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Balticconnector Offshore

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BALTICCONNECTOR OFFSHORE – OFFSHORE PIPELINE FEED REPORT

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1 Introduction

1.1 General introduction

Gasum Oy (Baltic Connector Oy) and Elering are planning for a bi-directional natural gas pipeline, which connects Finland and Estonia. The name of the project is Balticconnector. The connection of national gas transmission networks would significantly improve the regional availability and security of gas supply, and thus enhance the reliability of energy transmission in various circumstances in Finland and the Baltic countries.

The Balticconnector natural gas pipeline project is categorised as a priority project in the European Union (EU) and has therefore already been previously granted community financial assistance from the TEN (Trans-European Networks) – a programme founded by the EU.

Balticconnector is included in the list of Projects for Common Interest (PCI) and is co-financed by the EU's Connecting Europe Facility (CEF). The Balticconnector pipeline consists of three sections:

- Approximately 22 km onshore pipeline in Finland (including a compressor and custody metering station),
- Approximately 80 km offshore pipeline,
- Approximately 47 km onshore pipeline in Estonia (including a compressor and custody metering station)

This report is only concerning the Front-End Engineering Design (FEED) of the +/- 80 km offshore pipeline.

In this report, the starting point for the offshore pipeline is close to Inkoo in Finland, which is located approximately 50 km west of Helsinki. The offshore pipeline termination point is close to Paldiski in Estonia approximately 50 km from the capital Tallinn. In Finland, the landfall location will be on the peninsula of Fjusö. In Estonia, the landfall location will be in Lahepere Bay on the Paldiski peninsula.

Figure 1-1 Proposed location of Balticconnector offshore pipeline

The offshore pipeline system consists of a 20'' pipeline and the total length of the pipeline is approximately 80 km, with the precise length defined during the FEED phase following route optimisation. The system schematic of the proposed Balticconnector pipeline is illustrated in Figure 1-2.

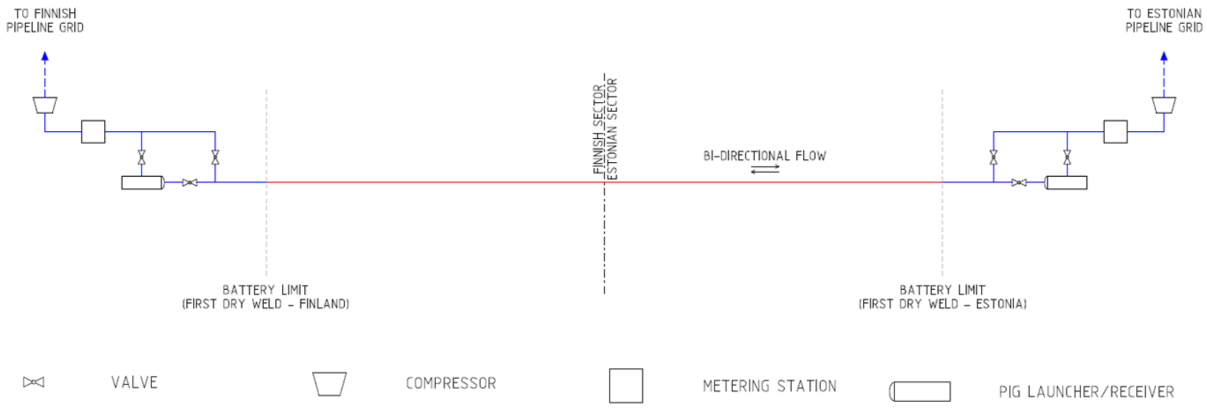


Figure 1-2 System schematic of Balticconnector offshore pipeline

1.2 Scope of this document

The scope of this document is to produce a FEED of the offshore pipeline to form a basis for the detailed engineering, procurement and management of the offshore services for the Balticconnector project.

The purpose of the FEED activities is to further develop, define and document the business case based on the selected concept to such a level that final project sanctioning can take place, the applications to the authorities can be submitted and the basis for detail engineering can be established. A success criterion for the FEED work is to enhance the technical definition of the pipeline by sufficient front end loading. This will reduce the number of late project changes during the detail engineering phase and pave the way for a successful project execution.

1.3 Acronyms

A&R	Abandonment & Recovery
AE	Asphalt Enamel
AIS	Automatic Identification System
API	American Petroleum Institute
BE	Best Estimate
BOP	Bottom-Of-Pipeline
CD	Concrete Density
CEF	Connecting Europe Facility
CP	Cathodic Protection
CWC	Concrete Weight Coating
DAF	Dynamic Amplification Factor
DCC	Displacement Controlled Criteria
DFI	Design, Fabrication, Installation
DNV	Det Norske Veritas
MDPE	Medium Density PolyEthylene
MSL	Mean Sea Level
MTO	Material Take-Off
N/A	Not Applicable
NORSOK	Norsk Sokkels Konkuranseposisjon
OD	Outer Diameter
OIMR	Offshore Inspection, Maintenance and Repair
OOS	Out-Of-Straightness
PCI	Projects for Common Interest
PE	PolyEthylene
PP	PolyPropylene
PU	PolyUrethane
QRA	Quantitative Risk Assessment
RBI	Risk-Based Inspection

EU	European Union
FBE	Fusion-Bonded Epoxy
FDW	First Dry Weld
FE	Finite Element
FEED	Front End Engineering Design
FMI	Finnish Meteorological Institute
GB	Global Buckling
GT	Gross Tonnage
HAT	Highest Astronomical Tide
HDPE	High Density PolyEthylene
HFW	High Frequency Weld
HSE	Health, Safety and Environment
ID	Inner Diameter
ISO	International Standardization Organisation
KP	Kilometre Post
LAT	Lowest Astronomical Tide
LB	Lower Bound
LBC	Local Buckling Criteria
LCC	Load Controlled Criteria
LDPE	Low Density PolyEthylene
RP	Return Period
SAWL	Submerged Arc Welding Longitudinal
SNCF	Strain Concentration Factor
SRI	Subsea Rock Installation
SSS	Side Scan Sonar
TBA	To Be Announced
TBD	To Be Decided
TEG	Tri-Ethylene Glycol
TEN	Trans-European Networks
TOP	Top-Of-Pipe
TPL	Trawl Pullover Load
TSS	Traffic Separation Scheme
UB	Upper Bound
UHB	UpHeaval Buckling
ULS	Ultimate Limit State
UT	Utilisation
UTM	Universal Transverse Mercator
WGS	World Geodetic System
WT	Wall Thickness

2 Summary, conclusions and recommendations

2.1 Summary

The Balticconnector offshore pipeline is characterised by:

- 80.392 km long 20" OD bi-directional gas transmission pipeline
- Routed along a predominantly soft to firm clay seabed with bedrock outcrops ranging between 0 m to 100 m water depth
- Highly uneven seabed resulting in numerous free spans
- Crossing of major shipping lanes
- Crossing of two (which may possibly become four) 48" Nord Stream pipelines and 17 known subsea cables

The FEED for the Balticconnector offshore pipeline comprises:

- Value engineering for pipeline dimensions, coating and material selection in accordance with DNV- OS-F101
- Final routing and landfall selection based on alternatives outlined in the pre-FEED report, Ref. /31/
- Route optimisation to minimise required seabed intervention, rock infill and/or dredging of free span shoulders. Alignment sheets for entire route have been prepared.
- General pipeline engineering and preparation of MTOs for line pipes, corrosion and concrete weight coating and anodes for ITT of contractors

2.2 Conclusions

2.2.1 System description

An offshore pipeline system consisting of a 20'' OD pipeline and the total length of the pipeline is approximately 80.392 km from shore to shore.

The design life of 50 years is designed to withstand design pressures of 80 barg and maximum and minimum design temperatures of 50°C and -10°C respectively.

2.2.2 Routing

The Balticconnector offshore pipeline route runs from KP 0.000 at the Finnish landfall on the Fjusö peninsula to KP 80.392 at the Estonian landfall on the Pakri peninsula.

Due to the rough seabed formed of a mixture of soft clays between bedrock outcrops, the total number of lay curves along the route is 50, with a total curve length of 27,735 m. All lay curves have radii greater than 1200 m to avoid curve instability.

2.2.3 Pipeline mechanical design

The pipeline material shall be procured as DNV HFW 450 F D Carbon steel, subject to pipe mill availability. Note that not all pipeline mills offer HFW pipe, in which case SAWL pipe is an equally accepted alternative.

A steel wall thickness of 12.7 mm is selected for the entire Balticconnector offshore pipeline route, governed by the propagation buckling design criteria and minimum required wall thickness for pipelines greater than 12” OD.

2.2.4 Cathodic protection and anti-corrosion coating

The pipeline will be coated in low density polyethylene (LDPE) anti-corrosion coating. A 3-layer thickness of 3.5 mm is suitable for pipelines with a design temperature less than 60 °C and the thickness adds robustness to the coating for transportation and handling. The density of the coating is estimated to be approximately 930 kg/m^3.

Given that the Balticconnector pipeline is not subject to heavy trawling and is protected (buried or rock covered) in areas with a high frequency of shipping activity, the field joint coating does not need to be able to sustain significant impacts. Therefore, it is proposed that polyurethane (PU) foam is applied as infill over a heat shrinkable sleeve between the adjoining concrete coatings at the field joints during installation. This is common practice on S-lay pipelay vessels and ensures a fast lay rate of the pipeline as the PU foam will cure sufficiently before the field joint passes over the stinger. A cutback length of 240 mm and 340 mm on the anti-corrosion coating and concrete weight coating is assumed based on project experience.

The cathodic protection requirement is calculated and designed using Al-Zn-In anode material. The number of anodes has been calculated for both the exposed and buried pipeline condition and the most onerous design (exposed) is adopted. The below Table 2-1 summarises the anode requirement for the pipeline, including one anode to protect the onshore pulled-in section at the Estonian landfall, which is below the water table. For pipeline joints with 55 mm or 80 mm thick concrete coating, the coating has be to tapered to the anode thickness with a 45 degree angle.

Pipe condition	Anode ID	Anode Thickness	Anode Length	Individual Anode Weight	Anode Spacing	Total No. Of Anode	Total Anode Weight
[-]	[mm]	[mm]	[mm]	[kg]	[Joints]	[No.]	[kg]
Exposed	515.20	40	600	104.56	12	551	57,613

Table 2-1 Anode summary

2.2.5 On-bottom stability

The concrete weight coating is selected so that the pipeline is laterally stable for the given environmental loading during the entire design life. The FEED is based on the latest directional wave and current modelling given in the *Metocean Data Report*, Ref. /35/.

The adopted concrete coating thicknesses are summarised in Table 2-2.

KP Locations		Distance [m]	Recommended Concrete Coating Thickness [mm]	Concrete Coating Density [kg/m^3]
From	To			
0.000	19.350	19,350	55	3400
19.350	26.000	6,650	80	3400
26.000	80.392	54,392	45	3400

Table 2-2 Concrete weight coating summary

The changes in concrete thicknesses and density along the pipeline have been reduced to aid in logistics. To facilitate procurement and installation, the same concrete density should be maintained for the entire pipeline, and the number of different coating thicknesses kept at a minimum.

Note that at two locations, between KP 19.350 – 19.835 and KP 20.860 – 21.000, a localised solution must be provided to meet the design stability criteria. Both locations have been identified in the local buckling assessment as areas in need of rectification. Therefore, the final solution depending on the potentially modified seabed water depth will need to be subject to localised solutions such as subsea rock installation of the pipeline or concrete mattress stabilisation methods.

The vertical stability of the pipeline, i.e. its buoyancy, is summarised in Table 2-3. Subsea pipelines must have a minimum specific gravity of 1.1. To ensure the pipeline is laid at the bottom of the trench during installation, a specific gravity of 1.6 is best practice (not including during pull-in operations).

Sr. No	Concrete Weight Coating (mm)	Concrete Coating Density (Kg/m^3)	Specific Weight S_g = *(submerged weight + buoyancy) / buoyancy*		
			Installation	Flooded	Operation
1	55	3400	1.61	2.20	1.64
2	80	3400	1.86	2.37	1.89
3	45	3400	1.49	2.12	1.53

Table 2-3 Pipeline buoyancy summary

2.2.6 Free span analysis and bottom roughness assessment

A free span analysis is performed to determine the allowable span lengths for the pipeline along the entire route.

The allowable span lengths are divided in sections based on varying input parameters to the analysis, such as pipeline coating thickness and weight, water depth, wave and current data, soil properties and heading. The pipeline is assessed in both the empty phase and operational phase for the FEED to determine whether pre-lay or post-lay rectification would be required.

The results provide a screening criterion to determine which spans identified in the bottom roughness assessment will need to be re-assessed using location specific details. Once this assessment is made in the detailed engineering phase, the decision to perform free span rectification through seabed intervention can be made. For the FEED phase, all spans greater than the allowable span length have been assumed to require free span rectification.

The allowable span lengths for given KP ranges and the number of spans not meeting this criterion are provided in Table 2-4.

KP Range		Min water depth (m)	Allowable span length (m)		Number of spans requiring pre-lay rectification		Number of spans requiring post-lay rectification		
To	From		Emp.	Op.	Fatigue	LBC	Fatigue	LBC	GB
0.038	0.155	-5.0	58	36	-	-	-	-	-
0.155	3.000	-8.7	60	36	2	-	2	-	-
3.000	6.000	-14.4	66	40	1	-	3	-	-
6.000	8.200	-17.6	64	35	-	-	-	-	1
8.200	13.200	-17.0	35	26	5	-	6	1	11
13.200	14.120	-23.1	64	29	2	2	6	-	8
14.120	19.350	-24.9	68	36	1	14	7	3	21
19.350	19.812	-16.2	35	21	5	2	7	2	7
19.812	20.860	-23.5	46	26	4	5	8	3	11
20.860	21.028	-17.2	33	20	2	4	3	-	1
21.028	22.400	-29.6	46	28	2	3	2	-	3
22.400	24.700	-38.2	65	35	-	7	2	-	3
24.700	25.400	-27.9	44	27	2	16	4	-	9
25.400	26.000	-40.4	70	36	-	-	1	-	-
26.000	33.650	-50.2	70	41	-	-	-	-	-
33.650	43.700	-56.2	70	42	-	-	-	-	1
43.700	51.500	-54.7	70	42	-	-	3	-	10
51.500	62.250	-56.3	70	43	-	-	-	-	4
62.250	65.000	-73.1	70	54	-	-	-	-	2
65.000	73.300	-34.9	70	39	-	-	-	-	-
73.300	79.035	-11.7	38	22	-	-	-	-	-
79.035	79.564	-5.0	46	25	-	-	-	-	-
				Total	**28**	**53**	**54**	**9**	**92**
				Accumulated total	**70**		**56**		

Note:
The accumulated total includes overlapping spans between design criteria, i.e. if one span requires rectification due to both fatigue and local buckling design criteria, it is only considered to be one span in the accumulated total. The post-lay accumulated total incorporates spans that have already been rectified by pre-lay activities.

Table 2-4 Free span and bottom roughness summary

All spans in the fatigue analysis are considered as isolated, single spans. The coupling effect of adjacent free spans will be a consideration for the detailed engineering phase and therefore the allowable span length in Table 2-4 does not incorporate the changes in frequency and amplitude resulting from coupling. Conservatisms of the analysis through the safety factors, soil stiffnesses and fatigue damage distribution compensate for the lack of coupling effect; consequently, the overall quantity of free span rectification due to fatigue damage is expected to decrease.

The bottom roughness analysis has shown the need for pre-lay seabed intervention for a total of 70 free spans and post-lay seabed intervention for a total of 56. Based on this collected data, the seabed intervention required to mitigate the stress or fatigue in the pipeline will be estimated as a rock volume or blasting/excavation volume to determine an overall cost estimate for these offshore activities.

By assessing each free span location identified in this FEED phase that requires seabed intervention in detail in the following phase, the quantity of seabed intervention can be reduced.

2.2.7 Local buckling analysis

24 locations with a local buckling (LBC) utilisation (UT) ratio of more than 0.9 have been identified along the route of the Balticconnector pipeline, as shown in Table 2-5. It is found that all necessary seabed intervention to mitigate local buckling is located in the northern part of the Gulf of Finland between KP 12 - 26.

No.	KP @ max UT	MSL	LB CUT	Method of intervention[1)]			Design complexity	Comments
				Pre-lay SRI	Post-lay SRI	Soil/rock removal		
[-]	[km]	[m]	[-]	[-]	[-]	[-]	[-]	[-]
1	12.242	-19.6	1.12	X	X		Low	Potentially prone to upheaval buckling to be mitigated by post-lay SRI
2	13.919	-26.5	1.13	X			Low	
3	16.193	-24.9	0.93	X			Low	
4	16.981	-28.3	1.08		X		Low	
5	17.426	-26.5	1.58	X		X	Low	Potentially removal of soil/rock might be omitted – to be further investigated
6	17.840	-31.5	1.32	X			Low	
7	18.248	-26.5	2.17	X		X	High	Further mitigation option to be evaluated[4)]
8	18.490	-34.0	0.97		X		Low	
9	18.729	-26.5	1.71	X[2)]		X	Medium	Further mitigation option to be evaluated[5)]
10	18.795	-26.5	1.03	X[2)]			Low	
11	18.982	-25.8	1.40	X		X	Low	Potentially removal of soil/rock might be omitted – to be further investigated
12	19.364	-24.3	1.90	X		X	High	Further mitigation option to be evaluated[5)]
13	19.735	-20.9	1.12		X		Low	
14	19.894	-27.6	0.90		X		Low	
15	20.263	-23.6	1.45	X			Low	
16	20.915	-17.2	1.76			X	Medium	Removal of rock required[5)]
17	21.193	-29.6	1.03	X			Low	
18[3)]	22.288	-31.7	1.33	X			Low	
19[3)]	22.371	-36.0	1.66	X		X	Medium	Further mitigation option to be evaluated[4)]
20	24.277	-39.0	1.79			X	Low	Removal of rock required
21	24.391	-41.0	1.05	X			Low	
22	24.753	-35.8	0.95	X			Low	High accuracy pre-lay installation i.e. -0/+0.2 m
23	25.104	-28.4	1.21	X			Low	
24	25.324	-28.0	2.02	X		X	High	Further mitigation option to be evaluated[5)]

Notes:
1) Pre-lay refers to installation prior to the installation of the pipeline while post-lay refers to installation prior to water-filling.
2) SRI intervention to be performed will influence both locations
3) Outside survey corridor on geophysical survey, Ref. /32/ (Doc. ALIGN013)
4) Recommended mitigation action includes re-routing potentially by means of counteracts – to be further investigated
5) Recommended mitigation action includes blasting – to be further investigated

Table 2-5 Summary of high local buckling utilisation locations

18 locations are defined as low complexity, 3 as medium complexity and 3 as high complexity. It is noted that the uncertainty of estimated rock installation and removal volumes are associated with the complexity of the design.

Of the two types of pre-lay and post-lay rock installation, pre-lay rock installation is associated with the greatest level of uncertainties. This is because post-lay rock installation can be installed relative to the as-laid pipeline while pre-lay rock installation design has to include installation tolerances. Typically, also the line load carried by pre-lay supports is greater than the load carried by post-lay rock supports. This is mainly due to that pre-lay support will carry the pipeline from installation i.e. effectively changing the configuration of the pipeline compared to a free spanning pipeline. Post-lay support first becomes effective in subsequent phases i.e. water-filling, pressure testing and operation.

2.2.8 Global buckling design and trawl pull-over analysis

A global buckling analysis of the entire pipeline has been introduced at the FEED phase to determine the effects of the functional load (pressure and temperature) on the pipeline combined with potential trawl pullover loads. The key inputs to the analysis include a temperature and pressure profile of the pipeline, the pipeline vertical profile on the seabed and trawl loads and sizes to determine the trawl pullover loads.

By conservatively estimating the temperature profile of the pipeline given bi-directional flow, the analysis was separated into three sections; the Finnish nearshore region, the Estonian nearshore region and the offshore region (between the two).

The global buckling mitigation technique is to rock cover the pipeline at the nearshore regions until the temperature in the pipeline reduces sufficiently to decrease the effective axial force in the pipeline. With a lower temperature, the functional loads in combination with the trawl pullover loads do not result in global buckling failure of the pipeline.

However, in the offshore region the cooled pipeline may still trigger global buckling behaviour. In the highly utilised pipeline, where the utilisation ratio of the load controlled local buckling criteria is above 0.3, a large span height at the pullover location (resulting in an increased duration of the trawl pullover load) can lead to global buckling failure. The solution in the offshore region is to mitigate each span which is highly utilised with post-lay rock cover up to a certain height from Bottom-Of-Pipe (BOP) to ensure the trawl pullover load is minimised.

The result of the analyses for each section is presented in Table 2-6.

KP	Length (km)	Mitigation	Cover height above TOP (m)	Restrictions
0 – 4.5	4.5	Rock covered	0.5	N/A
4.5 – 12.0	7.5	Exposed	N/A	Max span height = 0.3 m
12.0 – 19.0	7.0	Exposed	N/A	0.7m span height for UT < 0.30 0.4m span height for UT < 0.45 0.3m span height for UT < 0.60
19.0 – 21.0	2.0	Exposed	N/A	Max span height = 0.8 m
21.0 – 26.0	5.0	Exposed	N/A	Max span height = 0.7 m
26.0 – 67.5	41.5	Exposed/rock covered	Various	0.7m span height for UT < 0.30 0.4m span height for UT < 0.45 0.3m span height for UT < 0.60
67.5 – 74.9	7.4	Exposed	N/A	N/A
74.9 – 79.2	4.3	Rock covered	0.0	N/A
79.2 – 80.4	1.2	Buried	N/A	N/A

Table 2-6 Summary of global buckling solution

2.2.9 Seabed intervention

Seabed intervention has been specified for the Balticconnector pipeline based on the following engineering activities:

- Load controlled local buckling design criteria of the empty, flooded and operational pipeline
- Crossing requirements for the Nord Stream pipelines and subsea cables
- Fatigue design criteria for the free spanning pipeline
- HSE protection requirements for dragged anchors
- Landfall design at both Finnish and Estonian ends
- Global buckling and upheaval buckling mitigation

A total volume of 244,539 m^3 is envisaged to be installed to fulfil the protection strategy defined. Approximately 30,838 m^3 is defined as pre-lay and 213,702 m^3 as post-lay rock installation. The requirement for removal of bedrock amounts to 1,325 m^3. Note that all volumes are theoretical design volumes, which does not consider installation equipment of contractor for excavation width or conservative “over-dumping” of subsea rock installation.

A summary of the seabed intervention required for each engineering activity is presented in Table 2-7.

Design requirement	Volume (m^3)		
	Pre-lay	Excavation	Post-lay
Local buckling free span intervention	23,100	1,325	3,700
Fatigue free span intervention	2,287	-	1,252
HSE protection requirements +0.5 m TOP	-	-	160,626
Landfall protection at 5-10 m water depth (+1.0 m TOP)	-	-	8,087
Global buckling - nearshore	-	-	35,080
Global buckling - offshore	-	-	3,501
Crossings	5,451	-	1,456
Total	**30,838**	**1,325**	**213,702**
Note that all volumes are theoretical design volumes, which does not consider installation equipment of contractor for excavation width or conservative "over-dumping" of subsea rock installation.			

Table 2-7 Total estimated subsea rock installation volumes and excavations volumes for the Balticconnector offshore pipeline

2.2.10 Landfall design

The selected Finnish landfall site is situated on the Fjusö peninsula close to Inkoo and the selected Estonian landfall site is situated to the south of Lahepere bay near Paldiski. Both sites were selected due to beneficial technical, environmental and social conditions including ease of permitting on land at the given locations. The coordinates are summarised in Table 2-8.

Landfall location	WGS84 - UTM 35N		Aerial photo of landfall location
	Easting [m]	Northing [m]	
Inkoo Finland	330 769	6 656 682	
Paldiski Estonia	339 933	6 581 949	

Table 2-8 Summary of selected landfall locations

At the Finnish landfall, the worksite for the winch and foundation is to be setup approximately 90 m from the shore, beyond a 10 m high ridge located adjacent to the coastline. The pipelay vessel will be located approximately 500 m from shore where the water reaches depths greater than 10 m. Prior to the pull-in operation between the winch and pipelay vessel, an offshore and onshore trench will have to be blasted and excavated, and then filled with gravel to avoid pipeline abrasion with the bedrock. Once the pipeline termination head has reached its destination on the worksite, the offshore trench can be backfilled with the excavated material and the onshore trench reinstated.

At the Estonian landfall, the worksite size is limited by the close proximity of a steep embankment and public highway close to the coastline. Therefore, to perform the pull-in operation the winch will be located offshore with the pull-in wire positioned around a sheave on the worksite. Onshore excavation will be required for the worksite to create enough length for the pipeline termination head to rest above MSL after pull-in.

The offshore trench for the pipeline up to a water depth of 5 m will extend 830 m from the shore due to the flat sandy seabed. In preparation for excavating the trench, all boulders within the trenching corridor must be removed. The initial length of the offshore trench and part of the onshore trench will be buttressed by a cofferdam to prevent natural backfill of the trench by sediment transportation before the pull-in operation. The cofferdam length is estimated to be 500 m long at this stage of the project. The pipelay vessel will be located approximately 1.3 km from shore for the pull-in operation where water depths reach 10 m. After the pull-in operation, the offshore trench will be backfilled and the onshore site reinstated, specifically at the beach location which is designated for public use, and the location of the gas pipeline should be marked with notices.

2.2.11 Pipeline installation

The pipeline installation analyses have been carried out for the Balticconnector offshore pipeline using a typical S-lay installation vessel in the software Orcaflex. As water depth along the pipeline route varies significantly, two stinger configurations have been identified for various load cases. The Balticconnector pipeline will be pulled-in from the pipelay vessel to landfall locations at the Finnish and Estonian shores, followed by standard pipelay on the seabed, and subsequently welded together with an above water tie-in (Davit lift).

The selected stinger configurations are based on project experience and static calculations with adjusted DAFs, and therefore a detailed dynamic study shall be carried out in the next phase of the project.

The selected stinger configurations with stinger radii of 160 m and 300 m are denoted R_160 and R_300, respectively.

The installation cases and detailed stress utilisations are described in section 12. The result summary of top tension, residual lay tension and minimum lay radius are outlined in Table 2-9.

20" Balticconnector S-Lay Analysis Results (Pipe Empty)											
Item		Pipeline installation cases									
	Unit	BCP0	BCP1	BCP2	BCP3	BCP4	BCP5	BCP6	BCP7	BCP8	BCP9
KP Range	-	0 – 11	11-19	72 – 80.4		19 – 26		26 - 36	36 – 55		55 – 72
Water depth	[m]	30	40	30	40	30	52	56	70	80	100
Soil properties	-	Clay	Clay	Clay	Rock	Rock	Rock	Clay	Clay	Rock	Clay
Stinger configuration	-	R_300	R_300	R_300	R_300	R_300	R_160	R_160	R_160	R_160	R_160
Pipeline top tension	[kN]	312	351	203	242	650	764	384	486	549	662
Residual lay tension	[kN]	253	278	143	174	609	654	133	174	205	295
Min. stable lay radius	[m]	826	908	450	251	402	431	575	754	296	1280

Table 2-9 Pipeline installation results for empty case

A Davit lift analysis has been carried out at the Estonian side of the pipeline route at approximately 25 m of water depth on a relatively flat seabed. A detailed result for the analysis is given in section 12, and summary of tensions on the pull-in winch and pipeline residual tension is outlined in Table 2-10.

It is assumed that the vessel will be equipped with a total of 6 winches, i.e. 3 winches will be used to lift a pipeline section. These winches are denoted as Winch A, B and C, where winch A is nearest to the centre of the vessel. It should be noted that a symmetrical distribution of winches has been assumed, which is also imitated in the results.

Pipeline profile	Water depth [m]	Pull-in load on winch [kN]			Residual Tension [kN]
		Winch A	Winch B	Winch C	
Pipeline_Finland	25	196	490	345	658
Pipeline_Estonia	25	196	490	345	658

Table 2-10 Pull-in loads on winches and residual lay tension during the davit lift procedure

2.3 Recommendations

It is recommended that the next phase of the project allows for the following considerations.

<u>Increased gradiometer survey</u>
A 'security corridor' of 2 x 50 m centred on the pipeline trace is requested in the Survey Specification, Ref. /37/, in which all munitions found shall be cleared, using a high-resolution Side Scan Sonar (SSS) and array of magnetometers (i.e. gradiometer). To provide the optimal engineering solution during the detailed engineering phase, a wider gradiometer survey corridor is essential at several locations where significant seabed intervention has been identified, allowing localised re-routing using concrete counteracts. The precise locations of the widened corridor are specified in the Survey Specification, Ref. /37/.

<u>Dragged anchor protection study</u>
To determine the optimum protection design for the pipeline, a study to determine the interaction between dragged anchors and subsea rock protection should be initiated. This should include a cost-benefit analysis using subsea rock installation compared to trenching

and backfilling, taking into account the limitations of trenching on a seabed with bedrock outcrops.

Trawling activity study
A detailed trawling activity assessment, specifically along the proposed Balticconnector offshore pipeline route, should be performed. If, as expected, trawling activities in the region and specifically around the Finnish archipelago are less severe than the design data used in FEED, significant optimisation to the global buckling design can be made, i.e. reduction in rock protection in the nearshore regions and the removal of span infills in the offshore section.

Soil investigations
To perform a safe and cost-effective design of pre- and post-lay pipe supports and crossings, additional soil investigations should be carried out to assess the strength parameters, in particular of the soft clay strata.

Pipe-soil interaction modelling
In order to ensure a robust design, conservative parameters have been used in the FEED, but project experience shows that significant improvements can be made to the design of the seabed intervention through greater understanding of pipe-soil behaviour. A detailed breakdown of soil properties and accurate modelling along the pipeline route should be performed, notably with respect to the pipeline penetration into clay, and hence the requirements for rectification of free spans.

Flow simulations
By determining accurate temperature and pressure profile along the routes for both flow directions, inputs to the global buckling analysis, local buckling analysis and free span analysis can be optimised.

Free span reassessment
All identified free spans should be subjected to a location specific reassessment of rectification, comprising detailed soil properties including damping effects as well as location specific functional and environmental parameters.

Free span coupling
Free span analysis in the FEED phase has been carried out assuming isolated, single spans. However, once all acceptable free spans have been identified, the effect of coupling between adjacent spans should be taken into account before determining if free span rectification is required.

Fatigue damage distribution
To reduce the need for post-lay free span rectification, the allocation of allowable fatigue damage to the operational phase should be maximised in close collaboration between pipelay contractor and pipeline design engineers.

Updated bottom roughness assessment
The bottom roughness analysis should be revisited in the detailed engineering phase and should include detailed geotechnical data based on the 2016 survey data, worst case temperature profiles and known exposure times of empty, flooded, system pressure test and operational phases.

3 Design basis

3.1 General

The Balticconnector pipeline is a 20'' pipeline routed from Inkoo (Finland) to Paldiski (Estonia), a distance of 80,392 km.

A more comprehensive basis for design activities is provided in the *Offshore Pipeline Design Basis*, Ref. /34/.

3.2 Coordinate system

The following coordinate system (also referred to as EPSG:32635) shall be used as common reference for the project:

Coordinate system:	WGS 84 / UTM zone 35N
Geodetic Datum:	World Geodetic System 1984 (WGS 84/EUREF 89)
Transformation method:	Transverse Mercator
Area of use:	24 deg East to 30 deg East; northern hemisphere
Unit:	Metres (m)

3.3 Design life

The design life is envisaged at 50 years.

3.4 Battery limits

The Balticconnector pipeline consists of a pipeline between Inkoo in Finland and Paldiski in Estonia and will be equipped with a compressor station in Inkoo and possibly in Estonia.

Two alternative routes into Inkoo and the two alternative routes into Paldiski are presented and will be assessed in the FEED. This design basis covers only the offshore part of the pipeline and does not include the part where the pipeline has reached shore and thereby not the compressor stations. The battery limits are considered to be at the first dry welds at each landfall location, cf. Figure 1-2.

For the landfall design, interface with onshore activities, particularly with respect to the onshore pipeline route, construction site specifications and land usage plans will be necessary to produce the optimum design. The extent of interfacing will be clarified during the landfall design procedure during the FEED phase.

3.5 KP system

A pipeline Kilometre Post (KP) system shall be established for the entire Balticconnector transmission system; however, the focus of this report is only on the offshore pipeline.

For the offshore pipeline KP 0.000 shall be defined at the first dry weld between the offshore and onshore pipeline at landfall in Finland. The KP numbering shall be increasing towards the south.

3.6 Seawater properties

According to the *Environmental Impact Assessment Report Finland*, Ref. /30/, the sea bottom temperature is 4 - 6°C, and the salinity is approximately 0.6%. The corresponding seawater density range is approximately 1005 kg/m^3.

In the *Metocean Data Report*, Ref. /35/, the variations in seabed water temperature, salinity and density are reported at four positions along the Balticconnector route; KP 15 (nearshore Finland), KP 25 (offshore Finland), KP 60 (offshore Estonia) and KP 73 (nearshore Estonia). At each position, the minimum, maximum, mean and standard deviation values are given for each month, as well as the yearly average.

The yearly average seabed temperature ranges from 4.4°C (offshore Finland) to 6.8 °C (nearshore Finland), the minimum being -0.4 °C (nearshore Finland in January) and the maximum 21.4 °C (offshore Finland in August). The corresponding values for Estonian waters fall in between.

The yearly average salinity ranges from 0.6% (nearshore Finland) to 0.9% (offshore Estonia), minimum and maximum being 0.4% (nearshore Finland in December), respectively 1.0% (offshore Estonia in April).

The average yearly seawater density range is from 1004 kg/m^3 (nearshore Finland) to 1007 kg/m^3 (offshore Estonia), with minimum and maximum being 1002 kg/m^3 (nearshore Estonia in August), respectively 1008 kg/m^3 (offshore Estonia in April). The corresponding seawater resistivity will lie in the range of 100 - 180 Ω cm. For the FEED phase the following values are adopted at seabed level:

Seawater temperature:	5 °C
Seawater density:	1005 kg/m^3
Seawater resistivity:	1.5 Ω m

3.7 Soil data

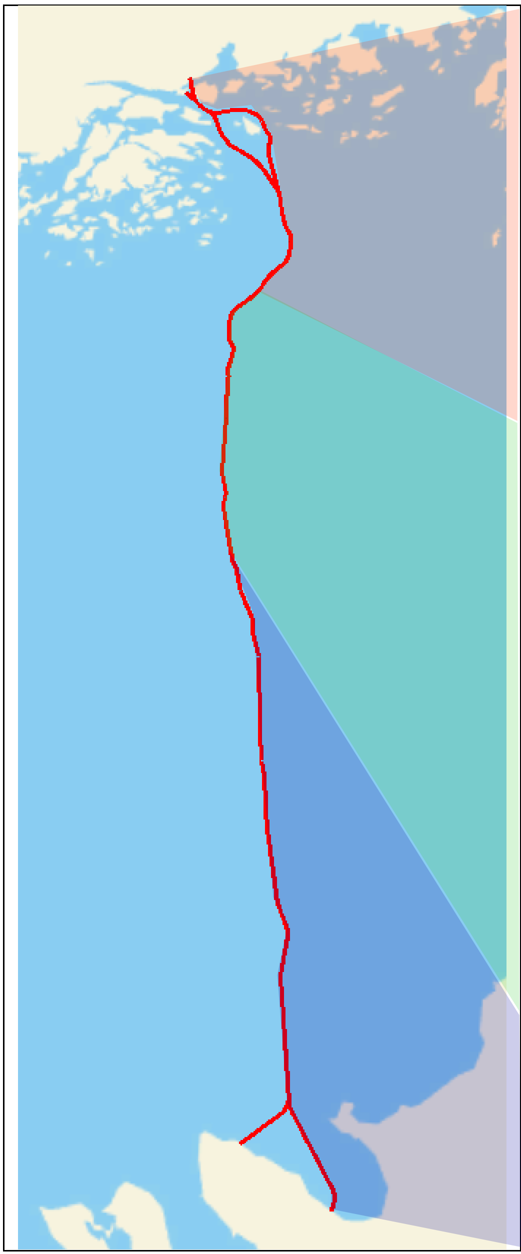
The northern part of the Gulf of Finland is characterised by crystalline bedrock (precambrian) with irregular relief and steep slopes. The bedrock is commonly observed at seabed surface as distinct outcrops. The basement depressions are filled with clay, with flat areas as a result.

The Estonian shelf is built up of a palaeozoic plateau of sedimentary bedrock overlying the crystalline bedrock. The sedimentary bedrock strata occur as an onlap sequence and are increasing in thickness southwards.

Practically all over the Gulf of Finland, till is deposited over the bedrock as ridges and infills in the basement depressions. In the northernmost part the till is dominated by a high content of large boulders, locally very large, while on the Estonian it is rich in clay and low coarse grained content.

Clay overlies the till. The lower part consists of late-glacial lacustrine deposits, represented by varved clays and over this marine sediments are found, represented by homogeneous clay. The late-glacial sediments are conformed to the underlying topography, while the postglacial clay deposits occur as basin fill-type sediments.

On the Estonian shelf the youngest deposits often consist of sands and aleurites. The seabed soil classifications are summarised in Figure 3-1.



KP 0 – KP 22

A crystalline BEDROCK relief is dominating as an underlying unit that in parts outcrops at or close to the route.

The BEDROCK is in parts draped by often relatively thin (<1m) TILL deposits with STONES and BOULDERS. In the vicinity of these hard seabed areas are normally surface layers of SAND and GRAVEL.

In larger depressions, CLAY is deposited. The uppermost CLAY unit is generally relatively soft, but is in places firm with grains of SILT and SAND, and may even include coarser sediments.

KP 22 – KP 38

A crystalline BEDROCK relief is dominating as an underlying unit that in parts outcrops at or close to the route in some sections.

The BEDROCK is in parts draped by often relatively thin (<1m) TILL deposits with STONES and BOULDERS. In the vicinity of these hard seabed areas are normally surface layers of SAND and GRAVEL.

At some locations coarse deposits with glacifluvial origin are noted. They differ from the TILL deposits, and are described in the classification as SAND and GRAVEL or GRAVEL and COBBLES. In some places these units may also include BOULDERS.

In larger depressions CLAY is deposited. The upper most CLAY unit is generally relatively soft or very soft. The very soft CLAY is mainly a GYTTJACLAY with high organic content.

KP 38 – KP 80

To a large extent the surface sediments are soft or very soft CLAYs, and in some parts SILT and FINE SAND are present. Firm CLAY is also present in sections. Closer to shore SAND and SILT are the dominating surface sediment.

Figure 3-1 Seabed surface geology along proposed route

Vibrocore samples taken along the proposed route reveal a mixture of SILT and CLAY up to a depth of 6 m. For the pipeline design, if the pipeline is not installed on bedrock, it is likely to be installed on one of the categories of soil listed in Table 3-1, where the saturated unit weight (in air) is stated.

Density	Range	Unit I Clay	Unit II Gyttja Silt and Clay	Unit III Silt and Clay
Saturated Bulk Unit Weight (kg/m^3)	Min	1260	1150	1210
	Average	1420	1300	1310
	Max	1660	1420	1380

Table 3-1 Summary of soil unit weights for cohesive soil units

More information is available in the *Geotechnical Report*, Ref. /21/.

For design purposes, a common profile for the undrained shear strength s_u (in kPa) may be used for all three cohesive soil units, as proposed by Ref. /21/:

$s_u = 4 + 1.5\ z$, where z is the depth in m.

For sandy seabed, where no soil parameters are proposed by Ref. /21/, the parameters for 'loose sand' in Table 7-1 of DNV-RP-F105, Ref. /5/, may be used.

The coefficient of friction μ for the concrete coated pipeline shall be taken as μ = 0.2 for clay and μ = 0.6 for sand and rock (SRI), in accordance with DNV-RP-F109, Ref. /8/. For bedrock the value μ = 0.2 is conservatively used.

For seabed roughness, the following values are used, cf. Ref. /8/:

Clay (including silt): 5×10^{-6}
Sand: 1×10^{-5}

3.8 Subsea rock installation properties

Properties of rock used for SRI are shown in Table 3-2, based on project experience. Pre- and post-lay SRI may be used to ensure a feasible design of the offshore pipeline.

Parameter	Rock berm design	Slope stability
Rock friction angle	45°	40°
Rock submerged density	8.73 kN/m^3	9.40 kN/m^3

Table 3-2 Rock properties for SRI

3.9 Linepipe dimensions and material properties

The linepipe properties are summarised in Table 3-3.

Parameter	Value
Linepipe Material	HFW or SAWL 450 F D
Yield Stress	450 MPa
Tensile Strength	535 MPa
Density	7850 kg/m^3
Pipe size (OD)	20" / 508 mm
Modulus of elasticity	207 GPa
Poisson's ratio	0.3
Steel design temperature (Min / Max)	- 10°C / + 50°C
Thermal expansion coefficient	1.17 x 10^{-5} °C^{-1}
Resistivity	0.2 × 10^{-6} Ω m
Corrosion Allowance	0 mm

Table 3-3 Steel linepipe data

The linepipe shall be delivered in accordance with DNV-OS-F101, Ref. /1/, and ISO 3183, Ref. /19/, for offshore service. Note that HFW can be used for 20" pipe diameters with a wall thickness less than or equal to 17.5 mm. It is considered the least expensive option, but SAWL may also be applied depending on pipe mill availability.

3.10 Pipeline coating

The applicable coating properties for the offshore pipeline are summarised in Table 3-4.

Parameter	Type	Thickness (mm)	Density (kg/m^3)
Internal flow coating	Epoxy Paint	0.1	1500
Anti-Corrosion coating	3-layer PE [1)]	3.5	930
Weight coating	Concrete	TBD [2)]	TBD [2)]
Field Joint Protection	Heat shrink Sleeve	1.5	1000
Field Joint Infill	PU foam	as concrete	1000 [3)]

Notes:
1) Justification for use of 3LPE coating is given in Appendix I.
2) Concrete coating thickness and density is to be defined in the on-bottom stability analysis, section 7.1
3) The dry foam density may be only 100 kg/m^3, but with 80% open cells the foam will be saturated.

Table 3-4 Pipeline coating properties

The coating cutback at field joints shall be assumed at 340 mm for the concrete weight coating and 240 mm for the anti-corrosion coating.

3.11 Operational data

The principal functional design data are summarised in Table 5-2 below.

Parameter	Value
Design Pressure	80 barg
Operating pressure (Min/Max)	TBA / 63 barg [1)]
Design Temperature (Min/Max)	- 10°C / +50°C
Operating Temperature (Max)	TBA
Design Gas Density	65 kg/m^3
Note: 1) Operating pressure specified in Ref. /38/	

Table 3-5 Offshore pipeline design data

3.12 Trawl data

The fishing activities in the region of the Baltic Sea do not include beam trawling, the principal methods being otter trawling and twin rig trawling. In the absence of specific data about fishing along the Balticconnector route, the relevant parameters can be taken from Table 3-6, representing the heaviest equipment in use.

Parameter	Trawl board	Clump weight
Type	Polyvalent	
Mass	3000 kg	3000 kg
Hydrodynamic added mass	6420 kg	1350 kg
Length x Height	4.5 m x 3.2 m	1.35 m x 1.0 m
Tow velocity	2 m/s	2 m/s
Warp line diameter	30 mm	30 mm

Table 3-6 Trawl equipment and pipeline data

The impact frequency is estimated to be < 1 events per km per year, corresponding to frequency class Low, as per DNV-RP-F111, Ref. /10/.

4 System design

4.1 Overall system schematic drawing

The Balticconnector pipeline project comprises a new offshore gas pipeline across the Gulf of Finland from landfall near Inkoo, Finland, to landfall near Paldiski, Estonia. The Balticconnector pipeline will allow flow in either direction, providing a transport route for natural gas to ensure delivery of energy to growing markets north and south of the Gulf. The operation objectives are to ensure that the delivery of gas to custody transfer point meets planned targets while safeguarding the technical integrity of the export pipeline.

The offshore pipeline system consists of a 20’’ pipeline and the total length of the pipeline is 80,392 km, which will be finalised during the routing exercise in section 5. The system schematic of the proposed Balticconnector pipeline is illustrated in Figure 1-2. The battery limits are considered to be at the tie-in locations to the onshore pipeline.

4.2 Protection philosophy

4.2.1 General

The pipeline will typically be installed on the seabed, but in some areas the pipeline will have to be protected by trenching and/or covering with seabed sediment or subsea rock installation. The main reasons for the pipeline protection requirements are maritime transport (dropped and dragged anchors), and ice gouging in coastal areas.

The protection methods considered to be relevant for the Balticconnector offshore pipeline are trenching (Figure 4-1) and subsea rock installation (Figure 4-2).

4.2.2 Trenching

If trenched and buried to a sufficient depth, the pipeline can obtain protection against anchor damage, grounding and sinking ships as well as ice scouring. The depth at which the pipeline should be trenched depends highly on the size of the vessels crossing the pipeline. Large vessels have anchors with large fluke lengths which can penetrate deep into the seabed. Trenching can be used where the surrounding seabed does not consist of soft mud. If the pipeline needs protection on locations where the seabed consists of soft mud, the mud should be replaced with more stable material (sand or clay) or consider a local re-routing if possible. If bedrock excavation is required, the use of underwater blasting shall be considered, as ploughing and jetting would not be able to trench at bedrock location.

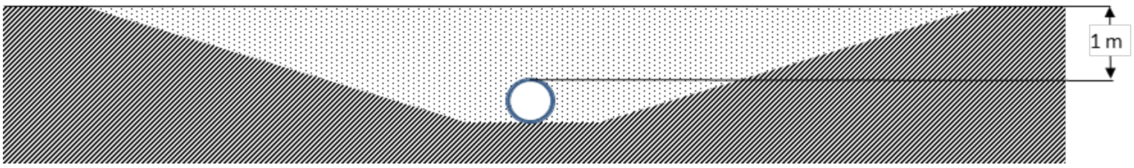


Figure 4-1 Buried pipeline cross-section

There are several trenching techniques that can be implemented for the Balticconnector offshore pipeline that can be utilised on the soft clay / bedrock seabed along the route:

- Post-lay ploughing and backfilling where bedrock does not appear near the surface of the seabed
- Post-lay jetting for sands and clays, although this results in more soil dispersion and does not allow for the possibility of backfilling
- Cutter suction dredgers or trailer suction hopper dredgers, effective in shallow water for shorter sections or between sections with bedrock outcrops

Shorter sections of excavation in soft to stiff clays can also be achieved using more localised dredging techniques. This includes the use of a remotely operated dredging vessel or a dredging barge and cargo vessel in shallow water.

4.2.3 Rock cover

In this context, subsea rock installation means that the pipeline remains on top of the seabed, but is covered with a layer of rock. The rocks can then protect the pipeline against anchor damage, grounding and sinking ships. It is to be noted that the rock aggregates will not sufficiently protect the pipeline from ice ridges, where trenching and burial under the seabed would be the preferred solution.

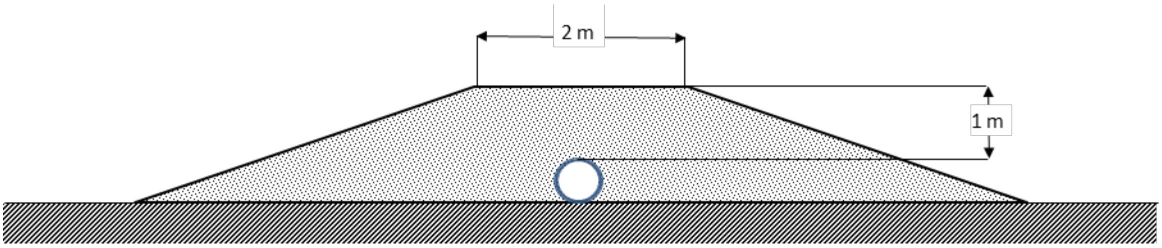


Figure 4-2 Rock covered pipeline cross-section

Rock cover can be achieved in very shallow water depths using a cargo vessel and long reach excavator. Otherwise a rock installation vessel with a fall pipe can be used to accurately install post-lay rock cover on the pipeline.

4.2.4 Increase steel wall thickness/concrete coating

Increases in steel wall thickness or concrete wall thickness to absorb larger impact forces are not pursued as options in the FEED.

4.2.5 Protection studies

The results of the *QRA report*, Ref. /33/, carried out in the FEED phase show that protection will be required at certain locations along the Balticconnector pipeline length due to the risk of dragged anchors. Given that the QRA report findings were based on the pre-FEED route, the exact locations are to be transferred to cover the optimised route in this FEED report.

According to the *QRA report*, Ref. /33/, the probability of dropped anchors on the pipeline is below the design failure frequency along the entire pipeline length, which reduces the need for pipeline protection above the Top-Of-Pipe (TOP). Therefore, dragged anchor protection determined by HSE requirements along the route has been assumed as 0.5 m above TOP with a 2 m wide crown width and 1:2.5 gradient slope, based on project experience of pipeline protection on soft clays.

