Appendix G.17
Hatch Report – Pacific NorthWest LNG
Lelu Island
LNG Terminal Marine Structure Scour

Pacific Northwest LNG
Lelu Island LNG Terminal

Marine Structures Scour

2014-12-11	1	Approved for Use	René Bérenger Roslin Arbuckle	Otavio Sayao	Otavio Sayao	
Date	**Rev.**	**Status**	**Prepared By**	**Checked By**	**Approved By**	**Approved By**
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1. Introduction

Hatch conducted a study of scour around the marine structures for the proposed PNW LNG Terminal (Figure 1-1). Scour is a local hydrodynamic effect in the seabed caused by the currents and waves interacting with underwater structures. In a dynamic seabed, upstream sediment is constantly replacing eroded sediment in the scour hole. The jetty design including bridge, trestle and berths is preliminary and conceptual and therefore likely to change over the course of the project.

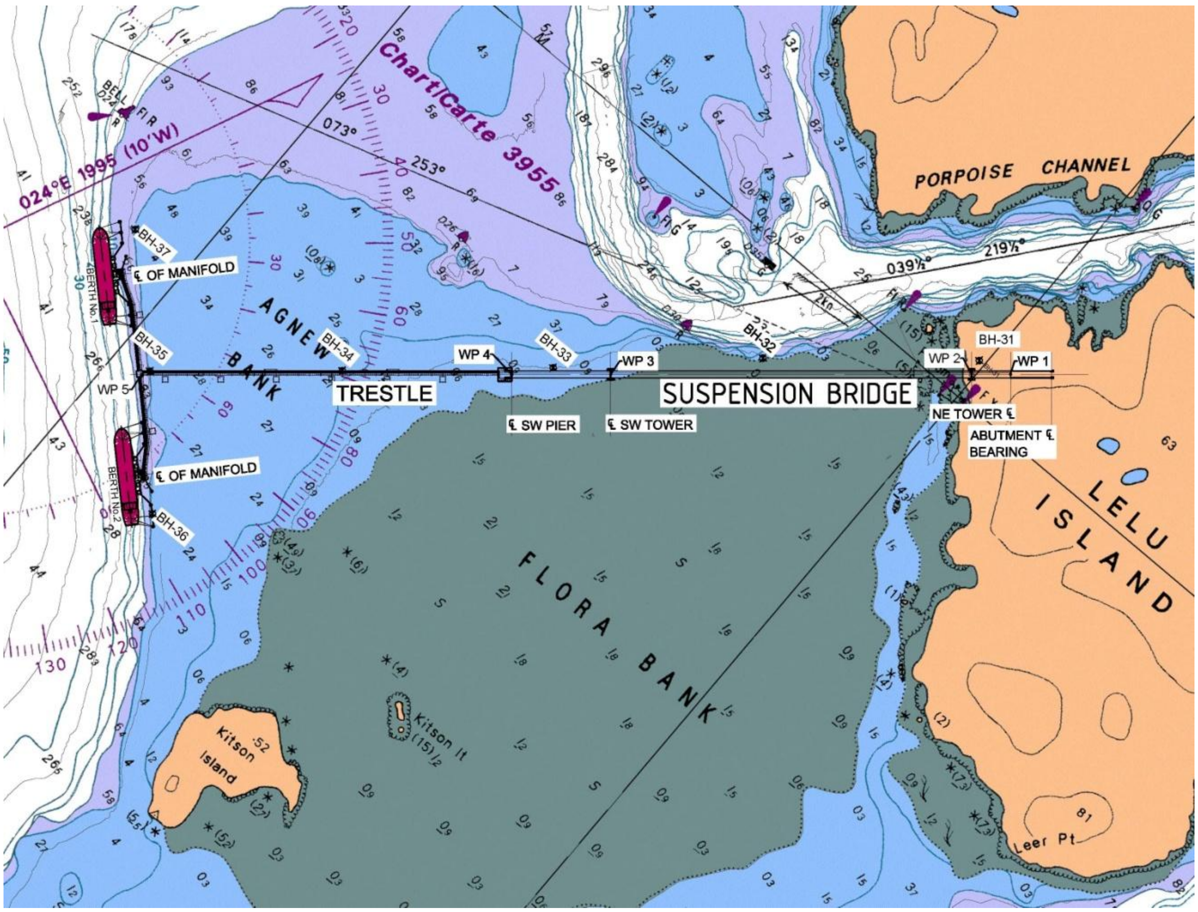

Figure 1-1: Proposed PNW LNG Marine Terminal Layout

- The bridge starting point, Work Point (WP) 1, is located on Lelu Island. The bridge extends seawards in a NE-SW direction, running adjacent to Flora Bank's NW flank and ending at WP 4. The trestle portion of the jetty extends from WP 4 to the jetty berth at WP 5.
- The coordinates of the working points for the LNG jetty structures are listed in Table 1-1.

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Table 1-1: Location of Work Points

Work Point	Associated Structures	Coordinates	
		Northing	Easting
WP 1	NE Abutment	6006718.81	415283.86
WP 2	NE tower	6006639.25	415186.21
WP 3	SW tower	6005900.24	414279.14
WP 4	SW anchor block	6005698.12	414031.06
WP 5	Jetty Head Platform	6004939.58	413100.02

- The potential impact of scour due to the proposed jetty and associated marine structures on Flora Bank is calculated to assess the following considerations:
- Local scour/global scour;
- Sediment characteristics: mud/sand;
- Pile arrangement: single pile/pile group;
- Hydrodynamic conditions: current and waves/current only; and
- Environmental mitigation measures including design modifications (scour protection).

2. Site Conditions

- The scour processes will be affected by the complex site conditions around Flora, Agnew and Horsey banks, the Skeena estuary and the various confined passages in the project area. The following physical environmental conditions were considered:
- Tide currents with opposite directions (ebb and flow) which interact with marine structures, with different direction, intensity and water depths, on a time scale of one tidal cycle (24 hours and 52 minutes long); tides are semi-diurnal (two high waters and two low waters each day);
- Wave action, mostly due to wind induced waves, and drift currents at the study area in water depths with tidal range of about 7.4 m;
- Wave and currents will interact over the marine structures according to the above mentioned processes;
- Bathymetry in the area of study is complex. The bridge is located from Lelu Island over Flora Bank to Agnew Bank. The trestle portion of the jetty is located through Agnew Bank to the berth located in the slope offshore of Agnew Bank; and
- Little information about sediment characteristics in the site is available but it is expected that cohesion and compaction are relevant.

Non-uniformity of the pile arrangements and interaction between structures was also considered;

The final objective of this study is to quantify the scour around the jetty and associated marine structures taking into consideration the above limitations, and to propose countermeasures to control scour.

2.1 Bathymetry and Topography

- Hydrographic survey data was collected by McElhanney Consulting Services Ltd (Contracted by KBR LLC) [40]. The survey was conducted August 2012.
- Bathymetric data was also collected from the Canadian Hydrographic Service (CHS) 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 was established from 1 m LiDAR contours produced by McElhanney Consulting Services [39].
- The jetty's starting point is located on Lelu Island, departing seawards on a NE-SW direction and running over the NW portion of Flora Bank and through Agnew Bank.
- Flora Bank is a relatively flat shoal with seabed level around +1.0 m Chart Datum (CD) sloping down towards NW.
- The trestle crosses Agnew Bank where the seabed level slopes gently down from +1.0 m CD to approximately -5.0 m CD, which represents a gradient of 0.25%.
- The berths area, at the extreme end of the trestle, is located at the southeast end of Agnew Bank where the seabed slopes rapidly down from -5.0 m CD to -50.0 m CD on a horizontal scale of 500 m, which represents a gradient of 10%.

2.2 Sediments

The study was developed considering Fugro geotechnical program described in detail in references [21] and [22].

The boreholes best located according to the jetty alignment are BH-32, BH-33, BH-34, BH-35, BH-36 and BH-37 as shown in Figure 1-1. The borehole descriptions are given below.

Borehole BH-32 (mid bridge area) first 10 meters of sediment are mainly soft with loose silty fine sand showing SPT's of 4 to 9, underlayed by loose sandy silt showing SPT's of 6 to 7 and soft to firm lean clay showing SPT's of 0 to 6. The surficial layer median diameter, D_{50}, is 0.038 mm.

Borehole BH-33 (SW anchor area) first 10 meters of sediment are consistently soft with very loose sandy silt showing SPT's of 1 to 3, underlayed by very loose silt showing no resistance, and interbedded with very loose silty fine sand and very soft sandy lean clay showing no resistance. The surficial layer median diameter, D_{50}, is 0.150 mm.

Borehole BH-34 (mid trestle area) generalized subsurface conditions at this location consists of about 6 m of loose silty sands and silts over 39 to 52 m thick under-to-normally consolidated lean and clay soils. Clay soils are underlayed by 5 m of glacial till soils over phyllite rock at -55m CD. The shear strengths in clay soils increased linearly from about 10 to 45 kPa. The glacial till soils are very stiff to hard.

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Borehole BH-35 and 37 (berth area at trestle alignment and berth 1 area respectively) generalized subsurface conditions at these locations consist of under-consolidated to normally-consolidated cohesive soils. Bedrock was encountered at about -82 m CD in Boring BH-37. Bed rock was not encountered at BH-35 within the drilled depth of 110 m. Shear strength of cohesive soils generally increased linearly from seafloor to about 40 kPa at a depth of about 45 m. At about 45 m below seafloor shear strength dropped to about 15 kPa and increased linearly with about the same slope to completion depth of the borings. The surficial layer median diameter, D_{50}, is 0.050 mm.

Borehole BH-36 (berth 2 area) subsurface conditions at this location consists generally of normally consolidated cohesive soils from seafloor to completion depth of the boring BH-36. The shear strength increased linearly from about 5 kPa at about 5 m below seafloor to 140 kPa at a depth of about 100 m below seafloor. Bedrock was not encountered within the drilled depth of BH-36. The surficial layer median diameter, D_{50}, is 0.085 mm.

MEG Particle Size Distribution data [54], present surficial sediment sample information at locations close to berth area. This information was used to cross checked with Fugro borehole information where available.

2.3 Tides and Water Levels

Tide levels near Lelu Island are established at the Port Edward station from Chart 3958 and CHS Canadian Tide and Current Tables [8]. The tidal range in this region is significant, with a variation in water elevation over 7 m (Table 2-1).

Table 2-1: Hydrographic Elevations 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 (Large Tide)	1.3
Lowest Normal Tide (Chart Datum)	0.0
Lower Low Water Level (Large Tide)	0.0

2.4 River Discharge Influence

The Skeena and Nass Rivers discharges have small influence on the hydrodynamic circulation around Agnew Bank, where the current pattern is mainly driven by the semi-diurnal macro tides.

River discharge influence is further described in Hatch's April 25, 2014 Project Memo[24].

2.5 Tidal 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) [26].

Tidal current circulation in the study area is presented in Figure 2-1 and Figure 2-2 below. These figures also show 5 selected points, three along the jetty (T1, T2, T3 and T4) and two in front of Berths (B1 and B2).

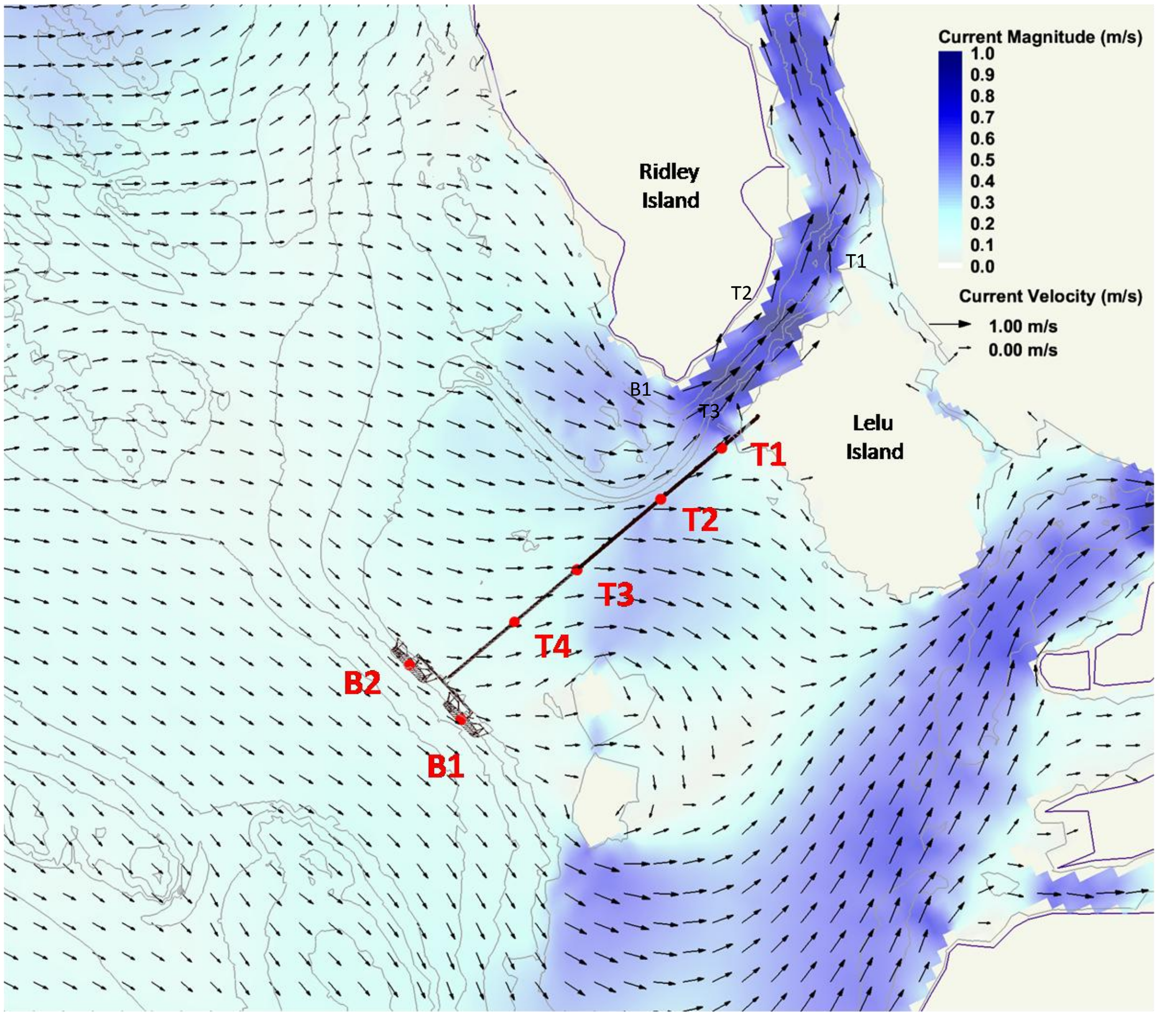

Figure 2-1: Maximum Spring Flood Currents (Depth Average)

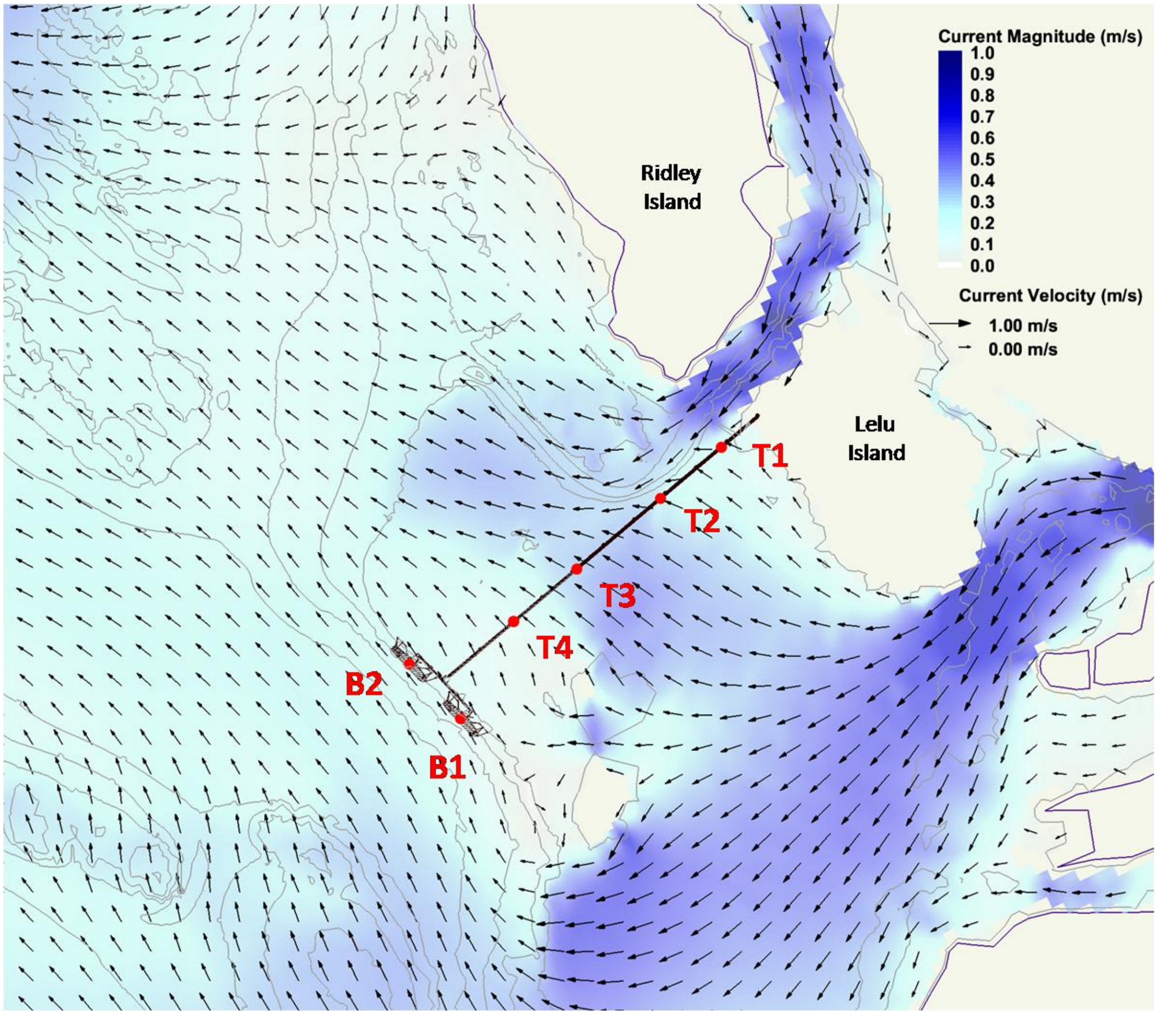

Figure 2-2: Maximum Spring Ebb Currents (Depth Average)

The current velocities magnitude from the maximum flood and maximum ebb during the simulation period are presented on Table 2-2.

