ≈24,000 $m^3$.

The abovementioned production rate was calculated based on the following information:

- Pump section area: 0.071 $m^2$;
- Pump speed: 3 m/s;
- Flow: 115 $m^3$/h;
- Effective flow: 69 $m^3$/h;
- Effective production hours: 350 hours per month;
- Monthly production: 24,150 $m^3$/month.

With the calculated monthly production, one marine DOP would need over 7.5 months to dredge the marine sediment volume below MSL.

### *4.3.2 Drilling and Blasting*

#### *4.3.2.1 Marine-Based Equipment*

A site visit to a drilling/blasting vessel was arranged with DEME on October 28, 2014 for the purpose of getting in-depth knowledge of D&B activities.

The Yuang Dong 007 (YD 007) vessel is one of the few rock drilling/blasting self propelled vessels in the world. The main vessel dimensions are 110 m LOA, 17 m beam and 4 m draft and contains 10 drilling towers on the portside of the vessel. The distance between the drilling towers can be adjusted over the installed rails. The drilling activities can be executed simultaneously and explosive water gel injection cassettes distribute the blasting material to the boreholes. Three cassettes service 10 drilling towers and all blasting activities are conducted by a specialized company (Maxam). Figure 4-10 shows the YD007 vessel in operation.

**Figure 4-10: YD007 Drilling/Blasting Vessel (Photo taken on October 28, 2014)**

The boreholes are drilled with a percussion and rotary method down to the top of rock through a sleeve that will prevent the overburden marine sediment from filling the borehole. The drilling continues to the design depth and the hole is cleaned with air pressure. The blasting gel is installed from the bottom of the hole upwards to a distance of approximately

0.75 m from top of rock level and detonation wires are installed. The borehole is then plugged with gravel up to top of rock level and the drilling sleeve is retrieved.

Positioning is done by adjusting anchor cables and spuds. An anchor handling vessel, as shown on the right hand side of Figure 4-10 (often called ‘multicat’), is necessary to support secondary operations such as anchor handling, installation of bubble curtain hoses, position the air compressor pontoon, deploy environmental mitigation measures, etc. Clearance of at least 100 m around the vessel is required to deploy anchors.

Figure 4-11 shows the instant of detonation taken from the YD007 vessel. Note the bubble curtain at the bottom of the photograph that surrounds the blasting area. Figure 4-11 also shows the air compressor pontoon that services the bubble curtain activity. The estimated rock volume blasted was around 15,000 $m^3$.

**Figure 4-11: Blasting Operation from the YD007 Vessel (Photo taken October 28, 2014)**

A typical operation cycle consisting of anchoring, positioning, drilling (maximum 10 boreholes), filling with explosives and connecting detonation wires takes about 5 hours. Local tidal analysis coupled with local bathymetry indicate that the vessel may work up to isobath +1.00 m CD.

Personal communication during site visit on October 28, 2014 indicates that the least volume worth for mobilization of such vessel is of 500,000 $m^3$.

Apart from rock volume for drilling and blasting do not fulfill minimum criteria for mobilization of such vessel, it is believed that drilling and positioning the vessel under tidal variations that can reach up to 1 meter per hour in spring tides can be unviable and could damage drilling equipment.

#### 4.3.2.2 *Land-Based Equipment*

Rock drilling/fragmentation above MSL could be performed with a hydraulic hammer adapted to the same equipment suggested as land-based dredging, the elevated excavator. This equipment is shown in Figure 4-12.

**Figure 4-12: Excavator with Hydraulic Hammer (www.besthammers.com)**

Commonly used in quarries and demolition works this equipment can remove the top of the weather rock profile. Nevertheless, this equipment will be efficient only for exposed upland and the intertidal rock volumes up to level +0.00 m CD. According to Table 1-2 the estimated rock volume above MSL is 354,000 m$^3$.

Rock drilling/fragmentation with a hammer equipped excavator will avoid blasting activities for intertidal and upland areas, but the rock quality will have to be investigated in more detail to assess if this equipment will be able to fragment the rock.

Rock drilling/fragmentation activities with hydraulic hammer tool will have to be supported by an extra elevated excavator with bucket in order to remove the fragmented material.