At the approaches to landfall, the pipeline will normally be trenched or covered with a layer of rock to ensure pipeline stability, and for sections close to the Inkoo fairway, to prevent ice ridge scouring. Ice scouring may only occur at the edge of the fairway where icebreakers will build up ridges of ice, or a little distance outside the archipelago where sea ice will build up into ridges. Rock cover will also be used at locations where existing pipelines and cables are crossed.

A trawl gear impact analysis shall be performed in order to estimate the impact force experienced by the pipe shell and the acceptable criteria for pipe dent shall be checked as per DNV-RP-F111, Ref. /10/. If the pipeline does not satisfy the above criteria, additional means of protection would be required on the exposed section of the pipeline which is not trenched or rock covered. A detailed impact analysis is carried out in section 6.3, and the recommendation of the study is outlined for further consideration.

Typically, protection against dragged anchors would be provided by ensuring trenching and burial of the pipeline below the penetration depth of the anchor flukes, Ref. /7/. Given that bedrocks outcrops exposed on the surface of the seabed or just underneath may prevent trenching and burial of the pipeline, it is recommended to perform a detailed study after the FEED phase to determine the size of rock protection required to lift anchors up and over the pipeline and locations where trenching is not possible and dragged anchor protection is required.

The detailed estimation of rock volume and trench configuration is presented in section 2.2.9, with a breakdown for each design discipline in section 11.

4.3 System operation philosophy

4.3.1 Maintenance

Operating procedure shall be developed during detailed design which shall include detailed system descriptions and step-by-step procedures for each part of the operational process. All operations will be performed by authorised and trained personnel.

In order to detect leaks, even small leaks that do not show on pressure monitoring, it is recommended to perform a visual inspection from ROV surveys, and such inspections be undertaken in accordance with long standing good practice. Operating procedure to be developed during detailed design will provide further guidance.

The maintenance philosophy has to be developed during the early operation stage. Potential maintenance tasks should be identified, optional approaches evaluated, and selections made for maintenance provisions to be incorporated into subsea systems and hardware. In some cases, simple and basic maintenance methods (i.e. divers with hand tools) are warranted. Maintenance can be limited by careful selection of equipment which is appropriate for the application and environment, and which embraces proven technology to minimise and reduce the maintenance burden.

The typical maintenance and monitoring activity/philosophy for the Balticconnector pipeline are given in Table 4-1.

System	Sub-System	Maintenance Type
Pipeline	Submarine	Visual, Intelligent Pigging, Side-scan sonar, Risk Based Inspection (RBI), Corrosion monitoring (Coupons, & Sampling)
	Buried	Intelligent Pigging, Risk Based Inspection (RBI), Corrosion monitoring (Coupons, & Sampling)

Table 4-1 Typical maintenance philosophy

With the exception of equipment governed by frequencies set by statutory regulations, inspections will be reviewed using tools such as RBI to optimise the inspection programmes.

For RBI, the most critical sections of pipeline are identified and subjected to a more thorough inspection programme following the guidance of DNV-RP-F116, Ref. /12/. These areas of inspection for a subsea pipeline would be where the risk of failure is highest, which is typically at locations with DFI threats (e.g. fabrication or installation errors), third party threats (e.g. trawl interference or anchoring) and structural threats (buckling or spanning locations). The evaluation will be carried out based on experience, historical data and criticality to derive a cost effective inspection frequency without compromising technical integrity.

Operational pigging is performed to maintain pipeline integrity. Operational pig runs using intelligent pigs at intervals of 5 years would be sufficient for the Balticconnector offshore pipeline given the non-corrosive gas composition. These runs would be preceded by cleaning/gauging pigging to ensure the passage is clear before the pipeline is put into operation. If any defects are noted during the pre-commissioning, consideration to more regular pigging intervals after the pipeline is put into operation should be given.

4.4 System corrosion protection philosophy

The coatings for the Balticconnector 20” pipeline are summarised in Table 4-2.

Description	Balticconnector
Pipe size [“]	20
Internal Coating	Epoxy paint
Anti-corrosion coating	3LPE 1)
Weight coating	Concrete
Field joint coating	Heat Shrink Sleeve
Field joint infill	PU foam
Note: 1) Justification for use of 3LPE coating as opposed to asphalt enamel is given in Appendix I.	

Table 4-2 Coating systems to be applied for the pipeline

A drag reducing internal flow coating is envisaged to reduce the pressure loss through the Balticconnector pipeline. A two-component epoxy paint of dry film thickness approximately 0.1 mm is normally applied. The internal coating shall comply with ISO 15741:2001, Friction-Reduction Coatings for the Interior of On- and Off-Shore Pipelines for Non-Corrosive Gases, Ref. /17/. Internal flow coating is not envisaged to offer any corrosion protection, which is not needed as the transported medium is dry gas, thus field joint coating is not internally coated.

The anti-corrosion coating is recommended as 3LPE coating, as the operating temperature is less than 80°C which is the limit for 3LPE coating. An evaluation of the external coating options is given in Appendix I. The required anti-corrosion coating thickness is estimated based on ISO 21809-1:2011 Ref. /18/. The anti-corrosion coating thickness is the function of coating class and the pipe weight. The coating class and coating thickness class are specified in Table 1 and Table 2, respectively, based on ISO 21809-1:2011 Ref. /18/.

The Balticconnector pipeline is mostly buried or rock covered, and no heavy trawling is envisaged. Hence, the field joint coating shall not necessarily be able to sustain trawl impact, and polyurethane foam is routinely applied as infill between the adjoining concrete coatings. A heat shrinkable sleeve is used as anti-corrosion coating, which shall be compatible with the PE coating, used as parent coating on the pipe.

Apart from the anti-corrosion corrosion coating, the submerged pipeline shall also be provided with the cathodic protection in case any damage to the anti-corrosion coating during construction. The submerged steel pipeline will suffer from anti-corrosion corrosion due to chemical reactions with the surroundings. Cathodic protection is applied using electrically connected sacrificial anodes, made from a less noble material than steel being aluminium alloy. The pipeline then acts as the cathode of the system, while the mounted aluminium acts as the anode being corroded.

Cathodic protection is provided by sacrificial anodes of the bracelet type which consists of two half-shells installed on the pipeline to form one anode bracelet. The anode material type selected shall be aluminium alloy Al-Zn-In with a proven chemical composition according to Table 5 in ISO 15589-2:2012, Ref. /16/, Section 8.4.

The anode requirements shall be analysed for two pipeline conditions;

- Exposed on seabed
- Completely buried

The condition resulting in the most conservative requirements shall be governing for the quantity of anodes.

The cathodic protection design is carried out in Section 6.4.

The anti-corrosion coating, field joint coating and cathodic protection design shall be applied based on codes and standards.

- DNV-RP-F106, Factory applied external pipeline coatings for corrosion control, May 2011, Ref. /6/
- DNV-RP-F102, Pipeline field joint coating and field repair of linepipe coating, May 2011, Ref. /4/
- ISO 15589-2 Petroleum and natural gas industries – Cathodic protection of pipeline transportation systems, Part 2: Offshore pipelines, 2012, Ref. /16/

Anti-corrosion coating, internal epoxy coating and concrete coating, as well as anode installation, are carried out at a dedicated coating yard of the pipe manufacturer. Alternatively, the pipe joints may be delivered from the pipe mill provided with internal epoxy coating and possibly anti-corrosion coating. Field joint coating (shrink sleeves and PU foam infill) is applied offshore, during installation of the pipeline.

4.5 RFO / pre-commissioning philosophy

Once the construction and installation of the pipeline is complete, there are a number of key activities that need to be performed before ownership is handed over and the operation of the pipeline can begin.

RFO (Ready for Operation) covers all activities from end of pipeline installation until first gas is pumped through the pipeline. RFO, also known as pre-commissioning and commissioning, comprises the following activities:

- Flooding
- Hydrostatic testing
- Gauging
- Cleaning
- De-watering
- Drying
- Nitrogen purging (if there is a substantial time interval between drying and gas filling)
- Gas filling

The activities of de-watering and drying are particularly important for the gas pipelines, because any remaining water may react with the gas to form hydrocarbon hydrates, which can obstruct the flow and in particular the proper functioning of valves. The pre-commissioning spread is envisaged to be located at one of the landfall sites.

A detailed breakdown of the RFO philosophy is included in Appendix II.

4.6 Material philosophy

The materials selection criteria are primarily focused on preventing both internal and external corrosion to withstand the process design conditions and to ensure non-contamination of product.

The default material choice for hydrocarbon systems is primarily carbon steel. Interal corrosion predictions are made to estimate the carbon steel corrosion rate for the given process conditions and a corrosion allowance is calculated for the design life. If this corrosion allowance is small enough to be economically and practically acceptable, i.e. less than 10.0 mm, carbon steel is usually adopted. For the Balticconnector pipeline no internal corrosion is envisaged, hence carbon steel is chosen.

The material selection is based on the following codes and standards:

- DNV-OS-F101, Submarine Pipeline Systems, amended October 2013, Ref. /1/
- NORSOK M-001, Materials Selection. Edition 5 – September 2014, Ref. /20/

The basis for determining the material selection, including the use of SAWL or HFW line pipe, is described in section 6.1.2. The external and internal corrosion protection is mentioned in the systems corrosion protection philosophy in section 6.4.

5 Routing

The route for the pipeline was defined during the pre-FEED study of the project by considering the defined survey data and other design constraints. The route was preliminary, as a detailed assessment of critical sections was not analysed. During the FEED study, the critical sections have been identified and detailed bottom roughness calculations have been carried out to outline the most efficient pipeline route with respect to technical, financial, environmental and social impacts.

The vertical seabed profile along the chosen offshore pipeline route is given in Figure 5-1.

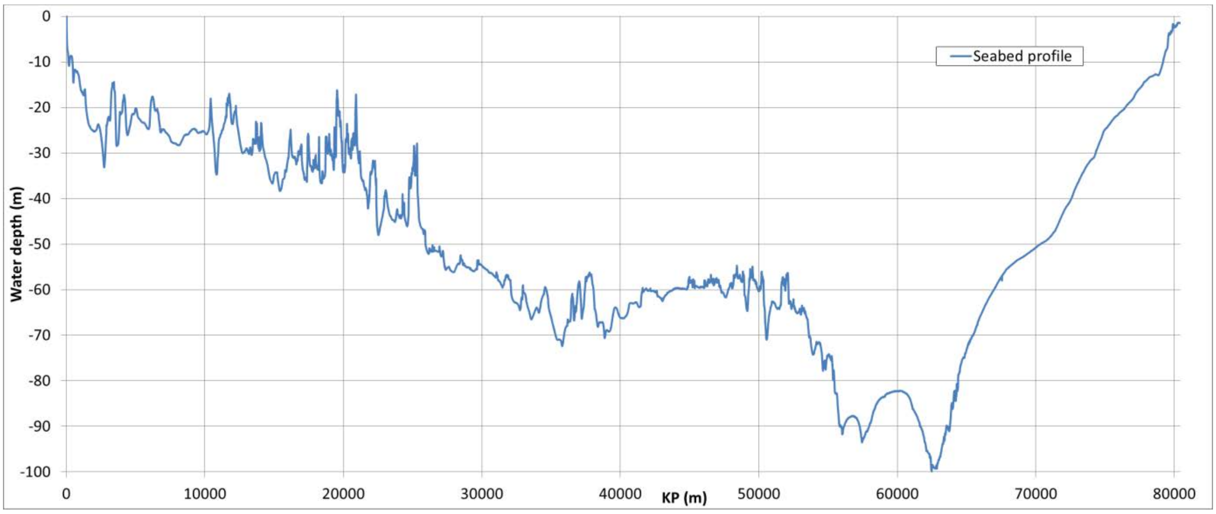


Figure 5-1 Seabed profile

5.1 Approach at Finnish shore

There are two landfall options in Finland to provide alternative choices for the environmental impact assessment. Coordinates for the two landfall options are given in Table 5-1.

#	Landfall location	WGS84 - UTM 35N	
		Easting [m]	Northing [m]
FIN 1	Inkoo Finland	330 985	6 657 677
FIN 2	Inkoo Finland (Base Case)	330 769	6 656 682

Table 5-1 Inkoo landfall coordinates

The base case landfall is located on the Fjusö peninsula (FIN 2) which provides a near clear line of sight to the Gulf of Finland. The alternative case (FIN 1), located adjacent to the Bastubackaviken shore, is located in a reed bed approximately 1 km north of the base case landfall location.

The two landfall locations at the Finland side of the Balticconnector are shown in Figure 5-2.

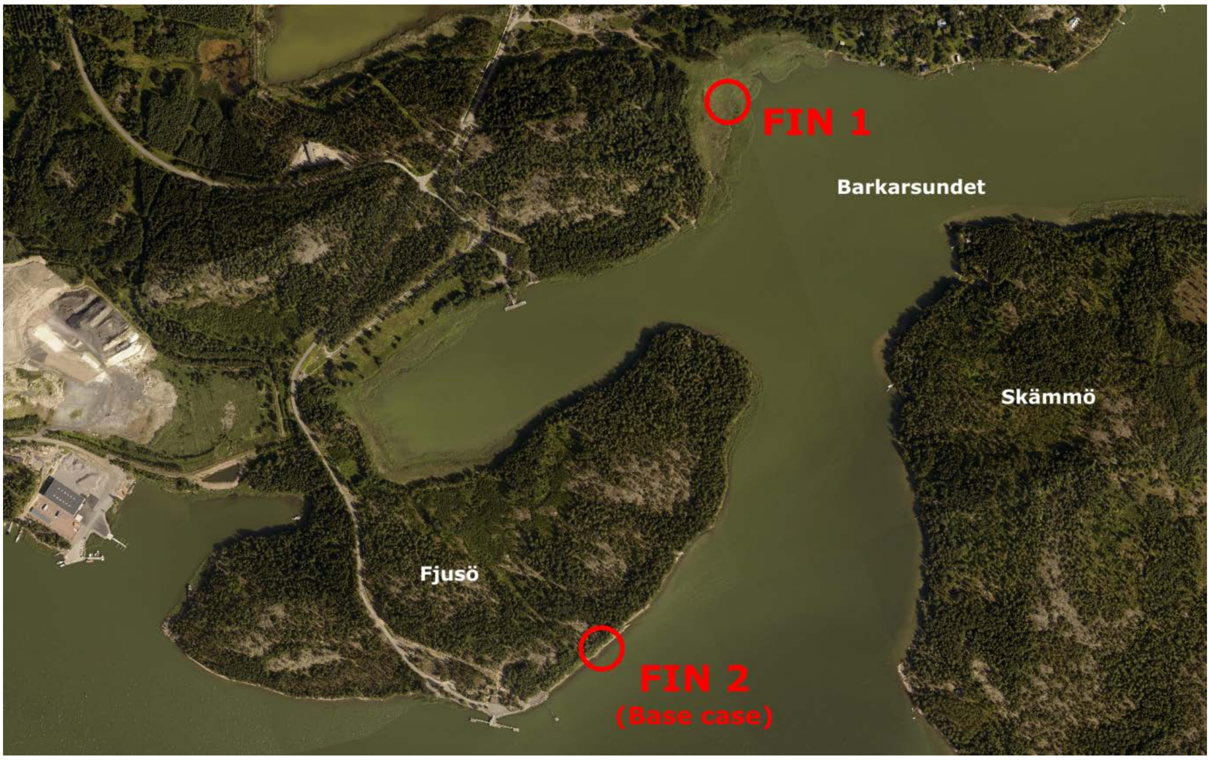


Figure 5-2 Inkoo landfall locations

The alternative case (FIN 1) landfall location was identified in the Finnish EIA report, Ref. /30/, to pass through a nesting area within the reed bed. Furthermore, closer proximity to private land and houses would result in more stringent permitting requirements and the resulting offshore route would be approximately 1 km longer. It was determined early in the FEED execution that the environmental, financial, social and technical benefits of the base case (FIN 2) outweighed the alternative case, and hence the design continued with consideration of the base case only.

The immediate approach to the coastline of the base case landfall shows a rapid decline in water depth in the first 200 m. After 400 m, the seabed along the pipeline route remains deeper than 9 m until the Estonian landfall, with a depth range of approximately 15 – 35 m for the first 10 km. It should also be noted that the water depth for the first 22 km of the pipeline remains between 15 – 38 m.

The seabed in the archipelago region of the route is a mixture of soft clay and firm clay layers in between outcrops of glacial till and bedrock. This geology results in a very rough seabed which contains both soft and stiff soil properties.

The nature of the channel formations in the archipelago combined with virtually no tidal range also results in a very calm sea state outside the winter months. During winter, the Finnish shore approach will be frozen with a layer of sea ice as seen in Figure 5-3.

Figure 5-3 Aerial photo Fjüso peninsula and the Inkoo landfall approach in winter 2014

The design of the landfall approach was performed by taking the following criteria into consideration:

- Remain within the survey corridor
- The need for a straight section of approximately 1 km length from landfall to ensure curve stability after pipelay commences following the pull-in operation
- Minimise interference with the fairway
- Avoid sections of shallow water which may limit the draft depth of the pipelay vessel

An extract of the resulting landfall approach drawing for Finland is shown in Figure 5-4, with the proposed pipeline route in red and the boundaries of the Inkoo fairway shown in a dashed blue line.

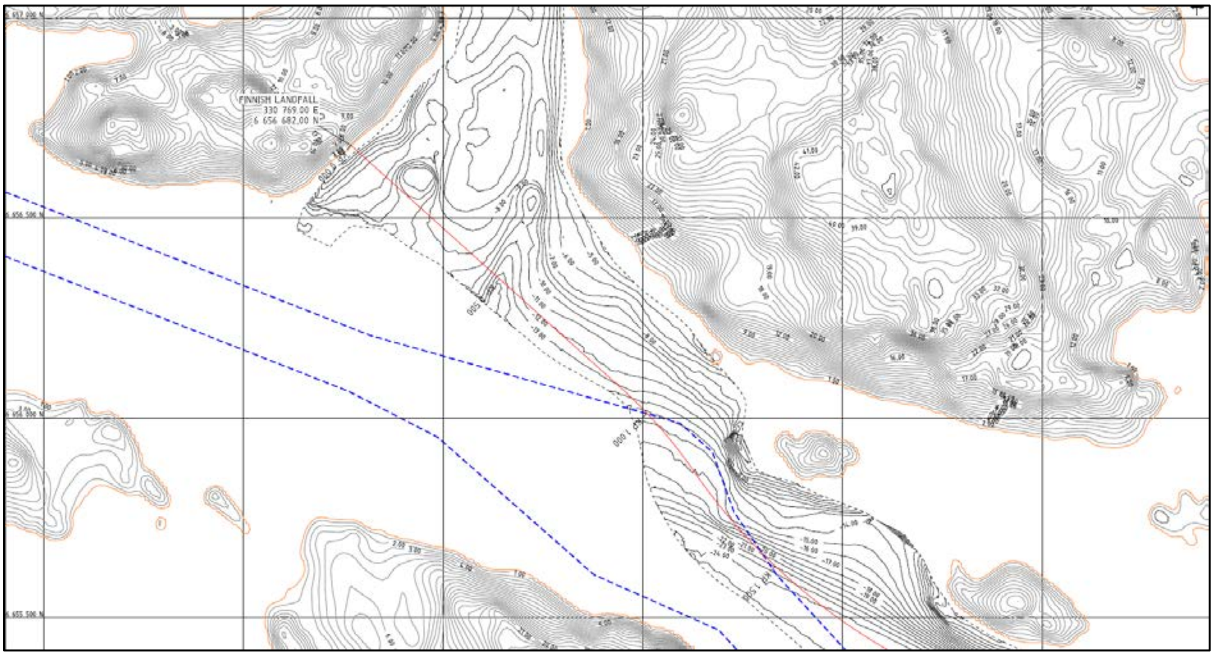


Figure 5-4 Finnish landfall approach, Ref. /39/

5.2 Alternative pipeline routes in Finnish waters

As seen in the *Design Basis*, Ref. /34/, the near-shore routing in Finnish waters shows two alternative routes split around the island of Stora Fagerö. The preferable option is to be determined based on the technical, financial, social and environmental challenges.

From a technical and financial perspective, by taking the eastern route, the length of the pipeline becomes approximately 1.3 km longer compared with the western route. Overall, the western route is slightly flatter and produces fewer critical free spans with less rectification required, although the crossing of the fairway is at a wider location. The route along the west is also generally 5-10 m deeper than the eastern route which would result in less exposure of the pipeline to the faster seabed currents and decreased wave induced loading compared to shallower water.

An approximate comparison of the seabed profile of both routes from the pre-FEED study is shown in Figure 5-5, with the respective fairway locations highlighted with colour coding.

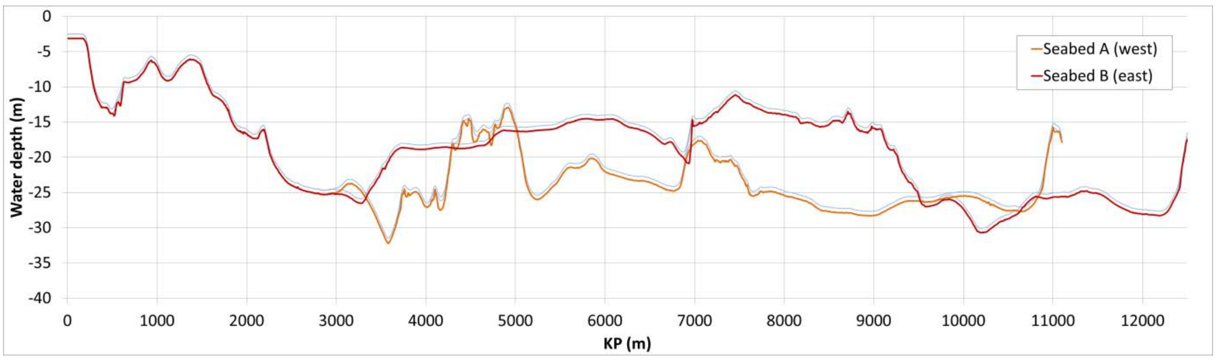


Figure 5-5 Seabed profile comparison of base case route (west) and alternative case route (east)

From an environmental perspective, the southern route provides the least interference given the existing fairway which is in close proximity, Ref. /30/. Both routes result in close proximity to private summer houses and public beaches; therefore, there are no distinguishing social benefits of either route.

In conclusion, the western route displays key technical, financial and environmental benefits in comparison to the eastern route, and hence the design continues with the western route as the base case.

5.3 Offshore pipeline route

The offshore pipeline route was designed to remain within a survey corridor that was determined in earlier phases of the Balticconnector project. A preliminary route was defined during the pre-FEED study, which is presented in Figure 5-6. The detailed routing section is described in *Pre-FEED Report*, Ref. /31/.

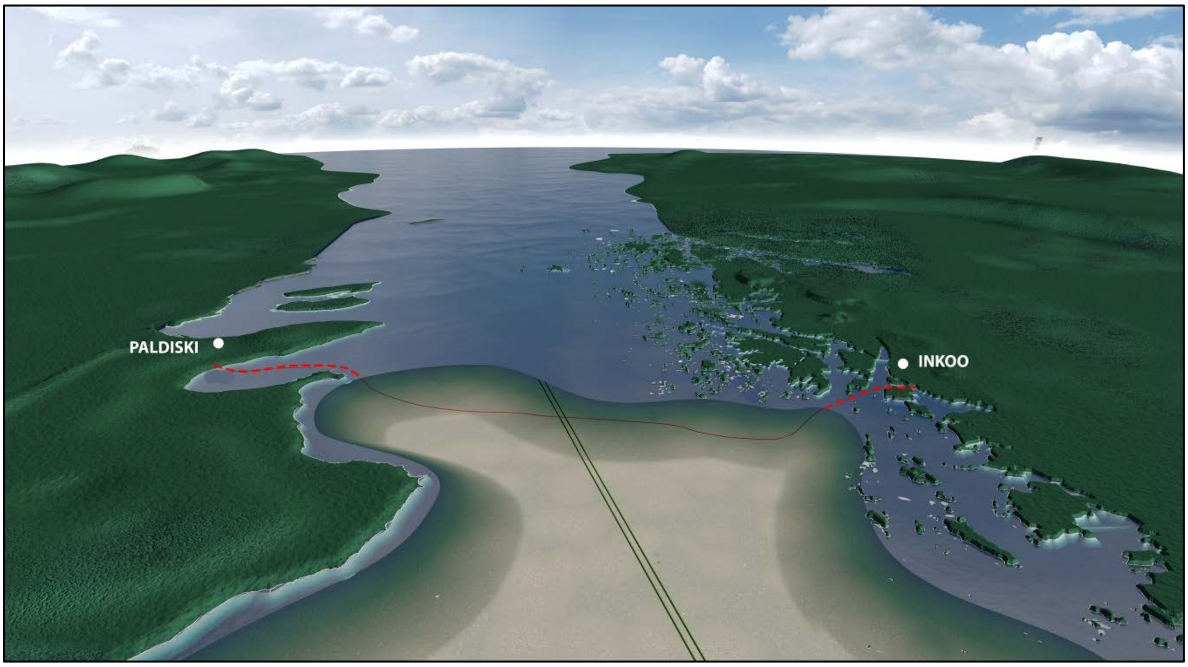


Figure 5-6 Visualisation of Balticconnector offshore pipeline route

During the pre-FEED, a routing exercise was carried out based on the previous conceptual route with design constraints from a technical, geographical and economical perspective. These constraints are summarised in the following list.

- Seabed morphology
- Installation constraints (pipelay vessel capabilities, curve stability, pipeline stiffness, etc.)
- To minimise pipeline length
- To minimise the number of curves but at the same time the number and heights of sea bottom-induced free spans
- To minimise required seabed intervention works

- Minimum radius of curvature shall result in equivalent stresses which shall not exceed 10% of the Specified Minimum Yield Stress (SMYS) or the minimum stable curve radius to avoid the use of counteracts if possible

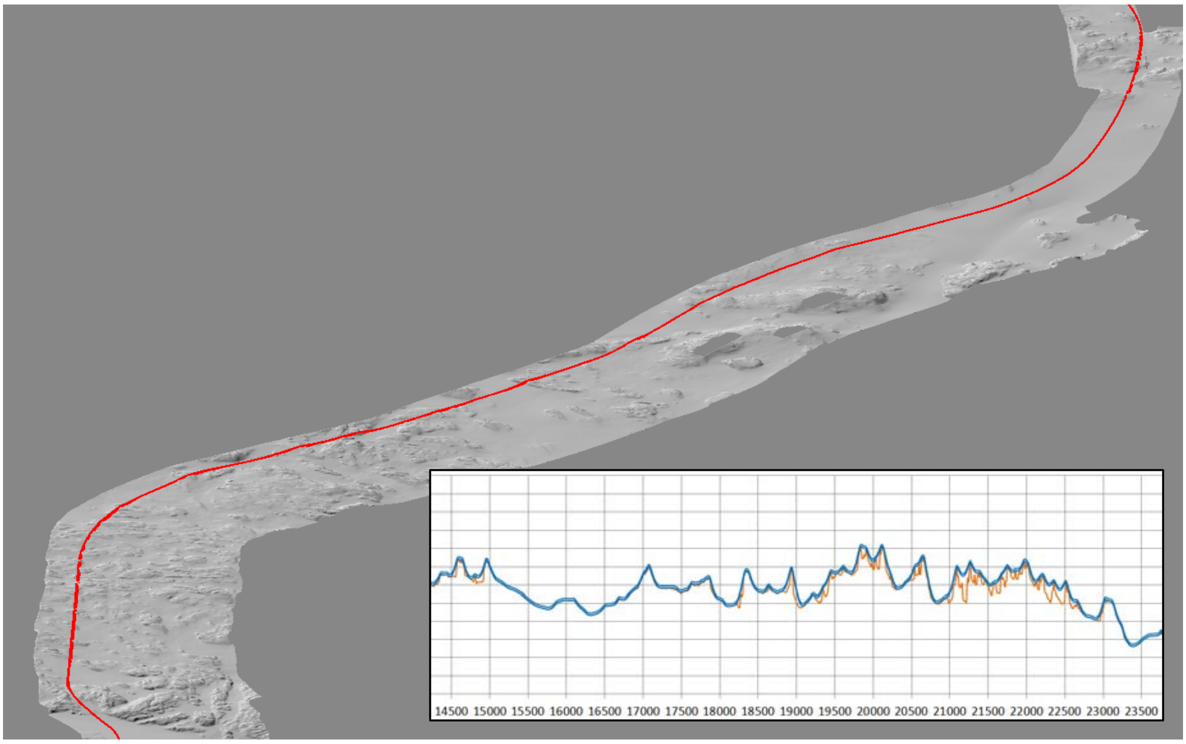


Figure 5-7 Pipeline routing performed in the pre-FEED phase

5.4 Route optimisation

The pipeline route defined in the pre-FEED study has been optimised to minimise the seabed intervention work and to ease the installation logistics of the offshore pipeline. Using the bottom roughness results from the pre-FEED study, the sections of pipeline where most seabed intervention is required were identified and defined as critical. For these critical sections of the route, one or more alternative routes were plotted and a bottom roughness assessment carried out for all the sections. The various route sections, with start KP, end KP and section length are listed in Table 5-3.

The bottom roughness calculations for the route optimisation exercise are carried out for both pre-FEED and alternative route options with pipeline and seabed properties as outlined in Table 5-2. Note that these properties are not representative of the final properties in the design, but are used to produce a consistent comparison between the alternative routes.

Parameter	Unit	Value
Pipe OD	mm	508
Pipe WT	mm	12.7
Steel grade	-	DNV 450
Anti-corrosion coating thickness	mm	5
Anti-corrosion coating density	kg/m^3	1300
Concrete coating thickness	mm	50
Concrete coating density	kg/m^3	3000
Clay vertical soil stiffness	kN/m/m	210
Lay tension	kN	500

Table 5-2 Specific bottom roughness model data for route optimisation exercise

Based on the bottom roughness results, the number of spans and span fill volume, i.e. volume between seabed and bottom of the pipeline with consistent berm crown width and slope has been estimated for the route options for all sections. The results from this assessment are depicted in Table 5-3.

Sections	Route section KPs		Length (km)	Pre-FEED results		Alternative results	
				Allowable span length of 30m		Allowable span length of 30m	
	KP start	KP end		Number of spans	Span fill volume [m^3]	Number of spans	Span fill volume [m^3]
Sec1	3.4	5.4	2.0	10	2585	5	802
Sec2	5.1	9.3	4.2	1	59	1	433
Sec3	8.9	12.5	3.6	5	1510	4	865
Sec4	11.9	13.8	1.8	5	1858	4	2235
Sec5	13.3	16.4	3.1	10	3606	5	6179
Sec6	16.9	25.6	8.7	58	60420	54	49421
Sec7	25.4	28.2	2.8	16	14387	18	8584
Sec8	28.3	34.0	5.7	9	1274	5	1007
Sec9	34.2	37.4	3.2	6	869	1	96
Sec10	37.8	40.5	2.7	2	228	7	586
Sec11	44.8	51.9	7.1	29	6438	30	7587
Sec12	52.3	53.2	0.9	6	3873	6	1462
Sec13	54.6	57.1	2.5	6	1005	10	2159
Sec14	60.3	66.6	6.3	6	1641	6	1829

Table 5-3 Bottom roughness results for route sections considered for optimisation in FEED

Note that there was no alternative route plotted for some sections of the route, where the pre-FEED study had already identified the most optimum route with no requirements for seabed intervention.

The route section 6 is presented in Figure 5-8, where the yellow lines signify several alternative route options and the red line represents the pre-FEED route.

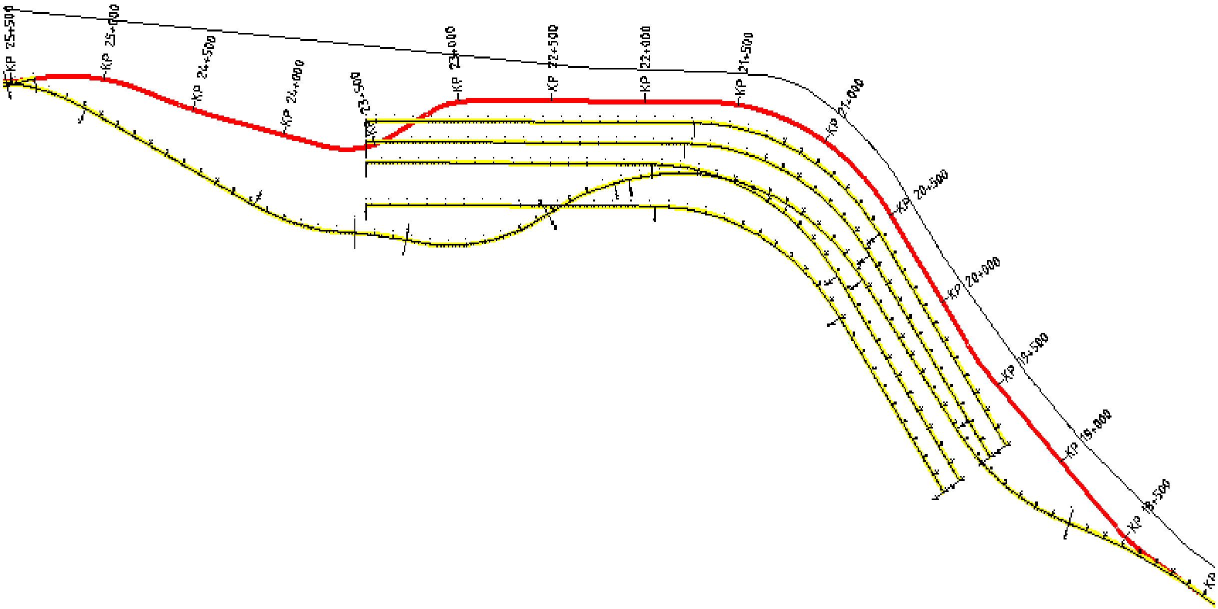


Figure 5-8 Alternative route options considered for section 6 between KP 16.9 to 25.6

At first, the alternative route options at critical locations were plotted and then compared to determine the optimal route within the defined KP range. Once the best alternative route was determined, a direct comparison with pre-FEED route section within the same KP range was made. The criteria used for the comparison were:

- Number of spans
- Span fill volume
- Minimum stable route curves

The number of spans in pre-FEED and alternative route sections are compared and presented in Figure 5-9.

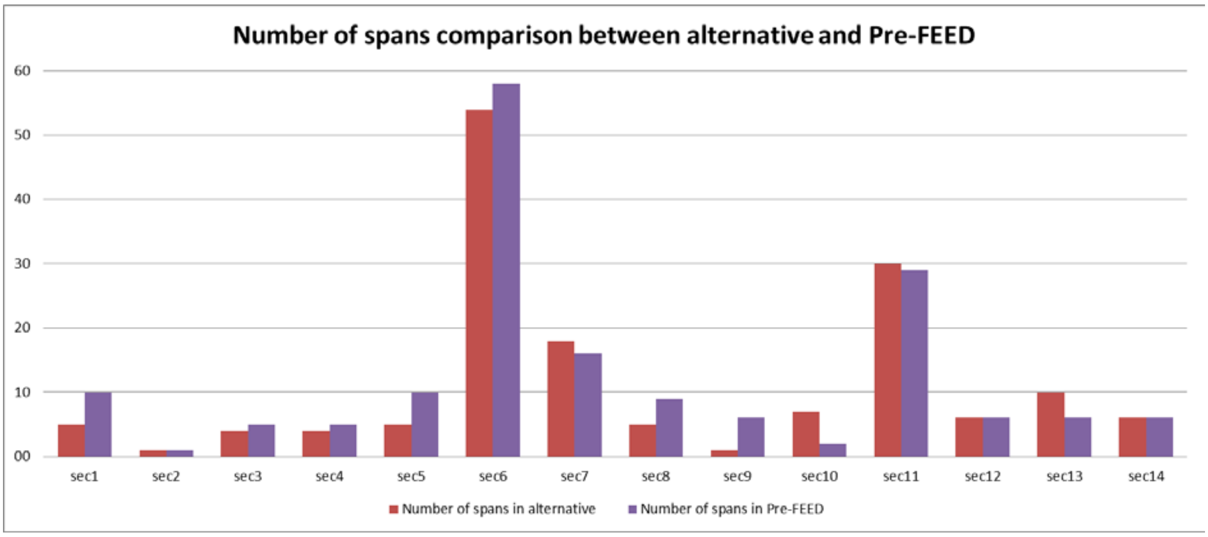


Figure 5-9 Number of spans for all route sections for pre-FEED and alternative, cf. Table 5-3

The span fill volume in pre-FEED and alternative route sections are compared and presented in Figure 5-10. The berm crown width of 10m and slope of 2.5 is used for the calculations.

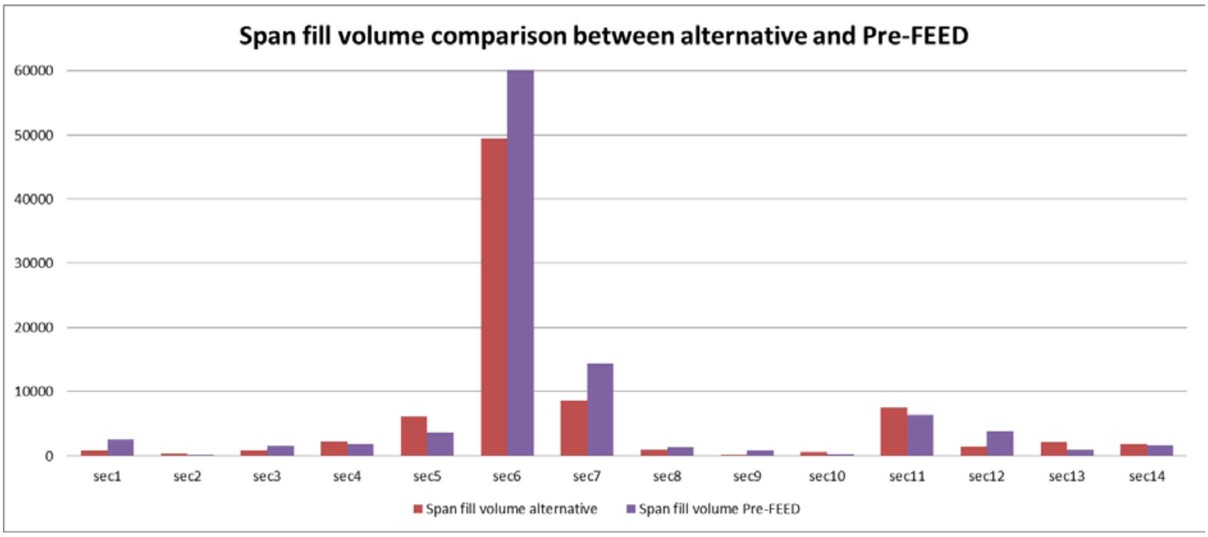


Figure 5-10 Span fill volume for all route sections for pre-FEED and alternative, Ref. Table 5-3

The route sections from pre-FEED and alternative have been compared in accordance with the defined criteria and they are outlined in Table 5-4. The selected route section is marked with 'X' and selection criterion is also outlined in Table 5-4. For the route section 4 and 5, the calculated number of spans and span fill volume was not consistent, and therefore both the options were assessed visually by going through survey data in order to determine the optimal route.

Sections	Route section KPs		Length (km)	Selected route		Selection criterion
	KP start	KP end		Pre-FEED	Alt.	
Sec1	3.4	5.4	2.0		X	Reduced number of spans and volume
Sec2	5.1	9.3	4.3	X		Reduced number of spans and volume
Sec3	8.9	12.54	3.6		X	Reduced number of spans and volume
Sec4	11.9	13.8	1.9	X		Lay curve and survey review
Sec5	13.3	16.5	3.2	X		Lay curve and survey review
Sec6	16.9	25.6	8.7		X	Reduced number of spans and volume
Sec7	19.3	23.5	4.3		X	Reduced number of spans and volume
Sec8	28.3	34.1	5.7		X	Reduced number of spans and volume
Sec9	34.2	37.4	3.3		X	Reduced number of spans and volume
Sec10	37.8	40.5	2.7	X		Reduced number of spans and volume
Sec11	44.8	51.9	7.2	X		Reduced number of spans and volume
Sec12	52.3	53.2	0.9		X	Reduced number of spans and volume
Sec13	54.6	57.1	2.5	X		Reduced number of spans and volume
Sec14	60.4	66.6	6.3	X		Reduced number of spans and volume

Table 5-4 Selected route sections from route optimisation study

5.5 Approach at Estonian shore

There are two landfall options in Estonia. The coordinates for the two landfall options are given in Table 5-5.

#	Landfall location	WGS84 - UTM 35N	
		Easting [m]	Northing [m]
EST 1	Paldiski Estonia (Base Case)	339 933	6 581 949
EST 2	Paldiski Estonia (Alternative Case)	334 033	6 586 405

Table 5-5 Paldiski landfall coordinates

The base case landing point (EST 1) is situated in the bottom of Lahepere Bay – a fairly shallow bay between the peninsula of Pakri and Lohusalu. The pipeline landing point is some 6.5 km east of the town of Paldiski. The alternative case (EST 2) is located closer to the tip of the Pakri peninsula, arriving at the edge of a protected area. The landfall options are shown in the aerial photo, see Figure 5-11.

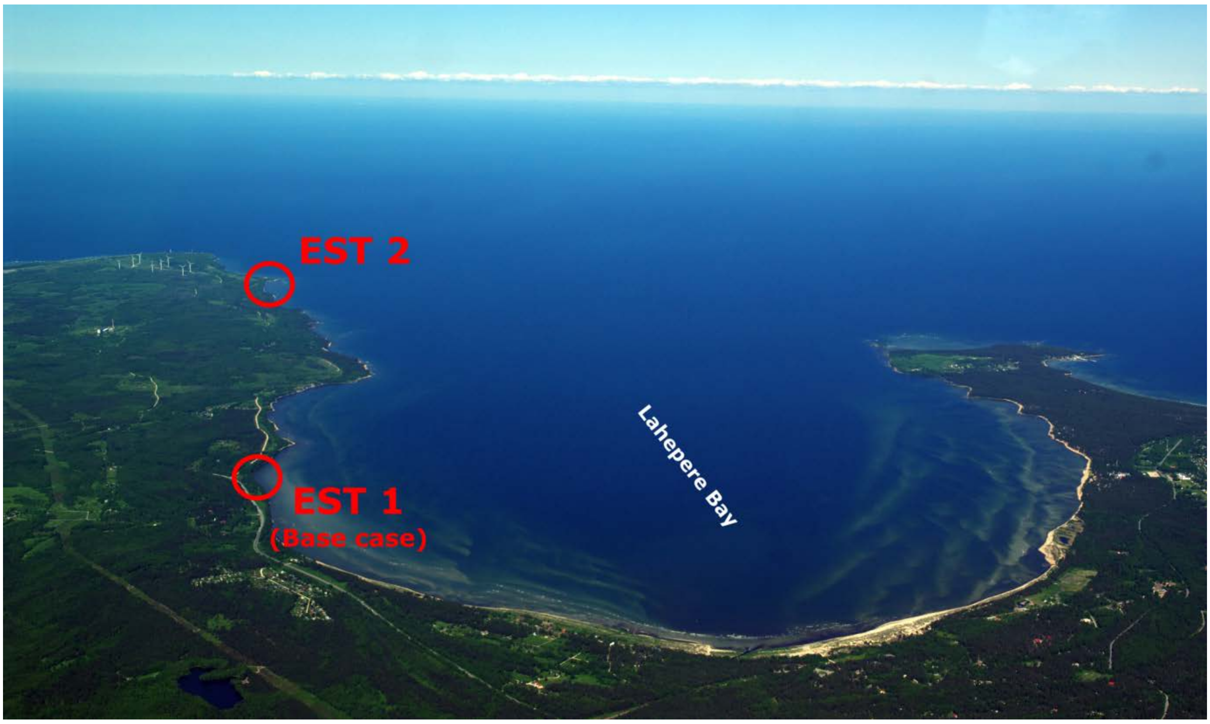


Figure 5-11 Paldiski landfall locations (Photo author: Mait Metsur, Aerofotod.ee)

The water depth at the entrance of the bay is approximately 27 m, but the major part of the bay is between 10 to 20 m deep. Outside the bay, the water depth drops to maximum 90 m. The near-shore profile at Paldiski in Estonia is quite different compared to the near shore bathymetry at the landfall in Finland. At Paldiski, the profile is smoother and it is observed that the depth of water increases quickly from the shore. Already 3.5 km out in the Gulf of Finland from Paldiski the depth of water reaches 20 m. The seabed gradients are around 0.5°.

The EST 1 option has been chosen as the base case landfall location.

It should be noted that a proposed LNG terminal in Estonia will add restrictions to the approach to the Estonian landfall sites. The restrictions are in the form of precautionary areas where anchoring is not permitted, and at an aquatorium limit linked to the LNG harbour. This is based on data received and shown in Figure 5-12.

As a result of these restrictions related to the future planned LNG terminal and harbour near the Pakri Cape and the EST 2 landfall option, EST 1 has been chosen as the base case landfall location for the Balticconnector at the Estonian side.

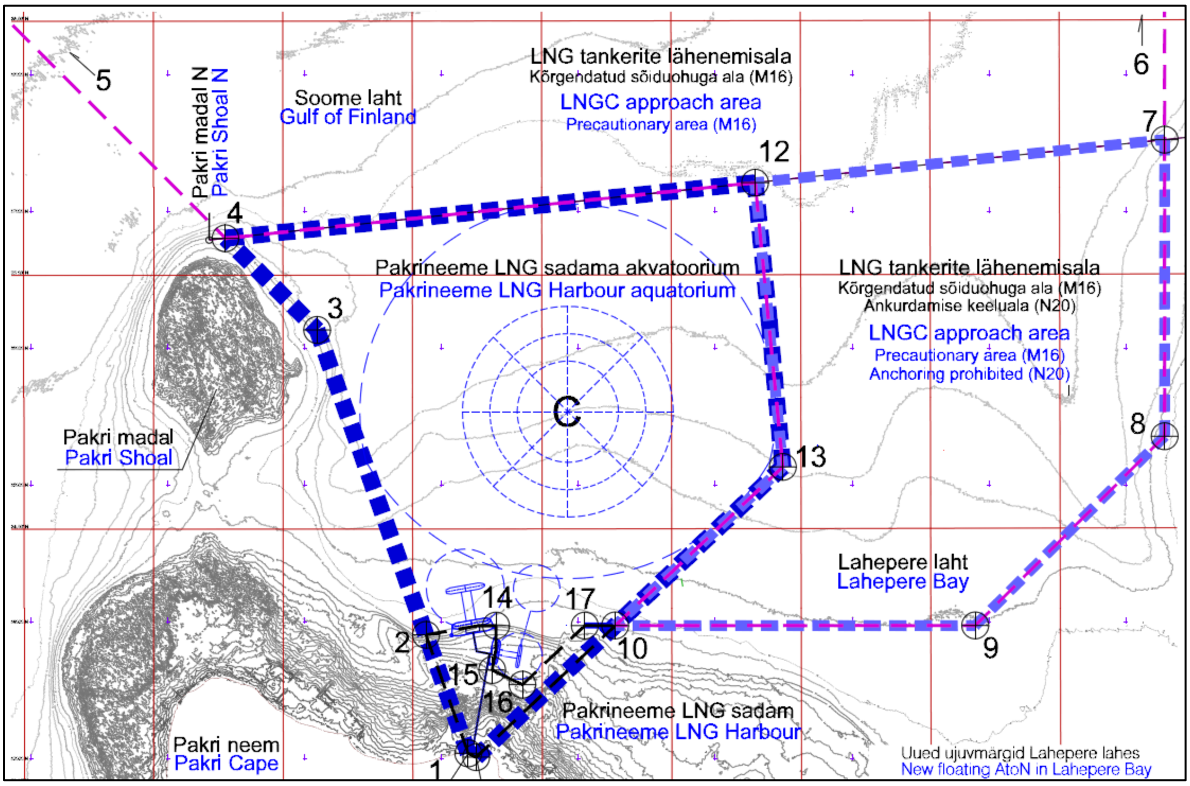


Figure 5-12 Restrictions to pipeline approach at Estonian landfall locations

5.6 Crossing coordinates

As a result of the routing optimisation task, the crossing coordinates between the Balticconnector pipeline and existing infrastructure along the pipeline route have been updated. The updated crossing KPs and coordinates are listed in Table 5-6 and Table 5-7.