Table 2-2: Current Magnitude (m/s)

Tide Level	Depth Averaged Current Magnitude (m/s)					
	T1	T2	T3	T4	B1	B2
Maximum Spring Flood	0.30	0.35	0.27	0.16	0.14	0.16
Mean Spring Flood	0.16	0.23	0.18	0.12	0.10	0.10
Maximum Spring Ebb	0.23	0.35	0.41	0.21	0.20	0.24
Mean Spring Ebb	0.17	0.29	0.29	0.13	0.09	0.11

2.6 Waves

Wave characteristics at the area of study are presented in the significant wave roses and wave statistics table below (Figure 2-3; Figure 2-4; and Table 2-3). The significant wave height (Hs) is defined as the mean wave height (trough to crest) of the highest third of the waves. The peak wave period (Tp) is defined as the duration of time between wave troughs or crests, with the highest energy.

These wave statistics were developed with Hatch's numerical wave model of the area [27]. Input data from Holland Rock and the National Oceanic and Atmospheric Administration (NOAA)'s hindcast model Wave Watch III were used to set up the model. Hatch's wave model was calibrated using PNW LNG's WatchMate Buoy deployed at the project site in December 2013. The offshore WW3 input was also compared with Department of Fisheries and Ocean (DFO)'s buoy from West Dixon Entrance (St 46205).

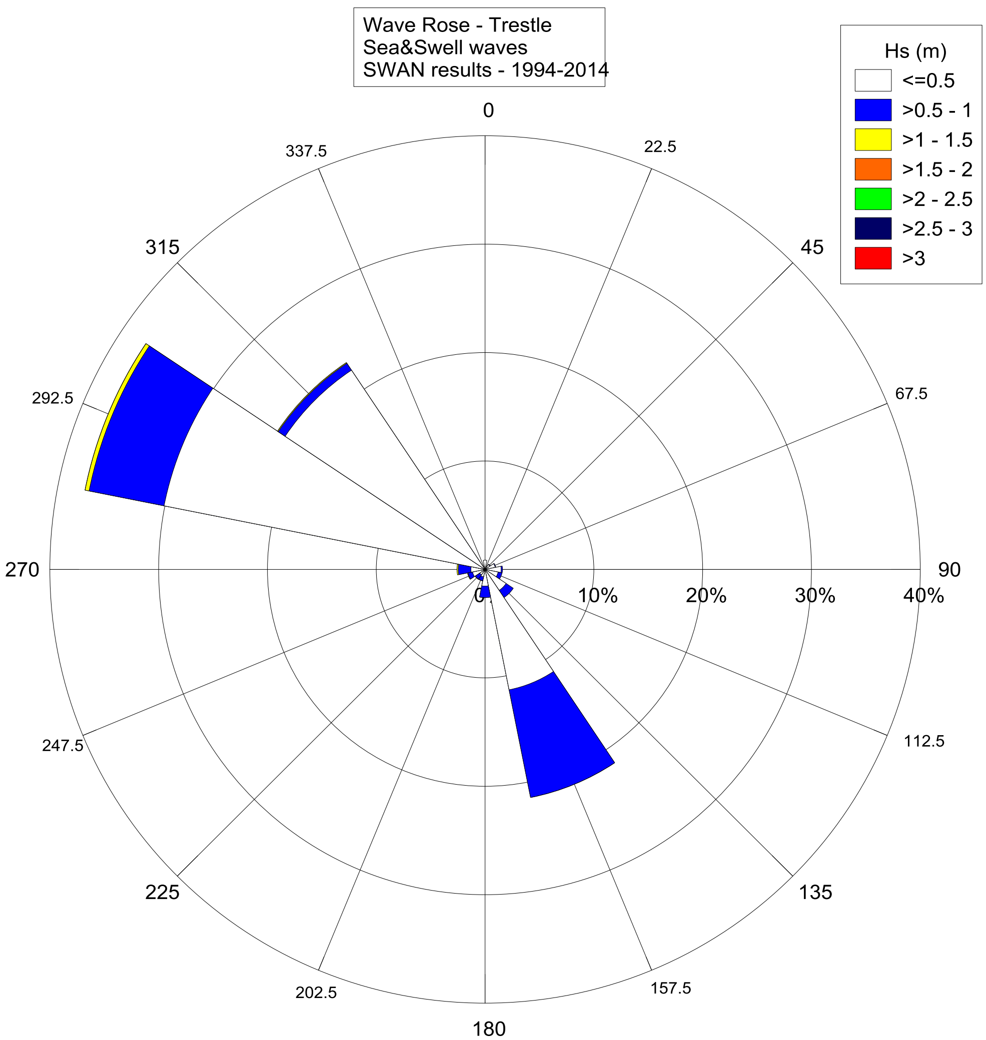

Figure 2-3: Wave Rose from Wave Modelling Results at Mid-Jetty [27]

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(UTM E: 414489, N: 6006039)

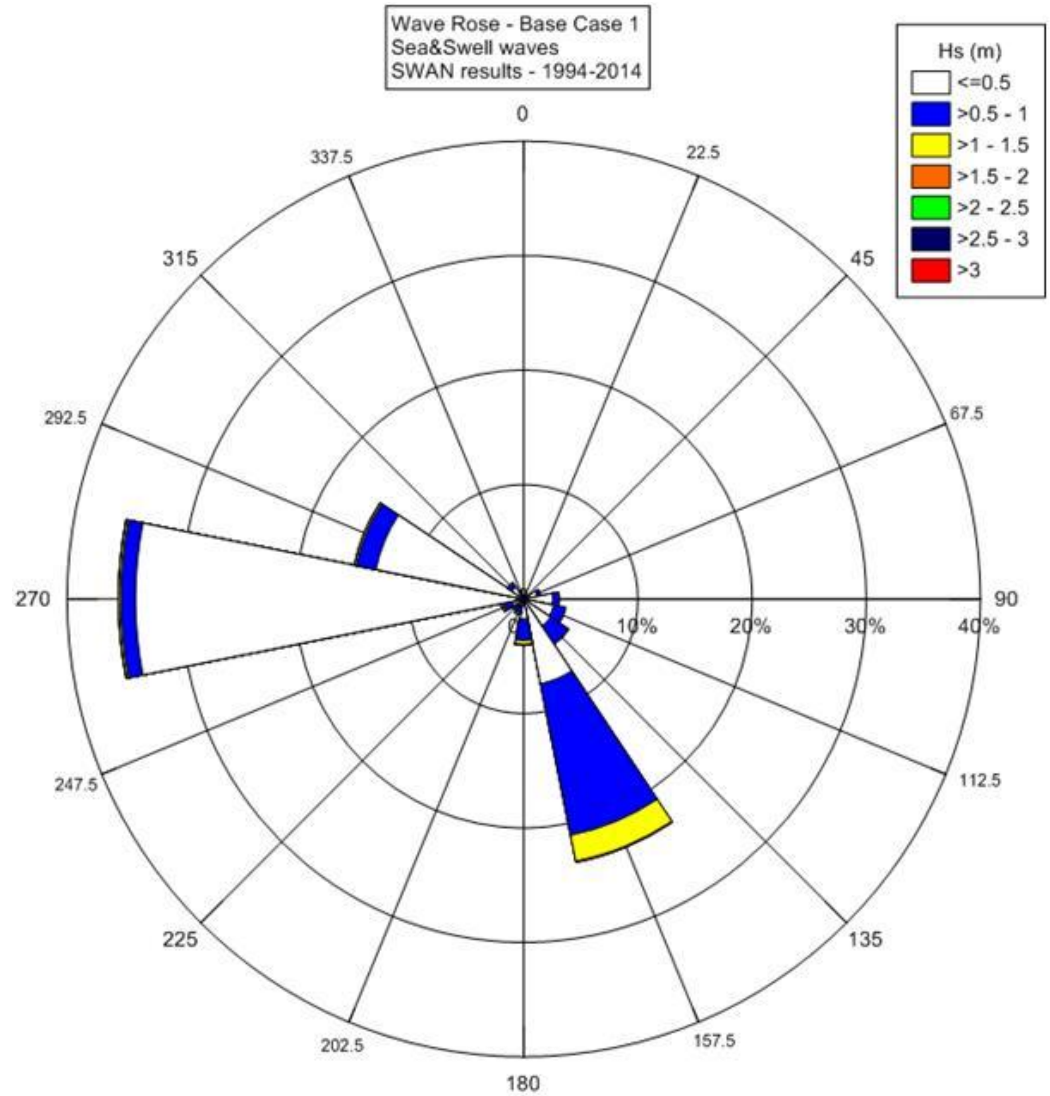

Figure 2-4: Wave Rose from Wave Modelling Results at Berth 1 [27]
(UTM E: 413694, N: 6005701)

Table 2-3: Wave Characteristics at the Area of Study

Wave statistics		Berth 1	Mid-Jetty
Storm	Hs (m)	2.6	1.6
	Tp (s)	6.6	6.2
Mean	Hs (m)	0.4	0.4
	Tp (s)	6.1	7.0

The local wave height data (Berth1 and Mid-Jetty) was extracted from the wave transformation modelling for a 20 year period (1994 to 2014) [27].

3. PNW LNG Trestle and Berth

3.1 Trestle and Berth Arrangement

The PNW LNG trestle portion of the jetty starts from the SW anchor block of the bridge (WP 4). The approximately 1300 m long trestle extends through Agnew Bank from WP 4 to WP 5 and intersects with the jetty berths. The berth section is located in the slope of Agnew Bank which slopes quickly from approximately -5.0 m to -50.0 m. A plan and profile view of the preliminary trestle and berth design are shown in Figure 3-1 and Figure 3-2.

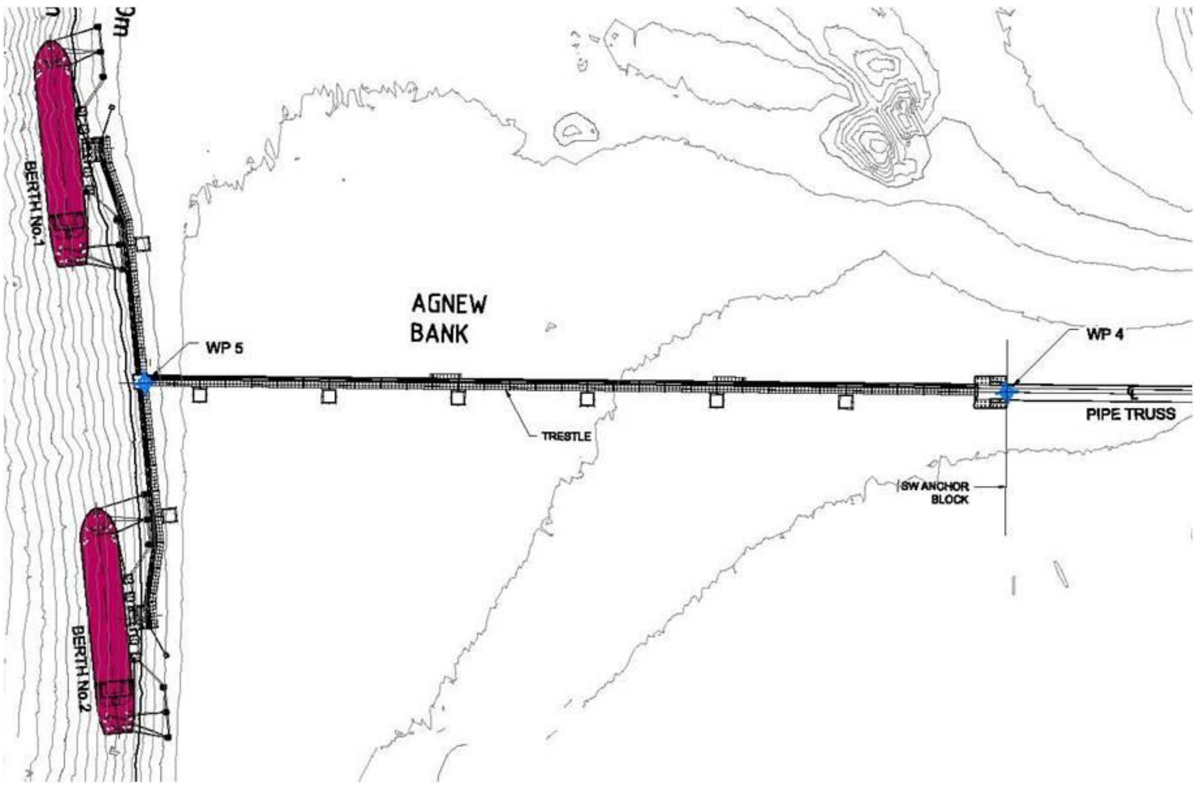

Figure 3-1: Jetty Trestle Plan View [24]

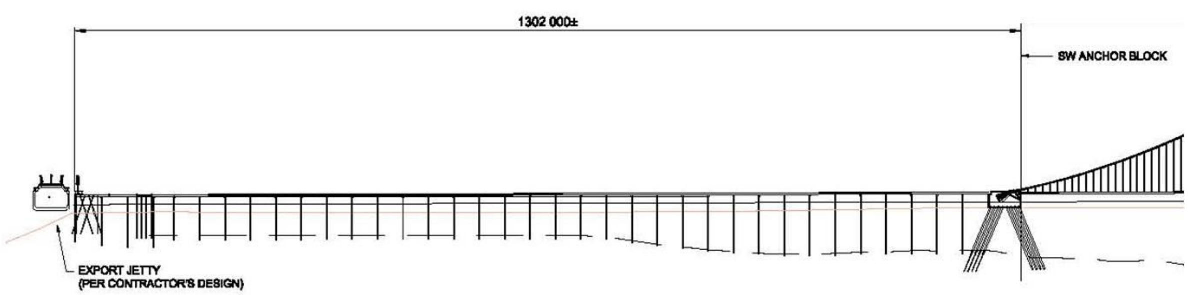

Figure 3-2: Jetty Trestle Profile View [24]

The PNW LNG trestle and berth structures that could lead to potential scour are described below.

The trestle layout comprises a total length of approximately 1300 m, where the following structures can be found:

- Trestle bents at intervals of 35 m;
- Anchor bents at intervals of at least 105 m;
- Expansion loops at intervals of 175 m; and
- Vehicle pullout bays at intervals of 385 m.

The berths layout comprises a total length of approximately 655 m, where the following structures can be found:

- Berth bents;
- Jetty head platform
- Loading platforms;
- Berthing dolphins; and
- Mooring dolphins.

As per Hatch preliminary's drawings and the FEED contractor's drawings, all piles from the trestle to the berth structures are battered.

Table 3-1 summarizes the preliminary number of piles and pile characteristics per typical structure.

A geometric analysis of the pile arrangement was conducted, taking into consideration the pile angles and distances from pile cap to seabed, to understand the layout projection of the batter piles on the seabed. This analysis helps define whether or not there are pile groups (global scour) according to the scour literature definitions.

Most of the trestle structures present the piles projection on the seabed around -2.00 m CD and the top of pile located around +9.40 m CD. Considering that the height from seabed to pile cap is approximately 12 m and batter pile angles for most of marine structures is 4V:1H it is expected that the piles will project 3 meters away from pile centerline at the pile cap.

The figures below (Figure 3-3 to Figure 3-6) represent the preliminary pile arrangement per typical trestle structure, described in Table 3-1, and their pile projection on the sea bed.