## 4.4 Summary Table

Table 4-2 and Table 4-4 summarize the volume and methodology information detailed in the previous sections.

**Table 4-2: Summary Table – Sediment Dredging**

<table>
<tr><th rowspan="2">Summary Table</th><th colspan="3">Sediment Dredging (Horizontal method)</th></tr>
<tr><th colspan="2">Marine (below MSL)</th><th>Land (above MSL)</th></tr>
<tr><th rowspan="2">Equipment</th><th>TSDH</th><th>DOP pumps</th><th>Elevated BHD</th></tr>
<tr><th>Suggested</th><th>Alternative</th><th>Suggested</th></tr>
<tr><td>Total Surface (m²)</td><td>53,867</td><td>53,867</td><td>53,867</td></tr>
<tr><td>Area Surface (m²)</td><td>37,566</td><td>37,566</td><td>16,301</td></tr>
<tr><td>Percentage Surface</td><td>70%</td><td>70%</td><td>30%</td></tr>
<tr><td>Total Volume (m³)</td><td>200,000</td><td>200,000</td><td>200,000</td></tr>
<tr><td>Area Volume (m³)</td><td>180,000</td><td>180,000</td><td>20,000</td></tr>
<tr><td>Percentage Volume</td><td>90%</td><td>90%</td><td>10%</td></tr>
<tr><td>Pros</td><td>- high accuracy through GPS positioning system</td><td>- high accuracy through GPS positioning system<br>- can be deployed from several equipment</td><td>- high accuracy through GPS positioning system</td></tr>
<tr><td>Cons</td><td>- needs to navigate 30 km to disposal<br>- higher capacity larger draft</td><td>- high water content disposal material<br>- downtime due to debris<br>- low productivity for large surface areas</td><td>- needs to dump excavated material on split barge</td></tr>
<tr><td>Auxiliary Equipment</td><td>- crew vessel</td><td>- excavator (by land)<br>- tug (by water)<br>- self propelled split barge (for disposal)<br>- crew vessel</td><td>- self propelled split barge (for disposal)</td></tr>
<tr><td>Production rate (m³/month)</td><td>150,000</td><td>24,000</td><td>60,000</td></tr>
<tr><td>Estimated schedule (months)</td><td>≈ 2</td><td>≈ 8</td><td>≈ 1</td></tr>
</table>

**Table 4-3: Summary Table – Rock Drilling/Blasting**

| Summary Table | Rock Drilling/Blasting (Vertical method) | | | |
|---|---|---|---|---|
| | Marine (below MSL) | | Land (above MSL) | |
| Equipment | Jack-up drill and blast barge | Self propelled drill and blast vessel | Drill rigs on tracks for blasting | Excavator with hammer (no blasting) |
| | Suggested | Alternative | Suggested | Alternative |
| Total Surface ($m^2$) | 53,867 | 53,867 | 53,867 | 53,867 |
| Area Surface ($m^2$) | 37,065 | 37,065 | 20,970 | 20,970 |
| Percentage Surface | 70% | 70% | 30% | 30% |
| Total Volume ($m^3$) | 590,000 | 590,000 | 590,000 | 590,000 |
| Area Volume ($m^3$) | 236,000 | 236,000 | 354,000 | 354,000 |
| Percentage Volume | 40% | 40% | 60% | 60% |
| Pros | - low draft reach shallow areas<br>- good stability on high seas and currents<br>- can drill several holes per position | - highly productive in optimum conditions<br>- can drill 10 boreholes at once | - can drill up to LLWL if earth fill is executed up to level ≈ +8.00 m CD | - no blasting required |
| Cons | - depends on assisting equipment to move | - draft limited<br>- tidal variation may impede drilling operations<br>- difficult positioning in tight areas<br>- mobilization below 500,000 $m^3$ not worth | - needs road access to site<br>- needs leveled terrain<br>- will probably need fill in shallow areas | - not able to dismantle rocks without visibility (below water)<br>- able to dismantle weathered rock only<br>- needs extra excavator to move rocks away |
| Auxiliary Equipment | - tugs<br>- crew vessel | - multicat vessel<br>- crew vessel | | - extra excavator with bucket |
| Production rate ($m^3$/month) | 60,000 | 112,500 | 100,000 | 50,000 |
| Estimated schedule (months) | ≈ 4 | ≈ 3 | ≈ 4 | 8 |