KP	Easting (m)	Northing (m)	Type	Pipeline Name/ Owner	Comment
42.175	335 205	6 619 236	Gas Pipeline	NS1 / Nord Stream	Shown only in 2013 survey
43.092	335 328	6 618 331	Gas Pipeline	NS2 / Nord Stream	Shown only in 2013 survey

Table 5-6 Updated KP and coordinates for Nord Stream crossings

KP	Easting (m)	Northing (m)	Type	Cable Name / Owner	Comment
1.428	331 780	6 655 680	-	-	Outside 2013 data coverage
1.496	331 829	6 655 631	-	-	Outside 2013 data coverage
N/A 1)	N/A	N/A	-	-	Outside 2013 data coverage
N/A 1)	N/A	N/A	-	-	Outside 2013 data coverage
31.071 2)	333 149	6 629 949	Tele	BCS B2/ Telia Sonera	
35.816 3)	333 730	6 625 355	Tele	Utfors 2/Telenor	
39.266	334 335	6 621 986	Tele	K – St/ Russian State	Not detected 2013
42.004	335 198	6 619 407	Unknown	NSP Cable? / Nordstream?	Not installed 2006
44.021	335 544	6 617 428	Unknown	Unknown	
44.178	335 556	6 617 271	-	-	Possible cable / seabed scar
44.781	335 569	6 616 668	Unknown	Unknown	92 m south of background information
46.905	335 614	6 614 546	Tele	DK–R1/ Tele Danmark Rostelecom	
48.184	335 642	6 613 266	-	-	Possible cable / seabed scar
52.641	335 819	6 608 843	Unknown	Unknown	Detected 10 m north of background information
61.811	337 224	6 599 825	Tele	Pangea-S4	
64.875	336 883	6 596 825	-	-	Possible cable / wire
65.870	336 906	6 595 833	Unknown	Unknown	Detected 138 m south of background information
68.679	337 077	6 593 029	-	-	Possible cable / wire
72.882	337 334	6 588 834	-	-	Possible cable / wire
n/a	n/a	n/a	Tele	C-Lion	The C-Lion cable is being installed between Finland and Germany in autumn 2015 will cross the pipeline route at an unknown location.

Note:
1) Pre-FEED crossing coordinates located on Stora Fagerö eastern route alternative
2) Pre-FEED crossing coordinate is 211 m from updated route, therefore KP based on extrapolation
3) Pre-FEED crossing coordinate is 76 m from route, therefore KP based on extrapolation

Table 5-7 Updated KP and coordinates for cable crossings

5.7 Shipwrecks

The location of shipwrecks in close proximity to the pipeline route can be found in Table 3-2 of the *Design Basis*, Ref. /34/. Following the optimisation of the route, Table 5-8 below shows the distance between the pipeline and the identified shipwrecks.

Name	Code	Location [UTM Zone 35N]		KP	Distance from pipeline [m]	Comment
		Easting [m]	Northing [m]			
Finnish waters						
Skämmö Northwest	none	331 308	6 656 437	-0.020	764	Unknown possible wreck, coordinates are exact
Skämmö West	id 1428	331 264	6 657 264	0.564	119	wooden vessel L 25 m
Pohjoinen Kotka	id 1426	333 113	6 653 793	3.890	154	Location is uncertain
Estonian waters						
F-20	Marked on sea chart	335 995	6 613 078	48.380	385	Location not confirmed
Nimetu-178	Nr infosüsteemis 52	336 580	6 595 435	57.077	4723	Tanker Železnodorožnik L 76 m, B 10 m, H 8 m
Železnodorožnik	Maritime Administration id 40	340 966	6 605 033	66.247	349	Unidentified wreck L 46 m, B 10 m, H 3.6 m

Table 5-8 Proximity of shipwrecks from pipeline route

6 Pipeline mechanical design

6.1 Material selection

This section includes corrosion assessment and material selection for the Balticconnector pipeline. The corrosion assessment is based on the fluid composition and functional requirement of the pipeline. The materials for the pipeline are proposed based on the sweet and/or sour corrosion level.

The material selection is based on the following codes and standards:

- DNV-OS-F101, Submarine Pipeline Systems, amended October 2013, Ref. /1/
- NORSOK M-001, Materials Selection. Edition 5 – September 2014, Ref. /20/

6.1.1 Gas Composition

The composition of the gas in the pipeline can vary in content, as seen in Table 6-1. The assumed gas composition is the nominal.

Component	Light (mole %)	Nominal (mole %)	Rich (mole %)
Methane, CH_4	94.89	90.33	85.70
Ethane, C_2H_6	4.75	5.00	6.82
Propane, C_3H_8	0.05	2.50	3.76
i-butane, i-C_4H_{10}	0.01	0.68	1.33
n-butane, n-C_4H_{10}	0.01	0.67	1.33
i-pentane, i-C_5H_{12}	0.04	0.15	0.27
n-pentane, n-C_5H_{12}	0.04	0.15	0.27
C_6+	0.02	0.17	0.17
Carbon dioxide, CO_2	0.19	0.20	0.20
Nitrogen, N	0	0.15	0.15

Table 6-1 Composition of the gas in the pipeline

6.1.2 Basis for material selection

The selection of material shall consider the following properties:

- Mechanical properties
- Hardness
- Fracture toughness
- Fatigue resistance
- Weldability
- Corrosion resistance

Sour service/H_2S corrosion – The gas is without H_2S content, hence there is no need to design the pipeline for sour service. Therefore, the supplementary requirement for suffix S is not applicable.

CO_2 corrosion – The gas is of sales quality and is dry. No CO_2 corrosion needs to be considered.

Mechanical properties – The line pipe is recommended to be carbon steel having SMYS 450 MPa (corresponding to X65) and the manufacturing process is HFW (high frequency welding) or SAWL (submerged arc welding). The pipeline is a high pressure gas pipeline, mainly carrying methane, therefore fracture arrest properties corresponding to suffix F should be applied. HFW is available up to a wall thickness of 17.5 mm and it can be used, as the S-lay installation method is adopted.

Anti-corrosion coating – The corrosion protection system is based on a 3-layer PE coating. The coating system, as well as surface preparations before coating, shall be in accordance with DNV-RP-F106, Ref. /6/. The anti-corrosion corrosion protection design is carried out in section 6.4.

Internal coating – The pipeline is provided with an internal epoxy coating of 0.1 mm which will reduce friction and turbulence in order to increase flow efficiency. The internal coating shall comply with ISO 15741, Friction-Reduction Coatings for the Interior of On- and Off-Shore Pipelines for Non-Corrosive Gases, Ref. /17/.

6.1.3 Supplementary requirement table

The material specification for linepipe shall meet the supplementary requirements as per DNV-OS-F101, Ref. /1/. A summary of all supplementary requirements for the 20" Balticconnector pipeline is given in Table 6-2.

Supplementary requirement	Suffix	Applicability	Reason
H_2S service (also referred to as sour service)	S	Not applicable	No H_2S content.
Fracture arrest properties	F	Applicable	High pressure gas carrying essentially methane.
Linepipe for plastic deformation	P	Not applicable	S-lay installation - no plastic deformation is envisaged.
Enhanced dimensional requirements for linepipe	D	Applicable	S-lay installation - facilitates offshore girth welding.
High utilisation, suffix U	U	Not applicable	No high utilisation is envisaged.

Table 6-2 Supplementary requirement

The testing requirement for Suffix F shall be as per DNV-OS-F101, Ref. /1/, Sec 7 I200.

6.1.4 Material selection table

The linepipe shall be delivered in accordance with DNV-OS-F101, Ref. /1/, and ISO 3183, Ref. /19/ for offshore service.

As the selected wall thickness is within the range of the HFW manufacturing process for 20" pipeline, the simpler manufacturing process and hence reduced cost for HFW would make it the preferred option subject to availability.

Hence HFW pipeline is considered as the preferred option compared to SAWL pipeline.

Table 6-3 contains the linepipe material specification for the Balticconnector pipeline.

Component	Code/Grade/Material
Line Pipe	DNV HFW 450 F D

Table 6-3 Material selection

6.2 Wall thickness design

The wall thickness calculations are based on input parameters from the *Design Basis*, Ref. /34/. Other relevant input parameters have been obtained from DNV-OS-F101, Ref. /1/.

Content in the gas pipeline is assumed to be “flammable and toxic fluids which are gases at ambient temperature and atmospheric pressure conditions”. Hence, the pipeline is classified as “Category E” and the safety classes outlined in Sec 2 C403 of DNV-OS-F101, Ref. /1/ are applicable.

DNV-OS-F101, Ref. /1/, Location Class 2 is defined as extending 500 m from areas with frequent human activity (the safety zone). Pipeline sections located within the safety zone shall be considered as high safety class for the operational phase, denoted Zone 2. The remaining pipeline sections are considered as Location Class 1 with medium safety class, denoted Zone 1.

The adopted material strength factor, α_U, is 0.96 and maximum fabrication factor, α_{fab}, is 0.93 as no supplementary requirement (Suffix U) is envisaged and HFW linepipe is considered.

The local incidental design pressure ratio is taken as 1.10 for the Balticconnector pipeline, Ref. /1/, Sec 3 D209. Thus the local incidental pressure is defined as $P_{li} = 1.1 \times P_{ld}$, where P_{ld} is the design pressure at the considered section of the pipeline.

The pipe wall thickness tolerance of ±0.7 mm is specified in accordance with Sec. 7 I400 of DNV-OS-F101, Ref. /1/. The tolerances are based on the specification of supplementary requirement D.

The pipe diameter out-of-roundness is selected as 1.5% of D as specified in Sec. 7 G200 of DNV-OS-F101, Ref. /1/, for a 508 mm OD pipeline.

6.2.1 Hydrotest pressure

The pipeline system shall be system pressure tested after installation. The local system test pressure (p_{lt}) during the system pressure testing shall fulfil the requirements based on the safety class during normal operation, Ref. /1/ Sec 5 B202, therefore:

$$p_{lt} = \alpha_{spt} \times 1.1 \times P_{ld} = 1.05 \times 1.1 \times 80 = 92.4 \text{ barg}$$

6.2.2 Characteristic material properties

The characteristic material strength for resistance calculations is dependent on the de-rating values due to an elevated temperature effect.

The characteristic material strength f_y and f_u, values to be used in the limit state criteria are given in DNV-OS-F101, Sec 5 C302, Ref. /1/.

The material de-rating value is only applicable if the design temperature is above 50°C for carbon steel, Ref. /1/ Sec 5 C304, Figure 2. As the 20" Balticconnector pipeline has a design temperature of 50°C, no de-rating factor is applicable.

6.2.3 Water depth

The maximum and minimum water depth along the route is presented in Table 6-4.

Description	Water depth (m)	Location (KP / coordinates)
Maximum water depth	99.86	KP 62.482 337 347 E, 6 599 171 N
Minimum water depth	0	KP 0.000 and KP 80.392 (Landfall locations) 330 769 E, 6 656 682 N Finland 339 933 E, 6 581 949 N Estonia

Table 6-4 Maximum and minimum water depth

6.2.4 Design Methodology

The design philosophy is to determine the required wall thickness in accordance with the requirements outlined in DNV-OS-F101, Ref. /1/.

The following design criteria are considered in order to determine the wall thickness:

- Pressure containment (operational condition and system pressure test)
- Hydrostatic system collapse
- Propagation buckling
- Trawl impact analysis (section 6.3)

An increase of wall thickness to meet on-bottom stability requirements is not pursued as an option in the FEED phase.

The adopted safety classes for the limit states are listed in Table 6-5.

Limit state	Safety Class	
	Zone 1	Zone 2
Pressure containment in design conditions	Medium	High
Hydrostatic system collapse	Medium	High
Propagation buckling	Low	
Displacement and load controlled condition	Medium	High

Table 6-5 Safety class for each limit state

<u>Pressure containment (bursting)</u>

The pressure containment verification calculations shall be performed for the proposed pipeline based on the material grade in accordance with DNV-OS-F101 Sec. 5 D200.

Minimum water depth and maximum content density are conservatively used to obtain the worst case scenario.

It is to be noted that the fabrication wall thickness tolerance (negative) as per DNV-OS-F101 shall be used in the wall thickness calculations.

DNV requires the tensile hoop stress, during both the operation and hydrotest conditions, to fulfil the criteria for yielding as a serviceability limit state (SLS) and bursting as an ultimate limit state (ULS). The pressure containment should fulfil the criteria specified in Sec 5 D200 of DNV-OS-F101, Ref. /1/.

Hydrostatic system collapse

The selected pipe wall thicknesses shall be able to withstand collapse due to external hydrostatic pressure. During installation and shutdown, the external hydrostatic pressure at the maximum water depth can cause collapse of the pipe. Hence the selected pipe wall thickness shall have adequate strength to prevent the collapse by taking into consideration the physical properties, ovality and external hydrostatic loads of the pipeline.

Local buckling may occur when the external pressure exceeds the internal pressure. This can occur during installation and decommissioning, or during the operational phase in case of shut-down. The external pressure collapse verification calculations shall be performed in accordance with DNV-OS-F101 Sec 5 D400.

Maximum water depth shall be used to obtain the worst scenario. The flattening due to bending, together with the out-of-roundness tolerance from fabrication of the pipe, is not to exceed 3% as defined in DNV-OS-F101 Ref. /1/, Sec 5 D1100.

The characteristic resistance for external pressure (P_c) collapse shall be calculated in accordance with DNV-OS-F101, Sec. 5 D402.

Propagation buckling

The buckling initiation and propagation verification calculations shall be performed for the Balticconnector pipeline in accordance with DNV-OS-F101 Sec 5, D500.

Maximum water depth shall be used to obtain the worst case scenario. In case local buckling has occurred and the external pressure exceeds the propagation buckling criterion, the initial buckle will start to propagate along the pipe. If propagation buckling is the critical design criterion, buckle arrestors can be installed with a given spacing determined by the failure consequences, cost and spare pipe philosophy. The external pressure should meet the criterion specified in DNV-OS-F101 Sec 5, D500.

The buckle propagation is typically critical for the installation case as the pipeline will be filled with pressurised product/content during operating condition. The minimum pipeline wall thickness and low safety factor has been used in calculations.

6.2.5 Results

This section outlines the results from the performed wall thickness calculations. Minimum required wall thickness for the 20” Balticconnector pipeline is listed in Table 6-6.

Characteristic condition	Safety class	Required wall thickness [mm] 20" Gas pipeline	
Safety zone	-	Zone 1	Zone 2
Material	-	DNV HFW 450 F D	
Pipeline size	-	20" (508 mm)	
Pressure containment (operational)	Medium/High	6.50	7.35
Pressure containment (system pressure test)	Low	6.08	
System collapse	Medium/High	8.19	8.46
Propagation buckling	Low	11.90	11.90
Selected wall thickness	-	12.70[1)]	12.70[1)]
Note: 1) API size has been selected. DNV minimum thickness is the governing design criterion.			

Table 6-6 Required wall thickness for the pipeline

For installation by S-lay the conventional limit of D/t < 45 is adopted. In accordance with DNV-OS-F101, Ref. /1/, the minimum nominal pipe wall thickness of 12 mm shall be used for all pipelines with nominal diameter equal to 8" and above with safety class High, and location class 2. Based on these requirements, the API standard wall thickness of 12.7 mm is proposed for the Balticconnector pipeline. The wall thickness of 12.7 mm satisfies the criteria for propagating buckling and therefore buckle arrestors are not required for the pipeline.

The selected wall thickness for the Balticconnector pipeline is presented in Table 6-7.

Pipeline	Material	Governing design criterion	Nominal diameter[1)] (mm)	Wall thickness[2)] (mm)	
				Zone 1	Zone 2
Gas	DNV HFW 450 F D	DNV Minimum Thickness	508	12.7	12.7
Note 1) and Note 2) API standard					

Table 6-7 Selected wall thickness for the pipeline

The wall thickness calculation can be found in Appendix III.

6.3 Trawl impact analysis

The trawl impact analyses are carried out in order to define the impact energy absorbed by the pipeline, to determine the penetration of trawl gear and clump weight into the concrete coating, and to determine whether any resultant pipe steel denting is acceptable. The analysis is carried out in accordance with DNV-RP-F111, Ref. /10/, and DNV-RP-F107, Ref. /7/.

In the following sections, an analytical approach to verify the pipeline integrity against a trawl gear impact is described.

6.3.1 Input data

The fishing activities in this region of the Baltic Sea do not include beam trawling, with the principal methods being otter trawling and twin rig trawling. In the absence of specific data about fishing along the Balticconnector route, the relevant parameters can be taken from Table 6-2, representing the heaviest equipment in use throughout the entire Baltic Sea.

Parameter	Trawl board	Clump weight
Type	Polyvalent	
Mass	3000 kg	3000 kg
Hydrodynamic added mass	6420 kg	1350 kg
Length x Height	4.5 m x 3.2 m	1.35 m x 1.0 m
Tow velocity	2 m/s	2 m/s
Warp line diameter	30 mm	30 mm

Table 6-8 Trawl equipment and pipeline data

The impact frequency is estimated to be < 1 event per km per year, corresponding to frequency class Low, as per DNV-RP-F111, Ref. /10/.

The coating properties used in calculation are mentioned in Table 6-9. The coating properties are taken from DNV-RP-F107, Ref. /7/.

Parameter	Unit	Values
Crushing strength of concrete coating	MPa	105
Energy absorption of concrete coating	kJ	40
Energy absorption of PE coating	kJ	0
Energy absorption of field joint coating	kJ	15

Table 6-9 Coating strength properties

6.3.2 Analytical approach

The energy from impacts with trawl boards and clump weights are calculated in accordance with DNV-RP-F111, Ref. /10/, Section 3.4.2. A conservative estimate of the kinetic energy absorbed by the local deformation of the coating and pipe wall is found by the maximum of the impact energy associated with the steel mass, $\mathrm{E_s}$, and the impact energy associated with the added mass, $\mathrm{E_a}$, of the trawl board:

$$\mathrm{E_{loc\_trawl}} = \max\begin{cases}\mathrm{E_s}\\ \mathrm{E_a}\end{cases}$$

The impact energy associated with the steel mass of the trawl board is given as:

$$\mathrm{E_s} = \mathrm{R_{fs}} \cdot \frac{1}{2} \cdot \mathrm{m_{t\_trawl}} \cdot (\mathrm{C_h} \cdot \mathrm{V}_t)^2$$

Where

$\mathrm{R_{fs}}$	= reduction factor depending on the pipe diameter
$\mathrm{m_{t\_trawl}}$	= trawl board mass
$\mathrm{C_h}$	= span height correction factor for the effective pull-over velocity
V_t	= tow velocity of trawler

The impact energy associated with the added mass of the trawl board is given as:

$$E_a = R_{fa} \cdot \frac{2 \cdot F_b^3}{75 \cdot f_y^2 \cdot t^3} \leq \frac{1}{2} \cdot m_{a\_trawl} \cdot (C_h \cdot V_t)^2$$

Where

R_{fa}	= reduction factor depending on the pipe diameter and soil type
f_y	= characteristic material strength for yield stress
t	= steel wall thickness
m_{a_trawl}	= trawl board added mass

The reduction factor for steel and added mass is conservatively considered as 1 as the soil along the pipeline varies from rock to soft clay. Considering the reduction factor as 1 would give conservative results for the trawl impact analysis.

F_b is the impact force associated with the hydrodynamic added mass of the trawl board and may be estimated as:

$$F_b = C_h \cdot V_t \cdot \sqrt{m_{a\_trawl} \cdot k_b}$$

Where

k_b	= lateral bending stiffness of the trawl board

The impact energy associated with clump weight is given as:

$$E_{loc\_clump} = R_{fs} \cdot \frac{1}{2} \cdot \left(m_{t\_clump} + m_{a\_clump}\right) \cdot (V_c)^2$$

Where

R_{fs}	= reduction factor depending on the pipe diameter
m_{t_clump}	= clump weight mass
m_{a_clump}	= hydrodynamic added mass for clump weight
V_c	= tow velocity of clump weight

The absorption of impact energy, E_k by the concrete coating is calculated in accordance with DNV-RP-F107, Ref. /7/, Section 4.6.1. The energy absorbed is a function of the penetrated volume and the crushing strength of the concrete. A formula for the energy can be written as a function of the penetration depth:

$$E_k = Y \cdot b \cdot 2 \int_0^{x_0} \sqrt{D \cdot x_o - x_o^2}\ dx$$

Where

E_k	= absorbed impact energy
Y	= crushing strength of concrete
b	= width of impacting object (footprint width)
D	= pipe outer diameter incl. coating
x_o	= penetration depth

The maximum acceptable dent size is calculated as per DNV-RP-F111, Ref. /10/, Section 6. The frequency class is assumed as low with an impact frequency of < 1 event per km per year. The maximum accepted ratio of permanent dent depth to the outer pipe steel diameter is:

$$\frac{H_{\mathrm{p,c}}}{D} = 0.05 \times \eta$$

Where

$H_{\mathrm{p,c}}$ = characteristic permanent plastic dent depth

η = usage factor

The acceptable permanent dent sizes are given in Table 6-10.

Frequency class	Usage	Dent depth, $H_{\mathrm{p,c}}$ [%] of D
High (>100)	0	1
Medium (1-100)	0.3	1.5
Low (<1)	0.7	3.5

Table 6-10 Acceptable dent sizes relative to outer diameter

The dent depth shall be estimated by using the force-dent pipe shell relationship given in DNV-RP-F111, Ref. /10/, Section 3.4.5

$$H_{\mathrm{p,c}} = \left(\frac{F_{sh}}{5 \cdot f_y \cdot t^{3/2}}\right)^2 - \left(\frac{F_{sh} \cdot \sqrt{0.005 \cdot D}}{5 \cdot f_y \cdot t^{3/2}}\right)$$

Where

f_y = characteristic material strength for yield stress

t = steel wall thickness

F_{sh} = maximum impact force experienced by the pipe shell

$$F_{sh} = \left(\frac{75}{2} \cdot E_{loc} \cdot f_y \cdot t^3\right)^{1/3}$$

E_{loc} = impact energy absorbed locally by the pipe shell

6.3.3 Results

The trawl impact assessment is carried out for the minimum concrete coating thickness along the entire pipeline, i.e. 45 mm, according to the On-bottom Stability analysis in Section 7.1. The effect from an impact with a trawl board and a clump weight is studied.

Results from the analytical trawl impact assessment are listed in Table 6-11, and the calculations are attached in Appendix IV. The concrete coating is found to absorb all the impact energy, and the maximum penetration depth into the concrete coating is found to be 26.13 mm and 25.39 mm for a trawl board and a clump weight impact respectively. Thus, it can be concluded that a concrete coating of 45 mm is sufficient to protect the steel pipe against a trawl impact.

Trawl gear	Parameter	Units	Values
Trawl board	Absorbed energy	kJ	9.08
	Penetration depth	mm	26.13
Clump weight	Absorbed energy	kJ	8.70
	Penetration depth	mm	25.39

Table 6-11 Analytical results of impact energies and concrete penetration depths

Efforts have also been made in order to estimate the acceptable trawl board and clump weight for the maximum allowable permanent dent depth on pipe shell as per DNV-RP-F111 section 6. The results are given in Table 6-12. The frequency class is assumed as low with an impact frequency of < 1 event per km per year. The impact energy transmitted to the pipe shell from the trawl board is reduced due to energy absorption by coating. Thus, the maximum acceptable trawl board and clump weight which can cause an allowable permanent dent on the pipe shell is 5670 kg and 8133 kg respectively.

Parameter	Units	Values
Allowable permanent dent on pipe shell	mm	17.78
Impact energy transmitted to pipe shell	kJ	8.59
Acceptable trawl board weight	kg	5670
Acceptable clump weight	kg	8133

Table 6-12 Analytical results of acceptable trawl board and clump weight

6.4 Corrosion protection design

6.4.1 Internal flow coating

Based on the recommendation in the system corrosion protection philosophy, Section 4.4, a drag reducing internal flow coating is envisaged to reduce the pressure loss through the Balticconnector pipeline. A two-component epoxy paint of approximately 0.1 mm dry film thickness is normally applied. The internal coating shall comply with ISO 15741:2001, Friction-Reduction Coatings for the Interior of On- and Off-Shore Pipelines for Non-Corrosive Gases, Ref. /17/. Internal flow coating is not envisaged to offer any corrosion protection, which is not needed as the transported medium is dry gas, thus field joints are not internally coated.

Coating	Type	Thickness (mm)	Density (kg/m^3)
Internal flow coating	Epoxy paint	0.1	1500

Table 6-13 Recommended Internal flow coating

6.4.2 Anti-corrosion coating

It is recommended to select 3LPE coating for the 20” Balticconnector pipeline based on the recommendation in the system corrosion protection philosophy in section 4.4 and the evaluation in Appendix I. The required coating thickness is estimated based on ISO 21809-1:2011, Ref. /18/. The coating thickness is the function of coating class and the pipe weight. The coating class and coating thickness class is specified in Table 6-14 and Table 6-15 respectively.

Coating class	A	B	C[1)]
Top layer material	LDPE	MDPE/HDPE	PP
Design temperature range (°C)	-20 to +60	-40 to +80	-20 to +110
Note: 1) Installation and transportation at temperatures below 0 °C can cause mechanical damage.			

Table 6-14 Coating class and design temperature range

Pipe weight P_m kg/m	Total coating thickness [1)] mm								
	Class A1[2)]	Class A2[3)]	Class A3[4)]	Class B1[2)]	Class B2[3)]	Class B3[4)]	Class C1[2)]	Class C2[3)]	Class C3[4)]
$P_m \leq 15$	1.8	2.1	2.6	1.3	1.8	2.3	1.3	1.7	2.1
$15 < P_m \leq 50$	2.0	2.4	3.0	1.5	2.1	2.7	1.5	1.9	2.4
$50 < P_m \leq 130$	2.4	2.8	3.5	1.8	2.5	3.1	1.8	2.3	2.8
$130 < P_m \leq 300$	2.6	3.2	3.9	2.2	2.8	3.5	2.2	2.5	3.2
$300 < P_m$	3.2	3.8	4.7	2.5	3.3	4.2	2.5	3.0	3.8

Notes:
1) The required total coating thickness may be reduced by a maximum of 10 % on the weld seam for SAW-welded pipes.
2) Class 1 is for light duty (onshore sandy soil).
3) Class 2 is for moderate duty (clay soils, absence of backfill).
4) Class 3 is for heavy duty (rocky soil or offshore).

Table 6-15 Minimum total coating thickness

Based on the recommended anti-corrosion coating of LDPE, the coating class is selected as A1 as the pipeline is well protected by the concrete coating thickness. The pipe weight considering the thickness of 12.7 mm is 155.1 kg/m. The recommended anti-corrosion coating thickness is presented in Table 6-16.

Description	Top layer material	Coating Class	Pipe weight, P_m [kg/m]	Selected/recommended coating thickness [mm]	Density [kg/m^3]
20” Balticconnector	3LPE	A1	155.1	3.5	930

Note:
The anti-corrosion coating shall comply with DNV-RP-F106, Ref. /6/.

Table 6-16 Recommended anti-corrosion coating

Based on the above Table 6-16, a 3.5 mm 3-layer polyethylene coating shall be adopted.

6.4.3 Field joint coating

The Balticconnector pipeline not subject to heavy trawling and is protected (buried or rock covered) in areas with a high frequency of shipping activity. Hence, the field joint coating does not need to be able to sustain significant trawl impacts, and polyurethane foam is routinely applied as infill between the adjoining concrete coatings. A heat shrinkable sleeve is used as anti-corrosion coating, which shall be compatible with the PE coating, in case this is used as parent coating on the pipe.

Coating	Type	Thickness (mm)	Density (kg/m^3)	Cutback length (mm)	
				Concrete weight coating	Anti-corrosion coating
Field joint coating	Heat shrinkable sleeve + PU foam	As per concrete coating thickness	1000	340	240

Note:
The field joint coating shall comply with DNV-RP-F102, Ref. /4/. Cutback length specified is based on the previous project experience, and girth welding machine requirement.

Table 6-17 Recommended field joint coating, Ref. /34/

6.5 Cathodic protection requirement

Apart from the anti-corrosion coating, the submerged pipeline shall also be provided with cathodic protection in case of damage and degradation of the anti-corrosion coating during installation and operation. The submerged steel pipeline will be subject to external corrosion due to chemical reactions with the surroundings. Cathodic protection is applied using electrically connected sacrificial anodes, made from a less noble material than steel such as an aluminium alloy. The pipeline then acts as the cathode of the system, while the mounted aluminium acts as the anode.

Cathodic protection is provided by sacrificial anodes of the bracelet type which consists of two half-shells installed on the pipeline to form one anode bracelet. The anode material type selected shall be indium activated aluminium alloy Al-Zn-In with a proven chemical composition according to Table 5 in ISO 15589-2:2012, Ref. /16/, Section 8.4.

The cathodic protection design is performed in accordance with *ISO 15589-2 Petroleum and natural gas industries – Cathodic protection of pipeline transportation systems, Part 2: Offshore pipelines, 2012*, Ref. /16/.

6.5.1 Input data

The cathodic protection requirement is designed for a pipeline design life of 50 years, as stated in the *Design Basis*, Ref. /34/.

The relevant pipeline properties are listed in Table 6-18.

Parameter		Unit	20” Gas pipeline
Pipe OD		mm	508
Wall thickness	Zone 1	mm	12.7
	Zone 2		
Pipeline length		m	80,392
Pipe temperature		ºC	50 [1)]
Anode temp.	Buried	ºC	50 [2)]
	Exposed		25 [3)]
Sea water temperature		ºC	4

Notes:
1) Pipe temperature is considered as the design temperature
2) For conservatism, the anode temperature is considered the same as content the temperature, i.e. design temperature.
3) The anode temperature for the exposed pipeline is conservatively considered as 25 ºC, instead of ambient seawater temperature.

Table 6-18 Pipeline system properties

The anti-corrosion coating system for the pipeline is provided in Table 6-19.

Pipeline	Coating System	Coating thickness (mm)
20” Gas pipeline	3LPE	3.5

Table 6-19 Pipeline anti-corrosion coating system

A heat shrink sleeve shall be applied at field joints along the pipeline system.

Coating breakdown factors are extracted from ISO 15589-2:2012, Ref. /16/, Section 7.5, Table 4 and are listed in Table 6-20.

Coating Breakdown factors		*fi*	*Δf*
3-layer PE	Heat shrink sleeve + infills with Multilayer 3LPE	0.004	0.0002

Table 6-20 Coating breakdown factors

The properties for the cathodic protection design for the pipeline system are presented below in Table 6-21.

<table>
<tr><th>Parameter</th><th>Symbol</th><th>Unit</th><th colspan="2">Buried condition</th><th colspan="2">Exposed Condition</th><th>Reference</th></tr>
<tr><td>Anode alloy</td><td>-</td><td>-</td><td colspan="4">Al-Zn-In</td><td>-</td></tr>
<tr><td>Protective mean current density</td><td>i_m</td><td>mA/m^2</td><td colspan="2">20$^{1)}$</td><td colspan="2">120$^{1)}$</td><td>Ref. /16/, Section 7.4.1, 7.4.3</td></tr>
<tr><td>Anode utilisation factor</td><td>u</td><td>-</td><td colspan="4">0.8</td><td>Ref. /16/, Section 8.4</td></tr>
<tr><td>Anode density</td><td>ρ_{anode}</td><td>kg/m^3</td><td colspan="4">2750</td><td>-</td></tr>
<tr><td>Potential of anode material$^{2)}$</td><td>E_a</td><td>mV</td><td colspan="2">-1000</td><td colspan="2">-1050</td><td>Ref. /16/, Section 8.3, Table 5</td></tr>
<tr><td>Minimum required potential for C-Mn steel</td><td>E_c</td><td>mV</td><td colspan="2">-900</td><td colspan="2">-800</td><td>Ref. /16/, Section 7.2.1, Table 1</td></tr>
<tr><td>Electrical resistivity of C-Mn steel</td><td>ρ_{CMn}</td><td>Ω·m</td><td colspan="4">0.2 x 10^{-6}</td><td>Ref. /34/</td></tr>
<tr><td rowspan="2">Anode electrochemical capacity$^{3)}$</td><td rowspan="2">ε</td><td rowspan="2">Ah/kg</td><td>< 30°C</td><td>1500</td><td>< 30°C</td><td>2000</td><td rowspan="2">Ref. /16/, Section 8.3, Table 5</td></tr>
<tr><td>60°C</td><td>800</td><td>60°C</td><td>1500</td></tr>
<tr><td colspan="8">Notes:
1) The protective mean current density shall be increased by 1 mA/m^2 for each degree Celsius of the metal temperature above 25°C as devised by ISO 15589-2:2012, Ref. /16/, Section 7.4.4.
2) The potential for anode material shall be selected based on the anode surface temperature as devised by ISO 15589-2:2012, Ref. /16/, Section 8.3, Table 5.
3) Electrochemical capacity for buried and exposed anode condition is as stipulated in ISO 15589-2:2012, Ref. /16/, Section 8.3, Table 5.
4) The electrochemical capacity has been linearly interpolated as stipulated in Ref. /16/, Section 8.3, Table 5 for the intermediate temperature.</td></tr>
</table>

Table 6-21 Anode design properties

The fraction of anode material that is assumed to supply adequate current cathodic protection is specified by the anode utilisation factor. Thus, when an anode is consumed beyond its utilisation factor, its capacity becomes unpredictable.

The electrochemical requirements shall be those selected by ISO 15589-2:2012, Ref. /16/, Section 8.3. The electrochemical capacity has been linearly interpolated as stipulated in Ref. /16/, Section 8.3, Table 5 for the intermediate temperature.

The seawater properties are presented in section 3.6. The seawater resistivity and the seabed mud resistivity are assumed as 1.5 Ω·m, as stated in the *Design Basis*, Ref. /34/. This is to be confirmed by survey contractors before the next phase of engineering.

6.5.2 Methodology

The CP design is carried out in accordance with the standard DNV-OS-F101, Ref. /1/, Section 6 D 500 which specifies the usage of the following standards:

- *ISO 15589-2 Petroleum and natural gas industries – Cathodic protection of pipeline transportation systems, Part 2: Offshore pipelines, 2012*, Ref. /16/.

Isolation joints can be provided at the interface between offshore and onshore sections at both landfall locations, in order to avoid current drained by the onshore section.

The anode applied in the CP design for the pipeline system is of the bracelet type. A sketch showing the anode bracelet type installed with concrete coating is shown in Figure 6-1.

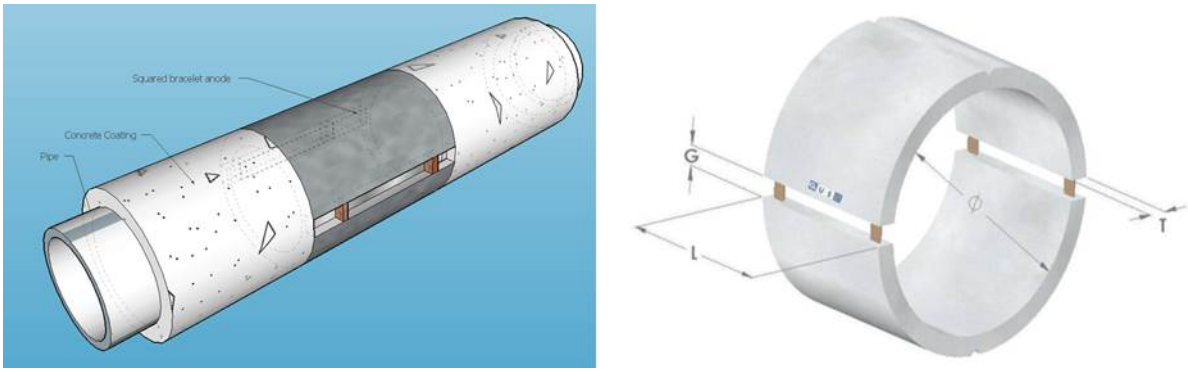


Figure 6-1 Squared bracelet anode type

Bracelet anode half shells shall have connection cables welded (via thermit welding process or pin brazing) to the anode insert extensions at the locations shown in Figure 6-2.

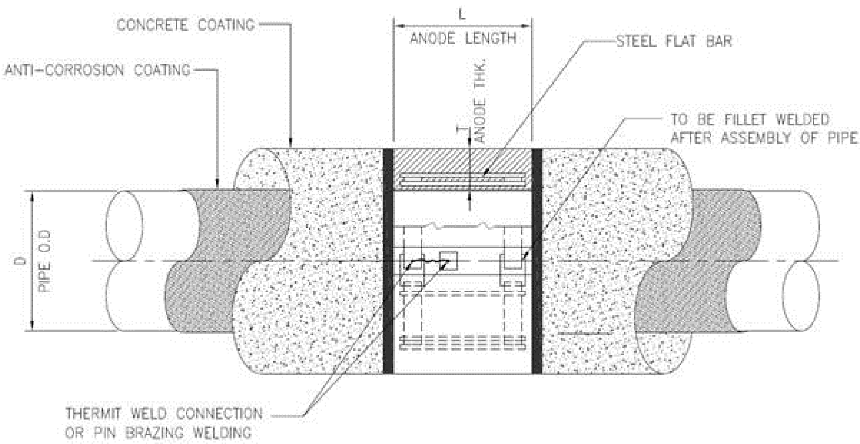


Figure 6-2 Electrical connectivity of bracelet anode

The anode material and dimensions used in this study are presented in Table 6-22.

Anode Material	Anode Gap G [mm]	Anode thickness T [mm]	Anode Length L [mm]	Internal Anode Coating [mm]	Anode mass [kg]
Al-Zn-In	80	40	600	0.1	104.56

Notes:
1. Anode thickness is considered 5 mm less than that of the minimum concrete coating thickness in order to accommodate the concrete coating tolerance.
2. For pipeline section with concrete coating thickness of 55 mm and 80 mm, the concrete coating has to be tapered to the anode thickness with a 45 degree angle.

Table 6-22 Anode material and dimension

ISO 15589-2:2012, Ref. /16/, Annex A provides the procedure for determining the mass and current requirements for the cathodic protection design. The mean current demand I_{cm}, the mean coating breakdown factor f_{cm}, the total final current demand and the final coating breakdown factor f_{cf} are used to calculate the cathodic protection mass requirement M_{req} and the current requirement I_{req} shown below.

$$M_{req} = \frac{\left(\frac{I_{cm} \cdot t_f \cdot 8760}{u \cdot \varepsilon}\right) \cdot \gamma_{sc}}{W_{anode}}$$

Where:

u = anode utilisation factor

ε= electrochemical capacity, A·hr/kg

8760 = number of hours per year

W_{anode} = anode mass, kg

γ_{sc} = safety factor = 1

$$I_{req} = \frac{I_{cf} \cdot \gamma_{sc}}{I_{af}}$$

Where:

$I_{af} = \frac{(E_c^{\circ} - E_a^{\circ})}{R_{af}}$

$R_{af} = 0.315 \frac{\rho}{\sqrt{A_{anode}}}$

ρ = resistivity

A_{anode} = anode surface area

E_c° = design protective potential,

E_a° = design close circuit anode potential

The attenuation check for anode spacing exceeding 300 m shall be carried out based on ISO 15589-2:2012, Ref. /16/, Section B3.

6.5.3 Results

The cathodic protection requirement is calculated based on the methodology stated in section 6.5.2. The nominal configuration, i.e. anodes per joint and spacing is obtained by rounding down the maximum allowable spacing to the nearest multiple joint lengths. The total number of anodes per section is conservatively rounded up to nearest integer. The anode spacing is maintained as an even number of joints in order to facilitate double-jointing. Al-Zn-In anode material has been considered.

No spare anodes are included in Table 6-23.

The anode requirement has been calculated for both the exposed and buried pipeline condition, and the most onerous results will be considered in the design. The calculations are attached in Appendix V. Table 6-23 summarises the anode requirement for the pipeline including one anode to project the onshore, pulled-in pipe section at the Estonian landfall, which is below the mean sea level.

Pipe condition	KP from	KP to	Anode ID	Individual Anode Weight	Anode Spacing	No. Of Anode	Total Anode Weight	Criteria for anode spacing
			ID_{anode}	W_a	$Joint_{anode}$	Nos	$Nos \times W_a$	
(-)	(km)	(km)	(mm)	(kg)	(Joints)	(No's)	(kg)	(-)
Exposed	0	80.392	515.20	104.56	12	551	57,613	Current requirement
Buried					16	413	43,183	Current requirement

Notes:
1. Anode thickness and configuration are as per Table 6-22
2. For the pipeline sections with concrete coating thickness of 55 mm and 80 mm, the concrete coating has to be tapered to the anode thickness with a 45 degree angle.

Table 6-23 Anode summary

7 Pipeline in-place design

7.1 On-bottom stability

7.1.1 General

The on-bottom stability analysis of the pipeline comprises lateral and vertical stability of the submarine pipeline during its operational life. The stability analysis for pipeline is carried out to determine the concrete weight coating required for both short and long-term stability against the environmental loading caused by waves and currents.

The method followed for the assessment of the pipeline on-bottom stability is based on DNV-RP-F109, Ref. /8/. The pipeline is checked for the following criteria:

- Lateral stability based on metocean data considering waves and currents
- Vertical stability, i.e. flotation in seawater

The input parameters, assumptions, methodology and results of the stability analysis are presented hereunder. The input parameters from the *Design Basis*, Ref. /34/, given in Table 7-1 were applied for the on-bottom stability analysis.

Description		Symbol	Unit	Value
Pipeline diameter		OD	inch (mm)	20 (508)
Pipeline wall thickness		t	mm	12.7
Steel density		ρ_{steel}	kg/m^3	7850
Anti-corrosion coating thickness		t_{coat}	mm	3.5
Anti-corrosion coating density		ρ_{coat}	kg/m^3	930
Concrete coating cut-back length (field joint coating length)		l_{FJC}	mm	340
Field joint coating density		ρ_{FJC}	kg/m^3	1000
Content density	Empty	ρ_{cont}	kg/m^3	0
	Flooded			1005
	Operation			65
Spectral spreading exponent		s	-	8
Reference height over seabed for current measurements		z_r	m	1.5
Peak enhancement factor		γ	-	1 [1)]
Storm duration		T_{storm}	h	3
Sea water density		ρ_{sea}	kg/m^3	1005
Soil type along the route		-	-	Clay/Sand
Seabed roughness parameter (Clay/Sand)		z_0	m	5x10^{-6}/1x10^{-5}
Friction coefficient (Clay/Sand)		μ	-	0.2/0.6
Saturated bulk unit weight (clay)		γ_s	N/m^3	11837.5 [2)]
Undrained shear strength (clay)		s_u	N/m^2	4000

Notes:
1) Pierson-Moskowitz spectrum is considered for wave and current loads calculation, hence peak enhancement factor is considered as 1.
2) Average of minimum saturated bulk unit weight for all soil types/units is considered in the calculation.

Table 7-1 Input parameters for on-bottom stability analysis

7.1.2 Metocean data

The metocean data applied for the stability analysis is presented in Appendix VI. Directional wave data and directional scaled current velocities were used for the on-bottom stability analysis. The metocean data is extracted from the *Metocean study report*, Ref. /35/. The significant wave height (H_s), peak time period (T_p) and the near seabed current velocity (U_c) for 1-year, 10-year and 100-year condition along with direction of action are summarised.

7.1.3 Lateral stability analysis

The lateral stability analysis considers the following assumptions:

- The design water depth considered for lateral stability analysis is based on minimum MSL (Mean Sea Level) along the pipeline section route.
- Stability analysis for the installation and flooded condition is performed with the empty pipeline and sea water filled pipeline respectively and subjected to 10-year and 1-year critical return period combinations (10-year RP wave + 1-year current and 1-year RP wave + 10-year current).
- Stability analysis for the operating condition is performed for the pipeline filled with product, minimum product density is considered and subjected to 100-year and 10-year critical return period combinations (100-year RP wave + 10-year current and 10-year RP wave + 100-year current).
- Wave spreading and directionality has been considered using a spectral spreading exponent of 8.
- Concrete cut back length of 340 mm for each pipe joint is considered in the analysis.
- Zero marine growth thickness is assumed, Ref. /34/.
- The generalised 10D stability criterion is considered along the entire pipeline length, to achieve lateral stability.
- A concrete coating density of 3400 kg/m^3, 3040 kg/m^3 and 2400 kg/m^3 shall be considered. Concrete coating thickness shall be estimated for all the above densities, and the most feasible and economical case would be recommended.
- Maximum concrete coating thickness shall be limited to 120 mm.
- Minimum concrete coating thickness shall be 45 mm due to limitation of impingement method, which fulfil the SG ≥ 1.1 requirement for vertical stability.
- Concrete coating thickness shall be rounded-up to the nearest multiple of 5.

The lateral stability analysis of the pipeline is carried out in accordance with the requirements of DNV-RP-F109, generalised method using DNV software “StableLines”, which determines the concrete thickness required for lateral stability of the submarine pipelines based on the design procedure stipulated in Section 3.5 of DNV-RP-F109, Ref. /8/.

<u>Generalised lateral stability method</u>
The generalised lateral stability method explained in DNV-RP-F109 is based on database results from dynamic analyses and simulations allowing for lateral pipe displacements.

For the pipeline to be stable, the actual submerged weight of the pipeline must be equal to or greater than the required submerged weight resulting from the required concrete thickness for lateral stability.

The design code DNV-RP-F109 provides design curves for on-bottom stability design with an allowed lateral displacement in the range from less than half a pipe diameter, i.e. for a virtually stable pipe, up to a displacement of 10 diameters during the given sea state. These curves are obtained from a large number of one dimensional dynamic analysis; i.e. on a flat seabed and neglecting bending and axial deformation of the pipe.

For a pipeline on clay, the generalised lateral stability method in DNV-RP-F109, Ref. /8/, is only valid for a strength parameter $G_c \leq 2.78$, where $G_c = S_u/(D \times \gamma_s)$. In the event a pipeline does not satisfy the aforementioned specific weight criteria, DNV-RP-F109 recommends the use of the absolute lateral static stability method.

Absolute lateral static stability method

The methodology of the absolute lateral static stability method is based on Section 3.6 of DNV-RP-F109, Ref. /8/. This approach is based on force equilibrium ensuring that the hydrodynamic loads are less than the soil resistance under a design extreme oscillatory cycle in the sea state considered for design.

7.1.4 Vertical stability analysis

In order to avoid flotation in sea water, the submerged weight of the pipeline shall meet the following requirement stipulated in Section 3.2 of DNV-RP-F109, Ref. /8/.

$$\gamma_w \cdot \frac{b}{w_s + b} = \frac{\gamma_w}{s_g} \leq 1.00$$

Where:

γ_w	= safety factor, 1.1
b	= pipe buoyancy per unit length defined as $b = \rho_{sw}\, g\, \pi\, D^2/4$ with:
ρ_{sw}	= density of seawater taken as 1005 kg/m^3
g	= gravity taken as 9.81 m/s^2
D	= outer diameter including coatings
w_s	= pipe submerged weight
s_g	= pipe specific gravity of the pipeline.

The density of seawater is taken as 1005 kg/m^3, which is characteristic for the brackish seawater in Gulf of Finland.

7.1.5 Sectioning of route for calculation

To optimise the stability requirements along the route, the pipeline route will be divided into many segments based on; water depth variations, pipeline orientation, soil data, environmental loading and protection requirements. For each section, minimum MSL will be used for the stability calculations. The route is divided into different segments as shown in Figure 7-1. The environmental data points and the KP's for each section are plotted along the pipeline route; this will help to identify the applicable environmental data point for each section. The sections are designated S1 to S10. If multiple environmental data points are applicable for a particular section and if there is a significant variations in water depth and soil type, those sections are further sub divided and are summarised in Table 7-2.