Table 3-1: Trestle and Berth Structures

Number of Structures	Structure	Number of Piles	Total Piles	Batter Angle	Pile Diameter
16	Trestle Bent	4	64	4V:1H	1219 mm
16	Anchor Bent	4	64	4V:1H	1219 mm
8	Expansion Loop	8	64	5.7V:1H	1219 mm
4	Vehicle Pullout Bay	5	20	4V:1H	1219 mm
1	Jetty Head Platform	16	16	4V:1H	1219 mm
2	Loading Platform	36	72	4V:1H	1219 mm
8	Berthing Dolphin	8	64	4V:1H	1219 mm
12	Mooring Dolphin	7	84	4V:1H	1219 mm

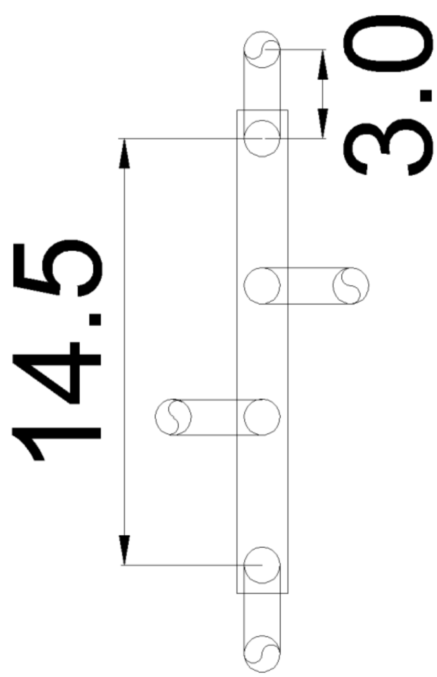

Figure 3-3: Trestle Bent (dimensions in meters)

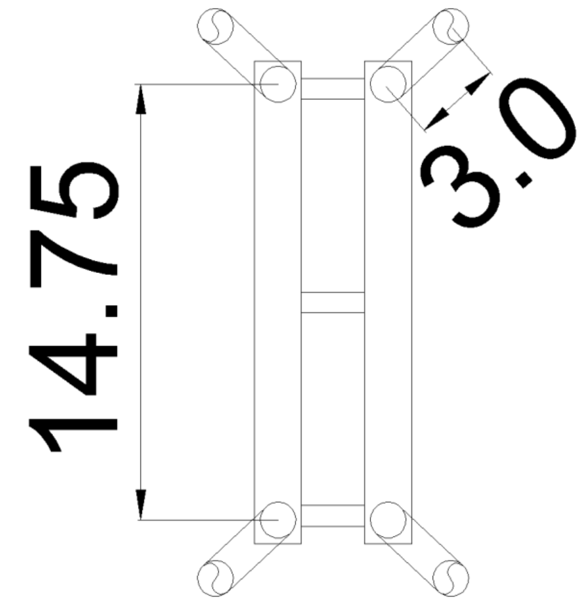

Figure 3-4: Typical Trestle Anchor Bent (dimensions in meters)

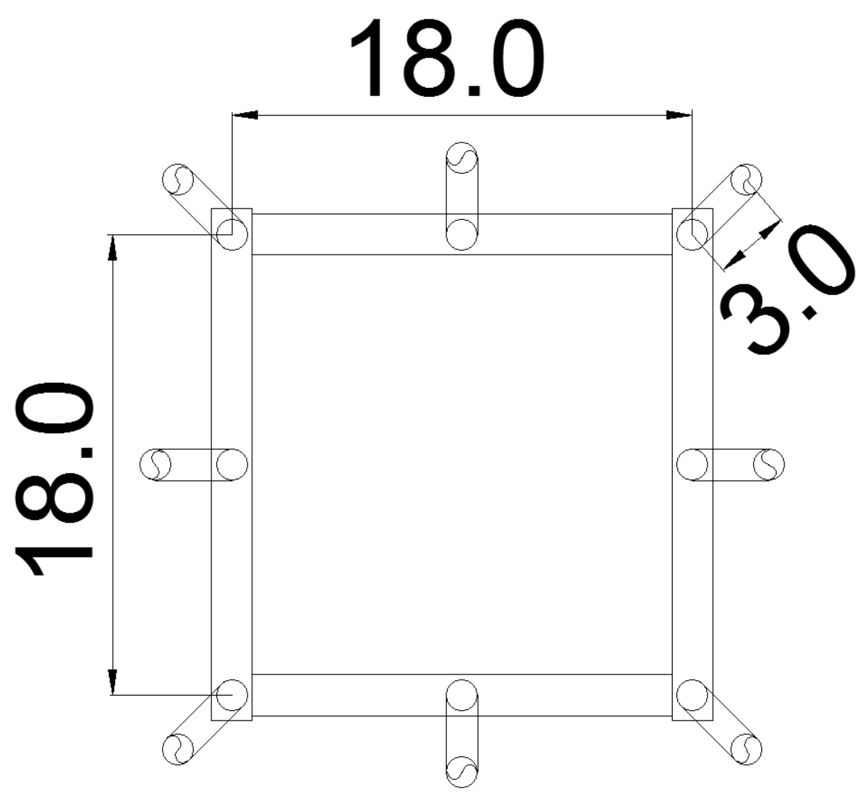

Figure 3-5: Typical Expansion Loop (dimensions in meters)

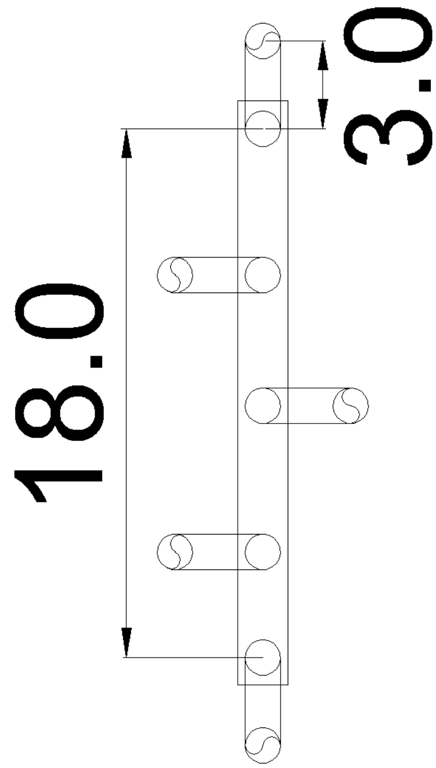

Figure 3-6: Typical Trestle Bent for Vehicle Pull Out Bay (dimensions in meters)

The berth structures are located below the -10.00 m CD bathymetric contour and the top of piles at +9.40 m CD. Considering the height from the seabed to the pile cap is approximately 20 m and the batter pile angles for most of marine structures is 4V:1H, it is expected that the piles will project 5 meters away from pile centerline at the pile cap.

The figures below (Figure 3-7 to Figure 3-9) represent the preliminary pile arrangement per typical berth structure and their pile projection on the sea bed.

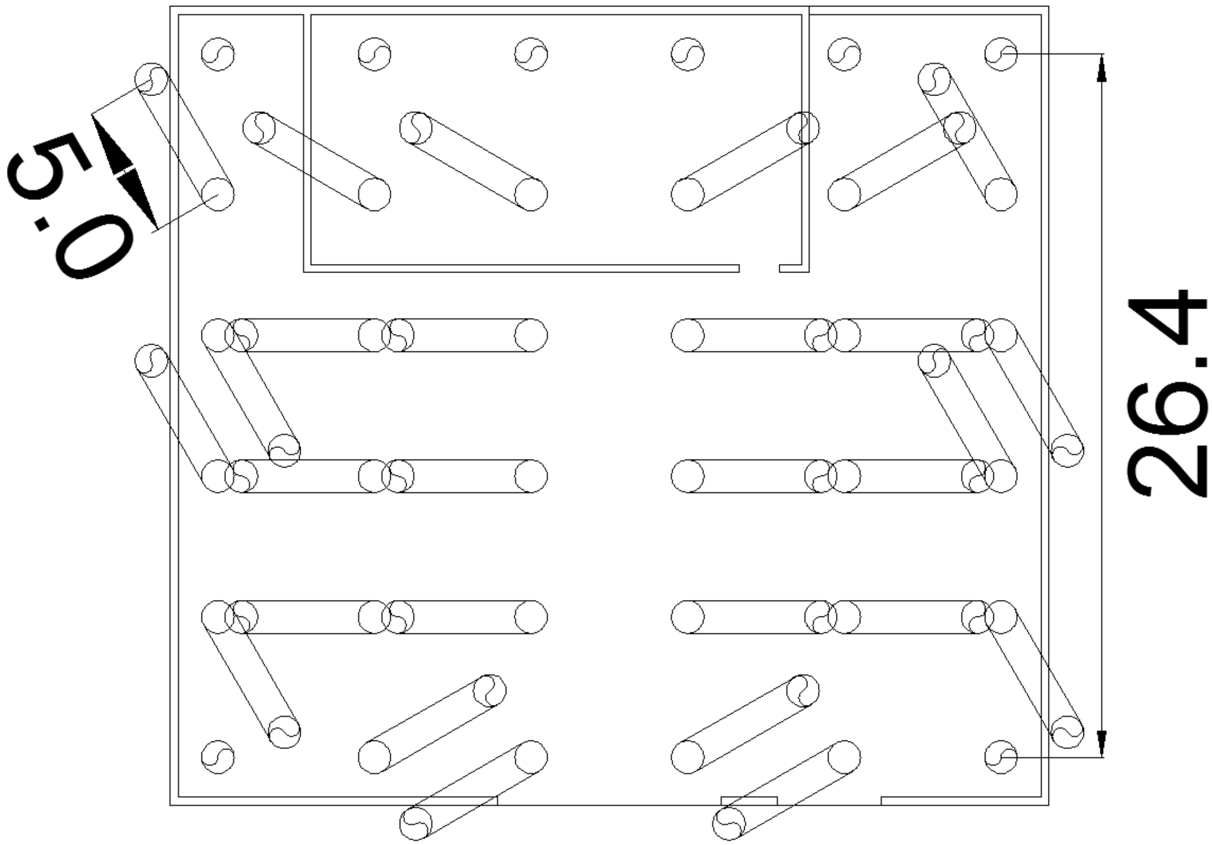

Figure 3-7: Typical Loading Platform (dimensions in meters)

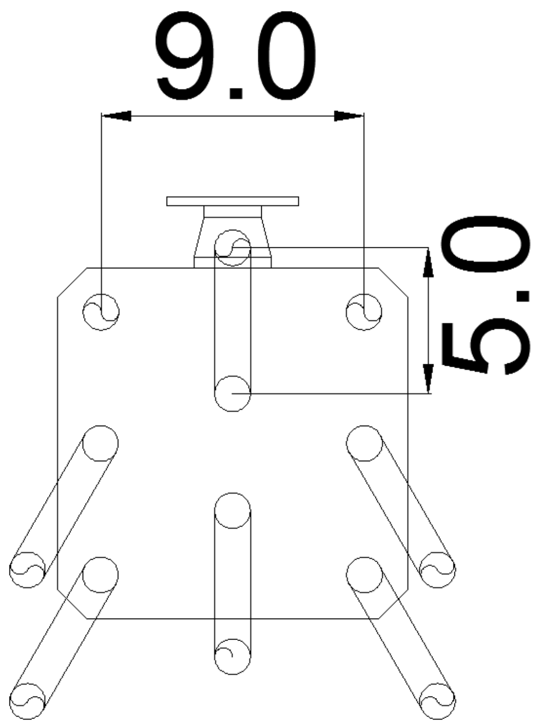

Figure 3-8: Typical Berthing Dolphin (dimensions in meters)

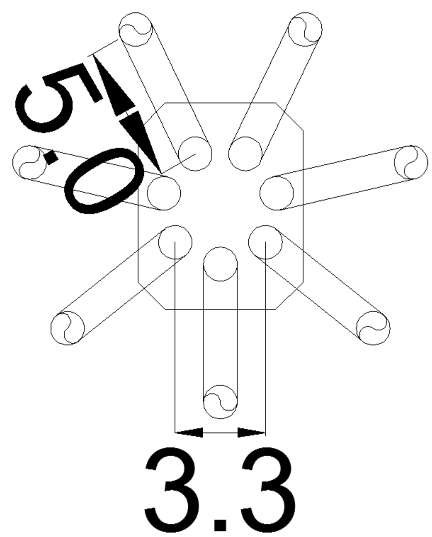

Figure 3-9: Typical Mooring Dolphin (dimensions in meters)

3.2 Scour Assessment

The scour around the piled structures will be assessed based on the occurrence of local or global scour.

Local scour can be defined as degradation of sea bed that is localized to a specific area due to a sudden change in the parameters associated mostly to the placement of a structure.

Global scour is accounted when a group of two or more piles increase the flow pattern between themselves. In the case of the trestle and jetty, global scour occurs when piles are in close enough in proximity that the potential scour for one pile will affect the scour of another nearby pile.

3.2.1 Local Scour

For the purpose of estimating the local scour depth in the study area, an analysis considering the literature methods applicable to the specific site conditions and an empirical formulation described in Appendix A were utilized. It is possible to conclude based on the information presented in Appendix A, that the correlation of the site conditions contribute to the reduction of scour effect over single piles.

Hatch's local scour adopted an attenuation factor of 0.5 to be applied over the empirical approach results to compensate for the positive effect of the site conditions that tend to decrease scour depths.

Table 3-2 summarizes the scour attenuation factor as a result of Hatch's local scour interpretation for the particular characteristics of Agnew Bank. Local scour depth rule of thumb, maximum scour depth is equal to about twice the pile diameter [12], was used as a base case.

Table 3-2: Scour Depth Factors

	Local Scour Depth	Reduction %
Steady Currents in sand	2*D	Base Case
Steady Currents in Soft clay	1*D	50%
Regular waves in sand	0.6*D	70%
Waves and Currents in sand	1.4*D	30%
Countermeasure Cable wrap	1.1*D	46.3%
Countermeasure splitter plate	0.4*D	61.6%
Hatch adopted attenuation factor	1*D	50%

3.2.1.1 Local Scour Analysis

An empirical formulation described in Appendix A was adopted for estimating local scour depth in the area of the trestle. A literature review applicable to the specific site conditions was also carried out.

Based on Breusers, 1977 [5] the scour depth varies two times the pile diameter up to about 3 m depth. In shallower waters less than 3 m, the ratio of scour depth to pile diameter decreases.

The scour volume at equilibrium state has also been calculated for a single pile according to the estimated scour depth and considering that the scour slope angle is equal to 45°.

The flow values have been determined as the maximum average current magnitude along the water column from 13 days simulation with CMS Flow hydrodynamic model using a spring tide scenario between the high and low water periods.

Hatch's scour analysis suggests that a reduction factor of 50% be applied over the empirical approach to compensate the positive effect of site conditions present in the study area, which tends to decrease the scour depth.

As a simplification, scour volumes have been calculated based on the solids of revolution method. In this method the scour volume equals a cone section volume minus the pile volume as depicted in Figure 3-10 below. The associated formula to calculate the scour volume is presented as:

$V=((\pi\ dS)/3)*(R^2 + Rr + r^2) - (\pi r^2 dS)$

where:

V is the scour volume

r is the pile radius (D/2)

R is the pile radius plus scour depth ($D/2 + d_S$)

H is equal scour depth (d_S)

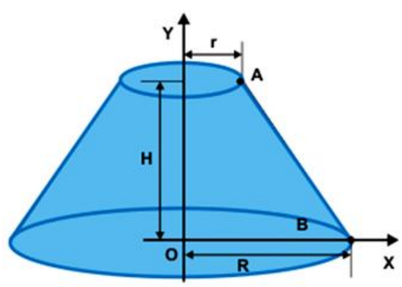

Figure 3-10: Scour Volume Calculation by the Solids of Revolution Method

Table 3-3 summarises the estimates for scour depth and volume for a single pile from the application of the empirical method.

Table 3-4 presents the interpretation results of local scour estimate for this particular site as described in Table 3.2.

Table 3-3: Single Pile Scour Geometry and Volume

Depth (m)	Current Magnitude (m/s)	Pile Diameter (m)	Scour Depth (m)	Scour Volume (m^3)
19	0.1	1.22	2.44	26.62
8.7	0.26	1.22	2.44	26.62
4.6	0.43	1.22	2.44	26.55
3.2	0.43	1.22	2.41	25.91
2.8	0.32	1.22	2.39	25.27
0.9	0.44	1.22	1.53	8.26

Using as input parameters the values shown in Table 3-3, the scour depth varies with water depth approximately two times the pile diameter up to about 3 m depth. In shallower waters less than 3 m, the ratio of scour depth to pile diameter decreases.

As a result, a scour depth at equilibrium state in the order of 1.25 m (approximately one pile diameter) is expected to occur throughout the trestle structures. The extent of the horizontal scour width is expected to be in the same order of magnitude.

These scour estimates of depths and volumes should be further investigated using a physical model study.

Table 3-4: Single Pile Final Estimates of Scour Geometry and Volume

Depth (m)	Pile Diameter (m)	Scour Depth Empirical Approach (m)	Scour volume Empirical Approach (m^3)	Scour Depth Final Analysis (m)	Scour Volume Final Analysis (m^3)
19	1.22	2.44	26.62	1.22	4.75
8.7	1.22	2.44	26.62	1.22	4.75
4.6	1.22	2.44	26.55	1.22	4.74
3.2	1.22	2.41	25.91	1.20	4.64
2.8	1.22	2.39	25.27	1.19	4.53
0.9	1.22	1.53	8.26	0.76	1.59

3.2.2 Global Scour

The site conditions considered for the local scour analysis were also used to calculate the occurrence of global scour around the trestle and berth piles.

The piled marine structures in the area of interest along the trestle are to be considered as ‘non-uniform pile groups’ since the direction angle of currents and waves are constantly changing.

The relevant literature research results that could be applied when assessing global scour and group of piles with these aforementioned site conditions are presented in Appendix A.

The pile group criteria adopted for this study is as conservative as the criteria suggested by technical literature. A particular pile will influence its surroundings with a pile spacing (S)-pile diameter (D) ratio of $S/D < 3$ [1]. For the purpose of this study the adopted criteria will be $S/D < 5$. The overestimation for the pile group criteria was selected due to the uncertainty of the pile arrangement especially around the berth area and constant change in flow direction due to tidal variation.

The scour depth results calculated at equilibrium state for local scour are also expected for global scour. The combination of scour processes in a global scour condition can be extended horizontally depending on pile arrangement, mainly on the inside of the group of piles.

As a result, from a geometric analysis it is possible to identify the places where the trestle and other marine structures present a group of piles where global scour will occur. The locations where global scour systems can be considered are listed below and further depicted in Figure 3-11, Figure 3-12 and Figure 3-13:

- All trestle expansion loops contact point with trestle anchor bents;
- Berthing platforms; and
- Berthing dolphins.

The solid circles represent the local scour projection (1.25 m from pile wall) whereas the dashed circles represent the pile group criteria that corresponds to 5 times the pile diameter. Global scour analysis is also identified.