**Table 4-4: Summary Table – Rock Dredging**

| Summary Table | Rock Dredging (Horizontal method) | |
|---|---|---|
| | Marine (below MSL) | Land (above MSL) |
| Equipment | Marine Backhoe Dredger (BHD) | Elevated BHD |
| | Suggested | Suggested |
| Total Surface (m²) | 58,035 | 58,035 |
| Area Surface (m²) | 37,065 | 20,970 |
| Percentage Surface | 70% | 30% |
| Total Volume (m³) | 590,000 | 590,000 |
| Area Volume (m³) | 236,000 | 354,000 |
| Percentage Volume | 40% | 60% |
| Pros | - high accuracy through GPS positioning system<br>- digs into hard soils up to 20 m water depths | - high accuracy through GPS positioning system<br>- can operate at water depths up to 5 m<br>- digs into hard soils up to 8-10 m below base line |
| Cons | - needs enough under keel clearance in order to allow safe water draft for dredging operations<br>- slow movement around the dredging area<br>- downtime due to high waves (>1.5 m) | - needs to put excavated material on the side for other equipment to collect or rework excavated material several times |
| Auxiliary Equipment | - multicat vessel<br>- self propelled split barge (for disposal) | - earth moving equipment (dozers) once rock is upland |
| Production rate (m³/month) | 37,500 | 60,000 |
| Estimated schedule (months) | ≈ 7 | ≈ 7 |

## 4.5 Phasing

### *4.5.1 Marine Sediment Dredging*

Marine sediment dredging equipment will have to be mobilized prior to D&B activities and rock dredging. It is recommended that the faster dredging method be selected in order to avoid delays in the schedule.

### 4.5.2 *Drilling and Blasting*

Drilling and blasting activities will be performed from the same equipment, either a jack-up barge with drilling equipment or an alternative equipment such as a specialized drill vessel.

Upland drilling and blasting activities can be executed simultaneous to marine drilling and blasting activities.

Equipment production will have to be assessed in more detail in order to determine the phasing of the works of the MOF dredging.

### 4.5.3 *Rock Dredging*

Since land-based excavator presents a higher production rate than marine BHD, ability to work up to water depths of 5 m and dig up to 10 m below its base line, it is possible that land-based excavator will dredge the rock volume from upland to around -8.00 m CD and the marine BHD will dredge the rock volume from deep waters up to final dredge level -12.00 m CD.

# 5. Conclusions and Recommendations

A more detailed geotechnical investigation should be carried out in the vicinity of the area of the proposed channel bench widening and further into the MOF area where the MOF is to be installed. The current level of information in the area is insufficient to make any meaningful estimates of the subsurface conditions or the stability. Investigations could consist of boreholes and Cone Penetration Testing (CPTs) and possibly expanding the geophysical survey to include the full bench area and possibly using a variable frequency approach to give a model of the sediment thickness.

Estimated upland rock volumes beyond MOF berth area (berth deck), where elevations exceed +24.00 m CD, could be very important and lead to much higher drilling/blasting and excavation activities increasing significantly the construction cost of MOF terminal. As discussed prior removal of rock volumes apart from the area depicted in Figure 1-3 is not part of the scope of this study.

Rock sample collection and lab testing could provide property of the phyllite/schist (i.e. GSI, joint properties, density, and etc), which could help optimize the rock excavation methods (i.e. underwater blasting, scaling, chipping, etc).

The following considerations are recommended for the next phases of this dredging study:

- Confirm the proposed dredging methodology, operational cycles and unit rates with contractors through Request For Proposal with Non Disclosure Agreement;
- Evaluate alternative sites for disposal of dredged material for this project;
- Review all regulations which will impact the schedule and timing of this work including Department of Fisheries and Oceans (DFO) and other local, provincial and federal authorities;
- Appraise the utilization of tides for optimizing dredging operations to help minimize impact of standby; and
- A sedimentation study can be conducted to assess the impact of turbidity due to blasting and BHD activities.