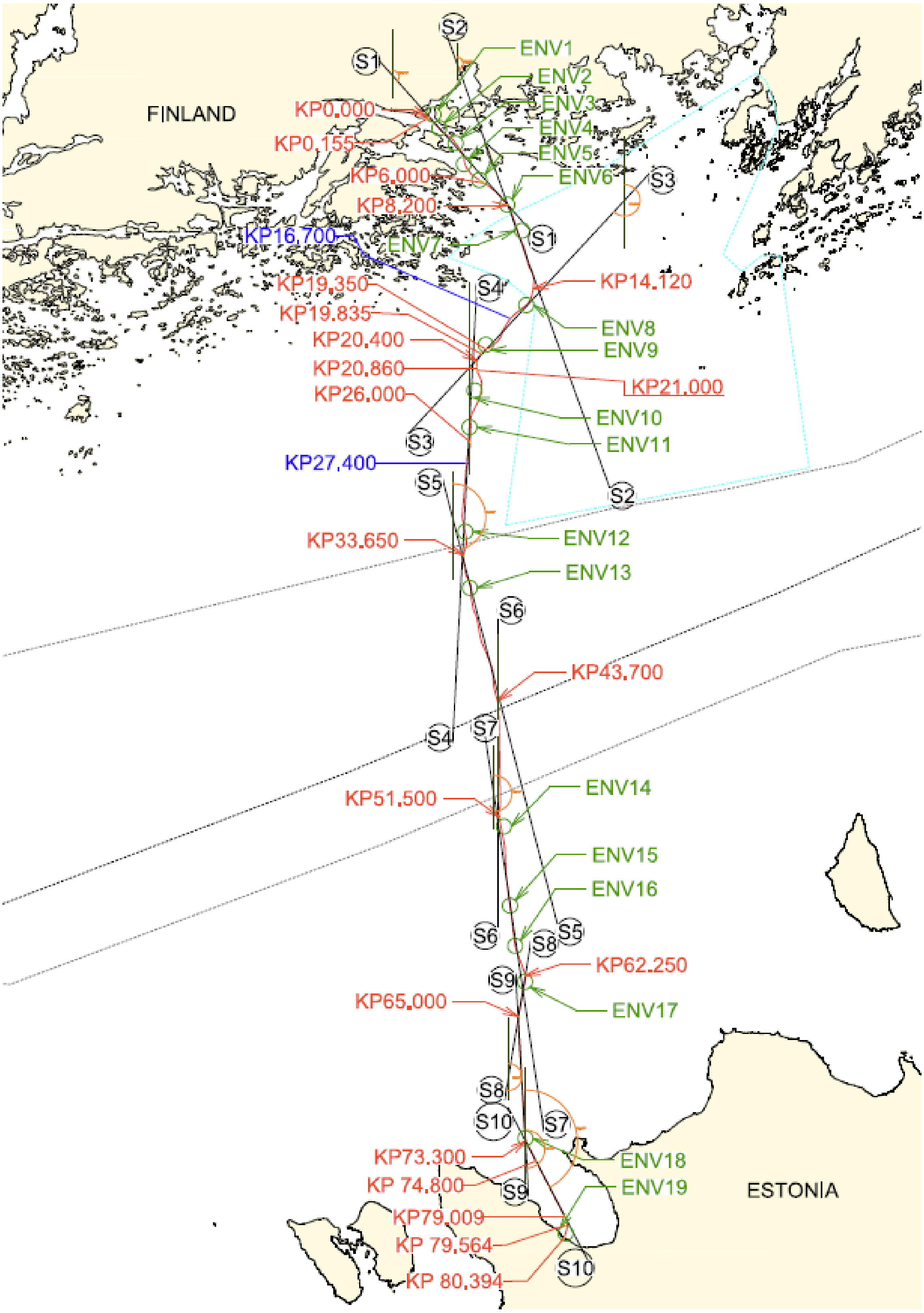


Figure 7-1 **Environmental data extraction points and segmentation on the pipeline route**

The pipeline from KP 0.000 to KP 0.038 and KP 79.564 to KP 80.392 will be trenched and protected by rock cover after landfall pull-in operations at the Finnish and Estonian shore respectively. The concrete coating thickness at these landfall locations will be considered the same as that of the adjacent section, which will ensure that the pipeline will be stable in the temporary phase. Hence these sections are not considered in the analysis. Long term stability for these sections is achieved by rock cover protection. The pipeline route segmentation, minimum water depth and wave/current data adopted in the stability analysis are in accordance with the Table 7-2.

Pipeline segments & locations				Direction of pipeline relative to geographic north angle (°)	Soil type	Minimum surveyed WD, (m)	Environmental condition data points ID's
Sr. No	Section ID	KP from	KP to				
1	S1-1	0.038	0.155	138	Clay	5.0	ENV1
2	S1-2	0.155	6	138	Clay	8.7	ENV 2, ENV 3, ENV 4, ENV 5
3	S1-3	6	8.2	138	Clay	17.6	ENV 6
4	S2	8.2	14.12	160	Clay	17.0	ENV 6, ENV 7
5	S3-1	14.12	19.35	223	Clay	24.9	ENV 8
6	S3-2	19.35	19.835	223	Clay	16.2	ENV 9
7	S3-3	19.835	20.4	223	Clay	23.6	ENV 9
8	S4-1	20.4	20.86	183	Clay	23.9	ENV 9
9	S4-2	20.86	21	183	Clay	17.2	ENV 10
10	S4-3	21	26	183	Clay	27.9	ENV 10, ENV 11
11	S4-4	26	33.65	183	Clay	50.2	ENV 12
12	S5	33.65	43.7	166	Clay	56.2	ENV 13
13	S6	43.7	51.5	180	Clay	54.7	ENV 14
14	S7	51.5	62.25	172	Clay	56.3	ENV 14, ENV 15, ENV 16
15	S8	62.25	65	189	Clay	73.1	ENV 17
16	S9	65	73.3	177	Clay	34.9	ENV 18
17	S10-1	73.3	74.8	152	Clay	26.2	ENV 18
18	S10-2	74.8	78.97	152	Sand	12.3	ENV 18
19	S10-3	78.97	79.564	152	Sand	5.0	ENV 19

Table 7-2 Pipeline route segmentation and wave/current data

Note that clay is predominantly used for the on-bottom stability analysis, as opposed to bedrock in some sections, as the friction factors provide the most conservative results.

Pipeline on-bottom stability has been performed for each pipeline segment and the environmental data points. Where several environmental data points are located in one segment, the worst case data point is considered for the on-bottom stability analysis.

7.1.6 Result of analysis

Lateral stability

The lateral stability of the pipelines has been examined as per DNV-RP-F109. The concrete weight coating is selected so that pipeline is laterally stable for the given unfavourable environmental loading during the entire design life. The lateral stability is carried out based on the methodology mentioned in section 7.1.3.

The calculated concrete coating thicknesses are summarised in Table 7-3.

Sect.	KP range		Met pt	Calculated concrete coating thickness (mm)								
				CD = 3400 kg/m^3			CD = 3040 kg/m^3			CD = 2400 kg/m^3		
	From	To		Op.	Inst.	Flo.	Op.	Inst.	Flo.	Op.	Inst.	Flo.
S1-1	0.038	0.155	1	22.33	20.09	0.00	26.24	23.57	0.00	38.10	34.09	0.00
S1-2	0.155	6.000	2	16.63	16.62	0.00	19.53	19.48	0.00	28.31	28.12	0.00
S1-2	0.155	6.000	3	22.88	20.44	0.00	26.90	23.99	0.00	39.12	34.70	0.00
S1-2	0.155	6.000	4	24.40	21.25	0.00	28.70	24.94	0.00	41.78	36.10	0.00
S1-2	0.155	6.000	5	48.26	23.10	0.00	56.66	27.11	0.00	82.28	39.25	0.00
S1-3	6.000	8.200	6	48.78	26.72	0.00	58.61	31.39	0.00	91.44	45.55	0.00
S2	8.200	14.120	23	44.35	28.91	0.00	53.05	34.06	0.00	81.98	49.86	0.00
S2	8.200	14.120	7	52.62	38.35	0.00	63.43	45.57	0.00	99.58	65.74	0.00
S3-1	14.120	19.350	8	45.85	27.28	0.00	54.45	32.12	0.00	81.51	46.90	0.00
S3-2	19.350	19.835	9	114.30	59.89	4.23	143.42	71.23	5.21	256.26	107.09	8.85
S3-3	19.835	20.400	9	62.55	32.36	0.00	76.06	38.22	0.00	123.04	56.27	0.00
S4-1	20.400	20.860	9	60.54	42.83	0.00	73.22	50.96	0.00	116.38	77.78	0.00
S4-2	20.860	21.000	10	192.75	105.70	44.68	251.40	131.52	57.38	480.13	225.61	114.41
S4-3	21.000	26.000	10	78.28	49.76	0.00	96.21	59.04	0.00	161.06	88.15	0.00
S4-3	21.000	26.000	11	73.43	47.89	0.00	89.95	57.41	0.00	149.00	87.79	0.00
S4-4	26.000	33.650	12	35.76	22.95	0.00	42.31	26.98	0.00	62.83	39.20	0.00
S5	33.650	43.700	13	33.80	24.76	0.00	40.01	29.28	0.00	60.04	43.11	0.00
S6	43.700	51.500	14	36.43	27.71	0.00	42.92	32.49	0.00	64.48	46.80	0.00
S7	51.500	62.250	14	38.09	28.06	0.00	44.83	32.84	0.00	67.59	47.30	0.00
S7	51.500	62.250	15	30.99	24.77	0.00	36.65	29.21	0.00	54.64	42.90	0.00
S7	51.500	62.250	16	35.76	26.19	0.00	42.14	30.79	0.00	63.20	44.74	0.00
S8	62.250	65.000	17	19.94	17.03	0.00	23.57	20.00	0.00	34.67	29.03	0.00
S9	65.000	73.300	18	37.31	23.14	0.00	44.22	27.19	0.00	65.78	39.50	0.00
S10-1	73.300	74.800	18	35.64	23.00	0.00	42.23	27.03	0.00	62.72	39.26	0.00
S10-2	74.800	78.970	18	38.22	22.61	0.00	44.53	26.43	0.00	63.06	37.81	0.00
S10-3	78.970	79.564	19	11.81	14.31	0.00	13.85	16.78	0.00	19.98	24.20	0.00

Abbreviations:
Sect. = Section
Met pt = Applicable metocean point
CD = Concrete density
Op. = Operation, Inst. = Installation, Flo. = Flooded

Table 7-3 Calculated concrete coating thickness

Based on the assumption that concrete coating thickness shall be rounded-up to the nearest multiple of 5 mm, the required concrete coating thicknesses are presented in Table 7-4. The required concrete coating thickness is based on the maximum of all the applicable metocean data points and the three pipeline condition cases; operational, empty and flooded. It is observed that the operational condition is governing as the pipeline is designed for a 100-year RP condition.

Sect.	KP range		Met pt	Calculated concrete coating thickness (mm)											
				CD = 3400 kg/m^3				CD = 3040 kg/m^3				CD = 2400 kg/m^3			
	From	To		Op.	Inst.	Flo.	Rec.	Op.	Inst.	Flo.	Rec.	Op.	Inst.	Flo.	Rec.
S1-1	0.038	0.155	1	25	25	0	**45**	30	25	0	**45**	40	35	0	**45**
S1-2	0.155	6.000	5	50	25	0	**50**	60	30	0	**60**	85	40	0	**85**
S1-3	6.000	8.200	6	50	30	0	**50**	60	35	0	**60**	95	50	0	**95**
S2	8.200	14.120	7	55	40	0	**55**	65	50	0	**65**	100	70	0	**100**
S3-1	14.120	19.350	8	50	30	0	**50**	55	35	0	**55**	85	50	0	**85**
S3-2	19.350	19.835	9	115	60	5	**115**	145	75	10	**145**	260	110	10	**260**
S3-3	19.835	20.400	9	65	35	0	**65**	80	40	0	**80**	125	60	0	**125**
S4-1	20.400	20.860	9	65	45	0	**65**	75	55	0	**75**	120	80	0	**120**
S4-2	20.860	21.000	10	195	110	45	**195**	255	135	60	**255**	485	230	115	**485**
S4-3	21.000	26.000	10	80	50	0	**80**	100	60	0	**100**	165	90	0	**165**
S4-4	26.000	33.650	12	40	25	0	**45**	45	30	0	**45**	65	40	0	**65**
S5	33.650	43.700	13	35	25	0	**45**	45	30	0	**45**	65	45	0	**65**
S6	43.700	51.500	14	40	30	0	**45**	45	35	0	**45**	65	50	0	**65**
S7	51.500	62.250	14	40	30	0	**45**	45	35	0	**45**	70	50	0	**70**
S8	62.250	65.000	17	20	20	0	**45**	25	25	0	**45**	35	30	0	**45**
S9	65.000	73.300	18	40	25	0	**45**	45	30	0	**45**	70	40	0	**70**
S10-1	73.300	74.800	18	40	25	0	**45**	45	30	0	**45**	65	40	0	**65**
S10-2	74.800	78.970	18	40	25	0	**45**	45	30	0	**45**	65	40	0	**65**
S10-3	78.970	79.564	19	15	15	0	**45**	15	20	0	**45**	20	25	0	**45**

Abbreviations:
Sect. = Section
Met pt = Applicable metocean point
CD = Concrete density
Op. = Operation, Inst. = Installation, Flo. = Flooded, Rec. = Recommended coating thickness

Table 7-4 Required concrete coating thickness

The required concrete coating thickness for the operational condition is plotted along the pipeline route in Figure 7-2. The final recommended concrete coating thickness is presented in Table 7-5.

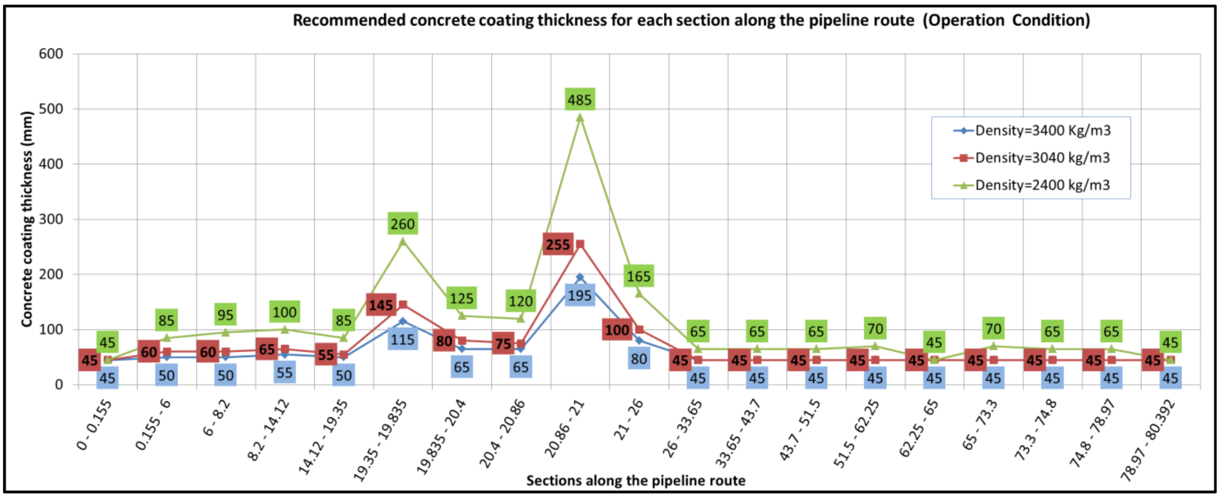


Figure 7-2 Required concrete coating thickness along the pipeline route for operation condition

Section	KP Locations		Length (m)	Recommended Concrete Coating thickness (mm)	Concrete Coating Density (kg/m^3)	Remarks (Selection Criteria)
	From	To				
S1-1	0.000	0.155	117	55	3400	Operation condition
S1-2	0.155	6.000	5845	55	3400	Operation condition
S1-3	6.000	8.200	2200	55	3400	Operation condition
S2	8.200	14.120	5920	55	3400	Operation condition
S3-1	14.120	19.350	5230	55	3400	Operation condition
S3-2	19.350	19.835	485	**80** 2)	3400	Operation condition
S3-3	19.835	20.400	565	80	3400	Operation condition
S4-1	20.400	20.860	460	80	3400	Operation condition
S4-2	20.860	21.000	140	**80** 2)	3400	Operation condition
S4-3	21.000	26.000	5000	80	3400	Operation condition
S4-4	26.000	33.650	7650	45	3400	Operation condition
S5	33.650	43.700	10050	45	3400	Operation condition
S6	43.700	51.500	7800	45	3400	Operation condition
S7	51.500	62.250	10750	45	3400	Operation condition
S8	62.250	65.000	2750	45	3400	Operation condition
S9	65.000	73.300	8300	45	3400	Operation condition
S10-1	73.300	74.800	1500	45	3400	Operation condition
S10-2	74.800	78.970	4170	45	3400	Operation condition
S10-3	78.970	80.392	594	45	3400	Operation condition

Note:
1) The concrete coating thickness for KP 0 to KP 0.038 and KP 79.564 to KP 80.392 is considered the same as that of the adjacent section, as the pipeline at landfall location would be protected by rock cover after landfall pull-in operation. Long term stability for these sections is achieved by rock cover protection. Short term temporary stability is achieved by the recommended concrete coating thickness.
2) For S3-2 post-lay intervention is required as it can be seen that the pipeline section is stable during the temporary condition and for section S4-2 both pre-lay and post-lay intervention would be needed as the section S4-2 is unstable during both temporary and operational conditions for the recommended concrete coating thickness.

Table 7-5 Final recommended concrete coating thickness

The changes in concrete thicknesses and density along the pipeline have been reduced to aid in material management and installation. It is to be noted that the concrete coating thickness for 2400 kg/m^3 concrete density is considerably high at some locations; hence the concrete coating thickness estimated with 2400 kg/m^3 concrete density is not recommended. To facilitate coating and installation, the same concrete density should be maintained for the entire pipeline, and the number of different coating thicknesses kept at a minimum.

Based on the results mentioned in Table 7-4, there are two small sections, S3-2 and S4-2, which are not stable during the temporary and/or operational condition for the recommended concrete coating thickness. Both those sections are noted as critical due to in the local buckling analysis of the free spans, therefore a combined solution with the seabed intervention requirements is recommended. Otherwise it is necessary to provide some localised stability by pre-lay or post-lay intervention.

For S3-2 only post-lay intervention is required as it can be seen that the pipeline section is stable during the temporary condition and for section S4-2 both pre-lay and post-lay intervention would be needed as the section S4-2 is unstable during temporary and operation condition for the recommended concrete coating thickness.

A sample calculation for lateral stability for section S1-3 (operational condition) with a concrete density of 3400 kg/m^3 is presented in Appendix VII.

Figure 7-3 is plotted in order to see the variation of the concrete coating thickness along the pipeline route for profile water depth as per Table 7-5.

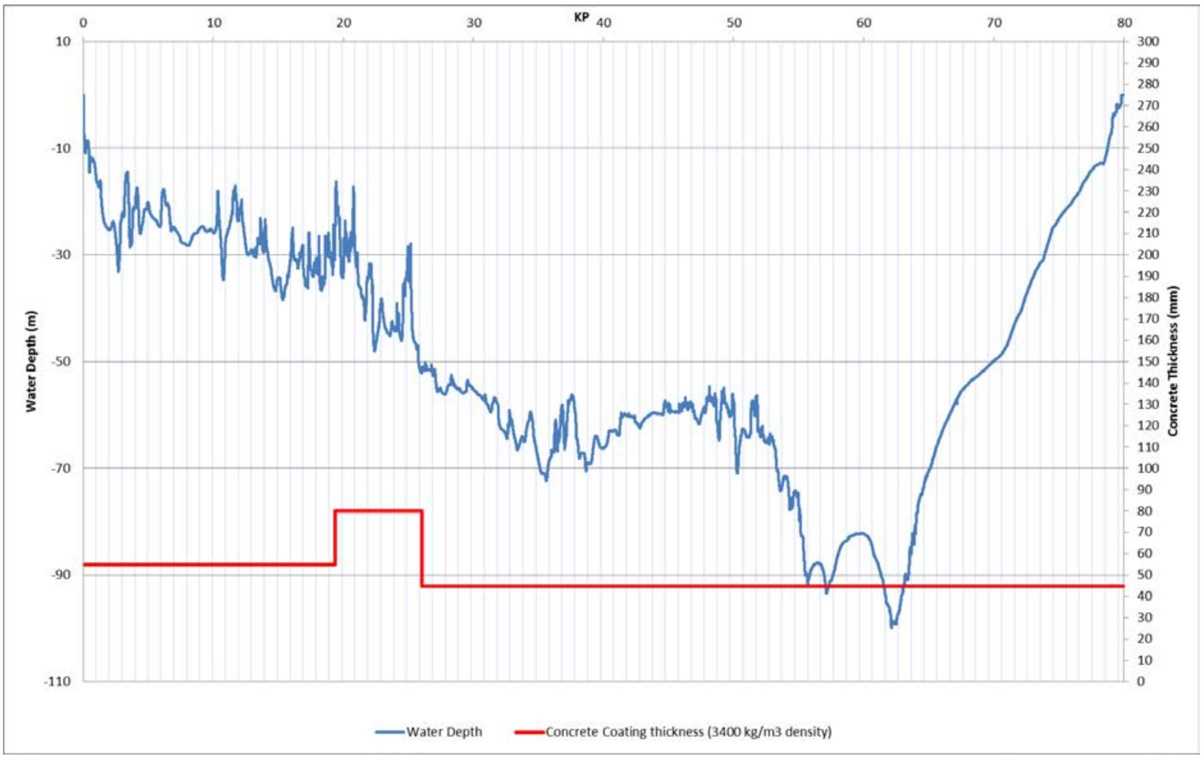


Figure 7-3 Recommended concrete coating thickness and water depth along the pipeline route

For section S3-2 and S4-2, it is recommended to provide pre-lay and post-lay seabed intervention to minimise the requirement for multiple concrete coating thicknesses. By maintaining the same concrete coating thickness as that of the adjacent section, it will also avoid changes in the tensioner settings during pipelay.

Efforts have been made to calculate the limiting water depth for the 80 mm concrete coating thickness with 3400 kg/m^3 density for section S3-2 and S4-2. The results show that at a water depth shallower than 20.2 m for section S3-2 and 27.6 m for section S4-2, the pipeline will not meet the lateral stability criteria.

If seabed intervention is not a feasible solution, a localised additional stabilisation method is recommended for the pipe sections S3-2 and S4-2. Localised additional stabilisation can be achieved either by concrete mattress installation or subsea rock installation i.e., post-lay seabed intervention. A detailed assessment for these sections shall be performed in the detailed engineering phase.

For the FEED study, a concrete coating thickness of 80 mm and density of 3400 kg/m^3 is considered for sections S3-2 and S4-2.

Vertical Stability

The vertical stability is carried out based on the methodology mentioned in section 7.1.4. The specific weight for the selected concrete thicknesses are summarised in Table 7-6. The detailed calculations of the vertical stability for the pipeline are included in Appendix VII. Vertical stability is calculated for the different selected concrete coating thickness and density.

Sr. No	Concrete Coating Thickness (mm)	Concrete Coating Density (kg/m^3)	Specific Weight S_g		
			Installation	Flooded	Operation
1	55	3400	1.61	2.20	1.64
2	80	3400	1.86	2.37	1.89
3	45	3400	1.49	2.12	1.53

Table 7-6 Results of vertical stability

From the analysis results it can be seen that floatation will not pose a problem since the specific weight of the pipeline is greater than 1.1, i.e., ${}^{\gamma_w}/_{s_g} < 1.00$.

7.2 Free span analysis

The free span assessment, performed in accordance with DNV-RP-F105, Ref. /5/, and DNV-OS-F101, Ref. /1/, calculates the allowable span length for the pipeline based on fatigue criteria under empty, water-filled, and operating conditions. Any spans below the allowable span length for the given condition will be deemed acceptable, whereas for spans greater than the allowable span length a location specific detailed re-assessment will be performed in the detailed engineering phase. To ensure a conservative design in the FEED phase, any spans greater than the allowable span length during operation will be rectified with post-lay rock installation.

Based on calculations performed using the DNV software Fatfree, a fatigue life of the pipeline sections for all span lengths were calculated.

Fatigue damage from Vortex Induced Vibrations (VIV) was assessed for both the cross-flow (vertical) and in-line (horizontal) directions and based on a minimum allowable fatigue life calculated from DNV-RP-F105, Ref. /5/, and DNV-RP-C203, Ref. /2/, an allowable free span length was determined.

The input data and assumptions used for the free span fatigue analysis of the offshore pipeline are listed in this section.

7.2.1 Pipeline dimensions and functional loads

The pipeline dimensions used throughout each sub-divided section are given in Table 7-7.

Parameter	Symbol	Unit	Value	Reference
Steel outer diameter	OD_{steel}	mm	508.0	Ref. /34/
Wall thickness	t_{steel}	mm	12.7	Section 6.2
Internal corrosion allowance	-	mm	0	Ref. /34/
Steel density	ρ_{steel}	kg/m^3	7850	
Thermal expansion	α	$°C^{-1}$	1.17×10^{-5}	
Poisson's ratio	ν	-	0.3	
Young's modulus	E	GPa	207	
Structural damping	ζ_{struc}	-	0.01	Ref. /5/ §6.2.11
Anti-corrosion coating				
Anti-corrosion coating thickness	$t_{coating}$	mm	3.5	Section 6.4
Anti-corrosion coating density	$\rho_{coating}$	kg/m^3	930	
Concrete coating				
Concrete coating thickness	$t_{concrete}$	mm	55 / 80 / 45	Section 7.1
Concrete coating density	$\rho_{concrete}$	kg/m^3	3400	
Concrete stiffness factor	k_c	-	0.25	Ref. /5/ §6.2.5
Total outer diameter	OD_{pipe}	mm	625 / 675 / 605	

Table 7-7 Pipeline properties

The gas pipeline will be filled with different contents during the temporary (air and water-filled phase) and the operational (gas-filled) phase. The respective functional loads are shown in Table 7-8.

Parameter			Symbol	Unit	Value	Reference
Temporary phase	Density	Empty	ρ_{air}	kg/m^3	1.3	Ref. /34/
		Flooded	ρ_{water}		1005	
	Pressure at seabed	Empty	P_{air}	bar	1	
		Flooded	P_{water}		$1005 kg/m^3.WD.g$	
	Temperature		T_{pipe}	°C	0	
Operational phase	Density	Operation	ρ_{cont}	kg/m^3	65	
	Pressure	Operation	P_{int}	barg	60 [1)]	
	Temp.	Operation	ΔT_{pipe}	°C	10 [2)]	

Notes:
1) Based on initial estimates of operational data.
2) Based on conservative temperature profile estimates, see Figure 8-1.

Table 7-8 Functional loads

Note that the assumptions for the operational pressure and temperature produce a conservative axial force in the pipeline for the fatigue analysis. From project experience, pressure profiles along low density gas transmission pipelines only show a minor decrease (approximately 5-10 barg) over a distance of 80 km. Temperature profiles decrease more rapidly, based on the external water temperature at the seabed, and given the bi-directional flow expected throughout the lifetime it is expected that a ΔT_{pipe} (difference in temperature of pipeline from installation) of 10 °C is conservative for the majority of the pipeline.

7.2.2 Safety factors

For the fatigue design criteria, performed in accordance with DNV-RP-F105 Table 2-2, Ref. /5/, the general safety factors for the normal safety class are to be applied, with the exception of the usage factor which is taken from the high safety class due to conservatisms explained in section 7.2.7.

Safety factors		Factor
Allowable fatigue damage ratio	η	0.25
Natural frequency (not well defined)	γ_f	1.20
Stability	γ_k	1.15
Stress range	γ_s	1.3
Onset for in-line VIV	$\gamma_{on,IL}$	1.1
Onset for cross-flow VIV	$\gamma_{on,CF}$	1.2

Table 7-9 Safety factors for fatigue criteria

The free span is categorised in compliance with Ref. /5/, Sec. 2.6.3, as being not well defined since information on soil conditions will be provided in the 2016 geotechnical survey and the environmental conditions are not well understood in this region. The classification of safety zones and factors follows the specifications given by DNV-OS-F101, Ref. /1/ Sec. 2, and DNV-RP-F105, Ref. /5/ Sec. 2.6.2. The corresponding applied values are listed in Table 7-9.

7.2.3 Design lifetime and fatigue damage distribution

The operational design life of the Balticconnector offshore pipeline is 50 years as stated in section 3.3.

For a given span length, the fatigue design life capacity T_{life} will be calculated using FatFree, in accordance with DNV-RP-F105, Ref. /5/. The fatigue criterion is given as:

$$T_{life} \geq \frac{T_{exp}}{\eta} \Rightarrow \eta \cdot T_{life} \geq T_{exp}$$

where η is the allowable fatigue damage ratio and T_{exp} is the exposure time of the considered pipeline for a given phase shown in given in Table 7-10. This yields the minimum allowable fatigue design life capacities. The allowable fatigue damage, D_{tot}, can be calculated as

$$D_{tot} = \frac{T_{exp}}{T_{life}} \leq \eta$$

The numeric value applied for the safety factor η depends on whether the safety class is normal or high.

For the fatigue damage distribution calculation, certain values are estimated for the time of exposure in the temporary phase period of the pipeline as well as the percentage of damage allowance given for installation activities.

Condition	Time of exposure	Allowable fatigue damage[1)]	Total allowable damage[2)] η = 0.25	Minimum required fatigue life (years)
Installation	n/a	40.00%	10.00%	n/a
Air-filled pre-hydrotest	6 months	3.00 %	0.75%	66.7
Water-filled	2 years	4.00 %	1.00%	200.0
Hydrotest	1 week	3.00%	0.75%	2.6
Operational phase	50 years	50.00%	12.50%	400.0

Notes:
[1)] Percentage of damage: Section 5 in Ref. /5/. D811 "Guidance Note". This percentage must be agreed through negotiation with pipelay contractor, but given the roughness of the seabed for this pipeline a larger percentage should be made available to the pipeline design life.
[2)] Based on the allowable design fatigue factor in DNV-RP-F105 Table 2-2, Ref. /5/

Table 7-10 Fatigue damage distribution and allowable fatigue life for various phases

This fatigue distribution has been used to determine the allowable span lengths. However, an analysis can be performed after installation using the as-installed data to determine the exact amount of fatigue damage used during the installation phase. Hence, this could allow for a potential increase in fatigue distribution to the temporary and permanent phase for any free spans identified to be critical and in need of rectification.

7.2.4 Environmental loads

The environmental wave and current data are extracted from the *Metocean Data report*, Ref. /35/.

The fatigue analysis is carried out by modelling the short and long term statistics for wave and current data; the former by incorporating the significant wave height H_s and for the current data by incorporating the current velocity U_c. Both H_s and U_c are being described by a 3-parameter Weibull probability distribution in compliance with Sec. 3.5 of Ref. /5/.

A Weibull analysis was carried out for 12 wave directional sectors and 12 current directional sectors at numerous points along the pipeline corridor representing differences in metocean conditions. A Weibull fit was found for directions with more than 10 events, hence in the archipelago region of Finland and in the Lahepere Bay in Estonia, some directions produce no results and a wave height of 0 m is assumed.

When considering the weibull distribution for the operational conditions, the tail of the data describing extreme conditions is not included for the fatigue analysis. These extreme data are covered by the extreme event analysis which is used for ULS checks, such as on-bottom stability.

The weibull parameters specified in the *Metocean Data Report*, Ref. /35/, are to be included in the FatFree software as input data. Table 7-11 and Table 7-12 show the directional significant wave heights and current velocities for these weibull parameters for each "free span section" considered for the fatigue analysis. A free span section is defined as a range of the pipeline length where metocean conditions, soil data, pipeline properties and water depths are similar and therefore the most conservative values of each parameter within the range are applied to provide the screening criteria required to determine the allowable span length.

FS Section	KP Range		Rtn period	Significant wave height for fatigue analysis, H_s (m)											
	From	To	yrs	0	30	60	90	120	150	180	210	240	270	300	330
FS1	0.038	0.155	1	0.00	0.00	0.00	0.00	0.00	0.59	0.02	0.04	0.03	0.00	0.00	0.00
			10	0.00	0.00	0.00	0.00	0.00	0.77	0.18	0.49	0.46	0.00	0.00	0.00
			100	0.00	0.00	0.00	0.00	0.00	0.96	0.50	0.96	0.98	0.00	0.00	0.00
FS2	0.155	3.000	1	0.00	0.00	0.00	0.84	1.26	1.11	0.13	0.00	0.00	0.00	0.32	0.00
			10	0.00	0.00	0.00	1.18	1.84	1.52	0.40	0.00	0.00	0.00	0.61	0.00
			100	0.00	0.00	0.00	1.46	2.47	1.95	0.59	0.00	0.00	0.00	0.75	0.00
FS3	3.000	6.000	1	0.00	0.00	0.00	0.39	1.98	1.68	0.40	0.00	0.00	0.00	0.00	0.00
			10	0.00	0.00	0.00	0.69	2.66	2.25	0.58	0.00	0.00	0.00	0.00	0.00
			100	0.00	0.00	0.00	1.03	3.36	2.83	0.76	0.00	0.00	0.00	0.00	0.00
FS4	6.000	8.200	1	0.00	0.00	0.00	1.75	1.89	2.25	1.68	0.71	0.00	0.00	0.00	0.00
			10	0.00	0.00	0.00	3.03	2.50	3.04	2.14	0.99	0.00	0.00	0.00	0.00
			100	0.00	0.00	0.00	4.54	3.10	3.82	2.60	1.27	0.00	0.00	0.00	0.00
FS5	8.200	14.120	1	0.24	0.00	0.48	2.38	1.82	2.21	2.06	1.07	0.02	0.33	0.34	0.00
			10	0.30	0.00	0.72	3.56	2.42	3.01	2.54	1.36	1.02	1.18	1.23	0.00
			100	0.32	0.00	0.90	4.81	3.01	3.78	2.98	1.65	4.96	1.81	1.94	0.00
FS6	14.120	19.350	1	0.60	0.84	1.06	1.90	1.56	1.87	2.60	3.55	2.67	0.66	0.41	0.45
			10	0.95	1.13	1.38	2.54	2.03	2.51	3.57	4.35	3.30	1.00	0.77	0.85
			100	1.18	1.35	1.64	3.17	2.46	3.11	4.49	5.08	3.89	1.23	0.96	1.08
FS7	19.350	20.860	1	0.61	0.98	1.19	2.29	1.79	2.00	2.31	3.37	3.49	1.22	0.31	0.59
			10	1.02	1.35	1.55	2.96	2.29	2.66	3.03	4.14	4.34	1.62	0.65	1.02
			100	1.27	1.64	1.85	3.61	2.75	3.26	3.68	4.83	5.15	1.93	0.82	1.29
FS8	20.860	21.028	1	0.81	0.98	1.58	2.43	1.73	1.95	2.22	3.31	3.93	1.84	0.73	0.87
			10	1.12	1.32	2.18	3.12	2.19	2.57	2.88	4.08	4.91	2.24	0.98	1.22
			100	1.34	1.57	2.72	3.78	2.59	3.11	3.47	4.76	5.85	2.57	1.15	1.46
FS9	21.028	26.000	1	0.86	1.26	1.71	2.46	1.76	1.93	2.17	3.23	4.12	2.20	1.09	0.86
			10	1.19	1.77	2.32	3.15	2.21	2.54	2.80	3.97	5.15	2.73	1.44	1.20
			100	1.43	2.19	2.88	3.81	2.62	3.07	3.35	4.64	6.13	3.20	1.71	1.45
FS10	26.000	33.650	1	1.20	1.57	2.04	2.59	1.68	1.84	2.06	3.12	4.45	2.92	1.65	1.28
			10	1.65	2.13	2.62	3.34	2.09	2.40	2.65	3.87	5.54	3.72	2.20	1.76
			100	2.01	2.60	3.15	4.06	2.44	2.87	3.16	4.52	6.57	4.46	2.68	2.17
FS11	33.650	43.700	1	1.30	1.66	2.26	2.51	1.64	1.74	1.99	2.95	4.43	3.16	1.84	1.42
			10	1.75	2.22	2.90	3.25	2.04	2.24	2.55	3.65	5.50	4.01	2.42	1.93
			100	2.13	2.69	3.50	3.95	2.38	2.65	3.03	4.26	6.51	4.81	2.92	2.36
FS12	43.700	51.500	1	1.67	1.78	2.56	1.42	1.46	1.40	1.65	1.96	4.04	3.88	2.37	1.78
			10	2.28	2.31	3.31	1.70	1.82	1.82	2.18	2.41	4.94	4.93	3.07	2.38
			100	2.82	2.78	4.02	1.94	2.12	2.15	2.62	2.78	5.78	5.93	3.70	2.91
FS13	51.500	62.250	1	1.82	1.64	2.21	1.16	1.19	1.10	1.20	1.54	3.22	4.16	2.80	1.89
			10	2.50	2.10	2.88	1.39	1.53	1.50	1.64	1.92	3.93	5.25	3.66	2.57
			100	3.12	2.50	3.52	1.59	1.79	1.81	1.98	2.24	4.57	6.30	4.46	3.18
FS14	62.250	65.000	1	1.82	1.64	2.17	1.06	1.09	0.95	0.94	1.28	2.92	4.15	2.93	1.93
			10	2.52	2.13	2.85	1.29	1.38	1.30	1.29	1.60	3.59	5.23	3.85	2.64
			100	3.16	2.58	3.51	1.49	1.60	1.56	1.54	1.85	4.19	6.27	4.71	3.28
FS15	65.000	79.035	1	1.80	1.59	1.32	0.34	0.32	0.31	0.00	0.00	0.09	2.43	3.07	2.24
			10	2.53	2.06	2.06	0.68	0.52	0.66	0.00	0.00	0.46	3.01	3.92	3.13
			100	3.23	2.50	2.83	0.90	0.60	0.86	0.00	0.00	0.80	3.56	4.73	3.99
FS16	79.035	79.564	1	1.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.89
			10	2.46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.08	1.13
			100	3.27	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.13	1.36

Table 7-11 Directional significant wave height (H_s) for fatigue analysis of free spanning pipeline

Table 7-12 presents the directional current velocities for each free span section considered for the fatigue analysis. Note that due to a poor resolution in the current model around the

Finnish archipelago, it was not possible to obtain directional data. Therefore omni-directional data from the model is used in all directions for conservatism.

FS Section	KP Range		Rtn period	Current velocity for fatigue analysis, U_c (m)											
	From	To	yrs	0	30	60	90	120	150	180	210	240	270	300	330
FS1	0.038	0.155	1	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16	0.16
			10	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22
			100	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
FS2	0.155	3.000	1	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12	0.12
			10	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
			100	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26
FS3	3.000	6.000	1	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
			10	0.14	0.14	0.14	0.14	0.14	0.14	0.14	0.14	0.14	0.14	0.14	0.14
			100	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19	0.19
FS4	6.000	8.200	1	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11	0.11
			10	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.17
			100	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22	0.22
FS5	8.200	14.120	1	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
			10	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21
			100	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27
FS6	14.120	19.350	1	0.09	0.17	0.21	0.12	0.06	0.05	0.08	0.16	0.19	0.06	0.03	0.04
			10	0.14	0.21	0.27	0.17	0.09	0.08	0.11	0.20	0.25	0.10	0.06	0.07
			100	0.18	0.24	0.32	0.21	0.12	0.10	0.14	0.24	0.31	0.13	0.08	0.10
FS7	19.350	20.860	1	0.02	0.03	0.14	0.25	0.03	0.01	0.01	0.03	0.13	0.20	0.03	0.01
			10	0.03	0.05	0.18	0.32	0.04	0.03	0.03	0.04	0.17	0.28	0.04	0.03
			100	0.04	0.07	0.22	0.39	0.05	0.03	0.03	0.05	0.21	0.34	0.06	0.04
FS8	20.860	21.028	1	0.05	0.11	0.17	0.07	0.04	0.04	0.06	0.12	0.15	0.08	0.04	0.04
			10	0.09	0.15	0.23	0.11	0.06	0.07	0.09	0.15	0.20	0.11	0.07	0.07
			100	0.12	0.19	0.29	0.14	0.07	0.09	0.11	0.18	0.23	0.14	0.10	0.09
FS9	21.028	26.000	1	0.09	0.15	0.14	0.07	0.05	0.05	0.09	0.15	0.14	0.08	0.05	0.05
			10	0.14	0.21	0.19	0.10	0.08	0.09	0.14	0.19	0.18	0.12	0.08	0.09
			100	0.19	0.26	0.23	0.12	0.10	0.12	0.18	0.23	0.22	0.16	0.11	0.13
FS10	26.000	33.650	1	0.13	0.14	0.06	0.03	0.03	0.04	0.12	0.14	0.07	0.05	0.04	0.07
			10	0.18	0.18	0.10	0.06	0.05	0.07	0.16	0.18	0.11	0.07	0.07	0.10
			100	0.22	0.23	0.13	0.07	0.07	0.09	0.19	0.21	0.14	0.09	0.09	0.14
FS11	33.650	43.700	1	0.03	0.05	0.10	0.07	0.04	0.03	0.03	0.05	0.13	0.11	0.05	0.03
			10	0.05	0.07	0.14	0.10	0.06	0.05	0.05	0.08	0.17	0.15	0.08	0.05
			100	0.07	0.09	0.17	0.12	0.08	0.06	0.06	0.10	0.21	0.18	0.10	0.07
FS12	43.700	51.500	1	0.09	0.14	0.17	0.13	0.09	0.08	0.08	0.23	0.24	0.12	0.08	0.07
			10	0.16	0.20	0.22	0.16	0.12	0.11	0.12	0.33	0.30	0.16	0.12	0.11
			100	0.21	0.26	0.27	0.19	0.15	0.13	0.16	0.43	0.36	0.20	0.15	0.15
FS13	51.500	62.250	1	0.06	0.16	0.21	0.10	0.05	0.05	0.07	0.16	0.20	0.09	0.04	0.04
			10	0.11	0.22	0.25	0.13	0.08	0.08	0.11	0.21	0.26	0.16	0.09	0.09
			100	0.16	0.27	0.29	0.16	0.11	0.10	0.14	0.25	0.31	0.21	0.13	0.14
FS14	62.250	65.000	1	0.04	0.12	0.11	0.04	0.02	0.02	0.04	0.11	0.11	0.04	0.02	0.02
			10	0.08	0.15	0.14	0.06	0.04	0.04	0.06	0.14	0.14	0.06	0.04	0.05
			100	0.11	0.18	0.16	0.08	0.05	0.05	0.08	0.17	0.17	0.09	0.07	0.08
FS15	65.000	79.035	1	0.05	0.04	0.04	0.05	0.05	0.04	0.04	0.04	0.05	0.06	0.05	0.05
			10	0.07	0.06	0.06	0.07	0.07	0.06	0.05	0.05	0.07	0.07	0.07	0.07
			100	0.09	0.08	0.08	0.09	0.10	0.08	0.06	0.06	0.08	0.09	0.09	0.09
FS16	79.035	79.564	1	0.02	0.01	0.02	0.03	0.05	0.06	0.01	0.01	0.01	0.02	0.04	0.05
			10	0.03	0.02	0.03	0.05	0.07	0.09	0.02	0.02	0.02	0.03	0.06	0.08
			100	0.03	0.03	0.04	0.06	0.09	0.12	0.03	0.03	0.03	0.04	0.07	0.10

Table 7-12 Directional current velocity (U_c) for fatigue analysis of free spanning pipeline

7.2.5 Soil data

As explained in section 4.7 of the *Design Basis*, Ref. /34/, the soil data along the surveyed corridor was collected in 2013 and presents a wide variety of soft to firm clays, glacial till, bedrock and sand along the pipeline route corridor. The complete classification is presented in the *Geophysical Survey Report*, Ref. /32/. For the FEED phase, a conservative simplification of the soil data has been summarised in Table 9-1.

Based on the simplified classification, the following soil data shown in Table 7-13 was applied for the fatigue analysis of free spans, with the most conservative results of clay or bedrock chosen where both soils appeared in a free span section.

Soil type	Vertical dynamic stiffness K_v (kN/m/m)	Lateral dynamic stiffness K_L (kN/m/m)	Vertical static stiffness $K_{V,S}$ (kN/m/m)
Clay	2,830	1,935	210
Bedrock	28,110	19,610	3,400
Sand	23,080	17,460	530

Table 7-13 Soil stiffness applicable for fatigue analysis of free spans

7.2.6 S-N curves

The fatigue analyses are performed using the S-N curves defined in DNV-RP-C203, Ref. /2/. Section 2.10 of the recommended practice specifies that for pipelines and risers, and S-N curve of either F1 or F3 is to be used for the weld root (inner diameter) and the D curve is used with a SCF for the weld toe (outer diameter).

To calculate which curve to use for the inner diameter, the eccentricity δ of the pipeline must be determined. The total eccentricity is calculated as an average of the tolerance at the weld cap and weld root incorporating fabrication tolerances.

For the FEED phase, the welding specifications are unknown therefore assumptions are made based on project experience. A total eccentricity of 1.3 mm due to welding has been assumed, and therefore the F1 curve is used in the fatigue analysis for the inner diameter of the free spanning pipeline.

Using this total eccentricity, an SCF can be calculated for the D curve on the outer diameter of the pipeline using Equation A.2 of DNV-OS-F101, Ref. /1/.

The result is an SCF of 1.273 to be applied with the D curve for the fatigue analysis of the outer diameter of the free spanning pipeline.

The F1 S-N curve for a weld is to be applied in air for the temporary air-filled and operation phase. For all phases, the D curve is to be applied with seawater and cathodic protection.

The characteristic parameters for the applicable S-N curves can be found in Table 2-4 of DNV-RP-C203, Ref. /2/ and are listed in Table 7-14.

S-N curve	$\log \bar{a}_1$ $m_1 = 3.0$	$\log \bar{a}_2$ $m_2 = 5.0$	SCF	Thickness exponent, k
F1 (air)	11.699	14.832	1.000	0.00
D (seawater CP)	11.764	15.606	1.273	0.15

Table 7-14 Characteristic parameters for the applicable S-N curves

7.2.7 Methodology

Free spans are defined as unsupported pipeline sections subject to dynamic loads from waves and currents. The resulting oscillations of the pipeline can result in failure of pipelines due to excessive yielding and fatigue. Depending on the current velocity and span length, the oscillations can be in-line with the flow direction, or cross-flow, i.e. transverse to the flow direction. The oscillations are termed Vortex Induced Vibrations (VIV) and have severe implications on the fatigue life of the pipeline. Following DNV-RP-F105, Ref. /5/, a fatigue assessment has to be performed in order to ensure sufficient capacity of the pipeline to resist fatigue failure.

The results given in this assessment consider the pipeline in its temporary phase and in its operational phase. The free span lengths have been assessed for a minimum gap height of 0.3 times the outer pipeline diameter including coating based on studies from *Hydrodynamics around cylindrical structures*, Ref. /25/, where it states that vortex shedding is suppressed for a gap-ratio less than 0.3D.

The fatigue assessment is performed according to the following DNV codes listed below:

- DNV-OS-F101, Submarine Pipeline Systems, Ref. /1/
- DNV-RP-F105, Free Spanning Pipelines, Ref. /5/
- DNV-RP-C203, Fatigue Design of Offshore Steel Structures, Ref. /2/
- DNV-RP-C205, Environmental conditions and environmental loads, Ref. /3/

In accordance with Ref. /5/ Sec. 1.6 and Sec. 1.7, the free span in this assessment is classified as an isolated single span (i.e. being independent of neighbouring span behaviour) having one single mode response. Furthermore, the span is assumed stationary (i.e. main span characteristics, gap height and span length remain the same).

Higher modes are activated in isolated pipeline free span vibrations when a combination of large span length, high axial force and high environmental loads are experienced. In case several vibration modes (in the same direction) may become active at a given flow velocity, multi-mode response shall be considered.

When two or more free spans are located adjacent to each other, the static and dynamic behaviour of each span is affected by the presence of the neighbouring spans. This is where multi-spanning behaviour should be considered, alongside multi-mode behaviour where the higher modes may be producing the more critical results.

Given the generic nature of this free span study, the results will only be performed for the isolated single span and the coupling effect of adjacent free spans will be a consideration for the detailed engineering phase. The consequence is that a few smaller spans than the calculated allowable span length may be subject to re-assessment in detailed engineering and subsequently possible free span rectification. However, the remaining conservatisms in the analysis compensate for these effects and the overall quantity of free span rectification due to fatigue damage is expected to decrease in the detailing engineering phase, despite the coupling effects of free spans.

FatFree software
The calculation of the allowable free span lengths is carried out in using DNV's "FatFree" program, version 10.6c. FatFree is in direct compliance with Ref. /5/.

The program is intended to be a tool used in the process of analysing and designing free spans of submarine pipelines in connection to computing the pipeline fatigue life T_{life}. The pipeline fatigue life in FatFree is computed with respect to combined direct current and wave action together with in-line and cross-flow vortex induced vibrations. Waves and current input for FatFree are given by directional long term distributions.

In FatFree, free span analysis scenarios are characterised by input parameters such as water depth, span length, span gap height, soil conditions, pipe heading, safety class, etc. together with the pipeline specifics such as material, geometry, S-N curves, etc.

The eigenfrequencies, computed by applying simplified beam theory expressions, and associated mode shapes for the pipeline span are established. Following this, the fatigue life due to in-line and cross-flow induced vibrations is calculated. The calculations include force and response model evaluations.

In the analyses only small to moderately long spans are considered. Typically, spans lengths are considered to be very long when the ratio between span length and outer pipe diameter exceeds 140, Ref. /5/. Long span lengths are considered to require rectification in this assessment.

Therefore, based on the FatFree software computation, the fatigue life T_{life} is assessed against the time of exposure, T_{exp}, to determine an allowable span length.

Based on this and combined with the FatFree "span-run" functionality, a free span fatigue screening is conducted. The "span-run" functionality is a batch where many different span cases can be set up and analysed making it possible to conduct a screening. The result of the screening is then an identification of the possible sub-ranges where the computed fatigue damage is less than or equal to the allowable fatigue damage.

Table 7-15 below highlights the key variables between pipeline sections that are applied in the screening assessment to determine the allowable span length for each pipeline section along the entire route.