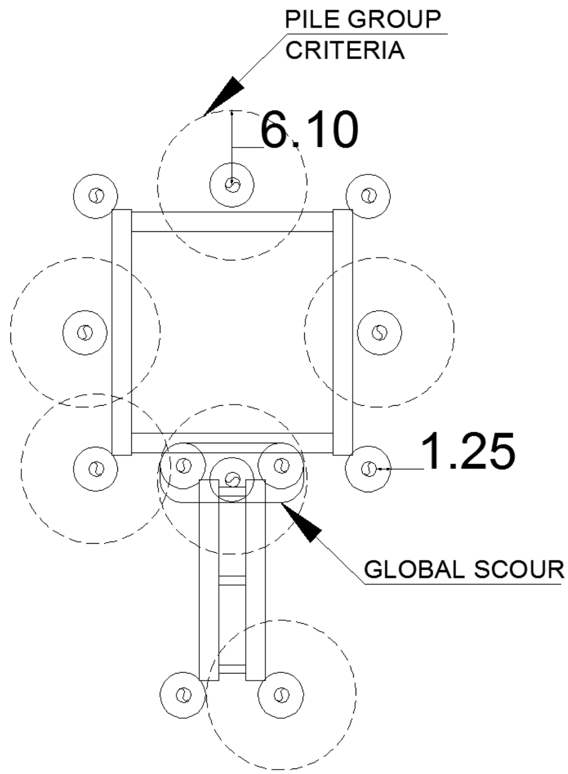

Figure 3-11: Anchor Bent Adjacent to Expansion Loop Pile Projection (dimensions in meters)

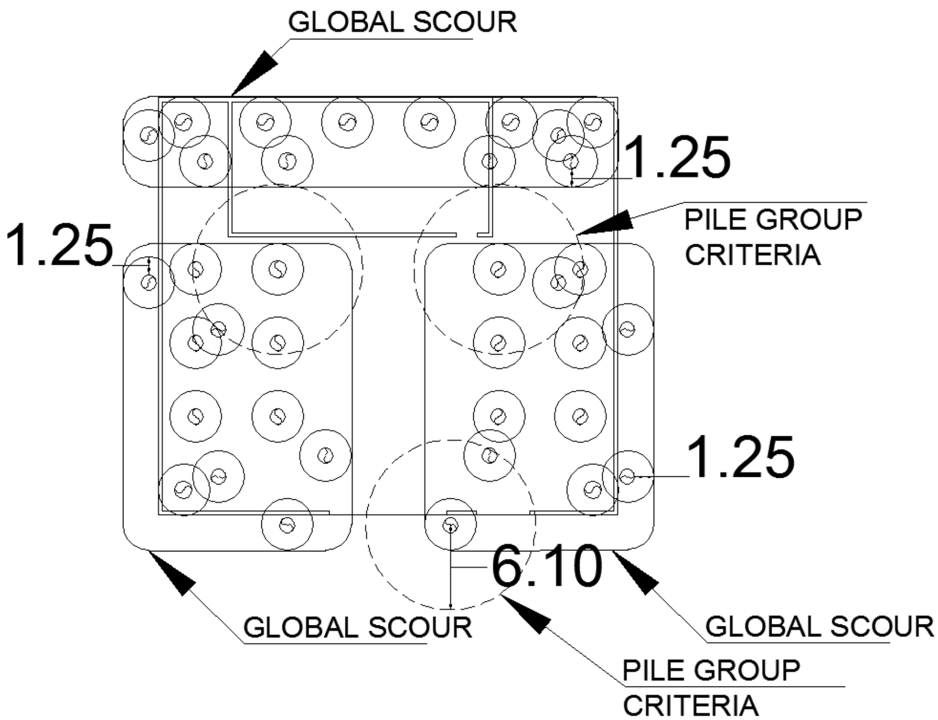

Figure 3-12: Berthing Platform Pile Projection (dimensions in meters)

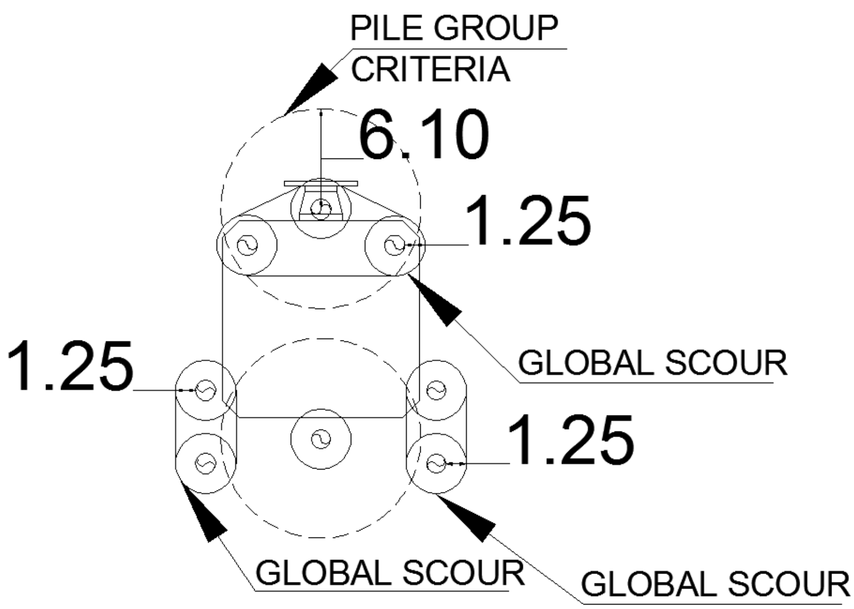

Figure 3-13: Berthing Dolphin Pile Projection (dimensions in meters)

3.2.3 Scour Volumes

Table 3-5 presents the scour volume results for each trestle and berth structure. Scour volumes for trestle structures were calculated for a water depth around -2.00 m CD and berth structures around -10.00 m CD. The scour volumes and countermeasure volumes were calculated based on a flat seabed. The effect of a sloping seabed would be better evaluated with a physical model discussed in Section 6.2.

The total expected scour volume at equilibrium state for the trestle and berth structures is approximately 2,100 m^3. These volumes are considering local scour only. The potential extra scour volume due to group scour was not yet predicted. As discussed in Section 6.2 it is best analysed with hydraulic physical modelling.

Table 3-5: Total Scour for Trestle and Berth Structures

Number of Structures	Structure	Number of Piles	Scour Volume per structure (m^3)	Total Scour Volume (m^3)
16	Bent	4	18.11	289.79
16	Anchor Bent	4	18.11	253.57
8	Expansion Loop	8	36.22	253.57
4	Pullout Bent	5	45.28	181.12
1	Jetty Head Platform	16	76.06	76.06
2	Loading Platform	36	171.14	342.28
8	Berthing Dolphin	8	38.03	304.25
12	Mooring Dolphin	7	33.28	399.32

3.3 Design of Scour Mitigation Measures

The scour protection, for both local and global scour, shall cover an area to ensure both the functional integrity of the scour protection and the protected structure. Scour outside the scour protected area is allowed as long as it does not jeopardize the stability of the protected structures.

Scour countermeasures can be generally categorized into two groups:

- armouring countermeasures; and
- flow altering countermeasures.

Deng and Cai, 2010 [11] compared the two types of scour countermeasures. Disadvantages for flow altering countermeasures include special design for particular site conditions and significant cost and construction of new structures. The disadvantages for armoring countermeasures include winnowing of sands through the armour, difficult to keep the armour in place and constriction causing additional scour.

Lauchlan and Melville, 2001 [38] concluded that the most effective armouring countermeasure is riprap.

Lagasse et al., 2001 [36] recommended placing the riprap layer at a depth below the average bed level.

For the purpose of this study the proposed scour protection consists of quarry stone protection.

Thickness of the scour protections shall be sufficient to:

- Mitigate edge scour by applying the falling apron principle;
- Prevent winnowing of sand out from between the scour protection material; and
- Allow natural horizontal movements of individual stones without locally reducing the scour protection layer.

The minimum design requirement for hydraulic stability of the scour protection shall follow some safety principles. It is suggested that the scour protection remains functional. Scour protection in the form of quarry run shall be allowed to launch in response to edge scour. A riprap scour protection with minimum thickness of about twice the median diameter is sufficient to prevent winnowing out of seabed material.

The proposed scour protection for trestle and berth structures consists of quarry stone protection covering a longitudinal extension of local and group scour as exemplified in Figure 3-11 through Figure 3-13.

3.3.1 Armouring Countermeasure Design

In order to reduce the scour around the PNW LNG structures, countermeasures are required. For the purposes of this study a riprap protection was selected as the preferred scour countermeasure since it is the most common methodology. Based on the wave and current conditions, a minimum size of a stable riprap stone gradation was selected.

The scour protection riprap will consist of quarried stone with selected gradation and no fines. The minimum stone size is estimated to be D_{min} = 0.16 m (with equivalent minimum mass M_{min} = 10 kg). The gradation of the riprap stone is shown in Table 3-6, in terms of stone diameters defined by D_{10}, D_{50} (median diameter) and D_{90}. Figure 3-14 shows the equivalent gradation in terms of stone mass.

Table 3-6: Scour Protection Riprap Sizing

	D (m)	M (kg)
D_{min}	0.16	10
D_{10}	0.21	25
D_{50}	0.39	159
D_{90}	0.57	500

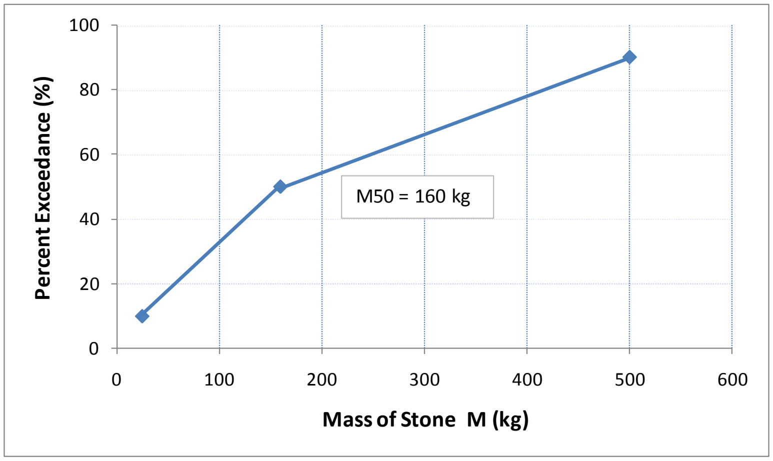

Figure 3-14: Scour Protection Riprap Mass Gradation

Two layers of riprap will be used for a thickness of 0.8 m (2*D_{50}). As will be discussed further in Section 6, settlement of this material will likely occur. The riprap around the structures foundations should be monitored and maintained as required.

A geotextile fabric should also be considered between the seabed and the riprap to prevent winnowing of sediment and armour sinking.

Table 3-7 presents the riprap volume to be installed over the seabed at the toe of a single pile. Figure 3-15 shows a 3D rendering of scour protection around a single pile.

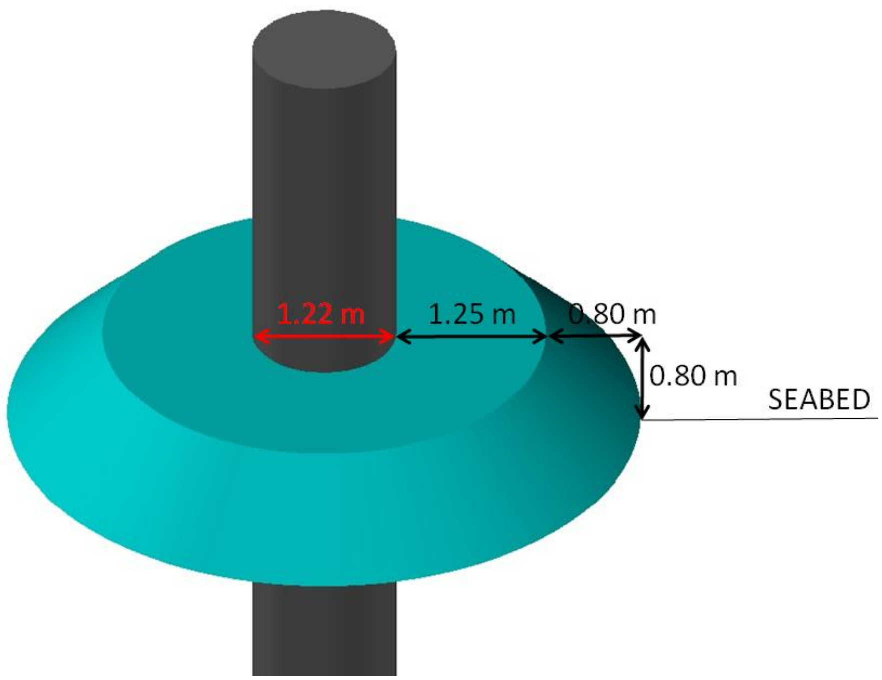

Figure 3-15: 3D Rendering Single Pile Scour Protection

Table 3-7: Scour Protection Volume - Single Pile

Scour Protection Height (m)	Scour Protection Crest Width (m)	Scour Protection Volume (m³)	Scour Protection Footprint (m²)	
			With Piles	Without Piles
0.8	1.25	12.0	22.2	21.0

Scour protection value demonstrated above corresponds to armour area only excluding the structure.

For piles where only local scour is present, the scour protection footprint with and without the pile cross-section is displayed in Table 3-8. The trestle/jetty design is conceptual and will be designed by the contractor, however the number of piles and configuration will likely not vary significantly and these values can be used as an estimate of the scour protection footprint.

Table 3-8: Trestle and Berth Local Scour Protection Footprint

Number of Structures	Structure	Number of Piles per Structure	Total Piles	Piles with Local Scour Only	Scour Protection Footprint (m^2)	
					With Piles	Without Piles
16	Trestle Bent	4	64	64	1421	1344
16	Anchor Bent	4	64	48	1066	1008
8	Expansion Loop	8	64	56	1243	1176
4	Vehicle Pullout Bay	5	20	20	444	420
1	Jetty Head Platform	16	16	16	355	336
2	Loading Platform	36	72	0	0	0
8	Berthing Dolphin	8	64	8	178	168
12	Mooring Dolphin	7	84	84	1865	1764
Total Scour					6571 m^2	6216 m^2

As described above, global scour is expected to occur at three different pile configurations along the jetty. The scour protection footprint with and without the pile cross-section for global scour is displayed in Table 3-9.

Table 3-9: Trestle and Berth Global Scour Protection Footprint

Structure	Number of Structures	Piles with Global Scour per Structure	Scour Protection Footprint (m^2)	
			With Piles	Without Piles
Loading Platform	2	36	2400	2316
Berthing Dolphin	8	7	1519	1454
Expansion Loop	8	2/1	634	606
Total Scour			4554 m^2	4376 m^2

The total area of scour protection, without the piles, is 10,600 m^2. The total footprint of scour protection including the piles is 11,150 m^2.

3.3.1.1 Riprap Toe Scour

Toe scour around the riprap scour protection is expected in the order of 0.3 to 0.5 m. Once toe scour develops, the riprap apron will fall and accommodate into the toe scour hole. This process will develop until it reaches equilibrium.

4. PNW LNG Bridge

4.1 Bridge Arrangement

The PNW LNG suspension bridge consists of an 1170 m main span above the Flora Bank with a vertical clearance of 11.3 m above Highest High Water Level (+7.40 m CD) between the SW tower and NE tower. The suspension cables are anchored at the NE side onshore on Lelu Island, 260 m from the NE tower. The suspension cables to the SW end are anchored at the SW anchor block located in Agnew Bank, 320 m from the SW tower (Figure 4-1).

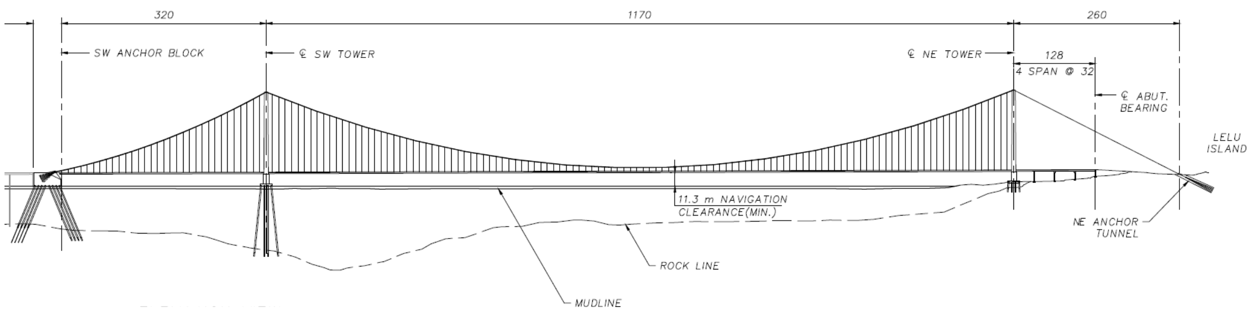

Figure 4-1: Jetty Bridge Profile [29]

The PNW LNG bridge structures that could lead to potential scour are further described below:

- SW bridge tower (WP 3); and
- SW anchor block (WP 4).

The bridge towers are composed of:

- Cast-in-place concrete tower base (bridge pier);
- Concrete tower foundation (pile cap); and
- Concrete filled steel pipe piles.

The SW anchor block is composed of:

- Concrete anchor block; and
- Concrete filled steel piles.

According to the designer's bridge drawings [29], the NE tower is located on land, above HHWL. Thus, no scour is expected at the NE tower.

Table 4-1 summarizes the preliminary number of piles and pile characteristics per typical bridge structure. Seabed elevations were determined by creating a longitudinal profile of the available bathymetric data along the bridge/trestle alignment.

Table 4-1: Trestle Structure Characteristics

Structure	Number of Piles	Batter Angle	Pile Diameter	Seabed Elevation
SW Tower	4x7 = 28	10V:1H	1800 mm	-0.35 m CD
SW Anchor Block	8x8 = 64	10V:1H	1800 mm	-0.73 m CD

4.2 SW Tower

The SW tower preliminary sections and foundation pile arrangement plans are shown in Figure 4-2 and Figure 4-3.

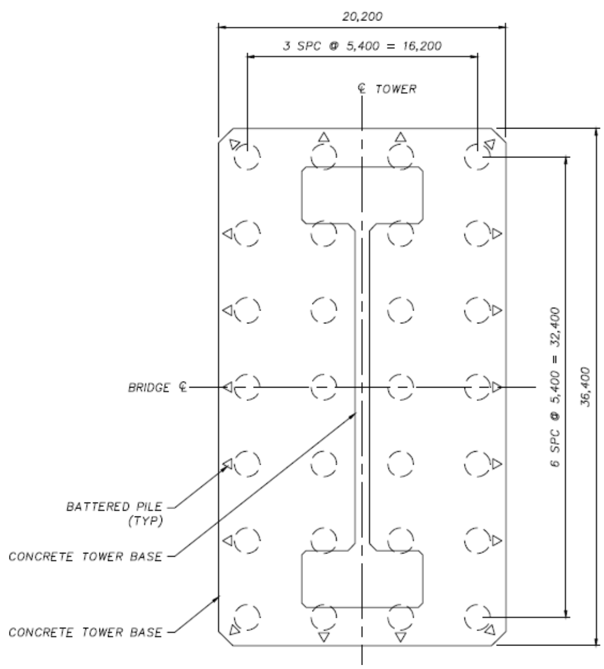

Figure 4-2: Bridge Tower Plan [30]

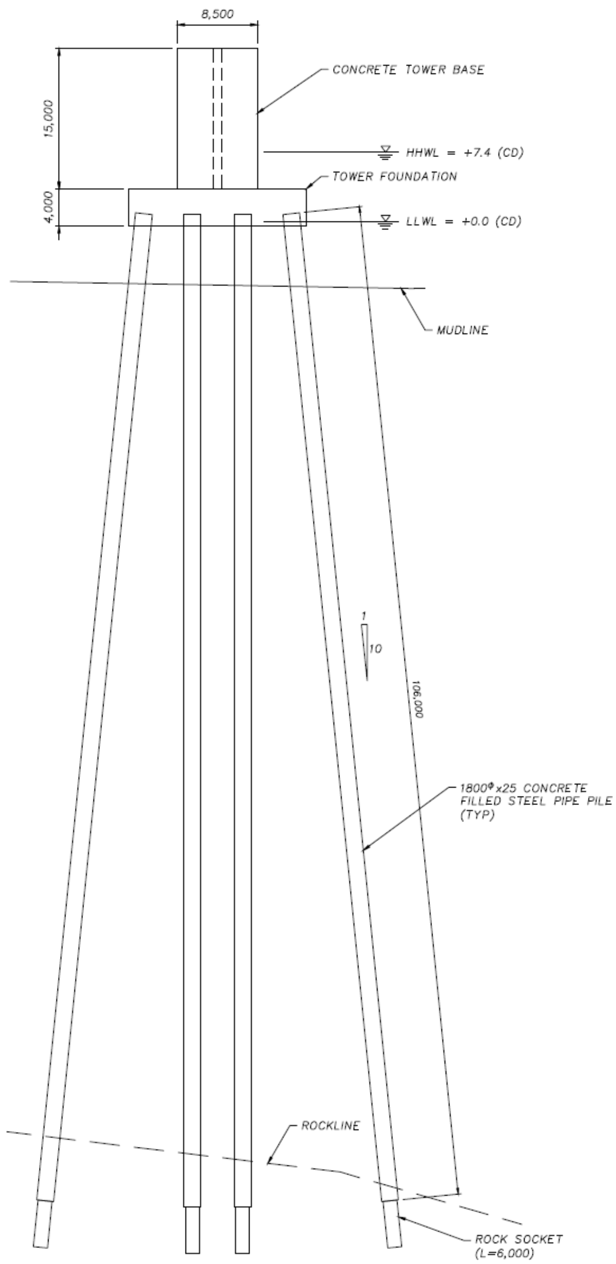

Figure 4-3: SW Tower Section [30]

As per Infinity Engineering's drawings [30], the bridge tower piles are comprised of 7 rows of piles aligned with the bridge center line and 4 columns of piles perpendicular to the bridge center line (Figure 4-2). The outer piles are battered (piles installed or driven at an angle with the vertical to resist lateral forces) in all directions.

The SW tower will be constructed using a cofferdam and will be placed over the seabed (pres. communication, PNW LNG – meeting November 4, 2014). As the seabed elevation at this location is approximately -0.35 m CD the base of the SW tower is assumed to be located at an elevation of -0.35 m CD.

4.2.1 Scour Depth Assessment

The scour calculation approach for complex bridge pier geometry is based on the method of superposition of scour components described in the Bridge Scour Manual [48].

A complex pier is frequently composed of up to three elements, presently referred to as bridge pier, pile cap and piles (or pile group). This method treats a complex bridge pier as a single cylindrical pier by using an equivalent pier diameter, *b**. This equivalent cylindrical pier is such that, for the same flow and sediment conditions, it produces the same scour depth, *ds*, as the complex pier (Figure 4-4). The equivalent pile diameter also depends on the height of the pile cap in relation to the initial bed level.

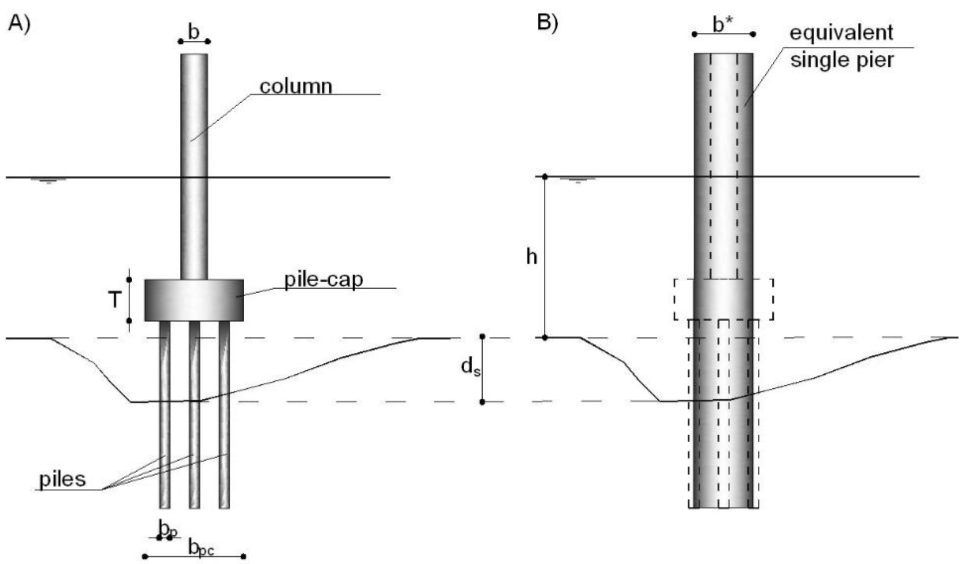

Figure 4-4: Complex Pier and Equivalent Pier Representation [23]

Estimates of SW tower scour depth and volume, based on empirical formulas from Sheppard and Renna, 2010 [48], are presented in Table 4-2.

Table 4-2: SW Tower Scour Depth and Volume

Location	Water Depth at HHWL (m)	Effective Pier Diameter (m)	Scour Depth (m)
SW Tower	7.75	9.63	15.35

Hatch's scour analysis suggests that an attenuation factor of 0.5 be applied over the empirical approaches to compensate the positive effect of site conditions present in the study area, which tends to decrease the scour depth.

Table 4-3 presents the final results of the scour depth at equilibrium state, considering Hatch's site characteristics approach including an attenuation factor of 0.5.

Table 4-3: Final Estimates of SW Tower Scour Depth

Location	Water Depth at HHWL (m)	Effective Pier Diameter (m)	Scour Depth Analysis (m)
SW Tower	7.75	9.63	7.67

4.2.2 Scour Width Assessment

It is important to determine the top width of a scour hole to evaluate if local scour holes between structures overlap and also to determine the extent of riprap coverage needed to protect bridge foundations from scour processes.

HEC-18 [63], informs that a top width of 2 times the scour depth is suggested for practical applications as studied by Richardson and Abed, 1993 [46] (Figure 4-5).

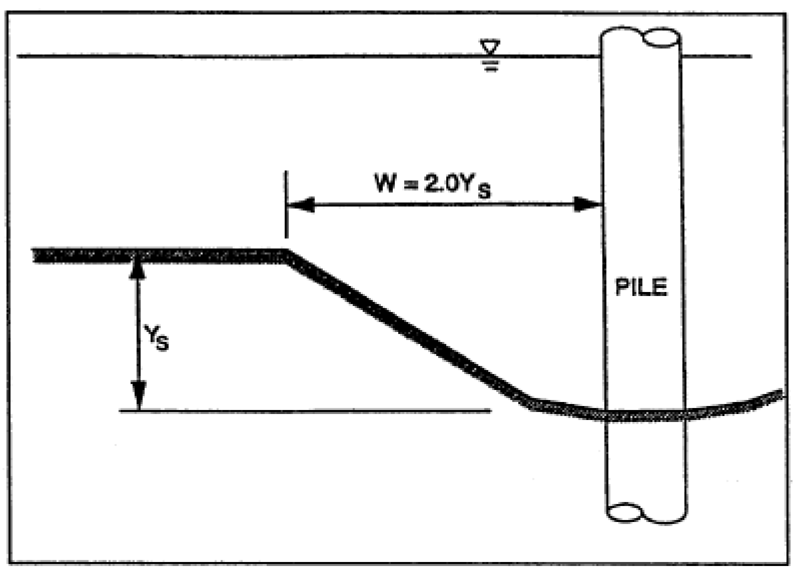

Figure 4-5: Top Width of Scour Hole [63]

Table 4-4 below presents the top width for the SW Tower as per HEC-18 63] recommendation.

Table 4-4: Top Width of Scour Hole SW Tower

Location	Scour Depth Final Analysis (m)	Associated Top Width (m)
SW Tower	7.67	15.35

Figure 4-6 below depicts the SW tower equivalent pier according to the method described in Section 4.2.1.

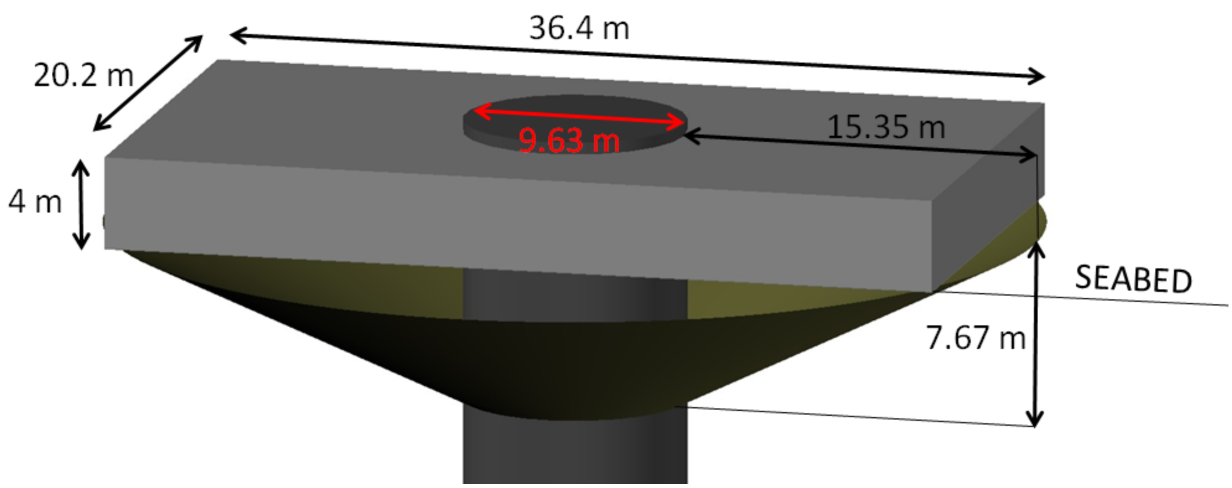

Figure 4-6: 3D Rendering SW Tower Equivalent Pier

4.2.3 *Scour Volume*

Table 4-5 presents the scour volume for the SW Tower.

Table 4-5: Scour Volume SW Tower

	Scour Depth Final Analysis (m)	Scour Volume (m^3)
SW Tower	7.67	1,300

4.2.4 *Countermeasures*

Similar to the trestle structures, the SW tower riprap scour protection gradation is shown in Table 3-6 (D_{50} = 0.39 m) and Figure 3-14 (M_{50} = 160 kg).

Two layers of riprap will be used for a thickness of 0.8 m (2*D_{50}). As will be discussed further in Section 6, settlement of this material will likely occur. The riprap around the bridge foundations should be monitored and maintained as required. A geotextile filter should also be considered between the seabed and the riprap.

Table 4-6 below presents the riprap volume to be installed over the seabed at the toe of the SW tower.

Table 4-6: Scour Protection Volume SW Tower

Location	Scour Protection Height (m)	Scour Protection Width (m)	Scour Protection Volume (m³)
SW Tower	0.8	10.00	1,220

Figure 4-7 depicts the SW tower scour protection according to the equivalent pier method.

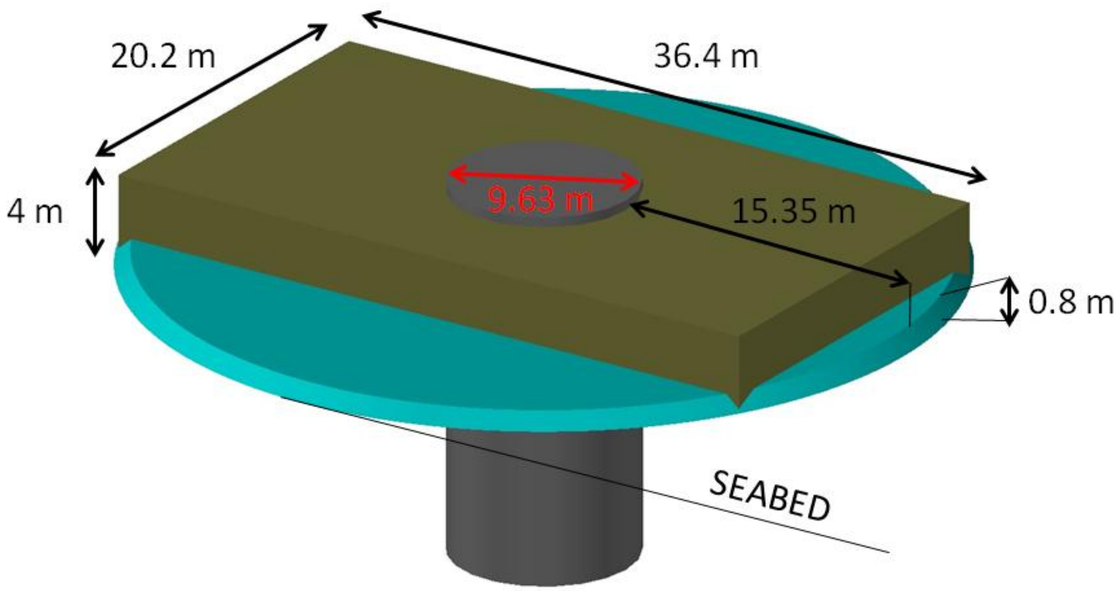

Figure 4-7: 3D Rendering SW Tower Scour Protection

4.2.4.1 *Countermeasure Final Analysis*

Figure 4-8 below depicts the plan view of a geometric analysis of scour protection results based on the equivalent pier method and the SW tower geometry.

The horizontal extent of scour protection over the longest side of the slab is 10 m. Therefore, this extent should be applied around the perimeter of the structure.

The associated scour protection surface area is of 1,600 m² and corresponds to the armour area only excluding the structure.

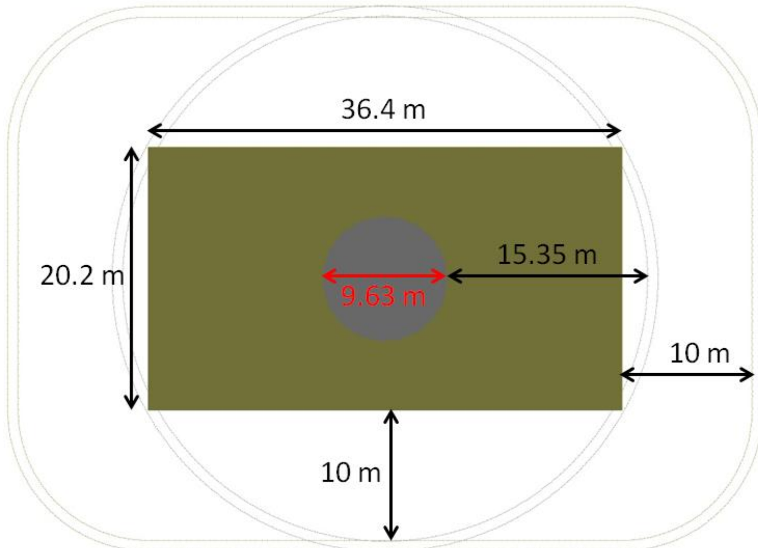

Figure 4-8: Overlaid Scour Protection Plan View

4.3 SW Anchor Block

Figure 4-9 and Figure 4-10 depict the SW anchor block foundation pile arrangement plan and section.