ID	KP Range		Min water depth (m)	Heading (deg)	Concrete thickness (mm)	Residual lay tension (kN)	Soil type	Applied metocean data point
	To	From						
FS1	0.038	0.155	-5.0	138.3	55	253	Clay	1
FS2	0.155	3.000	-8.7	138.3	55	253	Clay	2
FS3	3.000	6.000	-14.4	138.3	55	253	Clay	5
FS4	6.000	8.200	-17.6	138.3	55	253	Clay	6
FS5_1	8.200	13.200	-17.0	160.2	55	253	Clay	7
FS5_2	13.200	14.120	-23.1	160.2	55	253	Clay	8
FS6	14.120	19.350	-24.9	222.8	55	253	Bedrock	8
FS7_1	19.350	19.812	-16.2	183.0	80	609	Bedrock	9
FS7_2	19.812	20.860	-23.5	183.0	80	609	Bedrock	9
FS8	20.860	21.028	-17.2	183.0	80	609	Bedrock	10
FS9_1	21.028	22.400	-29.6	183.0	80	654	Bedrock	11
FS9_2	22.400	24.700	-38.2	183.0	80	654	Bedrock	11
FS9_3	24.700	25.400	-27.9	183.0	80	654	Bedrock	11
FS9_4	25.400	26.000	-40.4	183.0	80	654	Bedrock	11
FS10	26.000	33.650	-50.2	183.0	45	333	Clay	12
FS11	33.650	43.700	-56.2	165.5	45	350	Clay	13
FS12	43.700	51.500	-54.7	180.0	45	350	Clay	14
FS13	51.500	62.250	-56.3	171.5	45	350	Clay	16
FS14	62.250	65.000	-73.1	189.4	45	428	Clay	17
FS15_1	65.000	73.300	-34.9	176.6	45	333	Clay	18
FS15_2	73.300	79.035	-11.7	152.5	45	143	Sand	18
FS16	79.035	79.564	-5.0	152.5	45	143	Sand	19

Table 7-15 Summarised data input of key variables for each free span section

7.2.8 Results

A summary of the results in relation to the allowable span length is given in Table 7-16. The results are sub-divided into numerous sections of the pipeline, where the water depth, heading, lay tension and pipeline properties change. The division of sections roughly follows the division specified in Table 7-2 of the on-bottom stability analysis, with additional sections based on the varying water depth.

FS Section	KP Range		Min water depth (m)	Allowable span length (m)	
	To	From		Empty	Operation
FS1	0.038	0.155	-5.0	58	36
FS2	0.155	3.000	-8.7	60	36
FS3	3.000	6.000	-14.4	66	40
FS4	6.000	8.200	-17.6	64	35
FS5_1	8.200	13.200	-17.0	35	26
FS5_2	13.200	14.120	-23.1	64	29
FS6	14.120	19.350	-24.9	68	36
FS7_1	19.350	19.812	-16.2	35	21
FS7_2	19.812	20.860	-23.5	46	26
FS8	20.860	21.028	-17.2	33	20
FS9_1	21.028	22.400	-29.6	46	28
FS9_2	22.400	24.700	-38.2	65	35
FS9_3	24.700	25.400	-27.9	44	27
FS9_4	25.400	26.000	-40.4	70	36
FS10	26.000	33.650	-50.2	70	41
FS11	33.650	43.700	-56.2	70	42
FS12	43.700	51.500	-54.7	70	42
FS13	51.500	62.250	-56.3	70	43
FS14	62.250	65.000	-73.1	70	54
FS15_1	65.000	73.300	-34.9	70	39
FS15_2	73.300	79.035	-11.7	38	22
FS16	79.035	79.564	-5.0	46	25

Table 7-16 Summary of allowable free span lengths for fatigue criteria

All spans are considered as isolated, single spans in this analysis. The coupling effect of adjacent free spans will be a consideration for the detailed engineering phase and therefore the allowable span length in Table 7-16 does not incorporate the changes in frequency and amplitude resulting from coupling. The consequence is that a few smaller spans than the stated allowable span length may be subject to free span rectification. However, the conservatisms of the analysis through the safety factors and fatigue damage distribution compensate for these effects and the overall quantity of free span rectification due to fatigue damage is expected to decrease in the detailing engineering phase, despite the coupling effects of free spans.

It should also be noted that the contribution to the fatigue damage for the water-filled period is insignificant and is therefore not presented in the FEED phase. The water-filled period, particularly during the system pressure test, is more critical for the local buckling criteria and this phase is addressed in section 7.3.

7.2.9 Recommendations

The allowable span length specified in the results section provides a suitable screening criterion for the acceptable free spanning of the pipeline. However in the detailed engineering phase, the locations identified as needing rectification due to the free span analysis should be reassessed using the location specific criteria and incorporating the latest geotechnical survey data. Location specific data should include the operational functional data based on pressure and temperature profiles along the offshore pipeline route. The effects of coupling from adjacent spans should also be investigated.

The presence of boulders is observed in low to high density boulder fields in the *Geophysical alignment sheets*, Ref. /32/, throughout the route and particularly at the landfall approach in Estonia. Pipeline interaction with boulders will be a key consideration for the detailed engineering phase. Despite the planned removal of boulders from the pipelay corridor, in the event that the pipeline is installed on a boulder, the resulting interacting free spans should be considered and analysed. This analysis performed in advance of pipeline installation will help determine the size of boulders that should be removed which will help reduce the number of offshore hours removing boulders.

7.3 Local buckling analysis

In compliance with DNV-RP-F105, Ref. /5/, Section 2.5, the ULS criterion for free spans shall always be checked. The ULS criterion states that the combined equivalent (von Mises) stress from static and dynamic loads upon the pipeline in the in-line and cross-flow directions shall not exceed the yield stress of the material, applying both the material factor α_U and usage factor η. In compliance with DNV-OS-F101, Sec. 5 F202 the stress is obliged to fulfil the requirement:

$$\sigma_e \leq \eta \cdot f_y$$

where

σ_e	equivalent stress (von Mises)
η	usage factor
f_y	yield stress

For the Balticconnector pipeline, the cross-flow direction is governing, as the functional loads applied on a free spanning pipeline induce high stresses at the span shoulders. The contribution from the environmental load on the pipeline remains insignificant for the ULS.

7.3.1 Methodology

Using the in-house bottom roughness tool, Goliat, the axial force and vertical bending moments during the operational phase in the pipeline resting on the seabed is determined at one metre intervals. The forces and moments can then be used to determine the equivalent stress and hence the load controlled location buckling utilisation, as stated in Sec 5 D600 of DNV-OS-F101, Ref. /1/.

For the local buckling check, exceeding a utilisation ratio of 0.9 is defined as the threshold for seabed intervention. A threshold of 0.9 is chosen in order to have some additional safety to handle uncertainties like installation tolerances and lay tension.

Locations with utilisations ratios exceeding 0.9 are separately evaluated to determine if seabed intervention is required based on the calculated local buckling utilisation ratios and the local condition of pipeline and seabed. Where seabed intervention is required in order to lower the utilisation ratio, either dredging/blasting of peaks or installations of rock supports are specified.

The evaluation is based on the effectiveness of the method, i.e. the reduction of the utilisation ratio, the robustness of the selected seabed intervention and also the cost comparison of volumes for dredging versus SRI.

Rock supports are classified as either pre-lay or post-lay rock supports dependent on whether the support is to be installed prior to, or after the installation of the Balticconnector pipeline. Post-lay supports are preferred if possible, as they can be installed based on the as-laid position of the pipeline, and therefore the cost of post-lay supports is less compared to pre-lay supports.

It is necessary to evaluate if post-lay supports can be used to lower high local buckling utilisations in the phases following installation (air-filled), i.e. water-filling, pressure testing and operation. This is performed by assessing the air-filled configuration of the pipeline at the location where seabed intervention is required. As the free span gap height between Bottom-of-Pipe (BOP) and seabed decreases during flooding, pressure testing and operational phases it is often adequate to dump rock to a level below BOP in air-filled condition to mitigate high pipeline utilisation in the aforementioned phases. In case a post-lay support is found not to reduce the pipeline utilisation sufficiently, a higher support is required to change the configuration of the pipeline in the air-filled condition. This support will have to be installed prior to the pipeline and hence defined as a pre-lay support.

Locations where seabed rectification is performed are to be re-analysed during the detailed engineering phase, when accurate geotechnical data for each specific location should be available.

In areas where seabed intervention has been found to be particular challenging and/or costly, re-routing options have been suggested as alternative solutions with the use of counteracts if required.

Geotechnical calculations for pre-lay supports are addressed in Appendix VIII.

7.3.2 Results

24 locations with a local buckling criterion (LBC) utilisation (UT) ratio of more than 0.9 have been identified along the route of the Balticconnector pipeline. These are presented in Table 7-18. Each location has been separately investigated in order to determine the most optimal method of seabed intervention for lowering the LBC UT to below unity. It is found that all necessary seabed intervention to be performed is located in the northern part of the Gulf of Finland between KP 12- 26.

Each location in Table 7-18 is classified according to the complexity of the seabed intervention required to ensure pipeline integrity. The classifications are listed in Table 7-17.

Complexity level	Description
Low complexity	Cost effective seabed design can be carried out by means of rock installation
Medium complexity	Feasible seabed intervention solutions are present but design should be subjected to further investigation to identify potential cost savings.
High complexity	Further mitigation actions to be analysed and evaluated to find most favourable method for ensuring pipeline integrity during the design life.

Table 7-17 Seabed intervention classification terminology

18 locations are defined as low complexity, 3 as medium complexity and 3 as high complexity. It is noted that the uncertainty of estimated rock installation and removal volumes are associated with the complexity of the design. Estimated volumes are presented in Table 11-1 and sectioned according to design complexity level.

Of the two types of pre-lay and post-lay rock installation, pre-lay rock installation is associated with the greatest level of uncertainties. This is because post-lay rock installation can be installed relative to the as-laid pipeline while pre-lay rock installation design has to include design tolerances. Typically also the line load carried by pre-lay supports is greater than the load carried by post-lay rock supports. This is mainly because the pre-lay support will carry the pipeline from installation, i.e. effectively changing the configuration of the pipeline compared to a free spanning pipeline. Post-lay support first becomes effective in subsequent phases, i.e. water-filling, pressure testing and operation.

For this reason location #6, #15 and #21 have been chosen as representative locations used for the overall estimation of required rock volumes presented in Table 11-1. The geotechnical results and detailed design for one of these locations is found in Appendix VIII.

No.	KP @ max UT	MSL	LBC UT	Method of intervention[1]			Design complexity	Soil Type[3]	Comments
				Pre-lay SRI	Post-lay SRI	Soil/rock removal			
[-]	[km]	[m]	[-]	[-]	[-]	[-]	[-]		[-]
1	12.242	-19.6	1.12	X	X		Low	B	Potentially prone to upheaval buckling to be mitigated by post-lay SRI
2	13.919	-26.5	1.13	X			Low	B/FC	
3	16.193	-24.9	0.93	X			Low	B/FC/GT	
4	16.981	-28.3	1.08		X		Low	B/BL	
5	17.426	-26.5	1.58	X		X	Low	B	Potentially removal of soil/rock might be omitted – to be further investigated
6	17.840	-31.5	1.32	X			Low	B/SC	
7	18.248	-26.5	2.17	X		X	High	B/SC	Further mitigation option to be evaluated[5]
8	18.490	-34.0	0.97		X		Low	B/SC	
9	18.729	-26.5	1.71	X[2]		X	Medium	B	Further mitigation option to be evaluated[6]
10	18.795	-26.5	1.03	X[2]			Low	B/SC	
11	18.982	-25.8	1.40	X		X	Low	B	Potentially removal of soil/rock might be omitted – to be further investigated
12	19.364	-24.3	1.90	X		X	High	B/PC	Further mitigation option to be evaluated[6]
13	19.735	-20.9	1.12		X		Low	B	
14	19.894	-27.6	0.90		X		Low	B/FC	
15	20.263	-23.6	1.45	X			Low	B	
16	20.915	-17.2	1.76			X	Medium	B/SG	Removal of rock required[6]
17	21.193	-29.6	1.03	X			Low	B/SG	
18[4]	22.288	-31.7	1.33	X			Low	B/SG	
19[4]	22.371	-36.0	1.66	X		X	Medium	B/SG/FC	Further mitigation option to be evaluated[5]
20	24.277	-39.0	1.79			X	Low	B/S/GC	Removal of rock required
21	24.391	-41.0	1.05	X			Low	SC/S/GC	
22	24.753	-35.8	0.95	X			Low	B/SG	High accuracy pre-lay installation i.e. -0/+0.2 m
23	25.104	-28.4	1.21	X			Low	B/SG	
24	25.324	-28.0	2.02	X		X	High	B	Further mitigation option to be evaluated[6]

Notes
1) Pre-lay refers to installation prior to the installation of the pipeline while post-lay refers to installation prior to water-filling.
2) SRI intervention to be performed will influence both locations
3) Soil types close to location of high utilisation
B = Bedrock, SC = Soft Clay, PC = Partly silt/ fine sand Clay, FC = Firm Clay, GT= Glacial Till, SG =Sand/Gravel, BL=Boulder, GC = Gravel and Cobbles, S = Sand
4) Outside survey corridor on geophysical survey, Ref. /32/ (Doc. ALIGN013)
5) Recommended mitigation action includes re-routing potentially by means of counteracts – to be further investigated
6) Recommended mitigation action includes blasting – to be further investigated

Table 7-18 High local buckling utilisation locations

The locations of each pipeline section that exceeds the 0.9 utilisation ratio for the load controlled local buckling criterion are visualised within the survey corridor in Figure 7-4, Figure 7-5 and Figure 7-6.

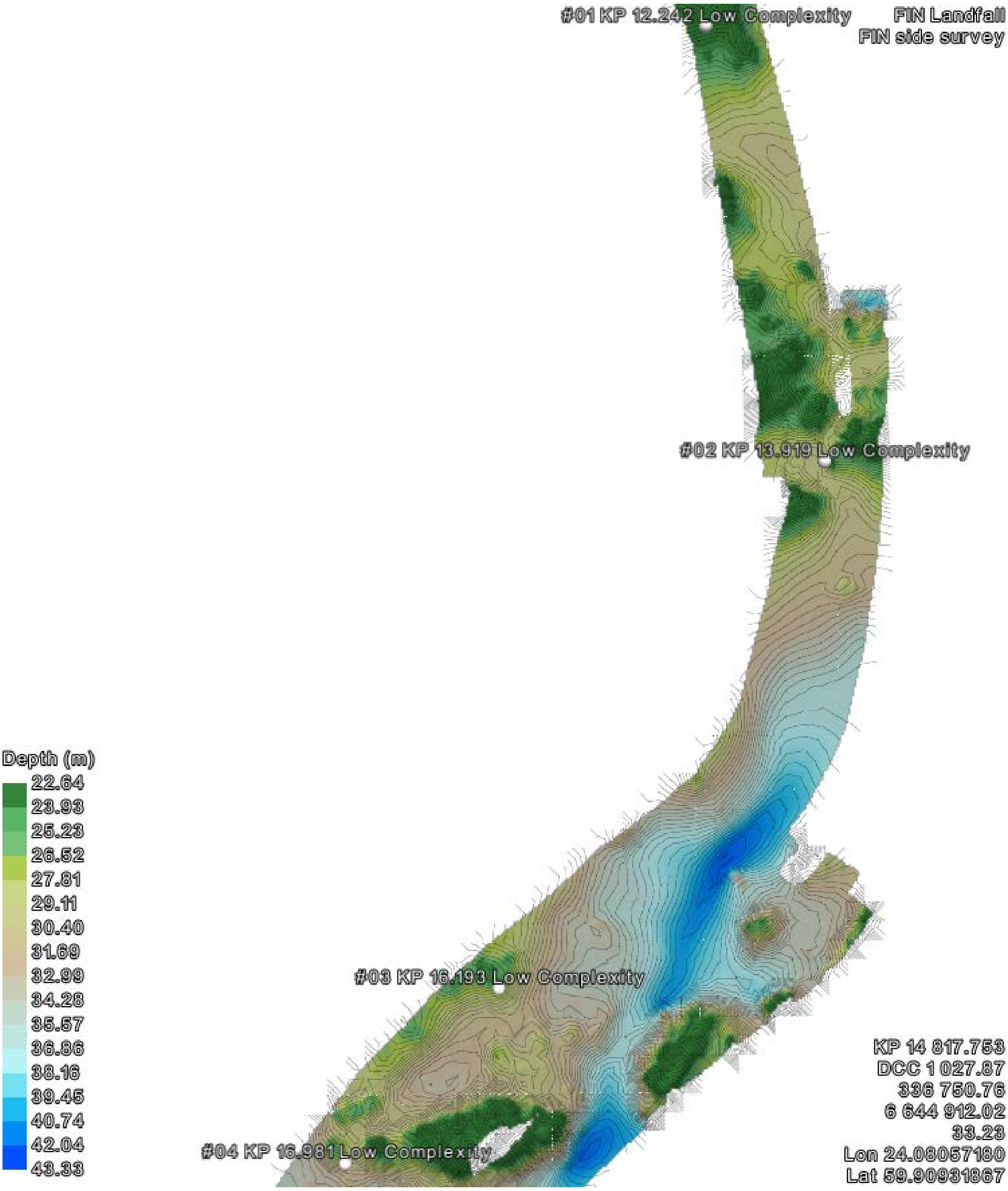


Figure 7-4 Location #01-04 exceeding the 0.9 local buckling utilisation ratio

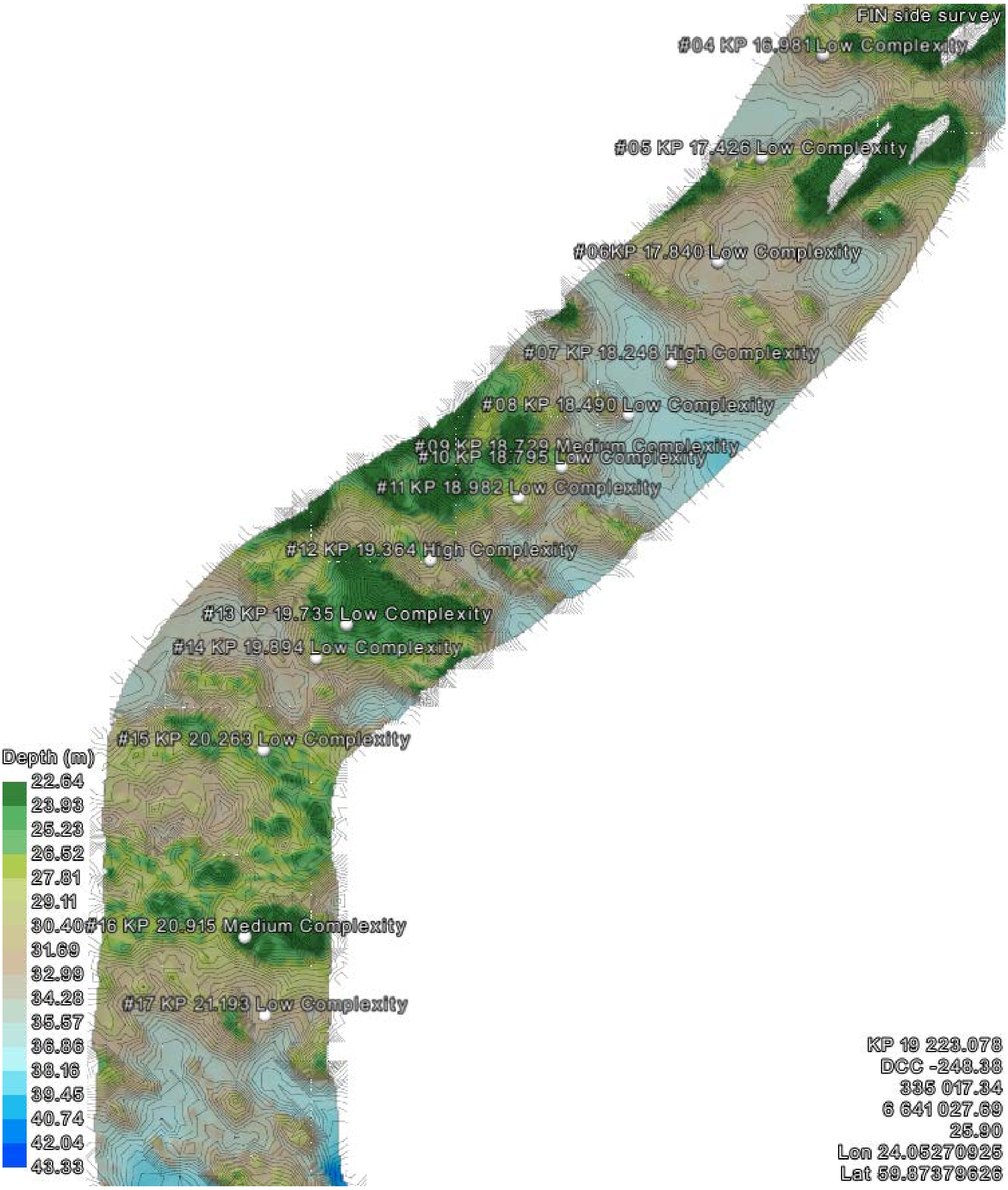


Figure 7-5 Location #04-17 exceeding the 0.9 local buckling utilisation ratio

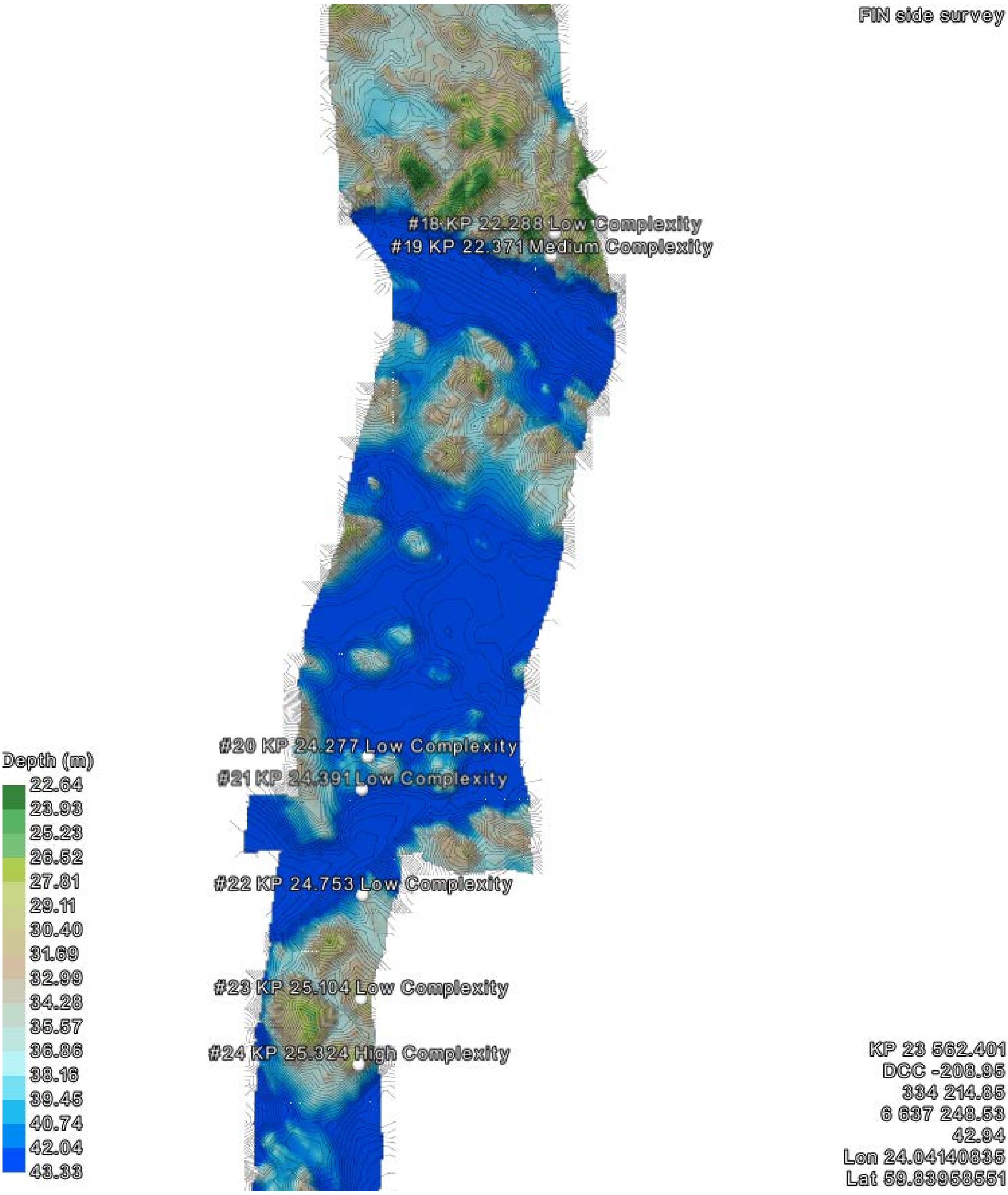


Figure 7-6 Location #18-24 exceeding the 0.9 local buckling utilisation ratio

7.3.3 High Complexity Locations

<u>Location #07</u>

At location #07 the pipeline crosses an 8 m high and 100 m wide bedrock outcrop at KP 18.248. The result is a 68 m and 59 m free span of the pipeline at the sides of the outcrop. In order to narrowly avoid additional bedrock outcrops close to KP 18.0 and KP 18.5, the pipeline route was designed to rest on the outcrop at KP 18.248. This was a result of the limitations to the pipeline lay radius due to curve stability. As a result, a localised solution will have to be provided at this location.

<u>Location #12</u>
At location #12, the pipeline spans across an 8 m bedrock outcrop, 30 m from the edge of the ridge. The result is a 78 m and 49 m free span either side of the bedrock outcrop. When investigating the best routing option, the ridge was unavoidable due to lay radius limitations and the need to thread the pipeline route between several other bedrock outcrops around KP 19.0.

<u>Location #24</u>
At location #24, the pipeline descends down a steep bedrock vertical of 10 m resulting in a 69 m free span. This free span is coupled with a 41 m and 37 m free span preceding the drop with short shoulder lengths on the bedrock resulting in a system highly susceptible to fatigue damage. An adjacent valley between the bedrock outcrops is shown in Figure 7-9, however a sharp bend would be required to lay the pipeline along that heading. The benefit of routing the pipeline over this steep decline is the access to a straight, flat clay seabed for the subsequent kilometre of the pipeline route.

The bedrock outcrop, route heading and vertical profile at locations #07, #12 and #24 can be visualised in Figure 7-7, Figure 7-8 and Figure 7-9 respectively.

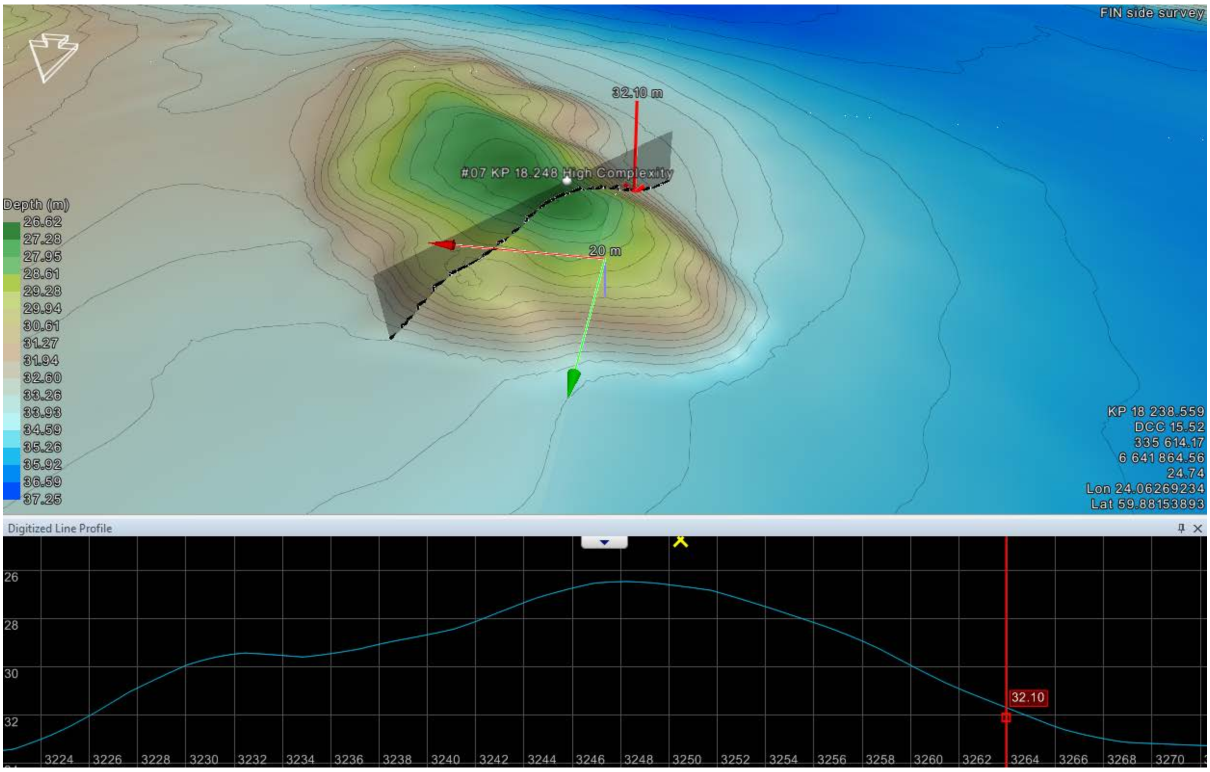


Figure 7-7 Navimodel visualisation of seabed including vertical profile at location #07 – KP 18.248

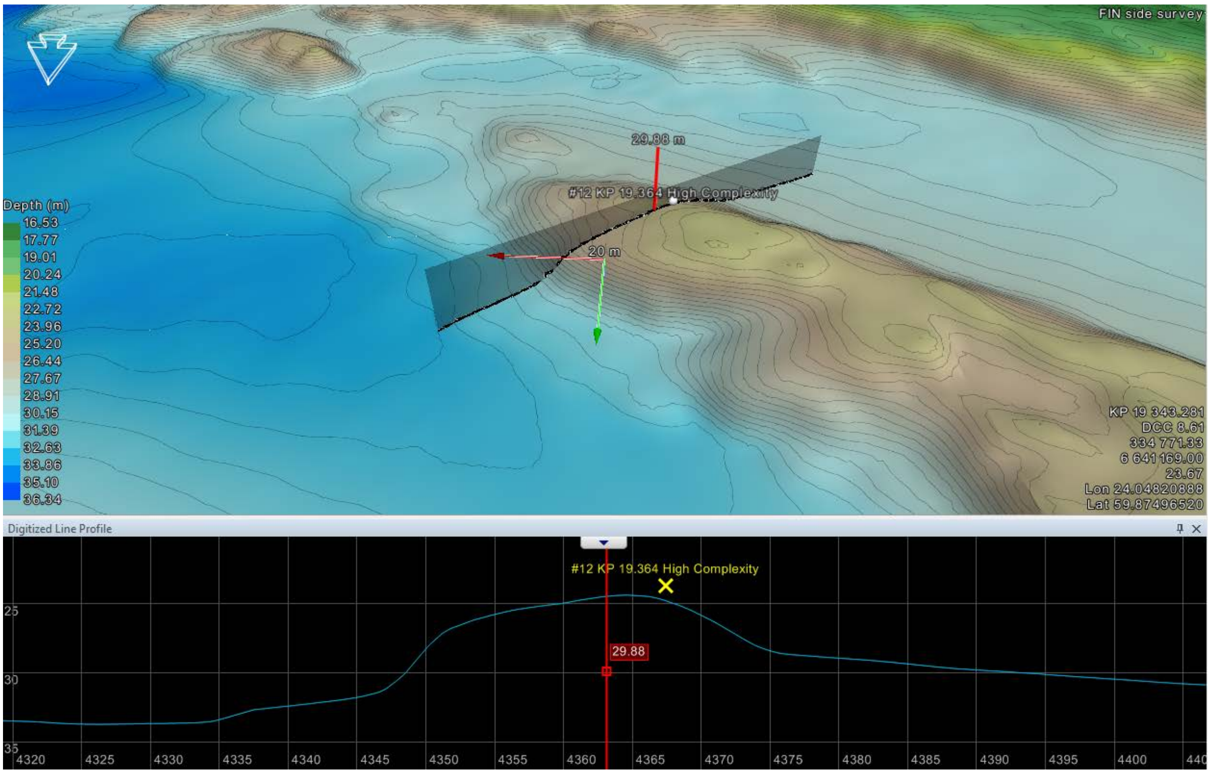


Figure 7-8 Navimodel visualisation of seabed including vertical profile at location #12 – KP 19.364

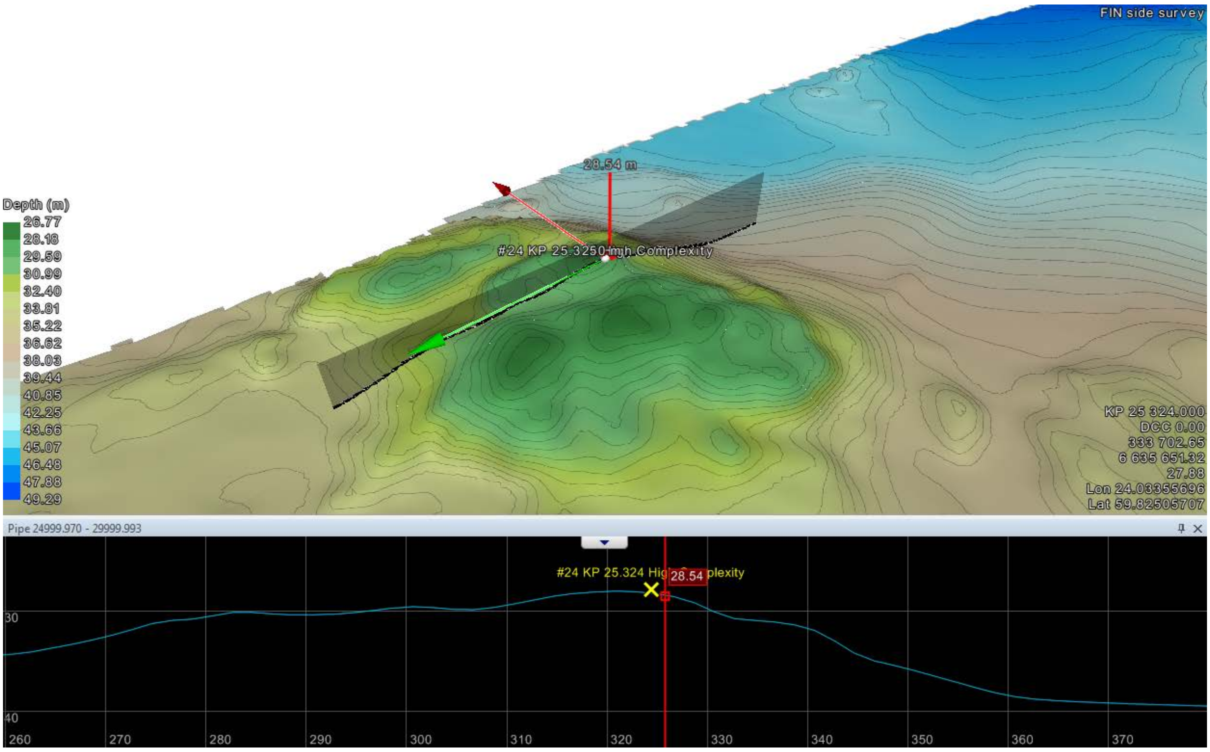


Figure 7-9 Navimodel visualisation of seabed including vertical profile at location #24 – KP 25.324

7.3.4 Medium Complexity Locations

Location #09

At location #09, the pipeline ascends an 8 m bedrock incline which results in a 67 m free span followed by a 59 m sagging free span between two bedrock ridges. The roughness of the seabed between KP 18.0 and KP 20.0 will result in the need for seabed intervention of any route chosen within the survey corridor.

Location #16

At location #16, the pipeline spans over a 10 m high bedrock peak, resulting in one of the largest free spans along the offshore pipeline route of 92 m in length. By routing the pipeline across this bedrock peak, the route gains access to a flatter section of clay seabed located to the east of the survey corridor to reduce the number of seabed intervention locations.

Location #19

At location #19, the pipeline route catches the edge of a bedrock outcrop resulting in a span of 65 m and 46 m in length either side. Investigations into the allowable lay radius of the pipeline at this location with the pipelay contractor may result in a re-routing solution as a lateral deviation of 10-20 m may remove the need for seabed intervention at this location.

The bedrock outcrop, route heading and vertical profile at locations #09, #16 and #19 can be visualised in Figure 7-10, Figure 7-11 and Figure 7-12 respectively.

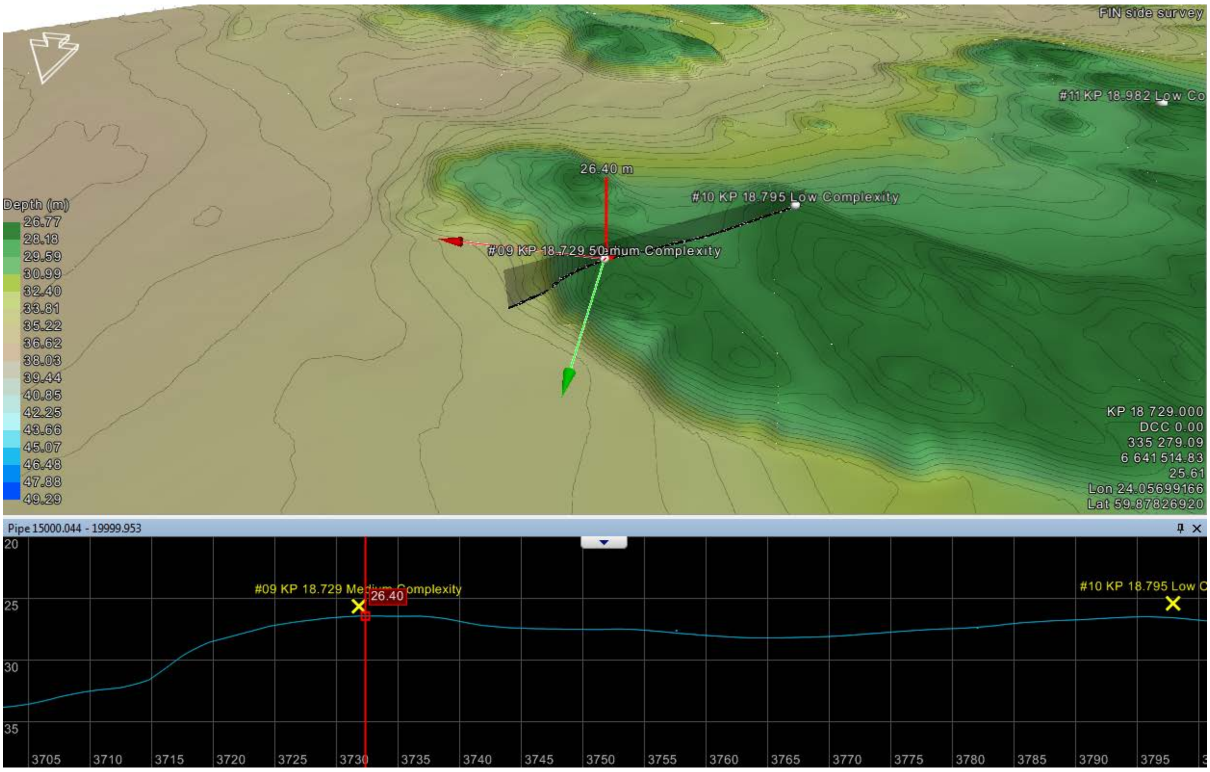


Figure 7-10 Navimodel visualisation of seabed including vertical profile at location #09 – KP 18.729

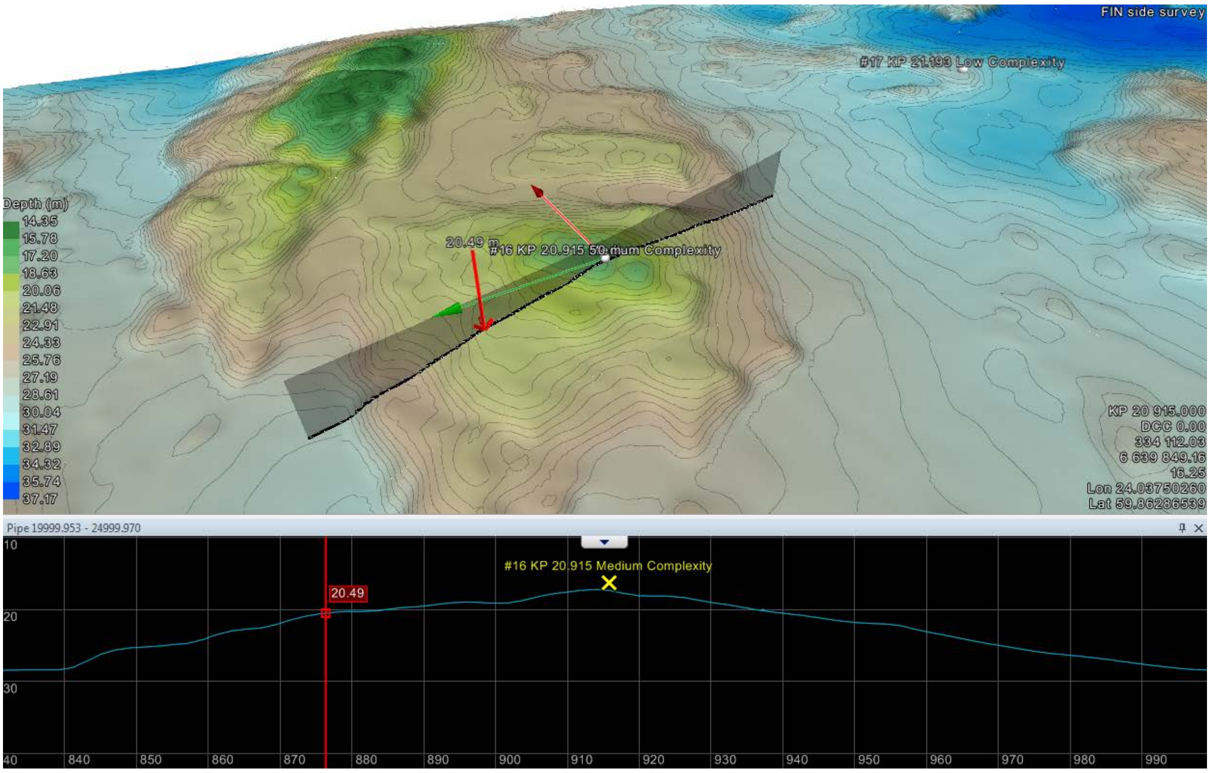


Figure 7-11 Navimodel visualisation of seabed including vertical profile at location #16 – KP 20.915

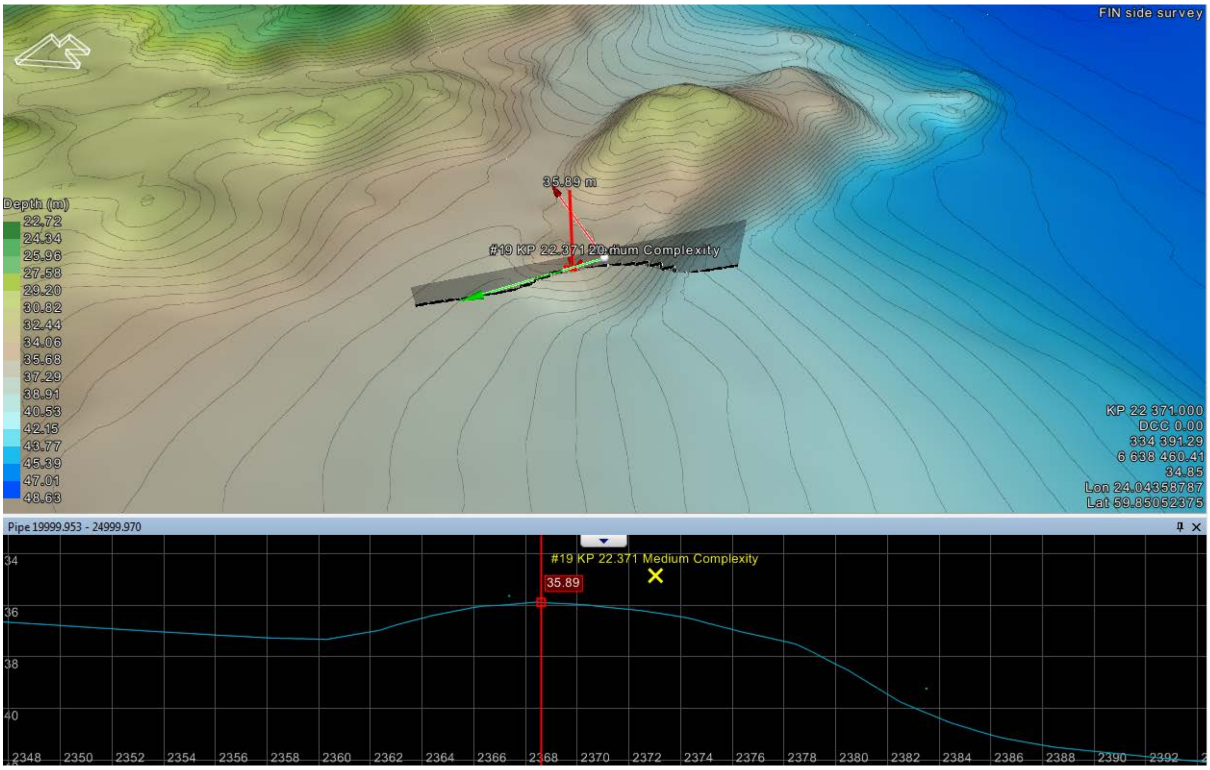


Figure 7-12 Navimodel visualisation of seabed including vertical profile at location #19 – KP 22.371

7.4 Bottom roughness assessment

The bottom roughness analysis is an FE analysis performed on ANSYS and presents the pipeline and seabed profiles along the entire route for the operational condition. The calculations are performed using the concrete thickness input from the on-bottom stability analysis (section 7.1) and the residual lay tension based on the static installation analysis (section 11). All other input data matches that shown in the free span analysis in section 7.2.

7.4.1 Results

The results for the operational pipeline provide the most conservative results for the bottom roughness analysis, although consideration to the empty, flooded and system pressure test profiles should be given in the detailed engineering phase. The results for the operational phase are to be conservatively used for the fatigue analysis of the empty phase to determine the requirement for pre-lay seabed intervention.

Table 7-19 shows the total number of spans modelled in the bottom roughness analysis.

Number of spans in operational phase										
		Span height greater than (m)								
		0	0.01	0.05	0.1	0.5	1.0	2.0	3.0	4.0
Span length interval greater than (m)	0	2,781	1,589	583	385	124	58	16	3	0
	20	311	311	304	269	120	57	16	3	0
	30	137	137	136	134	98	52	16	3	0
	40	66	66	66	66	59	37	14	3	0
	50	32	32	32	32	31	20	8	3	0
	60	17	17	17	17	17	12	6	2	0
	80	4	4	4	4	4	3	1	0	0
	100	0	0	0	0	0	0	0	0	0

Table 7-19 Summary of number of free spans along entire route

The majority of spans in Table 7-19 are still of little significance, or specifically that the span length and span height is too small for the pipeline to be subject to significant fatigue loads or bending moments.

For the FEED phase, all the spans can be divided into two categories:

- Acceptable
- Free span rectification required

In the detailed engineering phase, a third category would be included which defines that a span is in need of further assessment. In these cases, it is believed an engineering solution is available by applying location specific parameters to the individual span so that it meets the local buckling and fatigue design criteria.

Based on the pipeline profile and resulting free spans from the bottom roughness assessment, a summary of the spans requiring rectification is shown in Table 7-20.

FS Section	KP Range		Number of spans requiring pre-lay rectification		Number of spans requiring post-lay rectification		
	To	From	Fatigue	LBC	Fatigue	LBC	GB
FS1	0.038	0.155	-	-	-	-	-
FS2	0.155	3.000	2	-	2	-	-
FS3	3.000	6.000	1	-	3	-	-
FS4	6.000	8.200	-	-	-	-	1
FS5_1	8.200	13.200	5	-	6	1	11
FS5_2	13.200	14.120	2	2	6	-	8
FS6	14.120	19.350	1	14	7	3	21
FS7_1	19.350	19.812	5	2	7	2	7
FS7_2	19.812	20.860	4	5	8	3	11
FS8	20.860	21.028	2	4	3	-	1
FS9_1	21.028	22.400	2	3	2	-	3
FS9_2	22.400	24.700	-	7	2	-	3
FS9_3	24.700	25.400	2	16	4	-	9
FS9_4	25.400	26.000	-	-	1	-	-
FS10	26.000	33.650	-	-	-	-	-
FS11	33.650	43.700	-	-	-	-	1
FS12	43.700	51.500	-	-	3	-	10
FS13	51.500	62.250	-	-	-	-	4
FS14	62.250	65.000	-	-	-	-	2
FS15_1	65.000	73.300	-	-	-	-	-
FS15_2	73.300	79.035	-	-	-	-	-
FS16	79.035	79.564	-	-	-	-	-
		Total	**28**	**53**	**54**	**9**	**92**
		Accumulated total	**70**		**56**		

Note:
1) The accumulated total includes overlapping spans between design criteria, i.e. if one span requires rectification due to both fatigue and local buckling design criteria, it is only considered to be one span in the accumulated total. The post-lay accumulated total incorporates spans that have already been rectified by pre-lay activities.

Table 7-20 Total number of spans requiring rectification

Note that the start and end of the bottom roughness model is from the entry and exit points of the trenched landfall design, as explained in section 10.