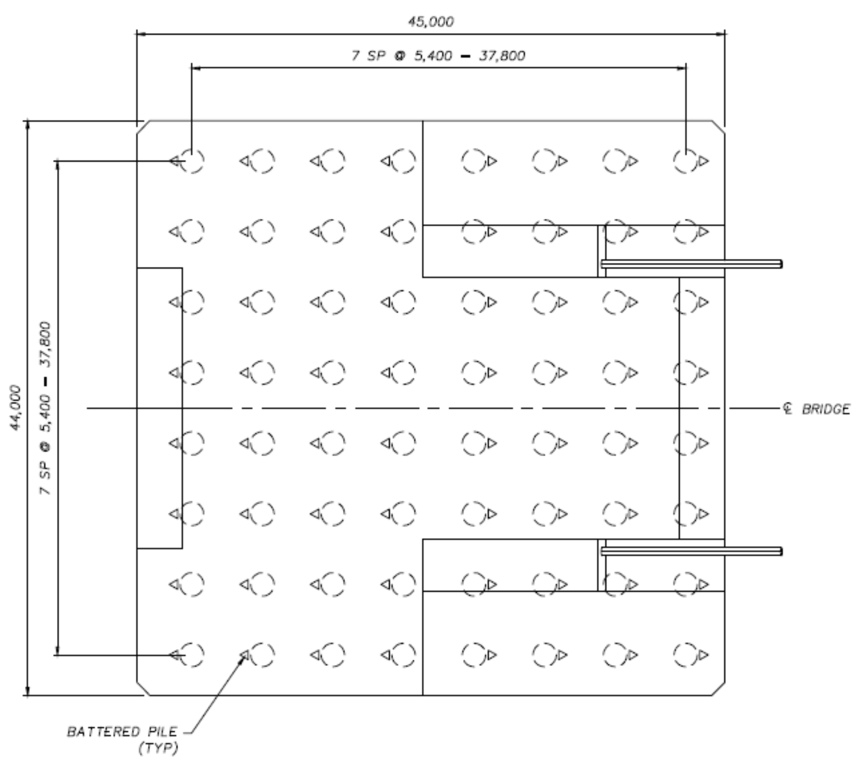

Figure 4-9: SW Anchor Block Plan [30]

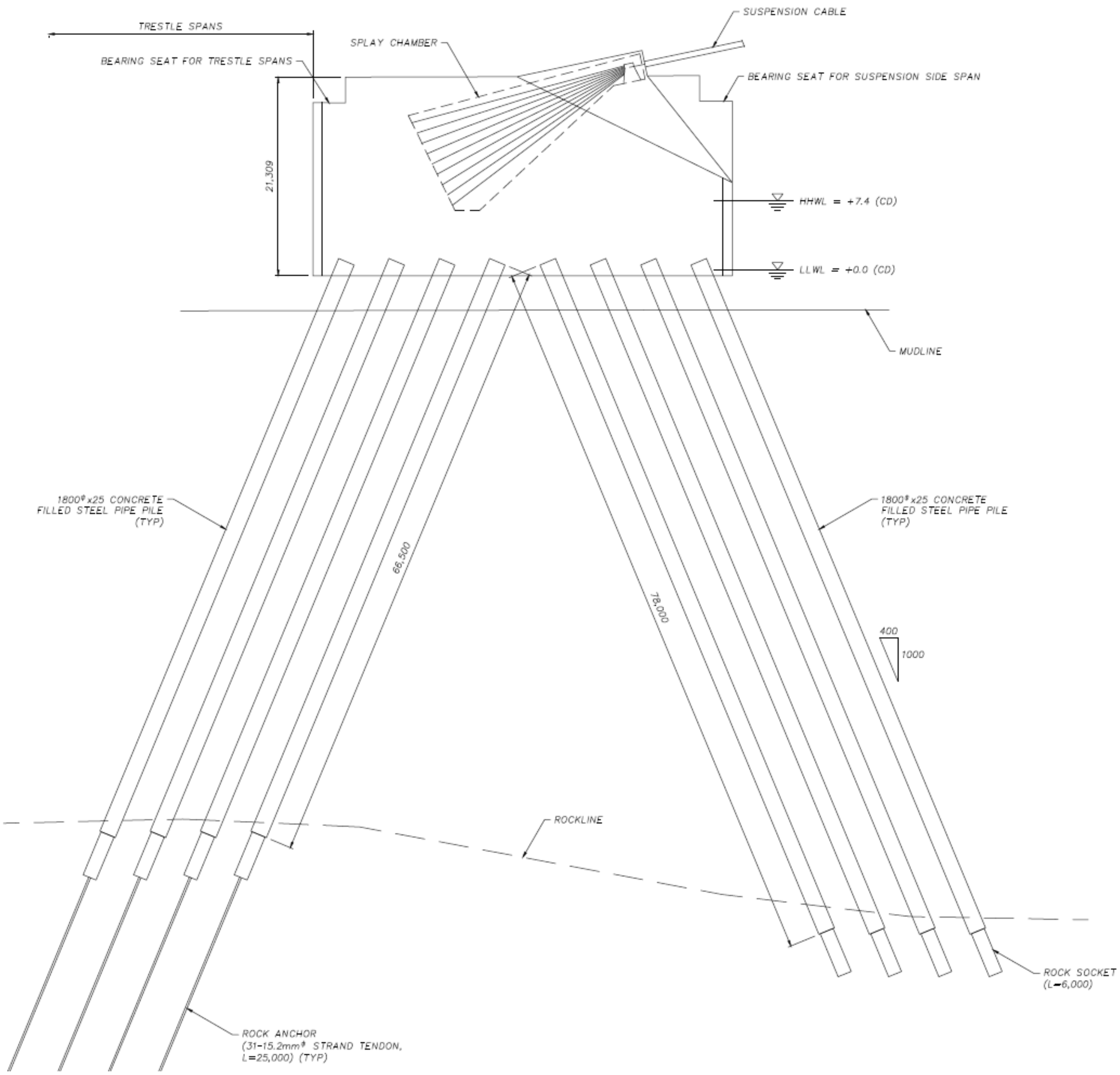

Figure 4-10: SW Anchor Block Section [30]

Figure 4-11 shows the anchor block during construction. The anchor block, which is constructed using a cofferdam, will be placed over the seabed (pers. communication, PNW LNG – meeting November 4, 2014). As the seabed elevation at this location is approximately -0.7 m CD, the base of the anchor block is assumed to be located at an elevation of -0.7 m CD.

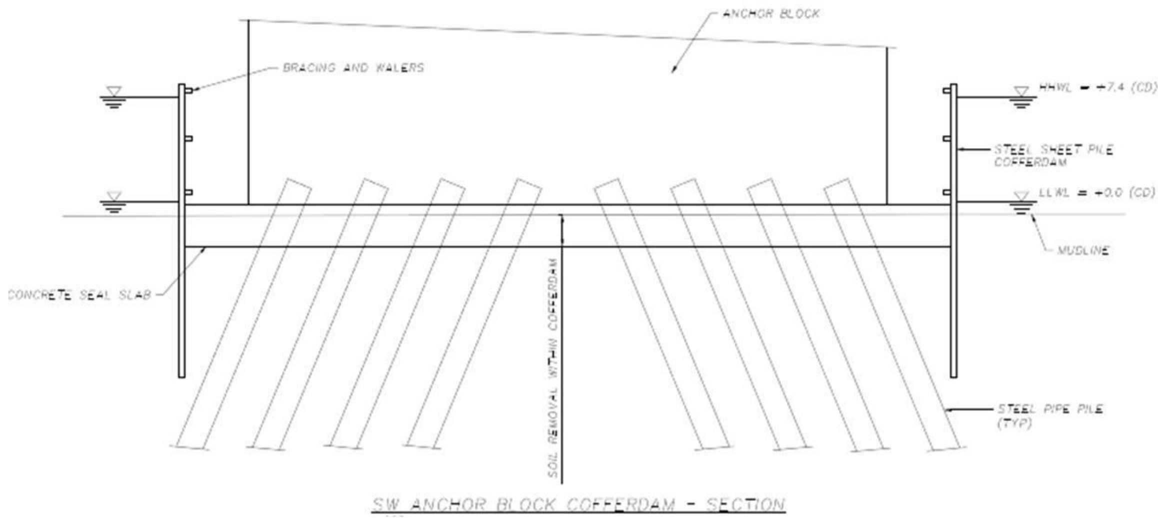

Figure 4-11: Anchor Block Construction [32]

4.3.1 Scour Depth Assessment

4.3.1.1 Equivalent Pile Diameter Method

Similar to the SW tower, the equivalent pile diameter method was applied to estimate scour for the SW anchor block.

SW anchor block scour depth estimates are presented in Table 4-7, based on empirical formulas from Sheppard (2010) [48].

Table 4-7: SW Anchor Scour Depth

Location	Water Depth at HHWL (m)	Effective Pier Diameter (m)	Scour Depth (m)
SW Anchor	8.13	29.21	34.50

Hatch's scour analysis suggests that an attenuation factor of 0.5 be applied over the empirical approaches to compensate the positive effect of site conditions present in the study area, which tends to decrease the scour depth.

Table 4-8 presents the final results of the scour depth at equilibrium state, considering Hatch's site characteristics approach including an attenuation factor of 0.5.

Table 4-8: Final SW Tower Scour Depth

Location	Water Depth at HHWL (m)	Effective Pier Diameter (m)	Scour Depth Analysis (m)
SW Anchor	8.13	29.21	17.25

4.3.1.2 Vertical Wall Method

Scour depth for the SW Anchor block will be also calculated considering the structure similar to a vertical wall, as it is composed of a concrete block sitting over the seabed and rising above the highest water level. The scour calculation method will be based on scour literature for vertical breakwaters and seawalls.

The scour in front of a vertical wall follows a simple rule that is widely used in engineering and states that maximum scour is proportional to the height of maximum unbroken wave at the toe of the structure (Shore Protection Manual, 1984) [11].

Since along-structure currents and oblique-incident waves (that generate mach-stem effect) will increase scour, an amplification factor of about 1.3 will be applied over the baseline recommendation, based on Asadi et al., 2014 [5].

The highest wave height expected in the study area is 1.60 m, according to Table 2-3. Therefore, the expected scour depth at the toe of the anchor block is approximately 2 m below the local seabed.

Table 4-9: SW Anchor Block Scour Depth

Location	Water Depth at HHWL (m)	Scour Depth (m)
SW Anchor	8.13	2.0

4.3.2 Scour Width and Volume Assessment

4.3.2.1 Equivalent Pile Diameter Method

As detailed in Section 4.2.2, the SW anchor block’s top width of the potential scour hole will be determined based on HEC-18, 2001 [63].

Table 4-10 below presents the top width for the SW Anchor as per HEC-18, 2001 [49] recommendation.

Table 4-10: Top Width of Scour Hole SW Tower

Location	Scour Depth Final Analysis (m)	Associated Top Width (m)
SW Anchor	17.25	34.50

As a simplification, the top width of scour hole on SW tower will be considered as 35 m.

Table 4-13 below presents the scour volume for the SW Anchor.

Table 4-11: Scour Volume SW Anchor

Location	Scour Depth Final Analysis (m)	Scour Volume (m³)
SW Anchor	17.25	19,000

Figure 4-12 depicts the SW anchor block equivalent pier according to the method described in Section 4.2.1.

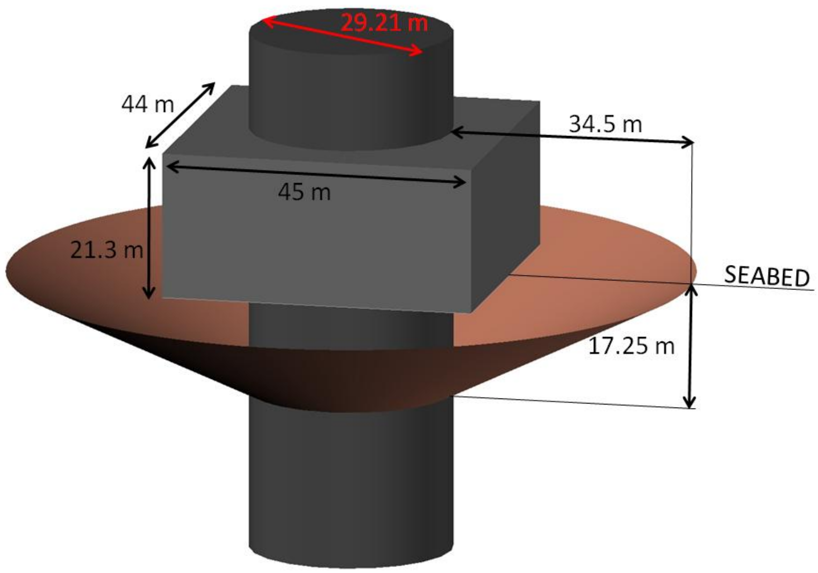

Figure 4-12: 3D Rendering SW Anchor Scour Equivalent Pier

4.3.2.2 Vertical Wall Method

As concluded by Steetzel,1988 [54] based on large-scale hydraulic laboratory experiments, toe scour near structures presents the maximum value of the landward slope of the scour hole between 1V:3H (18.26°) and 1V:5H (11.30°). Due to the combined influence of waves and currents over the structure in the area of study, the slope of the scour hole will be considered as 1V:5H (11.30°).

As the SW anchor block is not embedded in the seabed but sitting on it, it is expected that scour also takes place underneath the concrete structures.

Figure 4-13 and Figure 4-14 represent the expected shape of the scour at the toe of the SW anchor block structure.

A 3D Rendering Detail SW Anchor Scour based on the vertical wall method is shown in Figure 4-15.

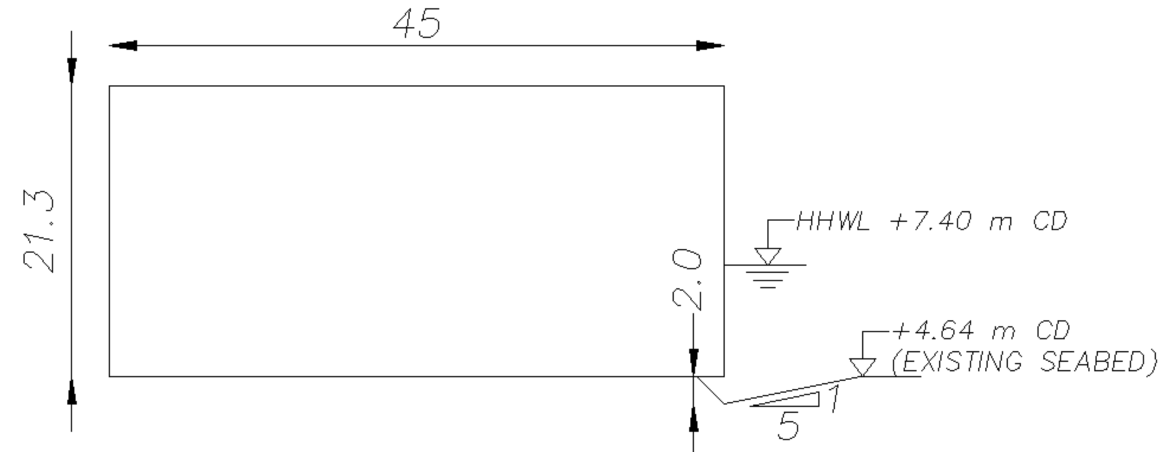

Figure 4-13: Expected Scour at Toe of SW Anchor Block, Section

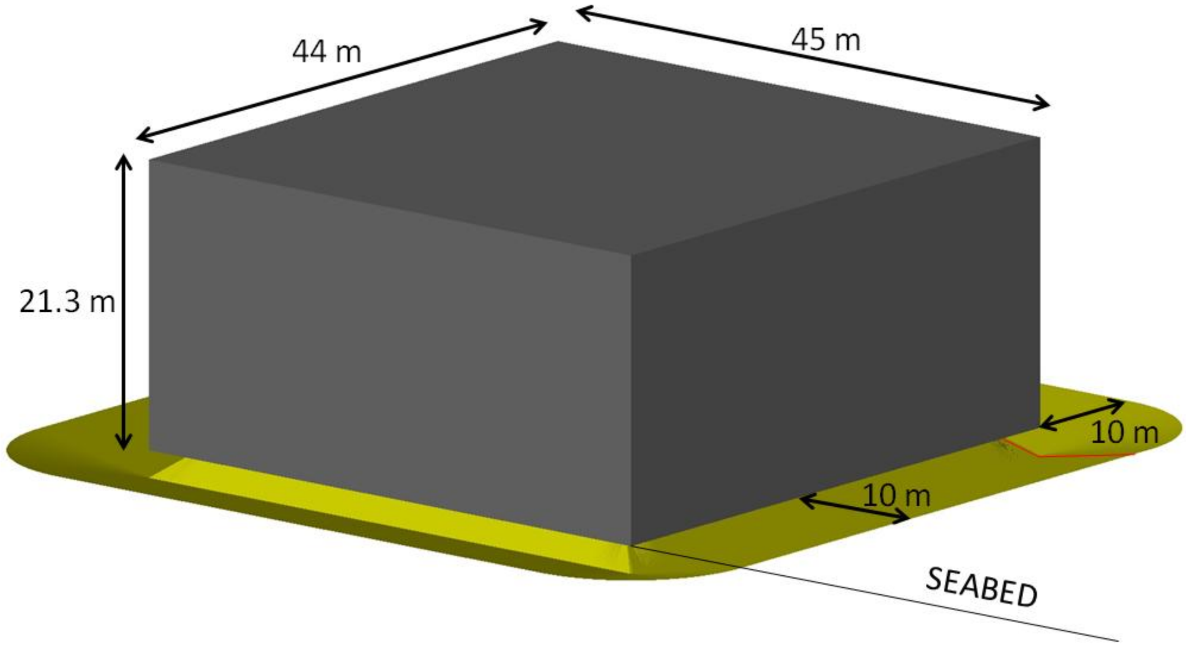

Figure 4-14: 3D Rendering SW Anchor Scour Vertical Wall

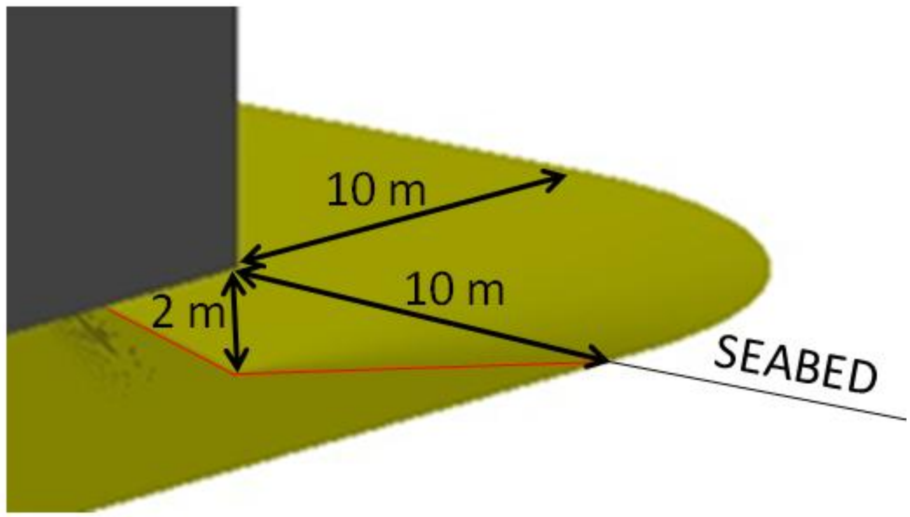

Figure 4-15: 3D Rendering Detail SW Anchor Scour Vertical Wall

Based on this empirical approach the estimated scour depth for the SW anchor block is 2.0 m; and the estimated scour volume is 2,890 m³, as shown in Table 4-12.

Table 4-12: Top Width and Volume of Scour Hole SW Anchor

Location	Scour Depth (m)	Scour Top Width (m)	Scour Volume (m³)
SW Anchor	2.00	10.00	2,400

4.3.3 Scour Volume Final Analysis

The important dimensions of the SW anchor block, both horizontally and vertically, led the study to assess the scour based on two different methods described in the Sections above. Both methods are well detailed in the literature and were applied to asses scour quantities.

Table 4-13 below presents the comparison between SW anchor scour depth, top width and volume for both methods.

The complex pier method considers the influence of each component. If scour is allowed to develop, the support piles underneath the block will impact the flow and generate extra scour.

As the scour process will develop since the first stages of construction, the vertical wall method is considered the most appropriate for scour countermeasures design.

Table 4-13: SW Anchor Scour Method Comparison

Method	Scour Depth (m)	Scour Top Width (m)	Scour Volume (m³)
Complex Pier	17.25	34.50	19,000
Vertical Wall	2.00	10.00	2,400

4.3.4 Countermeasures

4.3.4.1 Equivalent Pile Diameter Method

Similar to the trestle structures, the SW anchor block riprap scour protection gradation is shown in Table 3-6 (D_{50} = 0.39 m) and Figure 3-14 (M_{50} = 160 kg).

Two layers of riprap will be used for a thickness of 0.8 m ($2*D_{50}$). As will be discussed further in Section 6, settlement of this material will likely occur. The riprap around the bridge foundations should be monitored and maintained as required. A geotextile filter should also be considered between the seabed and the riprap.

Table 4-14 below presents the riprap volume to be installed over the seabed at the toe of the SW anchor.

Table 4-14: Scour Protection Volume for SW Anchor

Location	Scour Protection Height (m)	Scour Protection Width (m)	Scour Protection Volume (m³)
SW Anchor	0.8	34.50	5,600

Figure 4-16 depicts the SW anchor block scour protection according to the equivalent pier method.

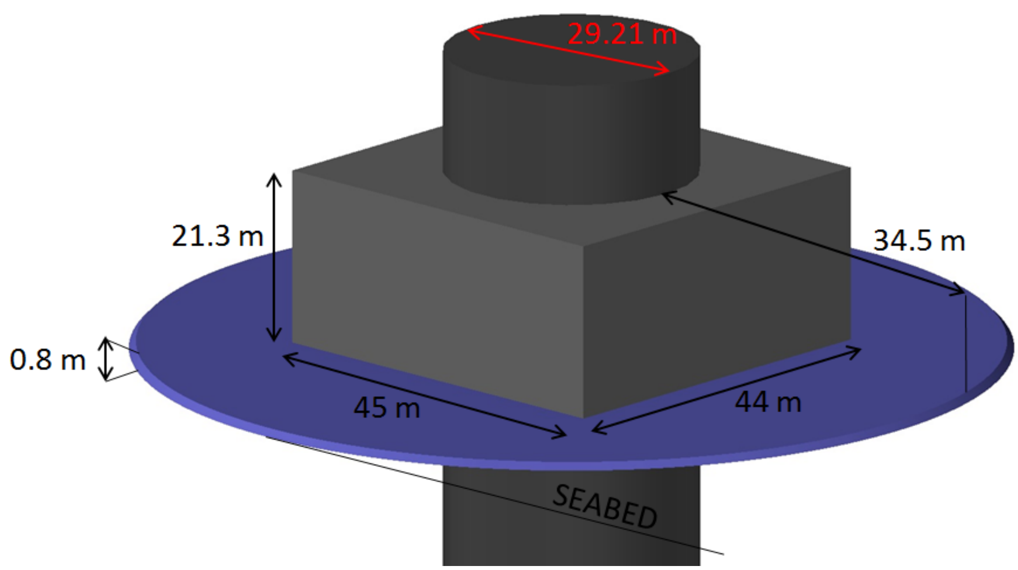

Figure 4-16: 3D Rendering SW Anchor Scour Protection Equivalent Pier

4.3.4.2 Vertical Wall Method

Irie and Nadaoka, 1984 [21] suggested through two- and three-dimensional laboratory models that toe protection should have a horizontal extent equal to about 0.25 the approaching wave length (L), which yields 10 m.

As maximum scour is expected at the sharp corners of SW anchor block, an amplification factor of 1.5 is applied to the horizontal extent of the calculated scour top width (10 m). In order to guarantee a constant construction width, the horizontal riprap coverage of 15 m should be applied throughout the perimeter of the SW anchor block.

Table below presents the riprap volume to be installed over the seabed at the toe of the SW anchor block.

Table 4-15: Scour protection volume SW Anchor

Location	Scour Protection Height (m)	Scour Protection Width (m)	Scour Protection Volume (m^3)
SW Anchor	0.8	15.00	2,800

Figure 4-12 depicts the SW anchor block scour protection according to the vertical wall method.

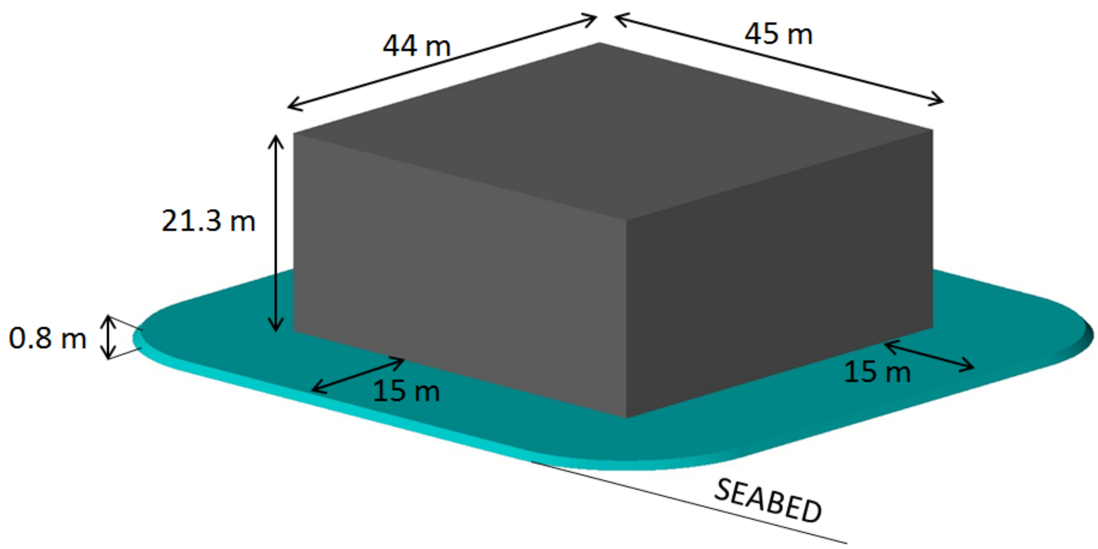

Figure 4-17: 3D Rendering SW Anchor Scour Protection Vertical Wall

4.3.4.3 *Countermeasure Final Analysis*

Figure 4-13 below depicts the comparison between scour protection results based on the two methods described above.

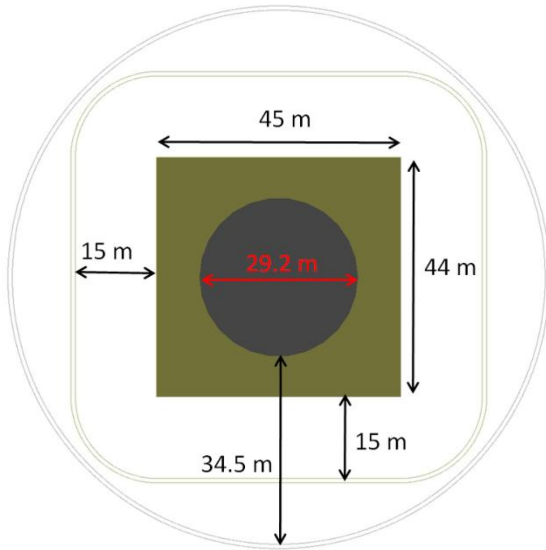

Figure 4-18: SW Anchor Block Scour Protection Plan View Comparison

As discussed in Section 4.3.3, the most adequate scour countermeasure for the SW anchor block is the vertical wall method with 15 m of riprap width from the border of the structure and 0.8 m of riprap thickness.

The associated scour protection surface area is of 3,600 m² and corresponds to the armour area only excluding the structure.

5. Distance to Eelgrass Beds

Figure 5-1 below depicts the overall extent in plan of the bridge structures scour countermeasures and the eelgrass locations in Flora Bank. The blue line shows the approximate limit of Flora Bank and the eelgrass bed area is shown in green and yellow colours [55]. For the SW tower structure, the minimum distance from the outer edge of the riprap to the eelgrass bed is 250 m, whereas for the SW anchor block structure, this minimum distance is 270 m.

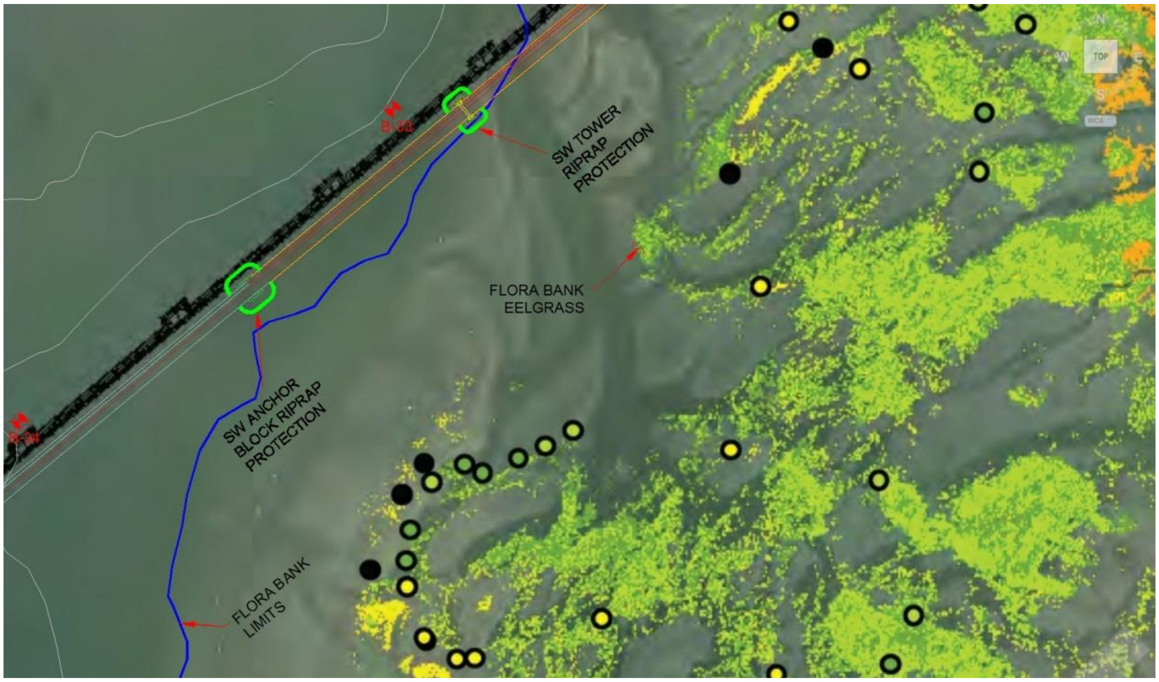

Figure 5-1: Bridge Structures Scour Countermeasures on Flora Bank [55]

A small amount of scour is expected at the perimeter of the riprap. This scour can be mitigated using the falling apron riprap approach. The riprap at the toe of the scour protection will adjust to the scour once it is setup and follow any bed erosion downwards.

Scour will cause the riprap to fall down into the initial hole, preventing the formation of an erosion slope that is too steep. The loose riprap will cover the scour slope to a thickness stable enough to retain the bed material in place and reduce further scour to negligible amounts.

6. Further Design Work

6.1 Geotechnical Information

Settlement of scour protection (riprap) may occur. Further review of potential settlements into the seabed and expected settlement values (initial and long term) should be considered once more geotechnical data becomes available.

Further design analyses of the need of geotextile material or filter material between the seabed and the riprap should be carried out.

Once construction is completed, the riprap protection should be monitored to determine if and at what frequency maintenance of scour protection is required.

It is recommended that the marine terminal area be surveyed prior to installation of the bridge structures and the trestle. Further geotechnical investigations will contribute to define if ground improvement measures are needed to reduce settlement of scour protection countermeasures.

6.2 Hydraulic Modelling

Further confirmation of scour estimates may be carried out with hydraulic physical models, particularly to review the potential for global scour. The empirical methods used to determine scour tend to be conservative and physical model testing may optimize the scour protection required.

As there are three pile structures identified along the marine terminal that have pile group characteristics, and as such would cause global scour, higher localized scour volumes may occur, as the example shown in Figure 6-1, where global scour is extended further away than the localized scour due to single piles.

The potential extra scour volume due to the group scour was not yet predicted. It is best analyzed in a physical model study during a future phase of the engineering design.

Figure 6-1: Global Scour Due to Multiple Piles [13]

Figure 6-2 shows a single pile after a test in a two-dimensional flume (2D tests). The scour hole is visible around the pile, which will develop with time until equilibrium is obtained. Alternatively, after pile installation a scour protection countermeasure may be constructed to reduce localized scour processes.

Figure 6-2: Single Pile Scour Test [28]

The design of the riprap scour countermeasures, both horizontal extent dimension and thickness, may be optimized with the use of hydraulic physical modeling. An example of such a model study is shown in Figure 6-3.

The slope of the seabed varies from approximately 0.25% along the trestle and 10% at the jetty. The impact of the slope on the scour protection at the project site will also be better evaluated and understood using a hydraulic physical model.

A procedure for the model study will be to test riprap designs in three-dimensional model basin (3D tests). Once the design is completed, it may be implemented from the initial construction phase, i.e., constructed at the same time as the bridge structures. In this way the equilibrium scour holes will not be allowed to develop in the model (and in the field).

Figure 6-3: Monopile Model with Riprap Scour Protection, Before Tests [62]

The trestle scour processes and scour protection countermeasures should be further investigated and confirmed using hydraulic physical modelling.

As the available empirical formulas and methodology for determining the scour due to the bridge tower and anchor block are limited, physical modeling should also be conducted to fully determine the potential extent of the scour and most cost effective and efficient design of scour countermeasure for each structure.

7. Conclusions

Potential scour along the bridge and trestle foundations were determined using available technical literature and adjusted to the site conditions along the marine terminal jetty structures. Scour mitigation measures, specifically using riprap to protect from scour, was also evaluated. With the use of scour protection around the marine structures foundations no significant adverse effects from scour are expected.

The potential scour, if no mitigation measures are implemented, was calculated for the piles along the trestle and berth structures. Scour around the trestle piled structures may develop to a depth at equilibrium state of about 1.25 m (approximately one pile diameter). The extent of the horizontal scour width is the same order of magnitude, i.e., 1.25 m. The expected scour volume at equilibrium state for the trestle and berth structures is approximately 2,100 m^3 considering local scour only.

The proposed scour protection for trestle and berth structures consists of riprap quarry stone covering a longitudinal extension of local and group scour, with a thickness of 0.8 m corresponding to two layers of stones. The total surface area of scour protection around one individual pile is approximately 21 m^2. The total volume of riprap scour protection around one pile is approximately 12 m^3. The footprint of the scour protection and pile for an individual pile is 22.2 m^2. The total footprint of scour protection including the piles for the marine terminal structures is 11,150 m^2.