7.4.2 Conclusions

The bottom roughness analysis has shown the need for pre-lay seabed intervention for a total of 70 free spans and post-lay seabed intervention for a total of 56 free spans due to the fatigue design criteria, local buckling design criteria or global buckling design criteria. Based on this collected data, the seabed intervention required to mitigate the stress or fatigue in the pipeline will be estimated as a rock volume or blasting/excavation volume to determine an overall cost estimate for these offshore activities.

By assessing in detail each free span location identified in Table 7-20 that requires seabed intervention in the following detailed engineering design phase, the quantity of seabed intervention can be reduced.

7.4.3 Recommendations

The bottom roughness analysis should be revisited in the detailed engineering phase and should include the following inputs:

- Detailed geotechnical data along the final route based on the 2016 geotechnical and acoustic survey data
- Worst case temperature profiles along the route for bi-directional flow
- Known exposure times of empty, flooded, system pressure test and operational phases

Once the pipeline is installed, its profile along the seabed should also be monitored regularly to ensure that scour from environmental loads does not cause any increase in free span lengths that may decrease the fatigue life of the pipeline or subject it to larger stresses. From data collected from the Nord Stream pipeline, it is known that scouring is a possibility along the route in the soft clay locations.

After the as-installed survey and the preliminary surveys once the pipeline is in operation, monitoring can be limited to every few years if no significant scour around free spans is shown.

7.5 Crossing design

The Balticconnector offshore pipeline crosses a number of subsea cables and the two exposed Nord Stream pipelines. Crossing designs have been carried out for the crossings of the two exposed Nord Stream Pipelines and one generic cable design – see *Crossing design drawings*, Refs. /41/ to /46/.

At both crossings locations of the Nord Stream pipelines, the Balticconnector offshore pipeline is resting on soft clay. For this particular reason, the crossings are carried out as continuous carpet designs. A carpet design is less sensitive to dynamic installation effects than bridge designs where pre-lay supports are installed on both sides of the crossings, similar to piers on a bridge. Furthermore, the line load from the crossing pipeline is reduced using a carpet design as the load is distributed over an increased length compared to a bridge design. This makes a carpet design particularly beneficial to use for the crossing of exposed products on soft soil conditions. An example of a bridge and carpet design can be seen in Figure 7-13 and Figure 7-14, respectively.

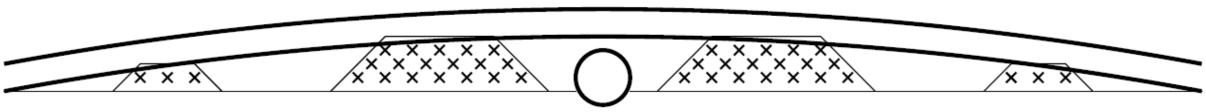


Figure 7-13 Bridge Design

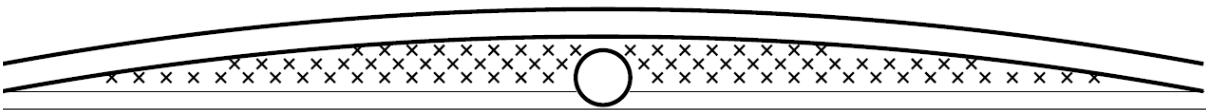


Figure 7-14 Carpet Design

From the Balticconnector seabed survey data, Ref. /32/, the as-laid position for Nord Stream 1 and 2 pipelines is observed at the future crossing locations. The survey data has been visualised in Figure 7-15

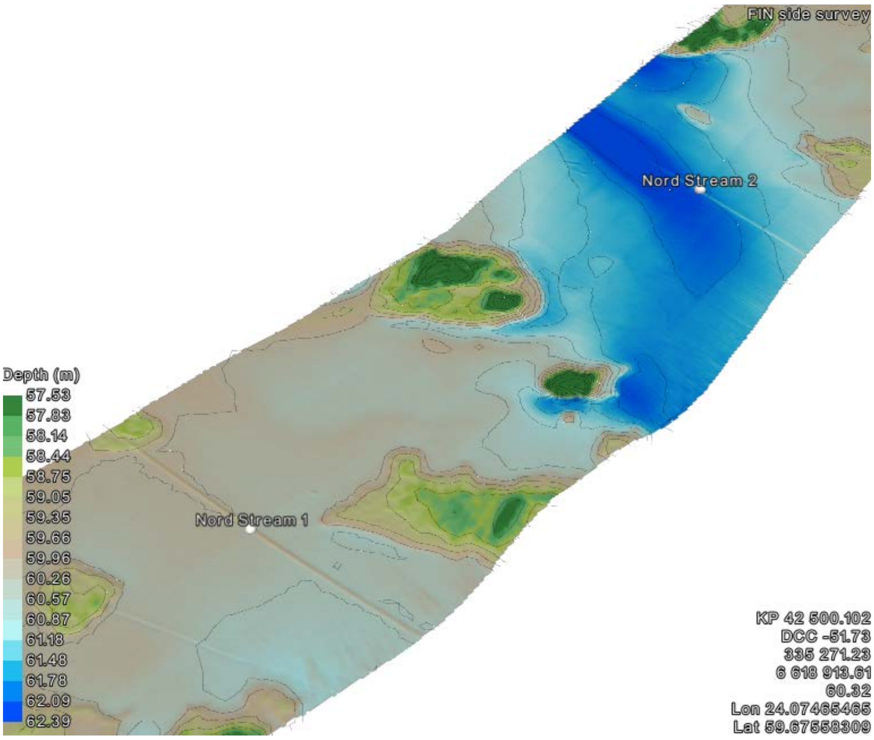


Figure 7-15 Bathymetric survey, Ref. /32/ - Nord Stream 1 and 2

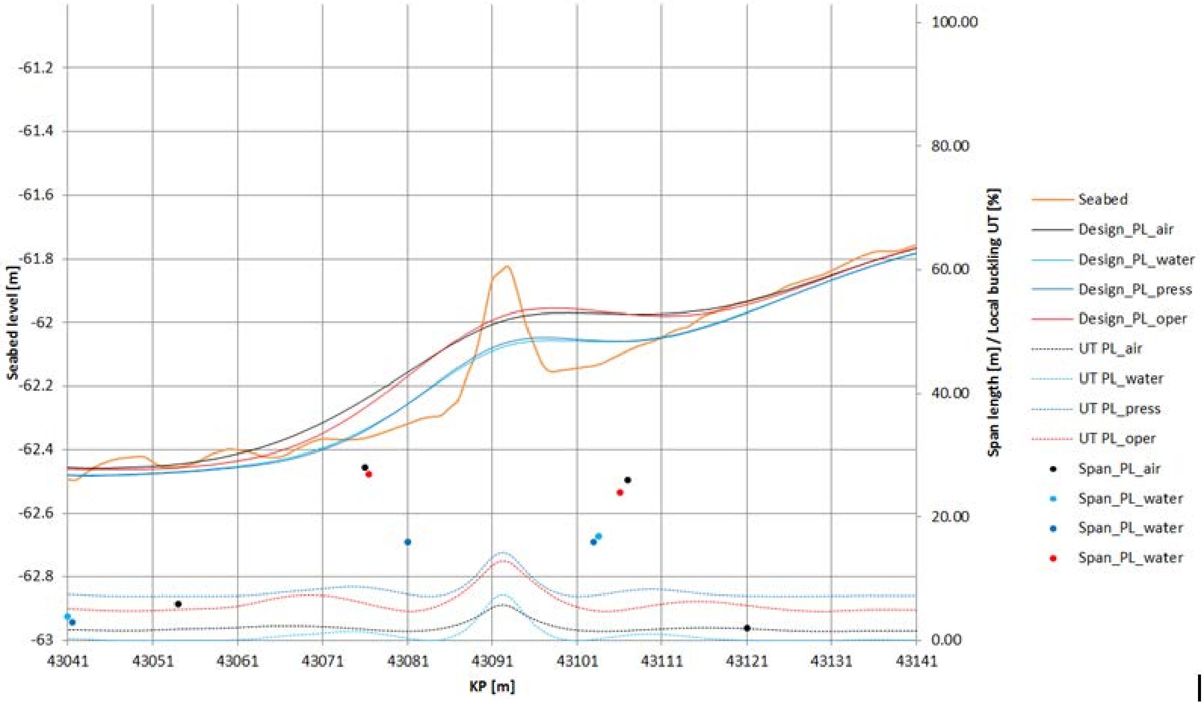


Figure 7-16 In-place analysis of Nord Stream 2 pipeline

Based on the survey, in-place analyses are carried out to determine the exact theoretical as-laid configuration of the Balticconnector pipeline at the crossing locations. This is performed for all phases of the design life; air-filled, water-filled, pressure test and operation, in order to design the optimal solution. Figure 7-16 depicts the initial in-place results of Nord Stream 2 with no carpet design, and in Figure 7-17 the final carpet design is presented. Comparing Figure 7-16 and Figure 7-17, it can be seen that the carpet crossing design is adjusted to both the natural curve of the pipe as well as the local topology of the seabed. Reference is made to the *Pre- and post-lay crossing design drawings,* Refs. /41/, /42/, /44/ and /45/.

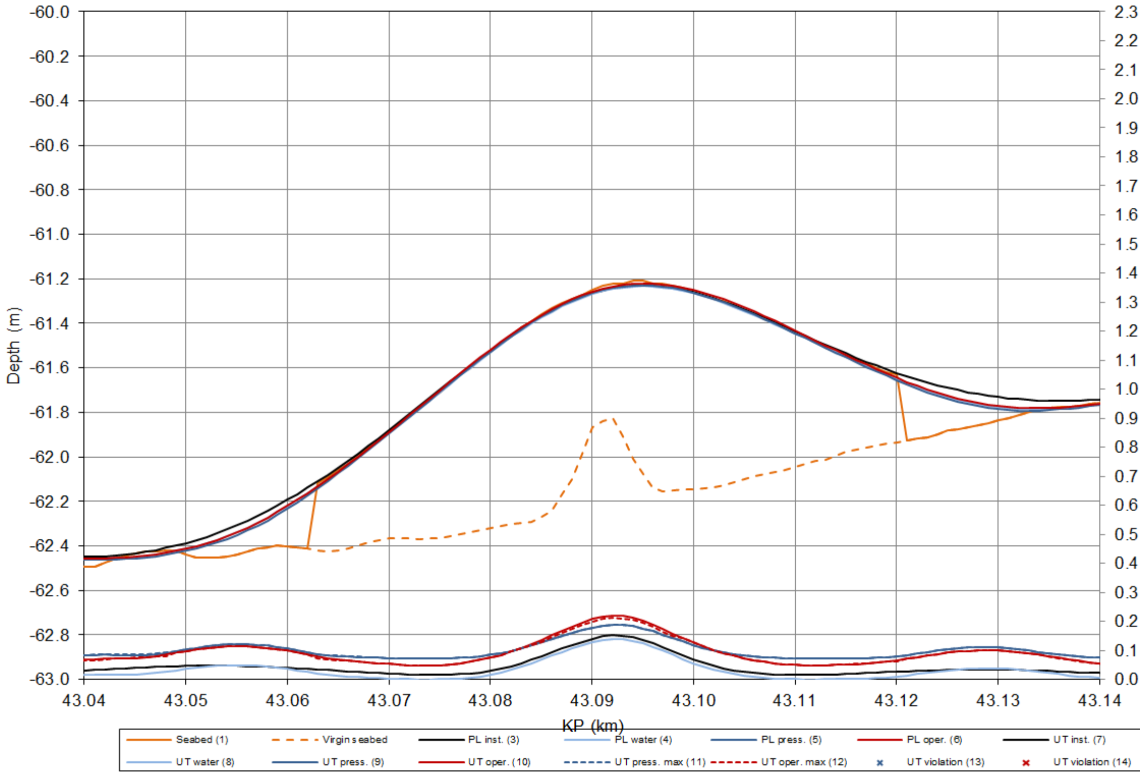


Figure 7-17 Carpet design for Nord Stream 2 pipeline (Solid orange line: seabed/pre-lay rock installation, Dashed orange line: virgin seabed)

For the crossing of cables, a more pragmatic approach is adopted. According to *DNV-OS-F101*, Ref. /1/, a minimum vertical separation of 0.3 m is required. For lines where it can be verified that the burial depth is deep enough to ensure a vertical separation from the Balticconnector pipeline of 0.3 m, no seabed intervention is required. If this cannot be verified, a pre-lay rock carpet is installed with a minimum height of 0.5 m. This contingency is in order to account for potential settlement and installation dynamics.

An assumption is made for the FEED phase that all buried cables are located in close proximity to the surface of the seabed.

Upon installation of the Balticconnector offshore pipeline, post-lay rock installation is applied to Top-of-Pipe (TOP) from touchdown to touchdown at all crossing locations to accommodate trawl gear interaction. Reference is made to the *Pre- and post-lay cable crossing drawings*, Ref. /43/ and /46/.

8 Global buckling and trawl pull-over analysis

8.1 Introduction

8.1.1 Scope of this chapter

The scope of this chapter is to screen the pipeline with respect to issues related to global buckling and trawl pull-over impact. Near the shores the screening includes a global buckling solution proposal based upon selected test locations. The methodology adopted is described and the results are summarised in this chapter.

The work is based on detailed finite element analyses and state-of-art design approaches, *DNV OS-F101, Submarine Pipeline Systems*, Ref. /1/ and *DNV RP-F110, Global Buckling of Submarine Pipelines - Structural Design Due to High Temperature/High Pressure*, Ref. /9/.

8.1.2 Lay-out

The GB (Global Buckling) / TPL (Trawl Pull-over Load) analysis is split into three different sections in which different approaches are used. The pipeline sections are summarised in Table 8-1, and explained in detail in section 8.3.5.

Pipeline section	KP	Length (km)
1 (Estonia)	67.5 – 80.4 (shore)	12.9
2 (Finland)	0.0 – 12.0	12.0
3 (Offshore)	12.0 – 67.5	55.5

Table 8-1 Definition of pipeline sections

8.1.3 Assumptions

Following assumptions have been used throughout the GB/TPL analysis.

- The design temperature profile is based on experience and not specific calculations
- The operational temperature profile is assumed to be identical to the design temperature profile
- The installation temperature is assumed to be 5 degree Celsius
- The seabed temperature during operation is assumed to be 10 degree Celsius
- The operational pressure is assumed to be identical to the design pressure
- The route is assumed to be straight
- The worst case scenario is assumed to correspond to UB (Upper Bound) TPL
- For even seabed condition the condition load effect factor is γ_c=0.9 and γ_c=1.07 for un-even seabed condition
- It is assumed that the seabed in pipeline section 3 is bedrock

8.1.4 Recommendations

In order to improve the accuracy and consequently reduce the conservatism of the design and screening, following recommendations are given:

- Calculate temperature profiles for both operation condition and design condition. A less conservative temperature profile lowers the effective compression force and thereby the susceptibility to GB and criticality of a TPL
- Obtain information regarding trawling activities i.e. frequencies, trawl gear and dimensions. Some sections may not be subjected to trawling; hence, no mitigation towards TPL will be required in these sections. In addition, reducing the trawl gear dimension etc. implies that the TPL is reduced

8.2 Design input

This section presents design input used specifically for the global buckling assessment. For general design input cf. section 3 (Design basis), section 9.1 (Pipe-soil interaction assessment) and section 11 (Pipeline installation). The sectioning is summarised in Table 8-1.

8.2.1 Pipeline configuration

Table 8-2 summarises the different pipeline configurations which have been analysed in this chapter. For a full description of the various pipeline configurations, cf. Ref. /34/ (Design basis) and section 12 (Pipeline installation).

KP start – KP end	OD_{steel} (mm)	WT (mm)	Concrete thickness (mm)	Submerged weight (N/m)	Residual lay tension (kN)
0.000 – 19.350	508	12.7	55	1941	253
19.350 – 21.000			80	3139	609
21.000 – 26.000			80	3139	654
67.500 – 80.400			45	1488	143

Table 8-2 Considered pipeline configurations

8.2.2 Temperature profiles

The temperature profiles are based upon an exponential profile, in which the parameters C_p and Q_f are used to calibrate the temperature profile against known profiles:

$$T = e^{-\frac{U\pi D x}{\rho C_p Q_f}} \cdot (T_{inlet} - T_{ambient}) + T_{ambient}$$

The insulating effect of the soil and rock covers is based upon knowledge of the insulating effect of a pipeline buried in soil, cf. Ref. /24/, section 6.6.5. A rock covered pipeline will be subject to a higher water ingress compared to a trenched pipeline. Thus the equivalent U[1] value of the rock cover is taken as the mean value of the fully exposed pipeline and the fully buried pipeline. Figure 8-1 shows the temperature profiles with and without rock covers for pipeline sections 1 and 2. Within pipeline section 3 a constant temperature profile of 10 degrees is used.

[1] Heat transfer coefficient

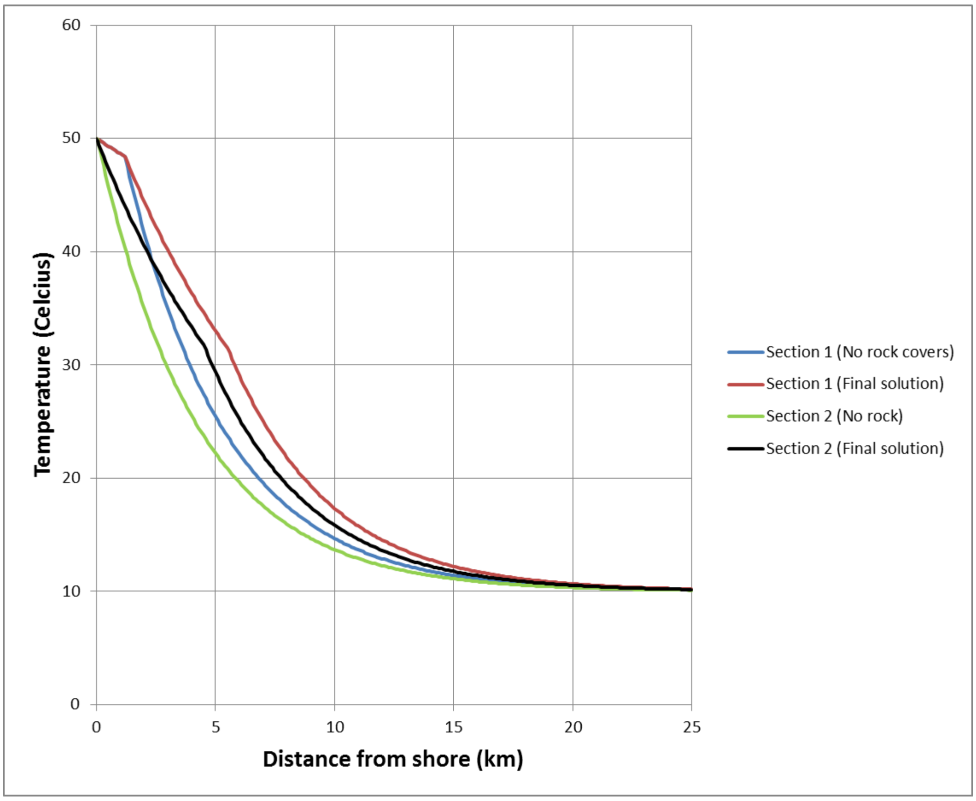


Figure 8-1 Design operation temperature profiles for pipeline sections 1 and 2

8.2.3 Pipe-soil behaviour

The geotechnical description and formulations are given in section 9.1 (pipe-soil interaction assessment), hence this section only summarises the relevant pipe-soil data as used in the report for each section.

Pipeline section 1 - Estonia

The soil is assumed to be sand, cf. the non-linear pipe-soil interaction behaviour in section 9.1 and Appendix X.

Pipeline section 2 - Finland

The soil is assumed to be clay. For the 2D analysis see the non-linear pipe-soil interaction behaviour in section 9.1. In terms of the 2½D[2] model the non-linear lateral pipe-soil behaviour is approximated by frictional elements which form a “resistance ladder”, cf. Figure 8-2 (force resistance ladder) and Table 8-3 (for corresponding “equivalent” friction coefficient[3] values). The axial resistance is approximated by a constant equivalent friction coefficient of 0.66, cf. section 9.1 for corresponding resistance force.

[2] Straight model located on the true seabed profile and able to move in a 3D space.

[3] The equivalent friction coefficient corresponds to the resistance force divided by the submerged weight of the pipeline and should not be confused with the friction coefficients given in section 9.1.

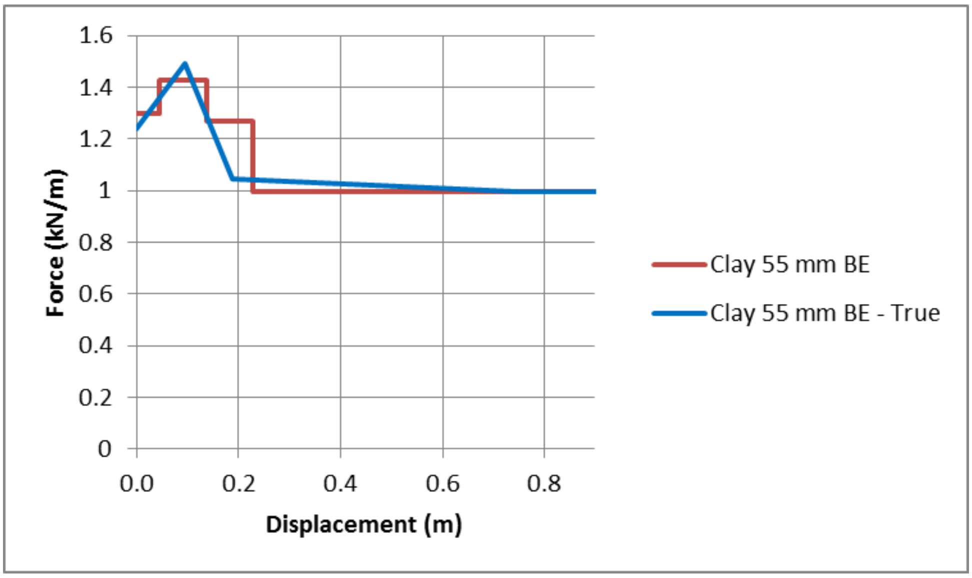


Figure 8-2 **Lateral pipe-soil force resistance ladder used as basis for the 2½D models**

Displacement (m)	Friction
0 – 0.045	0.67
0.045 – 0.136	0.74
0.136 – 0.227	0.65
> 0.227	0.51

Table 8-3 **Lateral equivalent friction coefficients corresponding to the force resistance ladder values**

Pipeline section 3 – Offshore

The soil is assumed to be rock. The axial and lateral resistance is approximated by a constant equivalent friction coefficient of 0.54, cf. section 9.1 (Pipe-soil interaction assessment) for corresponding resistance force.

8.3 Methodology

8.3.1 General introduction to global buckling

A pipeline which experiences a temperature increase and internal over-pressure will expand. Depending on the soil friction and boundary condition, this expansion will cause a build-up of an effective axial compression force. Eventually, the pipeline becomes unstable and buckles in case an asymmetry is present e.g. as an OOS or as a transverse force (TPL).

GB may appear either downwards into free spans, horizontal or vertically as UHB (Upheaval Buckling) at crests. During buckling the axial feed-in into the buckle will release the effective forces while bending moments start to emerge.

Depending on the feed-in, the integrity of the pipeline may suffer in the event of GB (or simply due to TPL), i.e. mitigation techniques are required. In this stage of the project only rock covers and rock in-fill will be considered since the project is in feed phase. Rock covers will be used to constrain the pipeline i.e. GB is prevented. Rock in-fill will be used to reduce the span height and thereby the TPL.

8.3.2 Codes

The principal design code for the pipeline is DNV-OS-F101 Submarine Pipeline Systems, Ref. /1/.

The DNV-RP-F110 design guideline, Ref. /9/, is used here for the GB methodology. This guideline complements DNV-OS-F101 and gives specific requirements for GB of high temperature/high pressure pipelines.

The trawl gear/pipeline interference will be based upon the approach outlined in the recommended practice DNV-RP-F111, Ref. /10/.

8.3.3 Load combinations

In line with Table 3-4 in DNV-RP-F110, Ref /9/, various load scenarios have to be considered. Table 8-4 summarises these load combinations for trawl frequencies below unity.

Trawling frequency	Design scenario	Functional load		Trawl load	Environmental
		Pressure load	Temperature load		
$10^{-4}<f_T<1$	Functional	Local incidental	Local design	No	No
	Interference	Local operating	Local operation	$F_T^{BE}=0.8$	No
	Environmental	Local operating	Local operation	No	100 yr.

Table 8-4 Load combinations to be considered for trawling frequencies less than unity

Since the operating temperature is unknown it is conservatively assumed that the operating temperature corresponds to the design temperature, while the local operating pressure is conservatively taken to be equal to the design pressure. Consequently, the interference design scenario governs, thus only this scenario will be treated.

8.3.4 Loads

Following sections briefly describe the loads which are considered.

Temperature and pressure

In line with section 8.3.3 the design temperature is used, cf. section 8.2.2. Conservatively, the design pressure is combined with the design temperature and trawl load, cf. section 8.3.3. Any decay in the pressure profile will not be accounted for in the load effect analysis, which is a conservative assumption.

Residual lay-tension

In section 8.2.1 the minimum residual lay-tension is given. For the integrity checks the minimum residual lay-tension is used since this gives the largest force build-up.

Trawl pull over load

For trawl gear information specified in Table 3-6, only polyvalent trawl boards are considered as these generate larger forces compared to the clump weights. The TPL is modelled using empirical formulas given in Section 4 of DNV-RP-F111, Ref. /10/. The method decomposes the TPL into a vertical and horizontal contribution. Both contributions vary linear within the duration time.

The TPL for various span heights, concrete coating thicknesses and water depths is summarised in Appendix IX.

8.3.5 Buckling design methodology

The GB analysis is made in accordance with DNV-RP-F110, Ref. /9/. Due to the preliminary phase of the project some simplifications and assumptions have been adopted compared to a full design report.

The GB analysis is split into three different pipeline sections in which different approaches are used:

- Pipeline section 1: Estonian section which is characterised by even seabed and high temperatures
- Pipeline section 2: Finnish section which is characterised by uneven seabed and high temperatures
- Pipeline section 3: In between the Estonian and Finnish section which is characterised by very uneven seabed but no temperatures

Pipeline section 1: even seabed

Even seabed conditions imply that the governing deformations will take place in the horizontal plane due to an OOS or TPL while the seabed undulations do not introduce any significant bending moments. For even seabed conditions a 2D model can be used. Pipeline section 1 is characterised by an even seabed without any significant bends of the pipeline. I.e. a 2D straight model is reasonable to use.

The methodology presented in DNV-RP-F110, Ref. /9/ assumes that lateral buckling can be initiated by either an OOS from laying the pipeline or a TPL. Depending of the GB susceptibility towards these trigger mechanism, a classification can be made along the pipeline;

- No buckling condition
- Maybe buckling condition
- Buckling condition: BE conditions are able to initiate GB. The post buckling configuration shall be checked as an ULS condition which requires a calibration of the condition load effect factor γ_c while the load effect factor for functional loads is γ_F=1.1.

In this report it is conservatively assumed that pipeline section 1 is classified as *Buckling* condition. A calibration of γ_c will not be performed and a value of γ_c=0.9 is preliminarily assumed. In case GB does not occur when using BE soil and BE TPL, the soil is changed to LB while the TPL is changed to UB which is conservative.

In regard to GB triggered by an OOS it is necessary to define a representative OOS and determine within which region an OOS is able to trigger GB. According to DNV-RP-F110, Ref. /9/, the susceptibility of GB triggered by random natural imperfections may be assessed using Hobbs infinite mode capacity. For more information see Hobbs, Ref. /22/.

In this report a maximum OOS and a minimum OOS are defined. In practice a sinusoidal OOS is included in the FE model[4]. The maximum OOS is calibrated so the trigger force is identical to the predicted Hobbs force *P*. The minimum OOS is calibrated to trigger at the maximum effective force at the particular location. The OOS is implemented as stress free.

The integrity is verified based upon the load cases defined in section 8.3.6 and the integrity check outlined in section 8.3.7.

<u>Pipeline section 2: uneven seabed</u>

For pipeline section 2 the seabed is uneven while the route is reasonably straight, hence a 2½D model is used (straight route but seabed undulations are included). For uneven seabed conditions a conservative γ_c=1.07 can be used, cf. Ref. /9/, section 7.2 in combination with a load effect factor for functional loads of γ_F=1.1.

Unlike the even seabed condition the un-even seabed condition implies that each location is unique due to both effective force level and local seabed profile. Consequently, the screening includes three different screening criteria:

<u>Even seabed screening</u>

Normal 2D screening identical to even seabed conditions i.e. lateral buckling triggered by an OOS or TPL. The purpose of this check is to insure that sections with contact to the seabed do not violate the integrity in case of GB. The outcome of the screening is to identify sections which have to be rock covered as they cannot withstand a potential GB.

Note that the cover heights are estimated based on the UHB requirement given in section 8.3.7. Soil springs are added in line with section 9.1 while a factor of 1.15 is multiplied to the temperature profile.

<u>Verification of critical span</u>

Selected spans along the length of the pipeline are examined for TPL i.e. a trawl is applied to the location of interest after temperature and pressure is applied. In general the spans are chosen based on maximum utilisation during operation condition. The outcome of this screening is to determine the maximum allowable span height and thereby identify sections at which rock infill has to be applied.

<u>Verification of critical crest</u>

Selected crests along the length of the pipeline are examined for TPL, i.e. a trawl is applied to the location of interest after temperature and pressure is applied. In general, the crests are chosen based on maximum utilisation during operational condition. The outcome of this screening is to determine the maximum allowable crest utilisation during operation.

<u>Pipeline section 3: uneven seabed</u>

For pipeline section 3 the seabed is uneven while the route is reasonable straight hence a 2½D model is used with γ_c = 1.07 and γ_F = 1.1.

The temperature is assumed to be constant within pipeline section 3 while the effective force is limited; hence, only two screening criteria are used;

---

[4] The wave length corresponds approx. to the wave length of the pipeline if the pipeline is lifted 1.5 m above the seabed i.e. the stiffness of the pipeline influence the length to avoid non-physical imperfections.

Verification of critical span
As stated above in *Pipeline section 2: un-even seabed.*

Verification of critical crest
As stated above in *Pipeline section 2: un-even seabed.*

Design approach
For sections 1 and 2 the design process is made up by following steps:

1) Normal temperature profile + no rock covers: Identification of sections where the integrity of the pipeline will be violated for load case 1 and 2, cf. Table 8-5
2) Temperature profiles including insulation (rock covers): Based upon the screening analysis rock covers will be added if the integrity is violated. Note that this is done iteratively due to the insulation effects of the rock covers and increased axial resistance
3) For pipeline section 2, load case 3 will be analysed for the above design solution. If necessary, additional rock covers will be added while a simplified UHB check will be carried out in order to verify the height of the rock covers. Note that load case 3 will provide restrictions towards the span heights and crest utilisations

For pipeline section 3, the screening will only prescribe some restrictions towards the span heights and crest utilisation.

8.3.6 Load cases

The load combination shall be applied to the most unfavourable situations (Load cases). The load cases are shown in Table 8-5 together with a description to where these load cases apply. A brief description of the load cases is given below Table 8-5. Due to zero corrosion allowance and since cyclic loading relaxes the bending moment, the number of load cases can be reduced to only three.

Pipeline section	KP	Load case	Model + location	Trigger
1 (Estonia)	67.5 – 80.4	1	2D model	OOS
		2	2D model	TPL
2 (Finland)	0.0 – 12.0	1	2D model	OOS
		2	2D model	TPL
		3	2½D model: Spans and crests	OOS if possible otherwise TPL. GB not required.
3 (Offshore)	12.0 – 67.5	3	2½D model: Spans and crests	OOS if possible otherwise TPL. GB not required.

Table 8-5 Summary load cases for all three pipeline sections

Load case 1
GB triggered by functional load: An OOS initiates GB while a TPL hits near the apex of the post buckle configuration[5]. This load case is only relevant when TPL are located within the section where an OOS is able to trigger GB.

[5] TPL is applied 5 m from the apex of the OOS which is in line with DNV-RP-F110, section 3.5.3, Ref /9/,

Load case 2
GB triggered by TPL: A TPL hit the straight pipeline and causes GB to develop. GB triggered by TPL can take place at any location along the pipeline.

Load case 3
Temperature and pressure is applied where after a TPL hits the location of interest. There is no requirement that GB must take place i.e. GB is not enforced if BE TPL is not capable of initiate GB.

8.3.7 Pipeline integrity

According to DNV-RP-F110, Ref. /9/, exposed pipelines must be examined for the following integrity checks (here only the GB/TPL related checks):

- Axial loading check
- Local buckling (load (LCC) and displacement controlled (DCC))
- Fracture
- Fatigue due to shutdown restart cycles

The local buckling (LCC) and fracture check will be used in this screening as the axial loading check and DCC[6] are usually not governing. Fatigue due to shutdown restart cycles is only applicable when the operational conditions are known. For load case 3 the fracture check will only be performed for the location of the largest LCC utilisation.

In regard to rock covered sections a simplified UHB check will be carried out to estimate the approximate required cover height. Note that UHB will not be examined for pipeline section 1 due to the even seabed condition in combination with a concrete coated pipeline.

Local buckling
According to DNV-RP-F110, Ref. /9/, pipelines subjected to GB must, among others, comply with the combined moment check (load controlled condition) which formally states:

$$M_F\left(f_L^{BE}; f_A^{BE}; f_c; F_c\right) \cdot \gamma_F \cdot \gamma_c \leq \frac{M_c}{\gamma_m \cdot \gamma_{SC}}$$

Here M_c is the characteristic moment resistance, γ_m is the material resistance factor while γ_{SC} is the safety class resistance factor.

Regarding exposed pipelines the combined moment check (LCC) is performed based on f_u and f_y, best estimate values of pipe-soil resistance and TPL.

The local buckling failure mode (LCC) will be checked for the ULS (buckling classification) for internal over pressure. The load controlled conditions are described in further detail in DNV-RP-F101, Ref. /1/.

Following assumptions and methodology have been incorporated:

- For LCC the $\gamma_c = 0.9$ for pipeline section 1 and $\gamma_c = 1.07$ for pipeline section 2 and 3

[6] A DCC check has been performed and found acceptable for the critical span however the result is not included

- TPL are considered as functional loads, cf. DNV-RP-F110 page 20, Ref. /9/
- Since the annual trawling frequency is less than 1, the BE TPL is scaled by a factor of 0.8 while UB TPL is scaled by a factor of 1.0

Fracture

The pipeline shall have adequate resistance against initiation of unstable fractures. One criterion is evaluated:

- Total longitudinal nominal tension strain $\varepsilon_{1,nom} \leq 0.4\%$ (DNV-OS-F101, Table 5-12, Ref. /1/)

For pipelines subjected to a nominal strain which exceeds 0.4 % it is necessary to perform an engineering criticality assessment (ECA) in order to confirm that unstable fracture will not occur.

The total strain is found as the sum of the axial plastic strain and axial elastic strain for worst case conditions. According to DNV-RP-F110, Ref. /9/, worst case scenarios correspond to the worst case found during the calibration of the condition load effect factor[7]; however, in the following, worst case scenario is chosen to correspond to UB TPL and BE soil conditions.

UHB check

Upheaval buckling (UHB) will be evaluated based upon a critical mobilisation distance of 25 mm, i.e. if the local uplift movement of the pipeline does not exceed 25 mm no UHB will take place, in accordance with Ref. /9/ section 5.4.1.

8.4 Numerical model

The FE program ANSYS (Ref. /23/) is used to perform the analyses of the pipeline buckling behaviour.

8.4.1 2D Lateral buckling (pipeline section 1)

A straight 2D model (flat seabed) is used to evaluate the lateral buckling behaviour and capacity for pipeline section 1. The assumption of flat seabed is conservative since there are no vertical undulations to help release the effective compression force. The seabed profile is shown in Section 5. From the seabed profile it is evident that the seabed will not influence the responses of the pipeline neither in terms of bending moments or trigger force. The TPL is applied as a concentrated nodal force which varies with time in accordance to Appendix IX. The vertical component is included as an increased contact pressure.

A summary of the model for the 2D transient analyses is given in Appendix IX.

8.4.2 2½D contact model (pipeline section 2 and 3)

A 2½D model with actual seabed topography is used to simulate the pipeline behaviour for pipeline section 2 and 3. The pipe-soil behaviour is implemented with discrete contact elements. The route is approximated by a straight route which is conservative since there are no bends to help release the effective compression force (this is true if the radii of the

[7] DNV-RP-F110, Ref. /9/ pp. 27

curvatures are not too small). The TPL is applied as a concentrated nodal force which varies with time as shown in Appendix IX.

Note that the 2D lateral buckling model is used for pipeline section 2 load case 1 and 2 while the UHB is assessed using the a 2D model in the vertical plane (Identical to the 2½D model except lateral movements are restricted).

A summary of the model for the 2½D transient analyses is given in Appendix IX.

8.5 Results

8.5.1 Pipeline section 1 - Estonia

Load case 1 and 2

A 2D design screening and local buckling check is performed in this section.

Initial screening

The Hobbs critical buckling force is 1299 kNm, i.e. sections which experience effective forces larger than 1299 kNm may buckle due to an OOS. The effective force along the pipeline together with Hobbs critical buckling force is shown in Appendix IX. According to the appendix an OOS may trigger GB from KP 74.4 to KP 79.2 if no rock installations are added and the pipeline is buried between KP 79.2 to KP 80.4.

The LCC for BE condition will be used to assess where mitigation techniques are required. Note that LB lateral soil condition is used in case BE TPL is not able to trigger GB[8]. The maximum allowable moment for an effective force of 600 kN is 1044 kNm. Locations at which the integrity is violated will be rock covered. Table 8-6 summarises the results of the design screening.

KP	Load case 1 - Moment (kNm)	Load case 2 - Moment (kNm)	Integrity
76.4	1072	967	Not OK
75.9	957	810	OK

Table 8-6 Summary integrity screening

From Table 8-6 it is evident that mitigation techniques must be introduced approx. between KP 75.9 and KP 80.4 in order to satisfy the integrity of the pipeline. Taking into account the insulating effect of the rock cover, it is found that the rock cover must be extended to KP 74.9 (i.e. between KP 74.9 and KP 80.4) in order to protect the pipeline against TPL and lateral movements.

Final design solution check

Table 8-7 shows the verification of the final design solution in terms of LCC i.e. the integrity of the post buckled pipeline outside the rock covered section.

[8] Since the pipeline is classified as buckling the post buckled configuration must be examined i.e. it must buckle.

KP	Load case	Moment (kNm)	Effective compression force (kN)	LCC utilisation	Integrity
74.7	1	1019	588	0.96	OK
	2	903	661	0.77	OK

Table 8-7 Summary local buckling check

No UHB check within the rock covers will be performed as the seabed is flat and the pipeline is concrete coated. Figure 8-3 illustrates the behaviour of the bending moment during GB for load case 1 and 2.

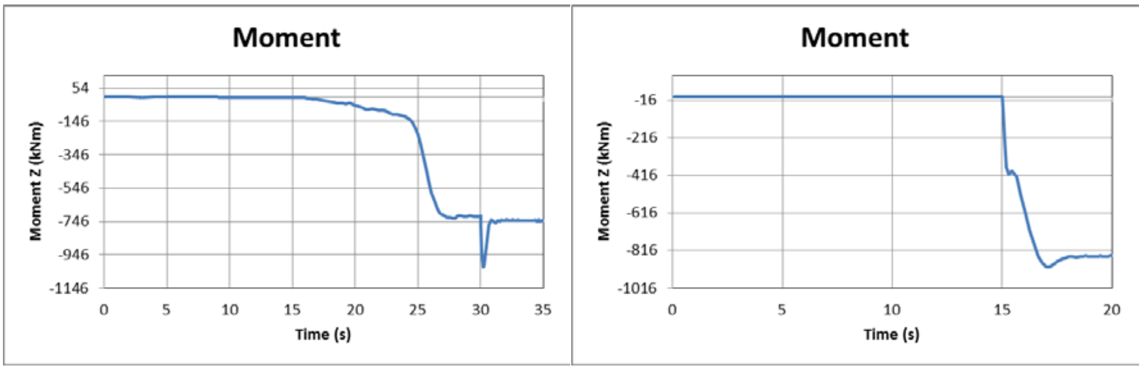


Figure 8-3 Moment as a function of time – Left: Load case 1 (OOS trigger), Right: Load case 2 (TPL trigger)

<u>**Fracture**</u>

The largest nominal tension strain for each load case is shown in Table 8-8 together with the utilisations. From the utilisations it can be concluded that the pipeline fulfils the fracture criterion, cf. section 8.3.7 for the chosen solution.

Load case	Characteristic strain ε (%)	Design strain ε_{Sd} (%)	Utilisation
1	0.26	0.29	0.73
2	0.20	0.22	0.55

Table 8-8 Fracture criterion

8.5.2 Pipeline section 2 - Finland

Between KP 0 and 1.5 the pipeline experiences GB and consequently over-utilisations due to the undulations of the seabed. Preventing these GBs simply moves the over-utilisations to adjacent spans and crests, i.e. a continuous rock cover is required between KP 0 and 1.5; hence, this section will not be part of the span and crest screening.

<u>**Load case 1 and 2**</u>

A 2D design screening and local buckling check is performed in this section.

<u>Initial screening</u>

The Hobbs critical buckling force is 1160 kNm which, without any post lay seabed intervention, implies that GB can be expected between KP 0 and 5.0, as shown in Appendix IX.

In line with section 8.5.1, the LCC for BE condition will be used to assess where mitigation techniques are required. The maximum allowable moment for an effective force of 600 kN is 875 kNm. Locations at which the integrity is not fulfilled will be rock covered. Table 8-9 summarises the results of the design screening.

KP	Trigger OOS - Moment (kNm)	Trigger TPL - Moment (kNm)	Integrity
2.0	890	764	Not OK
2.5	828	706	OK

Table 8-9 Summary integrity screening

From Table 8-9 it is evident that mitigation techniques must be introduced approx. between KP 0 – 2.5 in order to satisfy the integrity of the pipeline. Due to the insulating effect of the rock cover, it is found that a rock cover must be installed from KP 0 to KP 3 in order to fulfil the 2D screening criterion. Table 8-10 summarises the LCC for the chosen solution.

KP	Load case	Moment (kNm)	Effective compression force (kN)	LCC utilisation	Integrity
3.0	1	874	414	0.99	OK
	2	816	792	0.89	OK

Table 8-10 Summary local buckling check

<u>Load case 3</u>

The following section presents the screening results of the spans and crests in terms of LCC.

<u>Span screening</u>

Three different spans are considered. The spans have been chosen based on their utilisations and effective force level and are believed to cover the most critical and representative spans within exposed part of pipeline section 2, cf. Table 8-9. The screening results are summarised in Table 8-11 for various span heights.

KP 3)	Span length (m)	Span height (m)	Moment (kNm)	Effective compression force (kN)	LCC utilisation	Integrity
3.423	32	0.4	842 1)	895	0.96	OK
		0.3	1020 2)	300	1.32	Not OK
6.735	23	0.5	903	809	1.08	Not OK
		0.4	823	852	0.97	OK
11.541	40	0.4	924	314	1.09	Not OK
		0.3	870	319	0.97	OK

Notes:
1) Only TPL affects the span
2) A minor distributed load is applied during the temperature increase load step which initiate GB before the TPL is applied
3) Location of maximum utilisation within the span

Table 8-11 Summary 2½D span screening

In regard to location KP 3.423 it is chosen to add rock between KP 3.0 and 3.5 since the location is very susceptible to GB and consequently integrity violations in case of a TPL hits the buckle.

For KP 6.735 and 11.541 the integrity is fulfilled for span heights below 0.4 m and 0.3 m respectively. Thus, assuming that the spans in Table 8-11 are the critical spans, it is concluded that the integrity of all the spans are fulfilled if the span heights do not exceed 0.3 m.

Crest screening

Three different crests are considered covering the most critical crest utilisations. Note that the crests between KP 0 and 4.5 have not been considered due to the presence of a rock cover.

KP	Crest utilisation	Moment (kNm)	Effective force (kN)	LCC utilisation	Integrity
10.417	0.56	645 [1)+2)]	-525	0.57	OK
11.573	0.64	801 [2)]	-407	0.84	OK
11.749	0.12	636 [3)]	-405	0.55	OK

Notes
1) Max utilisation is due to the deformation introduced by the crest and not the TPL
2) TPL is defined for a 0.0 m span height
3) ”spanning crest” i.e. location between two crest located closely together. TPL is defined for a 0.4 m span height

Table 8-12 Summary 2½D crest screening

From Table 8-12 it is concluded that all crests within the exposed part of pipeline section 2 have sufficient capacity.

Fracture check

The largest nominal tension strain and respective utilisations for each load case are shown in Table 8-13. From the utilisations, it can be concluded that the pipeline fulfils the fracture criterion, cf. section 8.3.7, if the span height is below 0.3 m in pipeline section 2.

Load case	Characteristic strain ε (%)	Design strain ε_{Sd} (%)	Utilisation
1	0.21	0.23	0.58
2	0.17	0.19	0.47
3	0.17	0.19	0.47

Table 8-13 Fracture criterion results

Upheaval buckling check

Figure 8-4 illustrates the vertical uplift movement along the pipeline for a cover height of 0.5 m and 1.0 m. From the figure it is seen that a cover height of 0.5 m is sufficient for the majority of the length while an increased cover height is required at few places. In this FEED report it is chosen to specify a cover height of 0.5 m on average between KP 0 and KP 4.5.

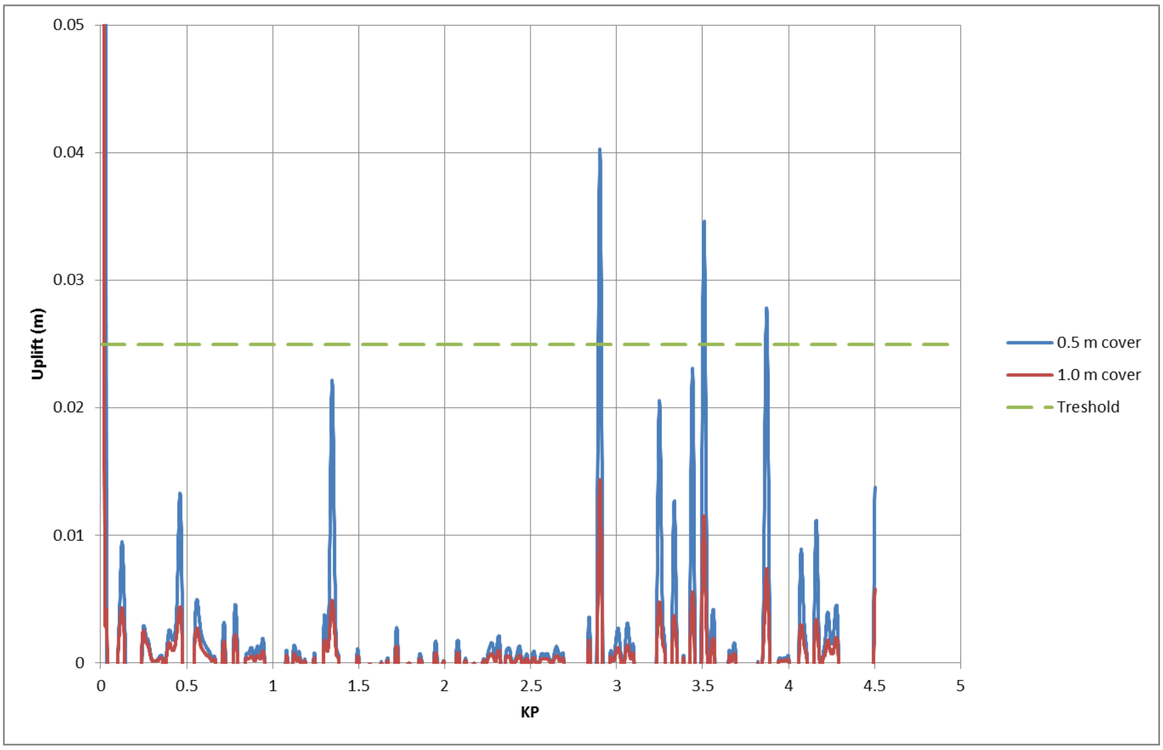


Figure 8-4 Uplift movements within rock cover near Finnish shore for design temperatures

8.5.3 Pipeline section 3 - Offshore

The screening only includes spans and crests which are able to fulfil the bottom roughness screening since these otherwise will be subject to changes which involves excavation or supports.

Since pipeline section 3 contains different pipeline configurations (residual lay tension, concrete thickness etc.), the screening is divided into four sub sections:

- Pipeline section 3A: KP 12.00 – 19.35
- Pipeline section 3B: KP 19.35 – 21.00
- Pipeline section 3C: KP 21.00 – 26.00
- Pipeline section 3D: KP 26.00 – 67.50

It is assessed that pipeline section 3A is the most critical section i.e. the conclusions for pipeline section 3A can conservatively be used for pipeline section 3D for the exposed parts without introducing any significant additional seabed intervention in pipeline section 3D. Thus, pipeline section 3D will not be examined.