Potential scour was calculated for the two bridge structures located in water, the SW tower and the SW anchor block. As per Infinity Engineering's drawings [32], scour estimates for these bridge structures were conducted considering bridge foundations located at the seabed. If no scour protection is used, the total expected scour volume at equilibrium state for the SW tower and SW anchor are 1,300 m^3 and 2,400 m^3, respectively.

For the SW tower, the total surface area of scour protection around tower is 1,600 m^2. The total volume of riprap scour protection around the SW tower is approximately 1,220 m^3. The total footprint, including the SW tower, is approximately 2,400 m^2.

For the SW anchor block, the total surface area of scour protection around the anchor block is 3,600 m^2. The total volume of riprap scour protection around the SW anchor block is approximately 2,800 m^3, using the vertical wall estimate method. The total footprint, including the SW anchor block, is approximately 5,600 m^2.

The scour prediction equations discussed in previous chapters are conservative, i.e., they were designed to predict scour depths under controlled laboratory conditions for simplified structures, bed sediments and flow situations. These simplified laboratory experiments that can influence local scour depths include narrow sediment size distribution and clear water (negligible fine sediments in the water column), both of which can produce larger scour depths than would occur under normal field conditions. In addition, the duration of the design flow event in the model tests may not be sufficient for developing an equilibrium scour depth.

As a natural process on a live bed environment, scour on an intertidal eelgrass environment is not expected to cause large disturbances to the existing dynamic equilibrium of Flora Bank depths. The complex hydrodynamic conditions of the site, namely multidirectional tidal currents, drift currents and wave action, in water depths with tidal range of more than 7 m will contribute to reduce any large scour hole development.

It is recommended in this report to implement the scour protection design at the beginning of construction. For practical reasons it is recommended that a scour evaluation monitoring program be implemented to follow up its development.

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Appendix A

Literature Review

A. Literature Review

A review of relevant literature research and theory that contributes to the understanding of the scour process involving the bridge foundations, pile arrangements, soil properties, hydrodynamics and countermeasures was conducted. The studies which applied to the site conditions are summarised below.

A.1 Overview

The parameters that influence the scouring process near individual piles, are characterized by the pile diameter (*b*), the ones related to the fluid characterization, such as acceleration due to gravity, fluid density and fluid kinematic viscosity; the ones related to the bed material, such as sediment diameter (D) and sediment density; and the flow parameters such as depth of approaching flow (d_0) and mean velocity (U) of the undisturbed flow.

For an initial practical application, Breusers, 1977 [6] based on dimensional analysis presents a method for estimating scour depth as presented below:

$$\frac{d_S}{b} = f_1(\frac{\overline{U}}{U_c}) \cdot [2.0 \tanh\left(\frac{d_0}{b}\right)] \cdot f_2(shape) \cdot f_3(\alpha_{attack})$$

where d_S is the maximum scour depth measured below the ambient bed level, and U_c is the critical flow velocity. This scour formula includes the influence of shallow water depths, which is relevant for the project site. This method neglects the influence of Froude number and bed material shape, density and gradation.

The use of dimensionless parameters in the formula demonstrated that it is possible to relate the scour depth to the diameter of the pile (or pier). This is explained physically by the fact that scouring is due to the horseshoe-vortex system whose dimension is a function of the diameter of the pile. A complete theory for computing the sediment transport rate in the scour hole hasn't been developed mainly because the flow field is too complex Breusers, 1977 [6].

Using the ratios f_1, f_2 and f_3 as described in Breusers, 1977 [6], the ratio of depth of scour to pile diameter may be estimated. If $f_1 = 1$, scour is present with sediment motion. However, the scour depth does not increase with velocity, apparently because the dynamic equilibrium between transport out of the scouring hole and the supply is not influenced by the magnitude of the transport rate. Also, $f_2 = 1$ for circular and rounded piles (piers), and there is no influence of the angle of current approach for a single circular pile ($f_3 = 0$).

According to the technical literature, based on accumulated evidence of both laboratory and field experiments, it appears that the scour depth may be regarded as a function of the geometry alone. Also, the effect of cohesion in the bed sediments is likely to increase the resistance of the bed to scour.

The effect on the local scour depth obtained by adding waves onto a steady current is still under research; however some investigators have found that when adding waves to the currents the tendency is a reduction of the scour depth, and that the scour depth is not increased above the current-only value.

A.2 Bridge Scour

In 1988 the Federal Highway Administration (FHWA) of the U. S. Department of Transportation issued a Technical Advisory to the States requiring them to evaluate all bridges over water as to their vulnerability to scour. The Advisory was the result of floods in the New England States in 1987, which destroyed and damaged 17 bridges and cost 10 lives. The outcome was the issue of the Hydraulic Engineering Circular HEC- 18 [63] from 2001 to form a unit for the evaluation, design, inspection, selection and design of countermeasures for stream instability and scour at bridges (Richardson and Davis, 2001).

Jones and Sheppard, 2000 [34] developed the Superposition of Scour Components Method of Analysis for complex piers. Each of the scour components is computed from the basic pier scour equation using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in a list of variables, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. The equivalent diameter of the complex pier can be approximated by the sum of the equivalent diameters of the complex pier components.

Two prediction methods, based on scour depth superposition concept, are the most used to assess equilibrium scour depth at complex piers: Florida Department of Transportation FDOT method (Sheppard and Renna, 2010 [48]) and HEC-18 method (Arneson *et al.*, 2012 [3]).

The Bridge Scour Manual, 2005 [7] presents a discussion about the Colorado State University (CSU) equation, currently used in the U.S. Federal Highway Administration (FHWA) Hydraulic Engineering circular Number 18 (HEC-18), and those published by Sheppard, 2004 [52], Melville, 2000 [41], and Breusers, 1977 [6] about their concept being empirical and based on laboratory-scale data. Many of these equations yield similar results for laboratory-scale structures, but differ significantly in their predictions for prototype scale structures. The over prediction of many of these equations for large structures in fine sands is well documented and is referred to as the “Wide Pier” problem.

Sheppard and Renna, 2010 [48] through the Bridge Scour Manual by FDOT (Florida Department of Transportation) proposed to calculate scour on live bed on complex bridge piers through the following formula and proposes a calculation methodology that was adopted for the purpose of this study.

$$\frac{y_s}{D^*} = 2.2\,tanh\left[\left(\frac{y_0}{D^*}\right)^{0.4}\right]$$

where,

y_s is the vertical scour depth;

y_0 is the water depth; and

D* is the equivalent pier diameter.

Escarameia,1998 [20] conducted a series of laboratory experiments studying the scour depth and time development of scour under reversing currents typical for estuarine environments associated with the construction of offshore wind turbine supporting structures. Equilibrium scour depths were found to be lower under reversing than unidirectional currents due to infilling of sediment back into the scour hole when flow reversed. Maximum scour depths were found to occur under live bed conditions.

A.3 Trestle Scour

Initially, the effect on the local scour depth was estimated based on the following conditions:

- One cylindrical pile;
- Non-cohesive granular bed material;
- One-way current, e.g., uniform, wide and steady flow;
- Flat initial bed.

A.3.1 Local Scour

Elsebaie, 2013 [19] presents classic conclusions about an experimental study of local scour around a circular bridge pier in sandy soil:

- Maximum scour depth was observed to occur at the upstream of the pier.
- It is observed that the coarse portion of the sediment is deposited at downstream zone of the pier.

Mostafa, 2011 [43] presents conclusions based on an experimental study of scour at single piles and pile groups due to waves and currents:

- For single piles, scour depth due to steady flow current is significantly larger than scour due to waves only. Scour depth in the case of waves against the current is larger than the case of waves only.
- The case of waves against the current leads to slightly reduced values of scour depth around a single pile compared to the situation of waves with current or waves perpendicular to the currents.

Mostafa, 2012 [44] presents structural aspects for pile groups supporting marine structures founded on cohesionless soils:

- Scour depth becomes less significant with the increase in pile batter angle.
- Scour has a significant impact on piles installed in sand and a less significant impact on piles installed in clay.
- Effect of scour is more pronounced for piles installed in stiff clay compared to piles installed in soft clay.
- It has been documented that local scour depth in sandy soils is 2 times the pile diameter and the ultimate scour depth around piles in soft clays is 0.75 to 1 times the pile diameter, representing a decrease in 62% to 50%.

Nielsen *et al.*, 2012 [45] presents conclusions based on an experimental study of the scour around a monopole in regular breaking waves:

- The maximum scour depth around a monopile in regular breaking waves is approximately 0.60D, where D is the monopole diameter.
- The scour was, for all cases, smaller than the maximum scour expected for the current alone.
- Considering that local scour depth in sandy soils is 2 times the pile diameter and the ultimate scour depth around piles in regular breaking waves is 0.6 times the pile diameter, the breaking waves effect alone represent a decrease in 70%.
- The scour over slender piles installed over beaches, bars, banks, and reefs when the potential exists for breaking waves, is approximately the same for breaking and non breaking waves. At the natural irregular sea state, the waves will break over a long distance and the pile will be exposed to both breaking and non breaking waves.
- When a pile is exposed to both breaking and non breaking waves, the following three scour processes will take place: scour caused by breaking waves, scour caused by non breaking waves, and backfilling by smaller waves (Sumer et al. 2012).

Dey *et al.*, 2011 [18] presents results of equilibrium scour and time variation of scour depths at circular piles embedded vertically in clay alone with different proportions of sand-clay mixtures as bed sediments under waves:

- The equilibrium scour depth under waves reduces with an increase in clay proportion (by weight) in a sand-clay mixture.
- The scour depth reductions for n = 0.3 and 1 are almost equal, suggesting that when the clay proportion in a sand-clay mixture becomes 0.3, the sand-clay mixture behaves as if it were clay alone.

Sumer *et al.*, 2013 [61] presents results of an experimental investigation of the backfilling of scour holes around circular piles either by a current or a wave (Figure A-1):

- Backfilling of a scour hole around a pile in the live-bed regime occurs when the flow climate changes from a steady current to a wave, from a steady current to a combined waves and current, or from a wave to a smaller wave.
- The time scale of the backfilling process is completely different from that of scour, because the scour process and the backfilling process are two entirely different processes. The time scale of backfilling can be larger than that of scour and vice-versa.

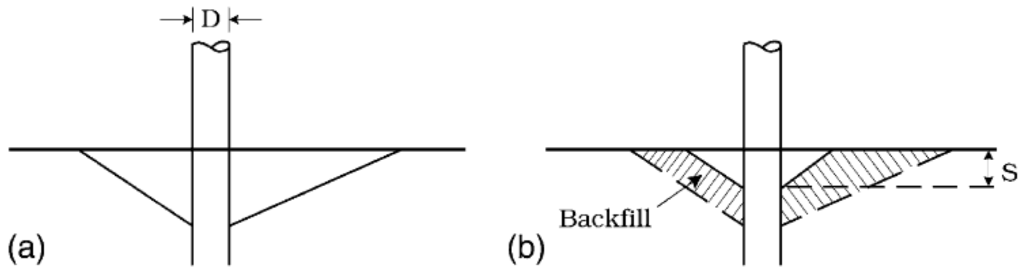

Figure A-1: Definition sketch (schematic): (a) scour hole generated by a current (or a wave); (b) scour hole after the initially generated scour hole is backfilled; S is the depth of the scour hole after the backfilling process attains its equilibrium; from Sumer *et al.* (2013)

A.3.2 Group Scour

Sumer and Fredsøe, 1998 [57] presents results of an experimental investigation on scour around pile groups with different configurations exposed to waves:

- The smaller the pile spacing, the larger the interference between piles. For very small pile spacing the pile group behaves as a single body. The interference affect disappears for a pile spacing ratio of S/D > 3.

By Sumer and Fredsøe, 2002 [60]:

- Sheltering by the front piles can decrease the approach velocity at the rear piles, resulting in decreased scour depths. This effect is augmented by sediment being deposited downstream of the first row of the pile group.

Ataie-Ashtiani and Beheshti, 2006 [1] also reported that the group scour influence for pile spacing ratio S/D ≥ 3 is negligible. In the same work, the scour for a pile spacing ratio of S/D = 2 is approximately 20% more than that for a ratio of S/D = 3.

Amini *et al.*, 2012 [2] performed experimental studies on clear-water scour at pile groups under steady flows and concluded the following:

- The scour depth tends to decrease with increasing S/D, S being the lateral spacing and D the pile diameter, because of a progressive decrease in the influence of neighboring piles on the development of scour and because the wake vortices shed from the front piles interfere with flow at the rear of the piles.

Mostafa, 2011 [43] presents conclusions based on an experimental study of scour at single pile and pile groups due to waves and currents:

- The scour depth around the pile groups subjected to waves against the current is substantially lower than those subjected to currents only.
- Measured scour depth with the presence of waves against current from this study indicates that scour decreases significantly from 16% to approximately 38% for the case of currents only.
- The case of side-by-side pile arrangement induced more scour compared to the case of tandem and the case of three piles with a triangular arrangement. The difference between measured scour depth in the case of side-by-side and tandem conditions became maximum at S/D =1 (difference of about 18% to 20%). For S/D between 2 and 4, the difference between measured scour depths in both conditions was between 3% and 8%.

A.4 Countermeasures

Scour countermeasures can be generally categorized into two groups:

- armouring countermeasures; and
- flow altering countermeasures.

Deng and Cai, 2010 [14] compared the two types of scour countermeasures. Flow altering countermeasures disadvantages includes: special design for particular site conditions and significant cost and construction of new structures. Armoring countermeasures disadvantages includes: winnowing of sands through the armor; difficult to keep the armor in place; and constriction causing additional scour.

Lauchlan and Melville, 2001 [38] concluded that the most commonly used armoring countermeasure is riprap.

Lagasse et al., 2007 [37] recommended placing the riprap layer at depth below the average bed level.

A.4.1 Bridge Scour

Literature references state that it may not be economical to design the elements of a bridge to withstand the maximum possible scour, therefore, the main alternative is to carry out scour protection works such as ground improvement or use of rip-rap to prevent or reduce scour of the seabed.

Mitchell and Cooke, 1995 [42] suggested that before installation of bridge structures, the whole seabed extension area that is expected to suffer scour should be improved by means of specific treatment methods on ground improvement for liquefaction mitigation.

Chen et al., 2012 [9] concluded from physical modeling that scour protection with different rock gradations was the most effective in preventing scour around a twenty-eight pier group of sea-crossing bridge foundation.

From Bridge Pier Scour Assessment for the Northumberland Strait Crossing Report [4], an extensive monitoring program has been recommended in order to quantify potential scour events over the design life of the Northumberland Strait Crossing Project, and to document scour which may occur around the base of the bridge piers. This monitoring program has been recommended for several reasons, as noted below:

- the scour assessment methodology utilized for that project was new, and has never been applied to bridge piers, waves and currents, or design of any kind;
- there were uncertainties associated with the estimation of both the driving forces for scour and the seabed resistance to scour;
- there was a desire to minimize seabed survey requirements around the base of the bridge piers associated with the monitoring program.

The monitoring program would consist of a near real-time wave and tide prediction system installed at the site. The prediction system would utilize numerical models similar to those used to develop the design database for that project. This system would run continuously, and would quantify the magnitude of potential scour events to which the bridge is exposed.

A.4.2 Trestle

Simarro et al., 2011 [53] presents results on riprap sizing for pile group protection against local scour and assumes the thickness of the sediment layer to be of minor importance as long as it is at least 2*D50 (median riprap diameter).

Dey et al., 2006 [16] presents results of an experimental study on the control of scour using cables wrapped spirally forming threads on the pile and a splitter plate at vertical circular piles under monochromatic waves and a steady current, as shown in Figure A-2. The splitter plate divides the flow by two sides of the pile and disrupts the vortex shedding from its usual frequency, whereas for threaded piles, the helical wires that form the threads disturbs the vortex shedding. In the study, the maximum reduction of scour depth obtained was 46.3% by using a triple threaded pile having a thread angle of 15° and a cable–pile diameter ratio of 0.1. In a steady current, the threaded pile proved to be effective to control scour depth to a great extent. The average reduction of the scour depth by the splitter plate was 61.6%. The methods recommended to control scour depth are easy to implement and inexpensive.

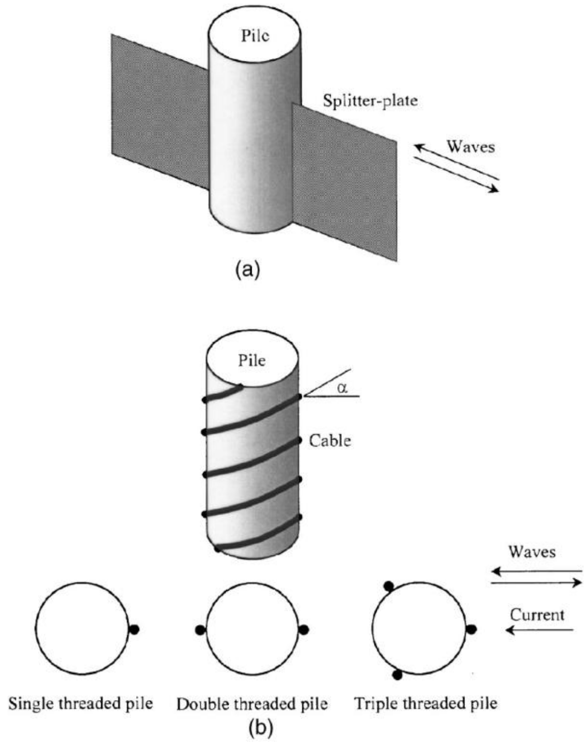

Figure A-2: (a) Splitter plate attached to the pile along the vertical plane of symmetry; (b) threaded pile (helical wires or cables wrapped spirally on the pile to form thread) from Dey *et al.* (2006)

B. References

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