Load case 3

The following section presents the screening results of the spans and crests. The screening is based upon the LCC criterion.

Span screening

In order to conclude on the behaviour of the spans, a number of different crests have been examined for pipeline section 3A while one span is used for pipeline section 3B and 3C.

These spans represent the most critical spans in terms of utilisation during operation. However, it is important to note that each span behaves uniquely to a TPL and the conclusions are simply based upon the shown locations. Especially the span lengths are seen to be important since this parameter among others determines the deformation shape when TPL are applied. The screening results are summarised in Table 8-14 for various span heights.

KP 1)	Pipeline section	Span length (m) / utilisation 2)	Span height (m)	Moment (kNm)	Effective force (kN)	LCC utilisation 3)	Integrity
13.885	3A	49 / 0.33	0.8	909	153	1.05	Not OK
			0.7	868	130	0.96	OK
16.039	3A	28 / 0.21	0.8	880	11	0.99	OK
17.364	3A	45 / 0.59	0.4	913	85	1.06	Not OK
			0.3	887	14	1.00	OK
17.943	3A	42 / 0.23	0.7	890	109	1.00	OK
18.683	3A	69 / 0.46	0.5	909	197	1.05	Not OK
			0.4	882	190	0.99	OK
20.668	3B	65 / 0.39	0.9	909	601	1.05	Not OK
			0.8	875	601	0.98	OK
25.372	3B	72 / 0.45	0.8	905	593	1.04	Not OK
			0.7	874	593	0.98	OK

Note:
1) Location of maximum utilisation within the span
2) These utilisations refer to the utilisations found in the Bottom roughness analysis i.e. the utilisations before a TPL hit the pipeline when the pipeline is in operation. These utilisations are used in order to relate the GB/TPL results with the bottom roughness results when the seabed intervention is considered
3) These utilisations are the maximum utilisations during/after TPL

Table 8-14 Summary 2½D span screening

From the results above it is obvious that several parameters, beside the initial utilisation, determine the pipeline response to a TPL. The span length, initial deformation shape and seabed condition adjacent to the span have a huge importance. The conclusions drawn from Table 8-14 are summarised in Table 8-16.

Crest screening

In order to conclude on the behaviour of the crests, three different crests have been examined for pipeline section 3A while one crest is used for pipeline sections 3B and 3C. These crests represent the most critical crests in terms of utilisation during operation. However, it is important to note that each crest behaves unique to a TPL and the conclusions are simply based upon the shown locations. The screening results are summarised in Table 8-15 for various span heights.

KP	Pipeline section	Crest utilisation [1]	Moment (kNm)	Effective force (kN)	LCC utilisation [2]	Integrity
13.764	3A	0.90	837	-77	0.90	OK
16.194	3A	0.85	834	-486	0.91	OK
18.982	3A	1.00	904	-66	1.04	Not OK
20.579	3B	0.96	875	358	0.98	OK
24.754	3C	0.93	862	93	0.95	OK

Note:
1) These utilisations refer to the utilisations found in the Bottom roughness analysis i.e. the utilisations before a TPL hit the pipeline when the pipeline is in operation. These utilisations are used in order to relate the GB/TPL results with the bottom roughness results when the seabed intervention is considered
2) These utilisations are the maximum utilisations during/after TPL

Table 8-15 Summary 2½D crest screening

In general the results show that a TPL only increases the utilisations at the crest slightly due to a large contact pressure. The conclusions drawn from Table 8-15 are summarised in Table 8-16.

Pipeline section	Utilisation	Maximum allowable crest utilisation [2]	Maximum allowable span height (m)
3A	0.00 – 0.30	0.90 [1]	0.7
	0.30 – 0.45		0.4
	0.45 – 0.60		0.3
	> 0.60		Individual examination required since none utilisations has been reported
3B	N/A	0.98	0.8
3C	N/A	0.93	0.7

Note:
1) No crest exists with a utilisation between 0.90 – 1.00 i.e. no conclusion can be made
2) These utilisations refer to the utilisations found in the Bottom roughness analysis i.e. the utilisations before a TPL hit the pipeline when the pipeline is in operation. These utilisations are used in order to relate the GB/TPL results with the bottom roughness results when the seabed intervention is considered

Table 8-16 Conclusions 2½D screening

Fracture check

The fracture criterion is verified based upon the location with maximum LCC utilisation (for UT≤1) i.e. KP 17.943 (this location has the largest strain). Table 8-17 summarises the relevant tension strains together with the utilisation. From the table is it concluded that the restrictions given in Table 8-16 also fulfil the fracture criterion, cf. section 8.3.7.

Load case	Characteristic strain ε (%)	Design strain ε_{Sd} (%)	Utilisation
1	0.2	0.22	0.55

Table 8-17 Fracture criterion

8.5.4 Design summary

Table 8-18 summarises the GB/TPL design in pipeline section 1 and 2 and the restrictions towards the maximum span height and crest utilisation in pipeline section 3.

KP	Length (km)	Mitigation	Cover height above TOP (m)	Restrictions
0 – 4.5	4.5	Rock covered	0.5	N/A
4.5 – 12.0	7.5	Exposed	N/A	Max span height = 0.3
12.0 – 19.0	7.0	Exposed	N/A	See Table 8-16
19.0 – 21.0	2.0	Exposed	N/A	
21.0 – 26.0	5.0	Exposed	N/A	
26.0 – 67.5	41.5	Exposed/rock covered	Various	See pipeline section 3 KP 12.0 – 19.0
67.5 – 74.9	7.4	Exposed	N/A	N/A
74.9 – 79.2	4.3	Rock covered	0.0	N/A
79.2 – 80.4 (shore)	1.2	Buried	N/A	N/A

Table 8-18 Summary of global buckling solution

9 Geotechnical engineering

9.1 Pipe-soil interaction assessment

The pipe-soil interaction assessment comprises the following:

- Pipeline penetration
- Axial and lateral resistances for a pipeline on SAND/CLAY/ROCK
- Uplift resistance for a pipeline trenched and backfilled with SAND/CLAY
- Uplift resistance for a pipeline covered with ROCK
- Soil stiffness for SAND/CLAY

The pipe-soil interaction assessment provides input to other design disciplines, which involves axial and lateral pipeline displacement. The primary use is thus in expansion and buckling design (the trawl pull-over load case) as well as installation in curves (curve stability).

9.1.1 Soil conditions

The soil information used for the Balticconnector pipeline is extracted from the geotechnical site investigation survey report, Ref. /21/.

A summarised description of the different soil conditions and their location along the pipeline route are presented in Table 9-1. The soil mainly consists of soft clay with outcropping bedrock. Sand is found close to the Estonian shore, approximately from KP 76 to KP 82.

KP range [km]	Soil description
0.0 - 11.5	CLAY
11.5 – 14.1	CLAY / BEDROCK
14.1 - 16.7	CLAY
16.7 - 27.4	BEDROCK
27.4 – 37.6	CLAY
37.6 – 55.4	CLAY / BEDROCK
55.4 – 74.8	CLAY
74.8 – 80.4	SAND

Table 9-1 Top soil description, based on *Seabed Survey Alignment Sheets*, Ref. /32/, and MMT Balticconnector Seabed Survey Report, Ref. /21/

General soil properties for SAND, CLAY and ROCK used in the assessment are according to the *Design Basis*, Ref. /34/.

Clay sensitivity (used to determine the remoulded shear strength) is assumed to be $s_t = 1.5$ based on project experience.

9.1.2 Methodology

The soil resistances are calculated for three different pipe contents:

- Air
- Seawater
- Product (gas)

And three different coating configurations:

- Medium: KP 00.000 - 19.350: 55 mm CWC @ 3400 kg/m^3
- Heavy: KP 19.350 - 26.000: 80 mm CWC @ 3400 kg/m^3
- Light: KP 26.000 - 80.392: 45 mm CWC @ 3400 kg/m^3

Pipeline penetration

The pipe-soil response for SAND and CLAY is highly dependent on the pipeline penetration into the seabed. However, as this parameter is difficult to assess experience data from other projects is used.

The three estimates of pipeline penetrations are determined as follows:

- Lower bound (LB)
  Minimum of experience-data and empirical calculated penetration.

- Best estimate (BE)
  Maximum of: BE experience-data and the average of LB and UB experience-data.

- Upper bound (UB)
  Taken as UB experience-data.

For ROCK, no penetration is assumed.

Soil stiffness

The static vertical soil stiffness is found in *Table 7-5 and 7-6 in DNV-RP-F105*, Ref. /5/ for loose SAND and very soft CLAY respectively.

Axial pipe-soil resistance

The axial soil response for a pipeline resting on SAND or ROCK is assumed to be pure Coulomb friction, where the force-displacement graph shows no marked peak, but is constant once full friction has been mobilised.

The upper and lower bound axial resistance for SAND is determined using upper and lower bound soil parameters, respectively. For ROCK the upper and lower bound resistances are determined by assigning a 20% variation to the best estimate.

For CLAY the axial soil resistance is, to a large extent, dependent on the pipe-soil contact surface. The axial resistance curve exhibits a marked peak ($F_{A,\,max}$) due to penetration/break-out response. Post break-out the pipeline-soil response will decrease to the residual resistance ($F_{A,\,res}$). The force-displacement relation is assumed piecewise linear defined by four points:

- 80 % of $F_{A,\,max}$ at an axial displacement of 1% *OD*
- 100 % of $F_{A,\,max}$ at an axial displacement of 4% *OD*
- 55 % of $F_{A,\,max}$ at an axial displacement of 40% *OD*
- $F_{A,\,res}$ at an axial displacement of 135% *OD*

The upper and lower bound soil resistance for CLAY is determined based on uncertainty treatment as defined by SAFEBUCK JIP, Ref. /26/.

Lateral pipe-soil resistance

The lateral resistance for both SAND and CLAY exhibits a marked peak due to the break-out response ($F_{L,\,max}$) followed by the residual resistance ($F_{L,\,res}$). Common for both soil types is that the break-out resistance is dependent on the initial penetration whereas the residual resistance is more dependent the submerged pipeline weight.

In both cases the break-out resistance is assumed to comprise two terms; one being the passive term due to the mound of soil being pushed ahead of the pipeline, the other being a frictional.

For SAND the peak resistance is assumed to occur at a lateral displacement of 0.5*OD* whereas the displacement required to mobilise the residual resistance is dependent on the initial penetration.

The upper and lower bound lateral resistance for SAND is determined using upper and lower bound soil parameters, respectively.

For CLAY the force-displacement relation is assumed piecewise linear defined by five points:

- 83 % of $F_{L,\,max}$ at a lateral displacement of 5% *OD*
- 100 % of $F_{L,\,max}$ at a lateral displacement of 15% *OD*
- 70 % of $F_{L,\,max}$ at a lateral displacement of 30% *OD*
- 44 % of $F_{L,\,max}$ at a lateral displacement of 120% *OD*
- $F_{L,\,res}$ at a lateral displacement of 240 % *OD*

The upper and lower bound soil resistance for CLAY is determined based on uncertainty treatment as defined by SAFEBUCK JIP, Ref. /26/.

The lateral soil response for a pipeline resting on ROCK is assumed to be pure Coulomb friction, where the force-displacement graph shows no marked peak, but is constant once full friction has been mobilised.

The upper and lower bound axial resistances for ROCK are determined by assigning a 20% variation to the best estimate.

9.1.3 Results

The following nine cases are considered in design:

	Case no.								
	1	2	3	4	5	6	7	8	9
Pipe configuration 1)	Medium	Medium	Medium	Heavy	Heavy	Heavy	Light	Light	Light
Content	Air	Gas	Water	Air	Gas	Water	Air	Gas	Water
Note: 1) See section 9.1.2 for definition of pipeline configuration									

Table 9-2 Design cases

Pipeline penetration

Pipeline penetrations are estimated from as laid surveys of the Polarled (CLAY) and Gjøa (SAND) pipelines in the Norwegian sector. The soil conditions for these pipelines are similar to those of the Balticconnector pipeline. The pipeline penetration values are presented in Table 9-3.

Relative pipeline penetration, *z/D* [%]			
Seabed material	LB	BE	UB
SAND	3	10	25
CLAY	11	20	32

Table 9-3 Design pipeline penetration

Static vertical soil stiffness

The static soil stiffness's are presented in Table 9-4.

Soil type	Static vertical soil stiffness [kN/m/m]		
	LB	BE	UB
SAND (loose)	200 1)	250	300 1)
CLAY (very soft)	50	75 2)	100
Note: 1) For SAND *DNV-RP-F105* only presents a BE value. A variation of. ±20% has been adopted to define the LB and UB values. 2) For CLAY *DNV-RP-F105* presents a range which is assumed to cover LB-UB. The BE value is thus assumed to be the average of the LB and UB values.			

Table 9-4 Static vertical soil stiffness

Note that these values are standard values for the given soil type as presented in *Table 7-5 and 7-6 in DNV-RP-F105,* Ref. /5/.

Axial pipe-soil resistances

All results are presented in Appendix X.

Note that for CLAY the axial soil resistance is independent of the submerged weight of the pipe. This is because the pipe resistance is determined from estimated penetrations, without considering weight variations.

For ROCK the axial and lateral soil response is assumed to be alike, since no penetration into the rock is foreseen.

Lateral pipe-soil resistances
All results are presented in Appendix X.

The lateral resistance for a pipeline resting on ROCK is assumed to be similar to the axial resistance.

Uplift resistance
All results are presented in Appendix X.

The uplift resistance for a trenched pipeline backfilled with CLAY is determined assuming the trenching method is ploughing (this only affects the upward displacement required to mobilise the uplift resistance).

The results for the uplift resistance for a pipeline buried in gravel are BE as only one set of rock parameters are used, Ref. /34/. If LB or UB are required a variation of ±20% can be applied to BE.

10 Landfall design

This section describes the design for the Balticconnector pipeline in the landfall areas on the Fjusö peninsula in Finland (Inkoo landfall) and in Lahepere Bay near Paldiski in Estonia (Paldiski landfall). The landfall areas are defined from the first dry weld (the battery limit between the offshore scope and the onshore scope) to approximately 0.5 km offshore from the first dry weld (FDW) at the Inkoo landfall and 1 km from the FDW at the Paldiski landfall.

The objective is to define the landfall design ensuring the integrity of the Balticconnector pipeline. The FEED scope of work comprises:

- Definition of first dry weld.
- Pull-in method and assessment.
- Pull-in trench profile and landfall approach design.
- Identification of work site areas for pull-in operation.
- Protection against grounding vessels and dragged anchors.

The onshore pipeline sections between the FDWs and the compressor stations in Finland and Estonia are part of the onshore pipeline design scope. Note, however, that due to the heavy wall linepipe required to resist the offshore pipeline design pressure, any change of direction can probably not be accommodated by on-site cold bending, but is likely to require a custom made hot formed induction bend.

Design challenges in the landfall design are:

- Construction of onshore trench at the Inkoo landfall, which will reach depth around 8 m.
- Hard bedrock at the Inkoo landfall.
- High density of boulders along the area for the pull-in trench at the Paldiski landfall.
- Narrow beach in combination with steep cliff at the Paldiski landfall.
- Location of the Paldiski landfall in close proximity of the Tallinn-Paldiski highway 8.
- No soil data available for both landfall locations at this stage of the project.
- Cofferdam design at the Paldiski landfall approach as the presence and depth of bedrock both on- and offshore is unknown at this stage of the project.
- Environmental impact during construction works.

Design of winch and anchor foundation, pull-in sheave arrangement together with assessment of engineering methods for preparation of the trench and installation of post-lay is not included in the FEED phase.

10.1 Finnish landfall

10.1.1 General

The landing point for the Balticconnector pipeline is located at the south east shore of the Fjusö peninsula, east of an existing oil offloading jetty. The landfall coordinate is given in Table 10-1. The Inkoo landfall location is in a wooded area which is not developed. The Balticconnector approach to the Fjusö peninsula is shown in Figure 10-1 and Figure 10-2. The offshore pipeline begins with a 900 m straight section from the landing point to ensure a straight pull-in from the pipe laying vessel and stability for the first lay curve.

Landfall	Easting	Northing
Inkoo landfall	330,769.00	6,656,682.00

Table 10-1 Inkoo landfall coordinates, Ref. /39/

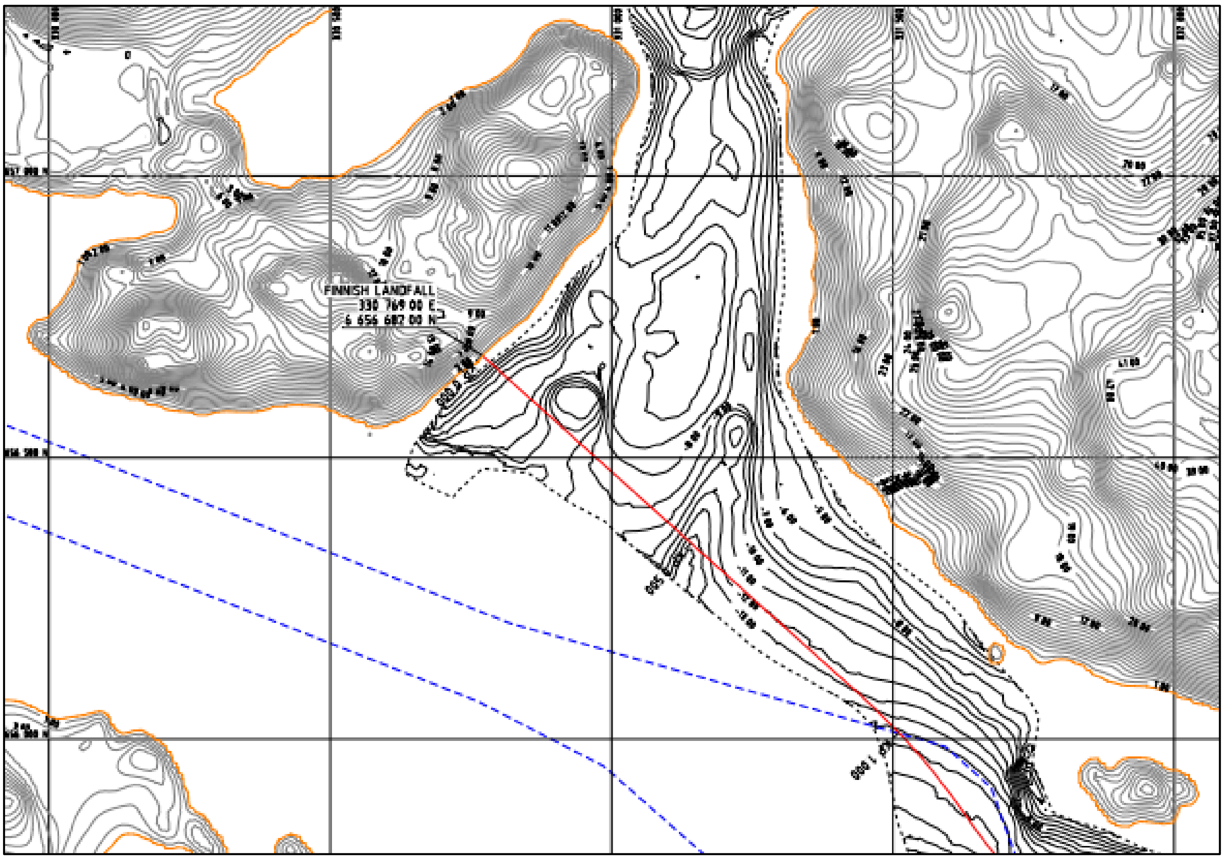


Figure 10-1 Inkoo landfall approach, Ref. /39/

Figure 10-2 Aerial photo of the Inkoo landfall location at Fjusö

The pipeline approach profile towards shore is shown in Figure 10-3. The first 500 m of the route have a water depth at around 10 m from where the seabed raises from a water depth at 7 m at around 60 m from shore to water level at the landing point. The first 60 m of the Balticconnector route have a seabed slope around 1:8, as seen in Figure 10-3.

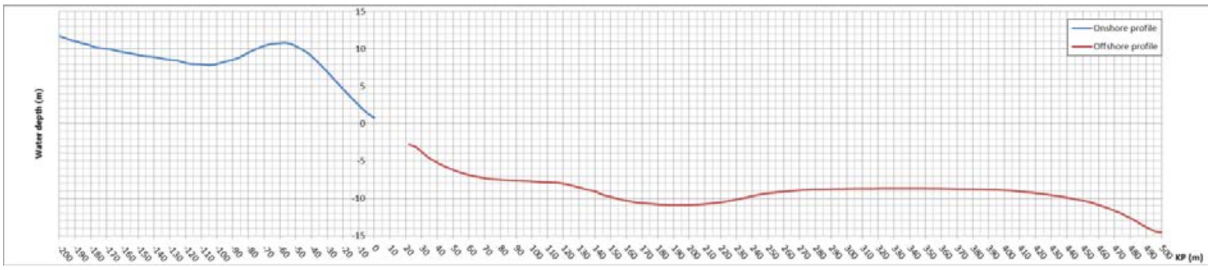


Figure 10-3 Fjusö shore approach profile at Inkoo landfall

Geotechnical information of the Inkoo landfall site is at this state very limited. However, the seabed along the shore approach is expected to consist of soft clay, Ref. /32/. Along the coastline in the area of the landing point visual observation indicates bedrock in combination with rocky terrain with a thin organic top layer. The coastline is dominated by boulders ranging from 100-700 mm in diameter with an average estimated size in the order of 300 mm, as seen in pictures taken at the landfall location, refer to Figure 10-4. It is assumed that this configuration of the shoreline is continued into the water until a clay dominated seabed is reached.

Figure 10-4 Photo from Fjusö – Coastline near landfall location

The coastline at the landing point of the Balticconnector pipeline has a very narrow beach (3-5 m wide) with a slope from the shoreline and inland at 1:5, as seen in Figure 10-3. The surface steepness inland from the landing point of the Balticconnector is continued from the shoreline and reaches a maximum elevation at 11 m around 50 m from the landing point. 80-90 m from the Inkoo landfall location relative even terrain is found at the centre of Fjusö. The area at Fjusö near the shoreline is densely packed with trees and requires clear-cutting prior to the worksite setup for the pull-in equipment (winch, back anchors, etc.) needed for the pull-in operation.

The worksite for the pull-in operation will be placed near the centre of Fjusö, 90 m from the Inkoo landing point. The pull-in will be performed in a pre-dredged open trench or alternatively in a tunnel created using horizontal directional drilling (HDD). The trench (or HDD borehole) will start at the worksite from where the pulling operation will take place. The exit location of the trench will be approximately 45 m from the shoreline at 5 m of water depth. The bottom profile of the trench is showed in Figure 10-5. A profile for a HDD borehole will look somewhere similar with variation to entry and exit angles.

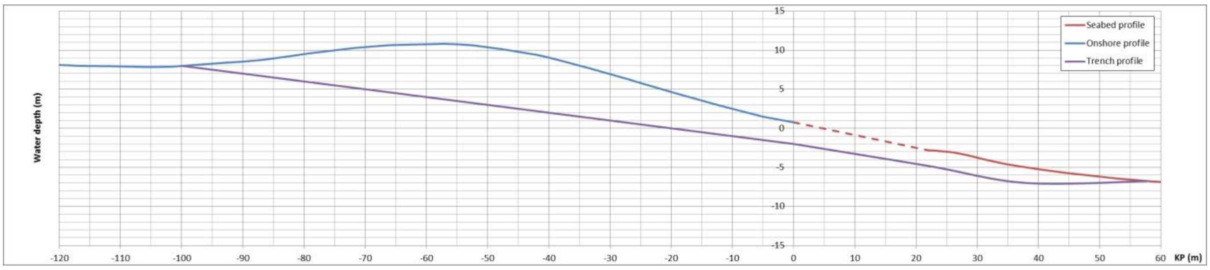


Figure 10-5 Trench bottom profile for the Inkoo landfall

The Balticconnector pipeline will be pulled by linear winches placed at the worksite on Fjusö. A guide wire will be established between the worksite to the lay barge. The guide wire will be connected to the pull-in wire on the lay barge, which then can be pulled on shore and connected to the linear winch. The onshore pull-in winch will then pull the pipeline towards shore and through the surf zone in a pre-dredged trench as the lay barge constructs and feeds the pipeline into the offshore trench.

The first dry weld (FDW) will be located at a position at the worksite close to the entry to the pipeline pull-in trench. The FDW will mark the intersection between the offshore and onshore codes and the transition from offshore to onshore pipeline.

10.1.2 Landfall construction activities

The pull-in operation of the Balticconnector pipeline at the Inkoo landfall will include a list of activities related to the establishment of the worksite for the pull-in operation and pipeline pull-in trench. The Inkoo landfall construction for the Balticconnector pipeline will include the following main activities;

- Setup of construction site including access roads. Levelling of worksite will include removal of bedrock at the worksite. Access road to worksite will include a temporary crossing of the oil pipeline routed parallel with the Fjusö access road.
- Establish of pipeline pull-in trench. The pull-in trench will be constructed by blasting and removal of rock and dredging of clay seabed offshore. Then trench will be established between the worksite and towards an exit point offshore at a water depth of 5 metres located around KP 0.045.
- Installation of bottom gravel layer in trench to avoid direct pull on bedrock.
- Construction pull-in winch foundation and hold-back anchoring.
- Installation of pull-in wires between the pull-in winch and installation vessel pick-up position.
- Position of installation vessel for pull-in operation and pick-up of the pull-in wire.
- Pull-in operation. Installation vessel will weld line pipe line simultaneous with the pull-in operation from shore.
- Locking of pipeline when pull-in head has reached its target position.
- Tie-in to onshore pipeline section at target location.
- Backfilling of pipeline pull-in trench, both onshore and offshore.
- Installation of pipeline protection rock cover until 10 metres of water depth.
- Demobilisation and re-establishment of construction site and landfall area to its former state, or as mandated by the authorities.

Alternative for the HDD option the following construction activities will be required:

- Setting up rig site for HDD drilling operation. Will include drilling rig, drill pipe storage, recycling pumps for drilling mud, mud tank, power units, entry pit, etc. Activities related to construction of a HDD borehole will include:
- Establishment of worksite for HDD drilling rig.
- Drilling of HDD pilot hole.
- Pull back of reamer head until desired HDD borehole diameter is achieved.

The following challenges will have to be considered for the Balticconnector pipeline at the Inkoo landfall;

- Removal of submerge bedrock when constructing the open offshore trench
- Hard bedrock at landfall location. Requirements for rock removal by blasting, both on- and offshore. May also challenge construction related to HDD.
- Crossing of onshore oil pipeline to gain worksite access.

10.1.3 Geology at landfall location

The geology of the landfall location is not extensively studied at this point, and supplemental explorations will be required to verify the landfall methodology and feasibility prior to execution of the work. However, publically available data for the site, and visual observation of the landfall conditions, indicate that the site area will be covered with a thin veneer of recent sediments consisting of humus and organic material supporting the forest cover, overlying glacial sediments, over Proterozoic rock strata. Based on visual observation, bedrock will be expected at or near the ground surface at the landfall site.

Rock at the landfall site is mapped by the Geological Survey of Finland, Ref. /27/. The rock is characterised as a supercrustal sequence of mica gneiss and mica schist, dating to approximately 1.9 billion years ago. The rock strata are considered to form a part of the accretionary arc complex of southern Finland, as part of the Sweco-Fennian Domain. Also observed in the area are microcline granites of younger age (approximately 1.4 billion years) but these units are likely to outcrop north of the site based on geological mapping data.

The gneiss and schist formations present at the Inkoo landfall site are metamorphic rock of great age. Both gneiss and schist are examples of high grade metamorphic rocks common worldwide, exhibiting a foliated (layered) structure.

No major faults are mapped immediately in the vicinity of the site, but this could be verified by site-specific mapping of the rock. Voids or caves are not considered likely to be present.

The rock type at the Inkoo landfall site is considered typical of rock on the southern coast of Finland in general. The rock will be expected to be Grade I, or fresh (e.g. no weathering degradation), due to the effects of comparatively recent glaciation.

The rock may be expected to be intact aside from joints or fractures. Likely characteristics of the rock will include extremely high hardness with high seismic refraction velocity. The rock will not be excavable by ripping or other standard digging methodology, and any rock excavation should be considered to require blasting.

10.1.4 Temporary worksite and access road

The landfall worksite will be located at the centre of Fjusö at around 80-90 m from the FDW of the Balticconnector and 150-200 m inland. The worksite is located in an area with relatively flat ground to reduce the requirements for levelling when setting up the worksite.

The preparation work for establishment of the worksite will include the following activities;

- Clearcutting of the worksite area
- Levelling and construction of platform for construction work and pull-in equipment
- Construction of foundation and back hold anchoring of the pull-in winch
- Temporary access road, incl. temporary crossing of the oil pipeline routed along the access road to the oil jetty on Fjusö
- Construction of pull-in trench / HDD borehole

The levelling of the worksite is expected to include some excavation/blasting of bedrock to obtain a level worksite in combination with layers of rock gravel.

The temporary worksite is estimated to be 50 m wide and 150-200 m long. The worksite size will be determined by, among others, the following factors:

- Pull-in winch foundation and anchoring size
- Equipment for the pull-in arrangement; winch, power unit, anchor, drum wheel for pull-in wire, etc.
- Work sheds
- Turning/parking area for verticals, trucks and other heavy equipment
- Storing of heavy equipment
- Temporary storing of excavated rock and top soil
- Or drill rig equipment for HDD construction

An overview sketch of how the worksite, winch arrangement and temporary access road could look like is shown in Figure 10-6.

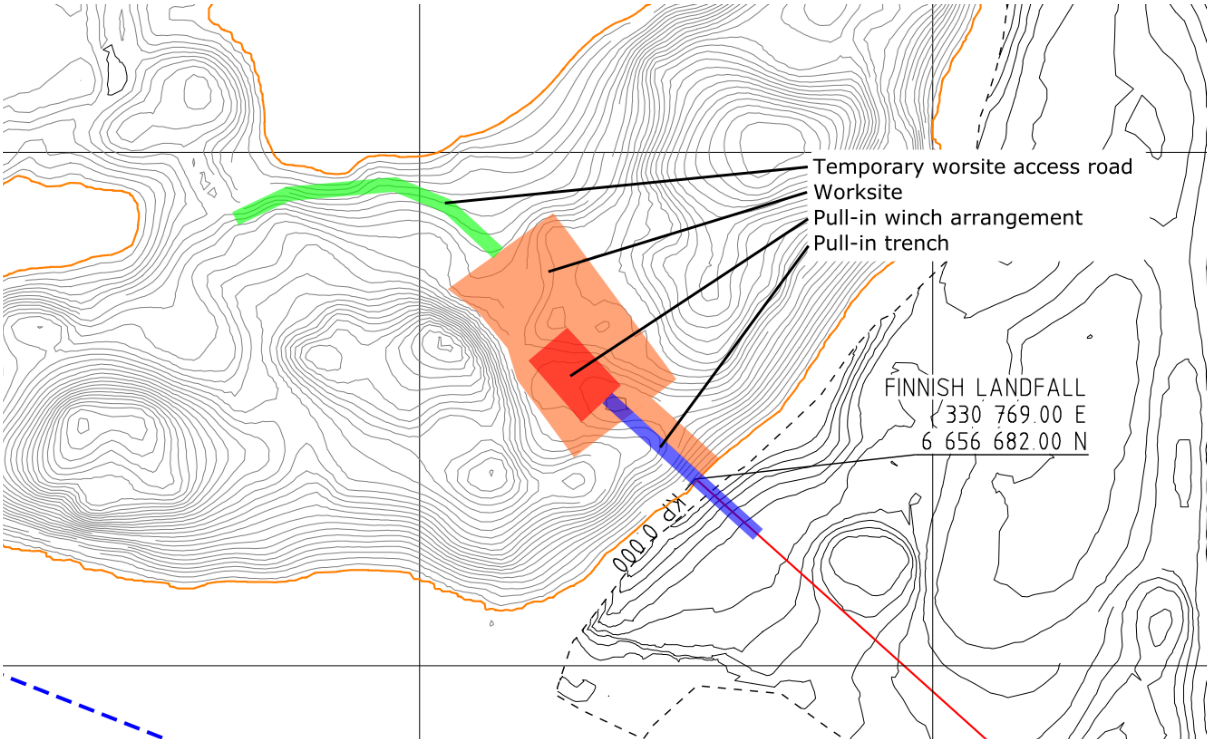


Figure 10-6 Sketch overview of the worksite at the Inkoo landfall

A temporary access road to the worksite will be routed from the existing access road to Fjusö. The worksite access road will start from the existing road near the isthmus which connects Fjusö to the main land and toward the worksite from the north side of the peninsular. The temporary worksite access road requires the construction of a temporary crossing of the existing oil pipeline routed parallel with the existing access road to Fjusö.

10.1.5 Winch foundation and anchoring

The pull-in arrangement needs an even foundation of either concrete or gravel depending on the winch to be chosen for the pull-in operation. It is assumed that the bedrock at the location has sufficient strength for anchoring of the pull-in winch with the use of rock anchors. Alternatively, a concrete anchor block can be used as pull-in winch anchoring.

The foundation and anchoring of the pull-in winch should be evaluated during the detailed design to find the best suited solution based on the pull-in arrangement chosen by the contractor and detailed investigation of soil and rock conditions at the location.

10.1.6 Inkoo landfall approach

The Balticconnector pipeline will be pulled in to Fjusö in a prepared open cut trench or alternatively a HDD borehole. Both methods are considered feasible for the shore approach at the Inkoo landfall site. Each method possesses various advantages and disadvantages, refer to Table 10-2. Both the open trench and HDD approach will follow a profile similar to the one given for the open cut trench in Figure 10-5.

Both landfall design methods are described and compared in this section.

Method	Advantage	Disadvantage
Trench Blasting	• Blasting is standard in Finland and being performed on a regular basis. • If pipeline is damaged or requires repair at some time in future, trench can be re-opened and pipeline fixed.	• Required trench depth may create challenges (noted to be partly up to 10 m deep due to surface topography at landfall) • Trench width may increase due to equipment access issues. At minimum 3 m shall be required, but potentially 5 m must be utilised in deep areas. • Trench restoration required, but trench in bedrock will be visible after construction when refilled.
Horizontal Directional Drilling (HDD)	• No trench restoration required, and landscape remains effectively untouched • Landside work area shall be limited to the drill platform and area for pulling equipment. • Minimal excavation spoil • Borehole only needs to be minimal size to fit pipeline	• May be challenging when voids exist in rock; however, local contractors indicate this can be overcome. Additionally the rock type at the landfall likely will not have appreciable voids. • Likely several stages of back-reaming are required due to the hard rock type found at the landfall location. • Casing may be required at entry and possibly exit to control release of drilling fluids into the environment. • Construction of drilled hole nearly 0.7 m diameter in very hard crystalline rock may be problematic; this should be discussed with the Contractors.

Table 10-2 Summary of advantages and disadvantages for Inkoo landfall approaches

Open trench

The open trench design will include:

- An offshore trench height of minimum 1.7 m to ensure a distance from TOP to seabed at 1.0 m.
- Maximum trench depth will be up to 8 m at its deepest section.
- A bottom width at 3.0 m. For deep sections (onshore) trench width may increase due to equipment access issues.
- The trench will be prepared from shore to a water depth of 5 m.
- A trench bottom rock cover for protecting of the pipeline under pull-in operation.
- Construction of trench is expected to be done by blasting.

The offshore part of the pull-in trench will extend approximately 45 m from the shoreline until a water depth at 5 m is reached. A theoretical cross-section sketch of the trench is shown in Figure 10-7. The trench will be backfilled after installation of the pipeline to protect the pipeline against any threat from ice ridges, Ref. /36/, and grounding pleasure boats. For further protection against grounding vessels and anchoring, the pipeline at landfall will be provided with 1.0 m TOP rock cover until 10 m water depth. The rock cover of the pipeline will be extended from KP 0.04 to KP 0.45 until a water depth greater than 10 m is reached.

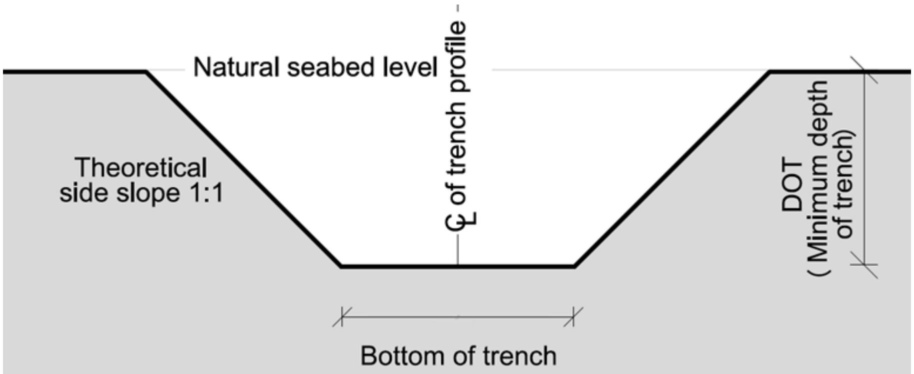


Figure 10-7 Pipeline pull-in trench cross-section sketch

The construction of the trench will include dredging of the top seabed layers, which are mainly consisting of a combination of a sand/gravel layer in the top layers and soft to firm clay down to bedrock, Ref. /32/. Blasting of subsea rock is expected as part of the construction of the offshore part of the trench.

The trench will extend from the coastline to the worksite where the target point for the pull-in head and the position of the first dry weld are located. The onshore construction of the trench will include blasting of bedrock to level out the landfall approach profile.

A rock bedding shall be installed in the trench bottom prior to the pull-in operation at section with bedrock. The rock cover will act as a protection to avoid direct pull-in on bed rock.

The construction of an open trench is considered as a proven and cost effective method. Blasting is standard method for rock removal in Finland and is being performed on a regular basis. Despite this, an open trench design will leave a permanent scar in the landscape and trench needs refilling after pipeline construction, and restoration of the landfall site may require re-filling of the trench.

Horizontal Directional Drilling (HDD) approach

Alternatively to a cut-open trench, horizontal directional drilling from the worksite to a position of 50 m from landfall is considered as a solid alternative to the open trench. The benefits of a HDD borehole for pull-in of the pipeline to shore are:

- Shoreline will be left untouched. Construction will not impact in the area between the HDD borehole two access points.
- Requirements for offshore trench will be omitted as the entry for the HDD will be located 5 m below the water surface.

The HDD option will induce additional requirements to the worksite area to accommodate drilling equipment needed for the HDD construction.

HDD is a technique used in hydrocarbon exploration and production whereby the drill bit at the end of the originally vertical drill string is diverted sideways to an eventually horizontal direction, which allows the tapping of a large and shallow reservoir area from a single production platform. In the context of pipeline installation, the term is used to designate an installation method in which the prefabricated pipe string is pulled through a hole in the ground made by a directed drill string. The method is illustrated schematically in Figure 10-8.

A drill rig is placed on shore, and a pilot string is inserted into the ground. The drill bit is hydraulically powered by bentonite drilling mud fed through the pilot string. The bentonite mud transports the soil away and fills the hole behind the drill head, preventing it from collapsing. The drill head is connected to the non-rotating pilot string by a swivel. The diameter of the cutting head is larger than that of the pilot string, which is encased by a drill string, and additional lengths of pilot string and drill pipe are added as the drill bit advances through the soil.

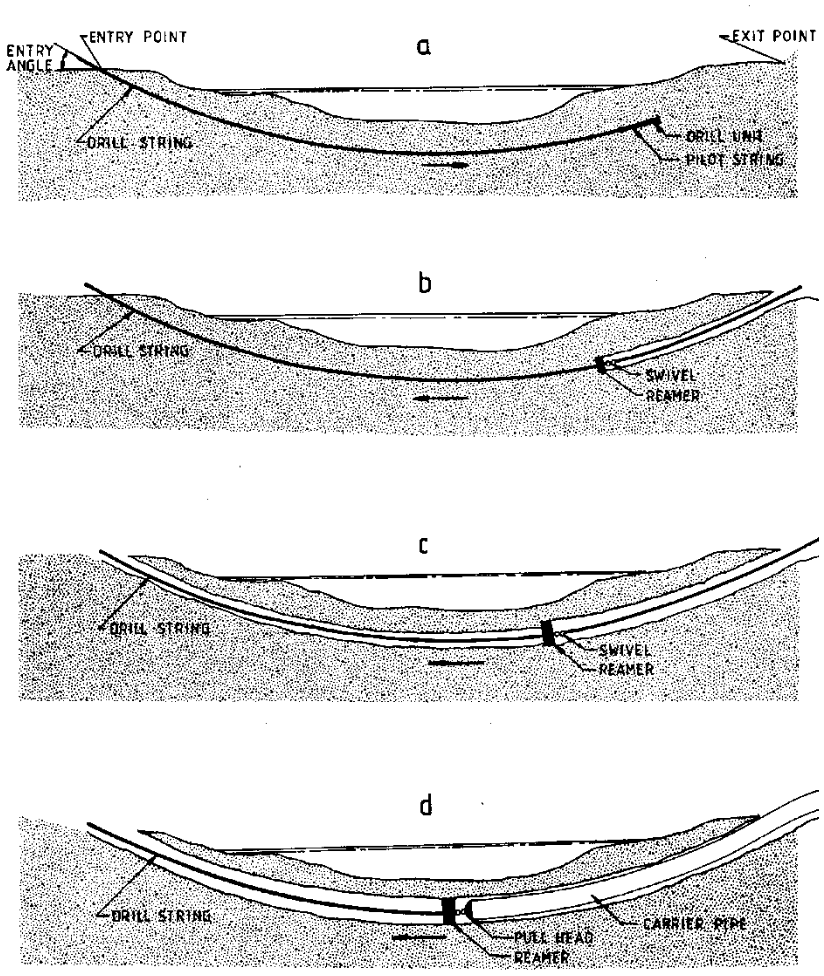


Figure 10-8 Principle of horizontal directional drilling

When the cutting head emerges at the exit point it is removed and the pilot string is withdrawn through the drill pipe. A reamer is then attached to the drill string, which is pulled back through the hole, wash pipe being attached behind the reamer. In the process the hole is enlarged by the reamer, and if necessary the process is repeated with larger reamers. When the hole is sufficiently large to accommodate the topical pipe string, it is attached to the wash pipe and pulled through the hole with a reamer attached to the pull head as a precautionary measure.

For landfall construction a pilot hole is drilled to a pre-dredged trench at the marine exit point. A crane barge with supporting equipment to handle drill pipe and hole openers (reamers) is positioned offshore, and the drill string is pulled onto the crane barge. A number of hole opening passes are carried out, until the drilled hole is sufficiently large to accommodate the topical pipeline, and the crane barge is then replaced by a laybarge.

As in the case of bottom pull, the pipeline produced on the pipelay barge is pulled into the drilled hole from the barge (shore pull).

The route of the pilot string is determined by the entry angle and by the design of the drilling unit. The cutting head includes a hydraulic motor that uses the energy of the circulating drilling mud to rotate the bit. The cutting head is mounted on a bent transition unit (bent sub), the angle of which determines the curvature of the pilot hole and forms the transition to the non-rotating pilot string. Any deviation from the prescribed path is corrected by rotating the pilot string, thus forcing the drilling unit into a revised direction. In this way the drill can be made to exit within a few metres from a target point located several kilometres away. If the exit point is unacceptable, the pilot string is withdrawn a certain distance and the route corrected.

Determination of the current position of the cutting head is accomplished by one or more of the following devices:

- A pendulum providing inclination with horizontal.
- A single shot survey camera providing tool face inclination and compass bearing.
- A plumb bob arrangement providing inclination.
- A triangulation system using sonar stations providing azimuth.

The success of the directional drilling method depends on the soil conditions, fairly uniform clay being the most appropriate; however, drilling through solid bedrock is perfectly feasible. To avoid damage to the anti-corrosion coating as the pipe string is pulled through the ground, the coating must be abrasion-resistant, and 3 - 4 mm polypropylene is a typical choice. Alternatively, a dual powder FBE system can be used, or a conventional fusion bonded epoxy coating may be protected by a layer of polymer epoxy concrete or similar. Concrete weight coating is obviously not needed, as the pipeline is deeply embedded in the soil.

Directional drilling does not involve any activities between the entry point and the exit point, and is therefore a preferred method for crossing heavily built-up or environmentally sensitive areas.

The hard rock type found at the Inkoo landfall location may be problematic for a HDD operation and could result in multiple stages of back-reaming to achieve the required tunnel diameter for the pull-in operation.

10.1.7 Seabed intervention work

The main objective is to establish a pipeline pull-in corridor for the pull-in operation of the Balticconnector pipeline at the Inkoo landfall site.

The construction of an open trench offshore will involve the following subsea intervention work;

- Dredging of sediment
- Removal of subsea rock by blasting
- Installation of rock carpet in bottom in trench to avoid pull-in on bedrock
- Backfilling of trench after installation
- Pipeline rock cover from trench exit point to a depth of 10 m

From the exit point of the pull-in trench (or exit point of HDD borehole) located at 5 m of water depth, the Balticconnector pipeline will be covered with a rock cover until a water depth of 10 m. The main purpose of the rock cover will be protection of the pipeline against grounding pleasure boats, ship anchors, etc. The rock cover shall be installed to a height of 1.0 m TOP for an estimated distance of 400 m, starting from around KP 0.04 to KP 0.45.

If HDD if chosen as the preferred method for the shore approach at the Inkoo landfall, subsea intervention work related to subsea rock removal is not considered necessary as the exit hole will be at a water depth of 5 m, from where the pipeline will be rock covered until 10 m of water depth.

10.1.8 Pull-in operation

A pull-in arrangement will be needed for the pull operation of the Balticconnector pipeline from the lay vessel to the Inkoo landfall location at Fjusö. The pipeline will be installed from the lay vessel and toward the target location of the pull-in head near the center for Fjusö around 110 m from the shoreline. The pipeline will be pulled through a pre-constructed open trench/HDD borehole which connected the offshore shore approach to the worksite at the centre of Fjusö. When the pull-in operation is completed and the pull-in head has reach its predetermined location installation of the Balticconnector pipeline will continue toward the Gulf of Finland and Estonia.

At Fjusö landfall the pull-in arrangement is expected to consist of a linear winch, winch anchoring, wire drum for storage of pill-in wire, power unit. The winch is expected to have length around 10-15 m, a width of 2-5 m and height of 1.5-2.5 m, depending on winch specification. The winch weight is estimated to be around 20-50 Ton.

The winch arrangement will include the construction of anchoring and foundation design for the winch arrangement. The anchoring can rock anchors or alternative a concrete anchor block. A wire drum is needed behind the pull-in winch. The winch foundation has to accommodate the placement of a power unit in close proximity to the winch.

The required pull-in force will depend on the following factors:

- Friction for pipe section above water level
- Friction for submerged pipe section
- Contribution from pulling on slope (height contribution/lifting of pipeline) for both above water and submerged sections
- Lay vessel hold-back tension

The required pull-in force will depend on the installation method, which will be developed by the installation contractor awarded the installation work.

During the pull-in operation, the lay vessel will maintain a hold-back tension. In the assessment of the needed pull-in force a hold-back tension at 700 kN (71 Ton) has been assumed. The pull-in design force is calculated to be 1400 kN (or 143 Ton). This is assumed to be a conservative pull-in force for the Balticconnector pipeline at the Inkoo landfall site.

10.2 Estonian landfall

10.2.1 General

The Balticconnector pipeline will terminate at the Estonian coastline at a landing point located on the east side of the Pakri peninsula near the bottom of Lahepere bay. The coordinate for the landing point is given in Table 10-3. The landing point is some 6.5 km from Paldiski. The Balticconnector approach at Estonia is shown in Figure 10-9 and Figure 10-10. The Balticconnector pipeline is routed 1,000 km in a straight section from the landing point before turning north/west out of the Lahepere bay.

Landfall	Easting	Northing
Paldiski landfall	339,933.00	6,581,949.00

Table 10-3 Paldiski landfall coordinates, Ref. /40/

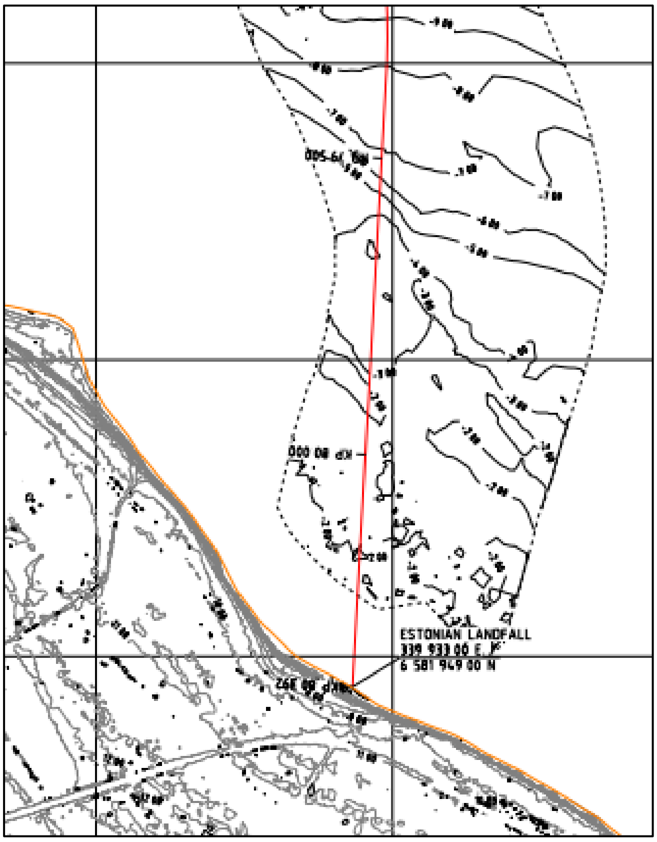


Figure 10-9 Paldiski landfall approach, Ref. /40/

Figure 10-10 Aerial photo of the Paldiski landfall location (Photo author: Mait Metsur, Aerofotod.ee)

The water depth at the entrance to the bay is around 27 m and in the major part of the route within the bay the water depth is at 10-20 m. The shore approach up to the landfall point is characterised by shallow waters and the last 800 m before landfall is in an area with a water depth of less than 5 m and with a seabed gradient around 0.5°, see Figure 10-11.

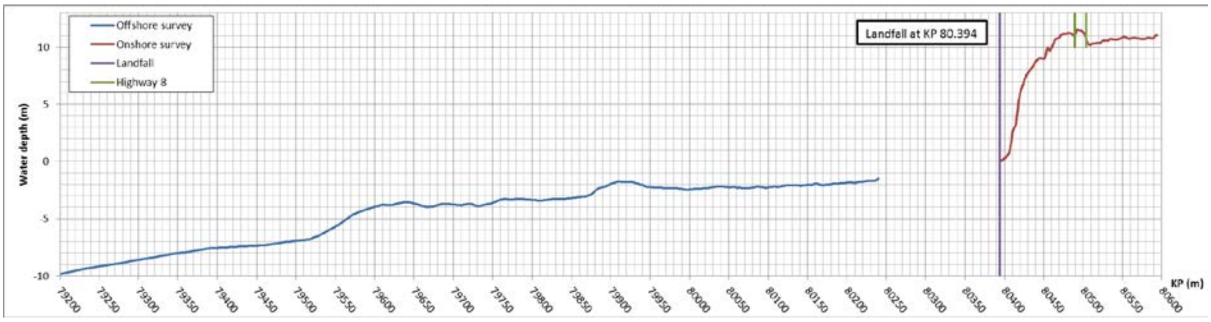


Figure 10-11 Paldiski Base case shore approach profile at Paldiski landfall

The seabed at the shore approach is characterised by a combination of glacial till with veneer of sand or combination of sand and gravel, Ref. /32/. The beach and nearshore environment at the landfall location has the presence of medium to large size sub-rounded boulders (estimated boulder size 200-1000 mm) within several metres from the coastline, as seen in the pictures in Figure 10-12 from the landfall location.

Figure 10-12 Photos from the Paldiski landfall. Beach and offshore view of the landfall location

The onshore part of the shore approach is characterised by a narrow beach (~10 m in width) followed by sloping cliff area with a slope at 1:3 from where the terrain is raised to a level about 10 m over the water level, seen in photos in Figure 10-12. Around 50 m from the shoreline the terrain flattens out. Approximately 95 m from the landfall point, the National Highway 8 (Tallinn-Paldiski highway) is located. The slope downward to the beach is covered with small trees and bushes. Similar conditions are found at the area behind the slope towards the highway. The onshore soil profile is not known at this stage, but is expected to consist of a soft top layer of sand, gravel and organic material with dolomitic limestone formations at shallow depth.

The Balticconnector pipeline will be routed from the landfall location in a one kilometre straight section going out north into the Lahepere bay, refer to Figure 10-9, to allow for a straight profile for the pull-in operation of the Balticconnector towards the Paldiski landfall.

The pipeline will be pulled towards the shore in a pre-dredged open trench. Near the shoreline a cofferdam design will protect the pipeline and the trench from backfilling in the surf zone. The pull-in arrangement will take place offshore from either the lay vessel or a lay barge. The pull-in wire will connect the pull-in winch with the pull-head via a sheave arrangement on the Paldiski landfall beach. The sheave arrangement will include ground anchoring of the sheave to achieve sufficient hold-back during the pull-in operation. The sheave arrangement has been chosen due to the lack of onshore space for a pull-in winch arrangement as a result of the near proximity of Highway 8 located 95 m from the landfall coordinate.

10.2.2 Landfall construction activities

The pull-in operation of the Balticconnector pipeline at the Paldiski landfall will include a list of activities related to the establishment of worksite and needed equipment for the pull-in operation and approach for the pipeline towards the target location of the pull-in head. The Paldiski landfall construction for the Balticconnector pipeline will include the following main activities;

- Setup construction site including access roads towards the beach area at landfall.
- Setup worksite at beach to use under construction of pull-in trench, cofferdam and sheave anchoring.
- Establishment of pipeline pull-in trench from the landing point and into the sea. The trench will be constructed from shore in a cofferdam out through the surf zone. Offshore the trench will be dredged by a dredging vessel approaching shore. The pipeline pull-in trench will include;
    - Removal of boulders in lay corridor
    - Cofferdam
    - Offshore open trench
    - Onshore trench through landing beach and cliff area
- Installation of bottom gravel layer in trench to avoid pull-in on bedrock.
- Construction pull-in sheave arrangement and back hold-back anchors.
- Position of installation vessel for pull-in operation.
- Installation of pull-in wires between and installation vessel, onshore sheave and pull-in winch to be used for the pull-in operation. The pull-in winch will be located either on the installation vessel or on a suitable pull-in barge.
- Pull-in operation. Installation vessel will weld line pipe line simultaneous with the pull-in operation.
- Locking of pipeline when pull-in head has reached its target position.
- Construction of temporary dam in the pull-in trench to drain the trench at beach level.
- Tie-in and welding of Balticconnector pipeline to a linepipe induction bend to overcome the change in trench slope between the onshore trench at beach level and through the landfall cliff.
- Backfilling of pipeline pull-in trench, both onshore and offshore.
- Installation of pipeline protection rock cover until 10 metres of water depth.
- Demobilisation and re-establishment of construction site and landfall area to its former state.

The following challenges will have to be considered for the Balticconnector pipeline at the Paldiski landfall;

- Removal of boulders in the lay corridor
- Unknown depth of bedrock at landfall location

- Anchoring of sheet piles used in the cofferdam design
- Anchoring of onshore sheave arrangement

10.2.3 Landfall pipeline trench and cofferdam design

The pull-in of the Balticconnector pipeline at the landing point on the Estonian site near Paldiski will be conducted in a pre-dredged trench that will consist of the following three structures; an offshore open trench, a cofferdam in the transition from offshore to onshore and an onshore open trench.

The offshore pull-in trench will be extending from the shoreline and until a water depth of 5 m, which has an estimated length of 830 m from the coastline. The first part of the open trench will be in a cofferdam to protect the trench from backfilling and stability of the pipeline in the surf zone. The estimated length of the cofferdam for this stage of the study is at 500 m but the precise cofferdam length should be addressed during the detailed phase. Parameters such as longshore sediment transport, location of the surf zone where wave breaking will take place and the probability for backfill of an open trench during the construction window will determine the cofferdam length. Therefore, there will be opportunity for optimising and reducing the cofferdam length during the next phase of the study.

The pull-in trench shall have a depth of minimum 1.7 m to ensure a pipeline cover of minimum 1.0 m from TOP. If bedrock is reached, an extra depth should be applied to allow for trench bottom rock carpet for protecting of the pipeline under the pull-in operation.

The construction of the pull-in trench will include offshore dredging, potential removal of bedrock both offshore and onshore and removal of boulders.

Pull-in trench

The Balticconnector will be pulled-in to shore in a pre-dredged open trench. The trench will be constructed by a dredging vessel approaching the cofferdam that protects the last section of the pull-in trench towards the shoreline and landing point. The trench should have a depth of minimum 1.7 m to ensure a cover of minimum 1.0 m of the pipeline after backfilling of the trench. If bedrock is present at the bottom of the pull-in trench, further depth should be added to allow for a bottom rock layer estimated being 0.3 to 0.5 m in thickness. The bottom rock cover will protect the pipeline from a direct pull on bedrock. The bottom profile of the pull-in trench is shown in Figure 10-13.

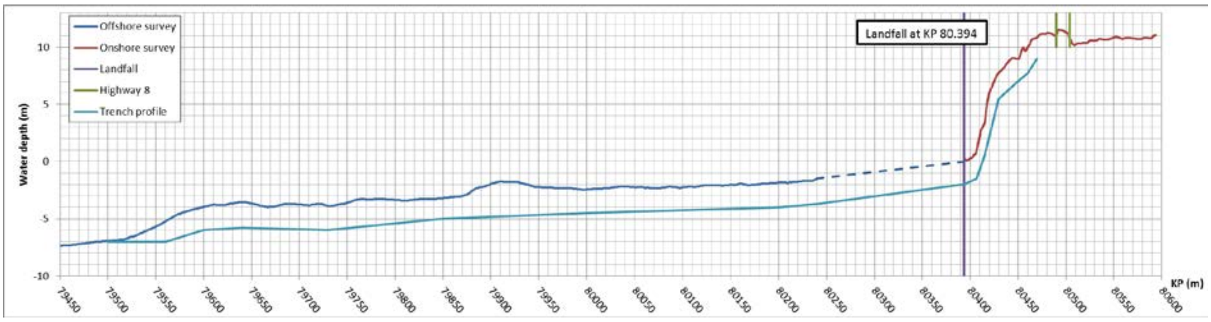


Figure 10-13 Trench bottom profile for the Paldiski landfall

The offshore environment where the pull-in operation is conducted, the prevalence of boulders is very high. It is therefore advised to inspect the trench for boulders prior to the pull-in operation. The presence of boulders can lead to pipeline integrity problems and the trench bottom profile should be cleared for boulders.

Due to limited geotechnical solid data of the shore approach, it is assumed that the trench bottom at some locations near the shoreline may go beneath bedrock level. It is expected that this will increase the work related to excavation of the trench.

Detailed design should also include investigation of any requirements for locking of the Balticconnector pipeline near the Estonian landfall. Requirements for locking may introduce the need for a trench bottom rock cover to increase pipe-soil friction.

Cofferdam design

A temporary cofferdam will be constructed at the beach of the Paldiski landfall and extend offshore to protect and prevent backfilling of the offshore trench before and during the pull-in operation. The length of the cofferdam will be determined by factors such as the wave and current climate and draft of dredging vessel. The final length of the cofferdam will be determined in the detailed design, but is estimated to be 500 m long at this stage of the design. The most optimal solution, and the aim in detailed design, will be to have a dredging vessel approaching the shoreline as close as possible to minimise the length of the cofferdam.

The dredging of the pull-in trench within the cofferdam can either be performed by excavators or diggers equipped with a long reach grab from an access platform constructed along the cofferdam side. Alternatively, to save cost, the dredging by long reach excavators can take place from a shallow water barge or vessel located along the cofferdam. This method would then be applicable beyond the cofferdam where an open trench would still be required. Otherwise the construction of the open trench up to the cofferdam mouth can be performed by a cutter suction dredger or trailer suction hopper dredgers, depending on cost and available equipment.

The cofferdam method considered most applicable to the landfall location is the braced cofferdam method, as illustrated in Figure 10-14. In this scenario the sheet piles (Larssen or similar type) are installed using vibratory hammer methods in two parallel rows until refusal or to minimum target depth. The piles are braced using a system of wide flange beams arranged as struts and walers, dimensioned according to height of wall at maximum dredging depth.

Figure 10-14 Pull-in head emerging from sea at shore pull

Lateral braces at the base of the wall are to be avoided due to potential complications with removal and possibility of inadvertent damage to the pipeline. As such, the cofferdam sheet piles must be installed to sufficient depth into the subgrade beneath dredge line so that they obtain capacity against passive failure (kickout of the pile) via passive earth pressure resistance, or they must be anchored in place.

As no soil investigation information exists to further specify in details the near-shore and onshore soil profile at this stage of the project, a soil profile as follows has been assumed:

- Near surface sand and gravel strata (characteristic of Estonian side beach deposits); over
- Paleozoic carbonate (dolomitic limestone) formations at “shallow” depth.

The anticipated soil profile at landfall is sand including some boulders over siltstone. It is preliminarily considered reasonable that the boulders are “rare” enough to allow installation of sheets into the sand subgrade and thus cofferdam can be considered. However, because thickness of the sand layer is not known, alternative installation methods are considered.

A standard cofferdam solution is not considered possible due to expectation of shallow bedrock, but several alternative options exist. The basic principle to be employed for the majority of the cofferdam options include fixing the sheet pile toes to bedrock or using internal support berms using various methods, including:

- Anchoring the toe of the cofferdam sheets using vertical rock anchors as shear elements
- External anchoring system
- Additional bracing row(s)
- Internal berm methodology

Each solution has both positive and negative aspects. The pros and cons for the various methods for sheet piling for the cofferdam construction at the Estonian landfall site are outlined in Table 10-4.

The single bracing method is not considered applicable as shallow bedrock is anticipated at the landfall location. The toe anchoring method is recommended at this stage of the project as the concept for sheet piling in the cofferdam design. Depending on future soil investigations of the landfall site, the different methods given in Table 10-4 should be re-evaluated in detailed design. The primary challenge is considered to be potential boulder presence in the subgrade, unknown depth to bedrock and the lack of detailed soil information. It can be expected that the design will require the need for excavation of bedrock if reached before the design depth of the trench.

Review of sheet piling methods applicable for the Paldiski landfall approach		
Method	**Pro**	**Con**
Single Bracing (Base Case)	• Simple • Fast • Construction entirely from dry side • Cheapest Solution	• Applies only to deep sandy areas • Cannot work with shallow bedrock
Toe Anchoring (Rock Anchor)	• Simple solution • Requires few extra anchors • Can be constructed fully from dry side	• Anchors remain in place after construction (environmental permit impact)
External Anchor (Tie Back)	• Functional • Can be constructed fully from the dry side	• Expensive • Time consuming • Anchor testing • Anchors remain in place after • construction (environmental permit impact)
Multiple Brace	• Simple • Fast • Proven method	• Will require underwater placement of bracing • Slightly more time consuming
Internal Berm	• Simple • Fast • Construction entirely from dry side	• Higher dredging amounts • Applies only if internal sediments are “good quality” sand • Much larger space needed

Table 10-4 Comparison of methods for sheet piling for the Estonian landfall site

As no specific soil profile is available at this state of the project, it is recommended to obtain more soil information on the landfall areas to consider possible solutions or limitations for the methodology. It is advised that the onshore and near shore soil investigations are completed prior to detailed design so that the methodology of landfall cofferdam construction can be verified as a suitable alternative, and that the cofferdam can be dimensioned according to actual conditions.

10.2.4 Temporary worksite

For the Paldiski landfall a temporary worksite needs to be established at or near the beach. The worksite will accommodate equipment needed for the construction of a pipeline pull-in trench, anchoring of the pull-in arrangement. The worksite is placed in an area worthy of preservation and requirement related to reestablishment of the area after pipeline installation can be expected, Ref. /29/.

The preparation work for establishment of the worksite will include the following activities;

- Establishment of an access road to the beach
- Construction of worksite at beach level
- Construction of hold-back anchoring for pull-in sheave
- Construction of pipeline trench and construction machinery access road along trench
- Allocations of space for temporary storing of excavated top soil from the construction of worksite and pipeline trench

The worksite area can be expected to be around 50 m wide and 100 m long at the beach, but will depend on the equipment needed for the construction of the trench and hold-back anchors. Additional space for construction of the onshore trench through the near shore cliff should also be taken into account, and can be placed on state-owned land. A sketch of how the layout of the worksite could look like is shown in Figure 10-15.

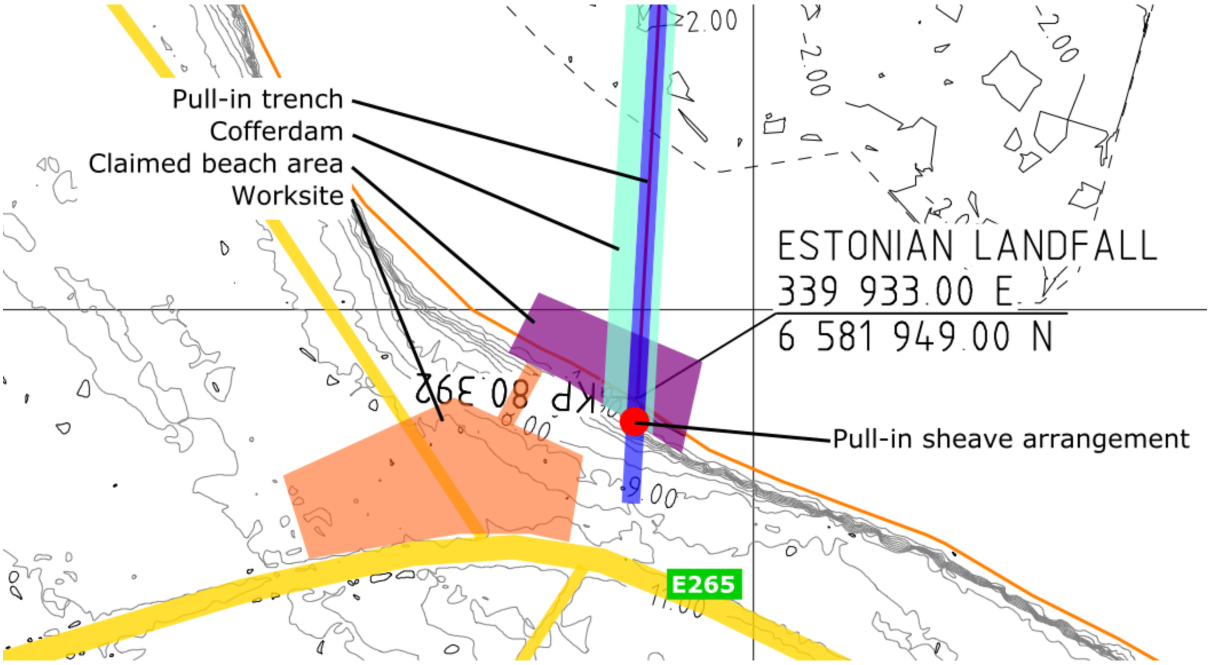


Figure 10-15 Sketch overview of the worksite at the Paldiski landfall

10.2.5 Seabed intervention work

The main objective is to establish a pipeline pull-in corridor for the pull-in of the Balticconnector pipeline.

The constructions of an open trench for pull-in involve the following subsea intervention work;

- Removal of boulders in lay corridor
- Dredging of offshore trench
- Removal of bedrock (limestone) may be needed to achieve the required trench depth
- Construction of cofferdam near shore. Construction will start from shoreline. Depending on soil conditions, demands for anchoring of sheet piles may be required.
- Backfilling of trench after installation
- Pipeline rock cover from trench exit point to a depth of 10 m

10.2.6 Pull-in operation

The Balticconnector will be installed from Finland towards the Estonian coast. At a given lay-down position off the Estonian coast, the lay vessel will lay down the Balticconnector pipeline and reposition for pull-in and installation from the Paldiski landfall. When the installation vessel reaches the lay-down site of the pipeline installed from the Finnish coast, the lay vessel or other suitable vessel will perform a Davit lift operation, refer to section 12.1.8, to connect and complete the Balticconnector pipeline.

A bottom pull will be the chosen method for the pull-in of the Balticconnector at the Paldiski landfall. Due to the limited space between the landing point and the Tallinn-Paldiski highway in combination with a narrow beach and change in terrain, the pull-in winch will be placed offshore.

To reduce construction work related to the establishment of an offshore pull-in trench onshore, the pull-in arrangement will include a sheave at the beach near the landfall point to connect the pull-in head and the pull-in winch via the pull-in wire. The sheave will then guide the pull-in head towards the target point of the pull-in head. The sheave arrangement will be anchored into the bedrock via rock anchors to achieve sufficient back hold during the pull-in operation. The construction including size and required number of rock anchors will depend on anchoring properties and rock quality at location. Detailed soil investigation is recommended to be obtained prior to detailed design phase for verifications of properties, dimensions and required amount of rock anchors required.

Depending on the lay vessel, the pull can be done by the A&R-winch on board the lay vessel if meeting the requirement for the pull-in force. Alternatively, a linear winch can be placed on the lay vessel or on a suitable barge.

Because of the limited space available on the beach at the Paldiski landfall area, the target point of the pull-in is expected to be submerged. Furthermore, a hot bend pipe piece will be required to accommodate the change in the trench slope between the beach and cliff area. To be able to weld the offshore pipeline to the linepipe induction bend going up the cliff area, a temporary dam in the pull-in trench is needed in order to drain the pull-in trench during the welding process. The different stages of the pull-in operation in the pull-in trench at the beach are sketched in Figure 10-16. Note that the location of the pipelay vessel approximately 1000 m from shore, which is where the winch will be located, is not included in the figure as the scale would eliminate the clarity of the sketch.

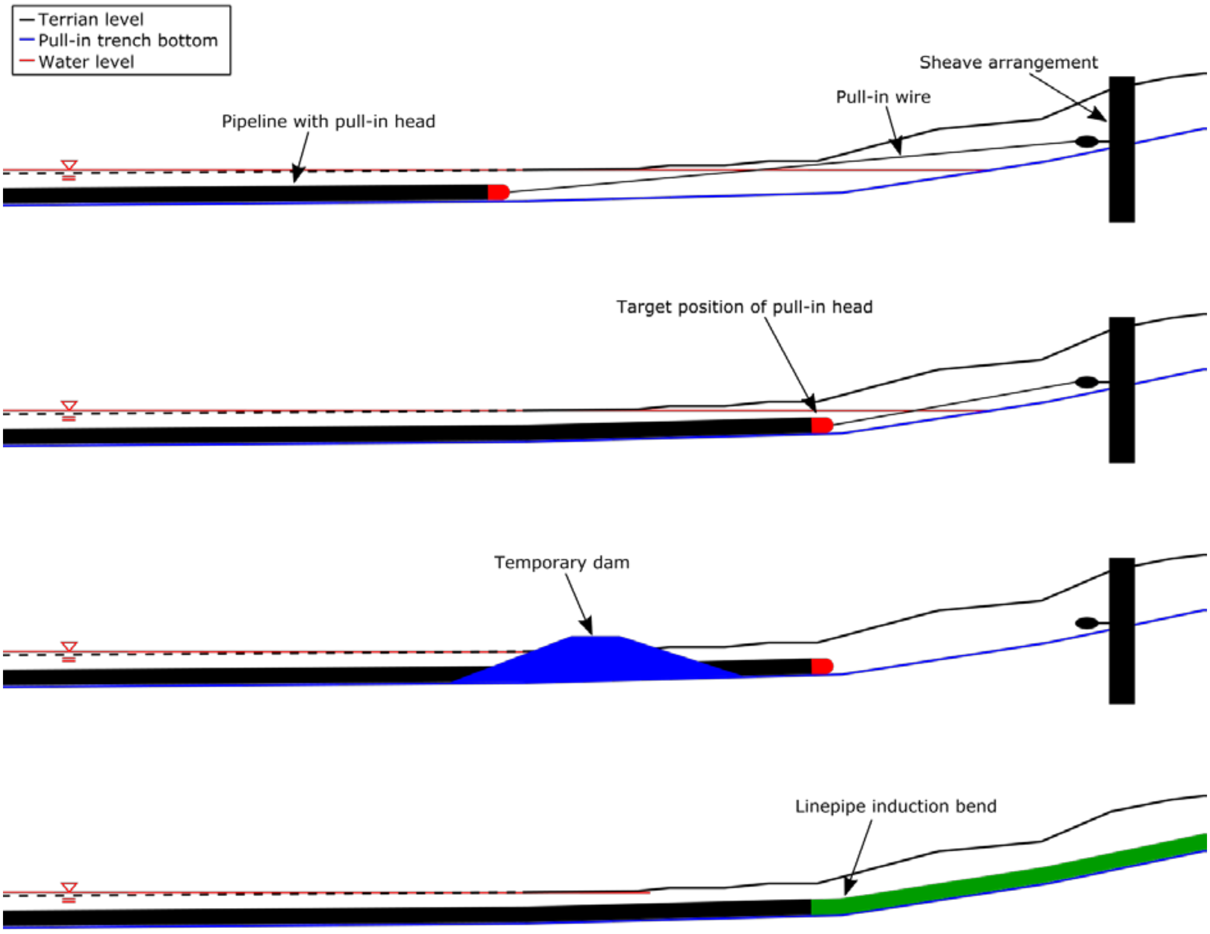


Figure 10-16 Sketch of the pull-in operation at the Paldiski landfall

The required pull-in force will depend on the following factors:

- Friction for submerged pipe section
- Contribution from pulling on slope for the submerged sections
- Lay vessel hold-back tension

The required pull-in force will depend on the installation method, which will be developed by the installation contractor awarded the installation work.

During the pull-in operation, the lay vessel will maintain a hold-back tension. In the assessment of the needed pull-in force a hold-back tension at 700 kN (71 Ton) has been assumed. The pull-in design force is calculated to be 2100 kN (or 205 Ton). This is assumed to be a conservative pull-in force for the Balticconnector pipeline at the Paldiski landfall site.

The lay vessel needed to be used for the installation of the Balticconnector at the Paldiski landfall should be able to operate at water depths down to 8-9 m in order for it to be placed at a position 1,000 m from the shoreline during the pull-in operation.

11 Seabed intervention

Seabed intervention has been specified for the Balticconnector pipeline based on the following engineering activities:

- Load controlled local buckling design criteria of the empty, flooded and operational pipeline
- Crossings requirements for the Nord Stream pipelines and subsea cables
- Fatigue design criteria for the free spanning pipeline
- HSE protection requirements for dragged anchors
- Landfall design at both Finnish and Estonian ends
- Global buckling and upheaval buckling mitigation

A total volume of 244,539 m^3 is envisaged to be installed to fulfil the protection strategy defined. Approximately 30,838 m^3 is defined as pre-lay and 213,702 m^3 as post-lay rock installation. The requirement for removal of bedrock amounts to 1,325 m^3.

Note that all volumes are theoretical volume estimates. Some contingency has been included to account for rock settlement, over-dumping, etc. however these effects are highly dependent on local soil condition and are hard to predict accurately.

11.1.1 Seabed intervention techniques

When defining seabed intervention to ensure a robust and reliable pipeline design, the choice is between subsea rock installation and excavation (trenching, dredging or blasting).

Subsea rock installation

Installation of subsea rock is the traditional method of rectifying free spans using a rock installation vessel and suspended fall pipe. The rock berms, installed either pre- or post-lay, act as a stable subsoil condition for the pipeline, or alternatively as protection against impacts and pull-over/hangover loads. An example of a rock installation vessel is shown in Figure 11-1.

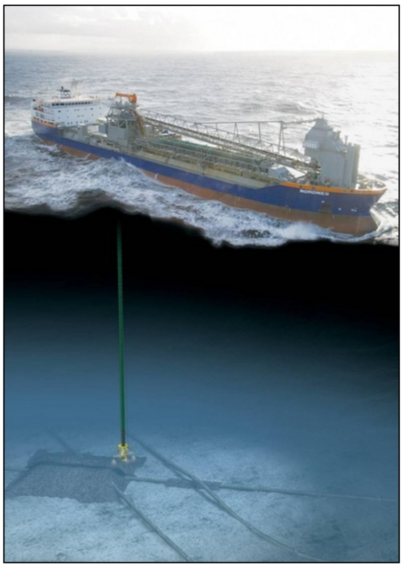


Figure 11-1 Subsea rock installation (SRI) – accurate positioning by fall pipe

The rock installation vessel shown in Figure 11-1 has a loading capacity of 24,000 t. The vessel has a maximum rock installation speed of 2000 t per hour, however, a typical average rock installation speed which takes into consideration transit times to and from quarry and between rock installation locations is 150 t per hour.

<u>Excavation – Trenching, dredging and blasting</u>
Mechanical protection of pipelines for long sections can be achieved by ploughing the pipeline into the seabed and subsequently backfilling the line. Trenching by ploughing and subsequent backfilling eliminates or reduces the requirements for continuous rock cover.

The plough is clamped around the pipeline, and pulled by the trenching vessel. Based on recent experience, a ploughing rate in the order of 200 - 400 m per hour may be obtainable and the rate for backfilling will be similar. The achievable trench depth depends on the soil conditions and the pull force available. Typically values (to bottom of trench) are 1.5 - 2.5 m.

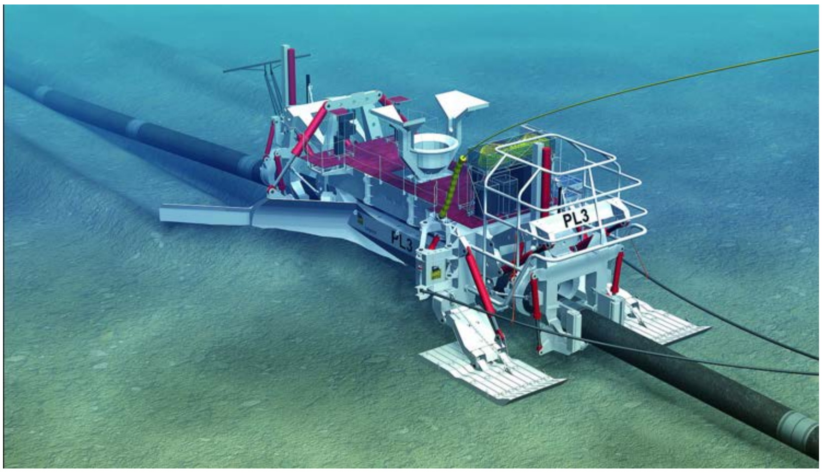


Figure 11-2 Subsea pipeline ploughing – image courtesy of Nord Stream AG

In soft or sandy soils, trenching can also be performed by water jetting equipment placed on the pipeline and pulled by the trenching vessel, which also delivers pressurised water for the jetting nozzles. The water jets bring the seabed soil into suspension, allowing the pipeline to sink into the trench, which may reach a depth exceeding 1.5 m, depending upon the soil conditions. As the jetting equipment is light-weight, the pipeline is not likely to be damaged by the equipment.

For more localised solutions, soil removal in soft to stiff clay is an effective method of free span rectification when a peak in the seabed profile leads to adjacent multi-spans. The removal of soil with a "spider" (remotely operated dredging vehicle) or surface-based dredging arm in shallow water areas is known as dredging, whereas the removal of soil using jetting or clay cutters is known as excavation in this context. These excavation methods are shown in Figure 11-3 and Figure 11-4.

When excavation of bedrock is required, removal of the peaks could be performed by using a traditional boring and blasting method with special restrictions applied with regards to water borne shockwaves and vibrations.

A barge can be deployed to be in the right position to drill and charge the holes for blasting. Once the explosion is triggered, loose rock will need to be excavated. The minimum trench width is dictated by the excavators used, and in this region 5-6 m wide buckets from 300 t excavators are used.

The water depth will determine whether drilling in the bedrock can be performed using a jack-up rig or divers. Typically it would be assumed the normal limits of the jack-up rigs are in 25 m of water depth although this would have to be verified with the chosen contractor. An example of the jack-up rig used for blasting is shown in Figure 11-5.

Figure 11-3 Dredging techniques using spider (left) and dredging barge and cargo vessel (right)

Figure 11-4 Excavation techniques using subsea jetting tools; T series (left) and ClayCutter (right)

Figure 11-5 Controlled blasting of subsea rock

11.1.2 Local buckling rectification

A geotechnical design has been carried out for 3 representative high local buckling locations; #6, #15 and #21 as classified in section 7.3. Each location has been rectified by means of rock supports effectively lowering the local buckling utilisation to below unity, i.e. UT ≤ 1.0. The locations are highlighted in Table 11-1 by use of bold borders. The geotechnical design can be found in Appendix VIII.

Locations #6, #15 and #21 have been chosen as representative locations for the estimation of required rock installation for mitigation of all high local buckling locations. This because the supports vary in height and rest on different soils; soft clay, firm clay and sand veneer over bedrock. The geotechnical results therefore form part of the overall estimation of method and size of seabed intervention performed for each independent location.

The estimates are, among others, based on:

- Geotechnical analyses for locations #6, #15 and #21
- Local soil conditions and topology
- Pipeline configurations during the design life (air-filled, water-filled, pressure-test and operation)
- Bottom roughness analyses based on FE modelling
- In-house project experience
- Robustness and method of seabed intervention

The estimate should be considered in its entirety, i.e. the total volume rather than sub volumes for each location. This as the overall estimate naturally has a higher level of accuracy than independent locations due to the numerous factors influencing the design as cited in the above bullet points.

It is noted that the estimated amounts of rock installation and removal for mitigation of high local buckling utilisation are based on that some re-routing will be performed, potentially by means of counteracts. Counteracts are typically large concrete cylindrical structures used during installation to re-route the pipeline beyond its stable lay radius. However, as indicated in Table 7-18 the complexity, with respect to optimal seabed intervention, varies for the identified high local buckling utilisations. Thus, further investigation during the detailed engineering phase is required to identify the most optimal methods of seabed intervention to increase the accuracy by which the estimated volumes are determined.

No.	KP @ max UT	MSL	Volumes			Design complexity	Comments
			Pre-lay SRI	Post-lay SRI	Soil/rock removal		
[-]	[km]	[m]	[m³]	[m³]	[m³]	[-]	[-]
LB 1	12.242	-19.6		500		Low	Mitigation of potential upheaval buckling
LB 2	13.919	-26.5	2400			Low	
LB 3	16.193	-24.9	500			Low	
LB 4	16.981	-28.3		1700		Low	
LB 5	17.426	-26.5	1500			Low	Rock removal not included cf. Table 7-18
LB 6	17.840	-31.5	4000			Low	
LB 7	18.248	-26.5	N/A	N/A	N/A	High	Re-routing options to be further evaluated cf. Table 7-18
LB 8	18.490	-34.0		1000		Low	Potentially affected by re-routing
LB 9	18.729	-26.5	4600		50	Medium	Potentially affected by re-routing
LB 10	18.795	-26.5	N/A	N/A	N/A	Low	Potentially affected by re-routing Volumes included in #9
LB 11	18.982	-25.8	700		25	Low	Rock removal included cf. Table 7-18
LB 12	19.364	-24.3	2600		475	High	Based on recommended mitigation option cf. Table 7-18
LB 13	19.735	-20.9		300		Low	
LB 14	19.894	-27.6		200		Low	
LB 15	20.263	-23.6	1400			Low	
LB 16	20.915	-17.2			50	Medium	
LB 17	21.193	-29.6	200			Low	Potentially affected by re-routing
LB 18	22.288	-31.7	300			Low	Potentially affected by re-routing
LB 19	22.371	-36.0	N/A	N/A	N/A	Medium	Re-routing options to be further evaluated cf. Table 7-18
LB 20	24.277	-39.0			175	Low	
LB 21	24.391	-41.0	300			Low	
LB 22	24.753	-35.8	400			Low	High accuracy pre-lay installation i.e. -0/+0.2 m
LB 23	25.104	-28.4	400			Low	High accuracy pre-lay installation i.e. -0/+0.2 m
LB 24	25.324	-28.0	3800		550	High	Estimates associated with large uncertainty
Volume low complexity			15900	3700	200	-	-
Volume medium complexity			4600	0	100	-	-
Volume high complexity			2600	0	1025	-	-
Total volumes			**23100**	**3700**	**1325**	-	-

Table 11-1 Rock volume estimates for rectification of high local buckling utilisations

11.1.3 Crossings

The majority of the crossing locations are placed within sections where the pipeline has to be protected due to HSE protection requirements according to the *Quantitative Risk Assessment*, Ref. /28/, see Table 11-4. Post-lay rock installation at cable crossings located within the section is included in the respective sections in Table 11-4. Thus, only post-lay rock installation for crossing locations outside the sections defined in Table 11-4 has been included in the estimates given in Table 11-2.

Pipeline name / owner	KP	Volumes		Comments
		Pre-lay SRI	Post-lay SRI	
[-]	[km]	[m^3]	[m^3]	[-]
NS1/Nord Stream	42.175	2,074	- 1)	According to *Pre- and post-lay rock installation design drawings,* Refs. /41/ and /44/
NS2/Nord Stream	43.092	1,973	- 1)	According to *Pre- and post-lay rock installation design drawings,* Refs. /42/ and /45/
Cable crossings, Ref. Table 5-7	N/A	1,404 (78 x 18)	1,456 (364 x 4) 2)	Estimate based on generic cable design, Refs. /43/ and /46/
Total volumes	-	**5,451**	**1,456**	

Note:
1) Both Nord Stream pipelines fall inside the HSE protection zone, therefore post-lay rock volumes are incorporated in overall HSE protection rock volume
2) Only 4 cable crossings fall outside the HSE protection zone, therefore the post-lay rock volumes are only applied to these four locations

Table 11-2 Rock volume estimates for crossings

11.1.4 Seabed intervention for fatigue mitigation of free spans

This section includes both pre-lay and post-lay span infills for the mitigation of fatigue damage at free spanning locations of the Balticconnector pipeline. The volumes are calculated for spans that do not meet the allowable span length criteria for the empty condition and operational condition defined in section 7.2. For each identified location, a pre-lay free span support has been assumed to Bottom-Of-Pipe (BOP) with a width of 11 m and length of 10 m. Note that width is specified as perpendicular to the pipeline axis. If the resulting span lengths either side of the support are still greater than the allowable span length, an additional support of similar size is calculated.

Pre-lay span infills are estimated for any spans that do not fulfil the empty phase fatigue design criteria, i.e. there is a risk that excessive fatigue damage will occur at the pipeline free span while empty before post-lay seabed intervention can be applied.

Post-lay span infills are estimated for any spans that meet the empty phase fatigue design criteria but do not withstand the fatigue loads during the operational phase. Hence, post-lay rectification which can be installed to a greater degree of accuracy (due to the lack of uncertainty about the pipeline profile which is incorporated into the pre-lay span infill design)

Spans that have been identified to require local buckling rectification and global buckling post-lay seabed intervention are identified in Table 11-3 but the rock volumes are not included.

Infill No.	Span			Rock Volume	
	KP start	KP end	Length	Pre-lay	Post-lay
[-]	[km]	[km]	[m]	[m³]	[m³]
FS 1	1.297	1.379	82	40	-
FS 2	2.861	2.927	66	36	-
GB Finland	3.505	3.561	56	-	n/a
FS 3	3.821	3.898	77	30	-
GB Finland	4.226	4.279	53	-	n/a
FS 4	10.385	10.418	33	-	79
FS 5	10.420	10.460	40	35	-
FS 6	11.528	11.570	42	129	-
FS 7	11.606	11.644	38	75	-
FS 8	11.733	11.793	60	125	-
LB 1	12.195	12.291	96	n/a	-
FS 9	13.225	13.314	89	143	-
FS 10	13.473	13.514	41	-	65
FS 11	13.646	13.689	43	-	145
FS 12	13.693	13.764	71	190	-
LB 2	13.873	13.917	44	-	n/a
LB 2	13.920	13.967	47	-	n/a
FS 13 [1)]	18.177	18.245	68	167	-
FS 14 [1)]	18.251	18.310	59	-	214
LB 9	18.656	18.723	67	-	n/a
LB 9	18.735	18.792	57	-	n/a
FS 15	18.850	18.902	52	-	70
LB 10	18.934	18.980	46	-	n/a
FS 16	19.187	19.238	51	-	57
LB 12	19.269	19.347	78	n/a	-
FS 17 [1)]	19.367	19.416	49	128	-
FS 18	19.422	19.500	78	435	-
FS 19	19.563	19.596	33	-	94
FS 20	19.606	19.657	51	104	-
LB 13	19.697	19.732	35	n/a	-
LB 13	19.738	19.802	64	n/a	-
FS 21	19.830	19.867	37	-	39
LB 14	19.897	19.936	39	-	n/a
FS 22	20.101	20.152	51	114	-
LB 15	20.213	20.261	48	n/a	-
FS 23	20.424	20.469	45	-	76
FS 24	20.528	20.577	49	153	-
FS 25	20.590	20.629	39	-	191
FS 26	20.633	20.698	65	124	-
FS 27	20.781	20.873	92	121	-
LB 16	20.878	20.913	35	n/a	-
FS 28	20.926	20.955	29	-	55
LB 18	22.234	22.285	51	n/a	-
LB 18	22.301	22.366	65	n/a	-
FS 29	22.376	22.422	45	139	-
LB 20	24.223	24.274	51	-	n/a
LB 23	25.059	25.102	43	-	n/a
LB 23	25.152	25.199	47	n/a	-
LB 24	25.234	25.275	41	-	n/a
LB 24	25.284	25.321	37	-	n/a
LB 24	25.328	25.397	69	n/a	-
FS 30	48.854	48.909	55	-	47
FS 31	50.215	50.276	61	-	46
FS 32	50.341	50.402	61	-	73
Total				**2,287**	**1,252**

Note: 1) Uncertainty due to local buckling solution involving possible re-routing

Table 11-3 Rock volume estimates for free span rectification due to fatigue design criteria

11.1.5 HSE protection requirement

According to the *Quantitative Risk Assessment*, Ref. /28/ certain sections are to be protected against dragged anchors in order to reach an acceptable failure frequency level. The volumes presented in Table 11-4 are based on rock installation to 0.5 m above TOP as described in the protection philosophy in Section 4.2.

No.	Section		Hazard	Volume
	KP start	KP end		Post-lay SRI
[-]	[km]	[km]		[m^3]
HSE 1	2.800	3.800	Dragged anchors along Inkoo fairway and man-made ice ridge scouring protection	n/a[1)]
HSE 2	26.400	39.500	Dragged anchors along TSS east to west	76,656
HSE 3	40.500	46.500	Dragged anchors along TSS west to east	35,703
HSE 4	47.500	48.500	Dragged anchors from vessels taking “short cut” south of TSS	6,086
HSE 5	51.500	53.500	Dragged anchors from vessels taking “short cut” south of TSS	12,623
HSE 6	60.500	65.500	Dragged anchors from westbound traffic to/from Tallinn	29,558
Total volume				**160,626**
Note 1) Rock volume not included as the fairway protection is covered by the global buckling solution in section 11.1.7				

Table 11-4 Rock volumes to provide dragged anchor protection

11.1.6 Rock cover landfalls

The rock covers at landfalls are installed from -5.0 m to -10.0 m MSL. The landfall solutions also overlap with the global buckling mitigation to protect the pipeline against trawl gear at each end.

Table 11-6 only includes the rock cover for the landfall design. The rock cover has been specified as +1.0 m TOP with a crown width of 2.0 m. This will provide the pipeline with protection from subsea turbulence close to coastlines (caused by waves breaking in shallower water during storms), as well as additional protection against smaller dropped objects, e.g. anchors, from pleasure boats.

Location	Section		Installation height	Volume
	KP start	KP end		Post-lay SRI
[-]	[km]	[km]		[m^3]
Finnish landfall -5.0 m to -10.0 m MSL	0.042	0.450	1.0 m above TOP	4,226
Estonian landfall -5.0 m to -10.0 m MSL	79.186	79.556	1.0 m above TOP	3,861
Total volume	**-**	**-**	**-**	**8,087**

Table 11-5 Rock volumes for offshore landfall protection

11.1.7 Global buckling

The global buckling solution is divided into three sections; the Finnish end solution, the Estonian end solution and the offshore solution. Both ends require rock cover to ensure no buckling is triggered from imperfections or trawl gear interaction while the pipeline is potentially at a higher temperature depending on the flow direction of the gas. Between the two ends, when the pipeline has cooled, trawl gear pullover loads on a free spanning section can initiate a buckle hence the need for span infills up to 0.3 m from the Bottom-Of-Pipe (BOP).

This section includes span infills that are not already included in the estimates for local buckling rectification, HSE protection requirements and free span rectification for fatigue mitigation. Table 11-6 summarises the total post-lay rock volume for the solutions at the nearshore approaches, whereas Table 11-7 covers all the span infills up to 0.3 m from BOP for the offshore section in between.

No.	Section		Installation height	Volume
	KP start	KP end		Post-lay SRI
[-]	[km]	[km]		[m^3]
End Section Finland	0.451	4.500	0.5 m above TOP	24,497
End Section Estonia	74.900	79.185	TOP	10,583
Total volume	**-**	**-**	**-**	**35,080**

Table 11-6 Rock volumes for nearshore global buckling design

No.	Section		Volume	No.	Section		Volume
	KP start	KP end			KP start	KP end	
[-]	[km]	[km]	[m^3]	[-]	[km]	[km]	[m^3]
GB 1	6.734	6.741	3	GB 18	20.350	20.356	50
GB 2	11.585	11.586	0	GB 19	20.717	20.741	369
GB 3	11.659	11.665	7	GB 20	20.757	20.763	47
GB 4	11.718	11.724	8	GB 21	22.346	22.360	183
GB 5	11.823	11.835	46	GB 22	24.701	24.715	120
GB 6	11.869	11.874	3	GB 23	24.907	24.915	73
GB 7	13.786	13.797	59	GB 24	24.974	24.979	34
GB 8	14.020	14.031	149	GB 25	25.035	25.039	26
GB 9	17.465	17.484	208	GB 26	25.131	25.133	16
GB 10	17.502	17.509	55	GB 27	49.339	49.372	490
GB 11	17.956	17.967	113	GB 28	49.389	49.398	53
GB 12	17.986	18.000	160	GB 29	49.512	49.537	303
GB 13	18.808	18.825	159	GB 30	50.179	50.183	28
GB 14	18.912	18.928	130	GB 31	50.290	50.305	114
GB 15	19.488	19.494	6	GB 32	50.310	50.315	35
GB 16	19.506	19.514	107	GB 33	50.322	50.329	71
GB 17	19.525	19.539	212	GB 34	55.363	55.373	64
				Total volume			**3,501**

Table 11-7 Rock volumes for offshore global buckling design

12 Pipeline installation

Pipeline installation procedures described in this FEED report will cover the following key aspects of design:

- Determining the possible stinger configuration
- Required tension levels at various water depths
- Offshore procedures to be followed during execution
- Initiation methodology at landfall location
- Above water tie-in of pipeline
- Pipelay vessel availability

Further on, only the static analyses for various pipeline profiles have been carried out, i.e. sea wave and current effect on pipelay installation vessel have not been considered in the FEED phase of the project.

The results from normal pipelay by S-lay installation method will be described in this section. Other detailed analyses such as installation initiation, installation laydown, abandonment and recovery (A&R), and dynamic installation analyses will only be discussed on generic level and should be carried out during the detailed design study of the project.

12.1 Pipeline installation methodology

The pipeline installation methodology has been outlined in this section. The possible pipeline installation method for the 20" gas pipeline is by S-lay vessel, typical S-lay installation is presented in Figure 12-1.

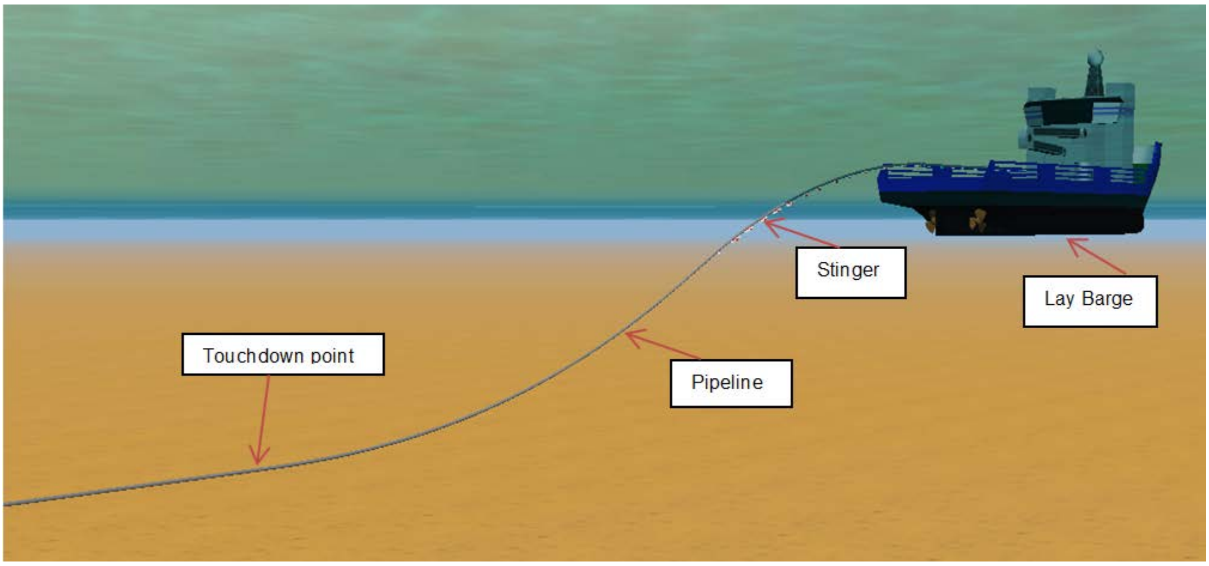


Figure 12-1 A typical pipeline installation by S-lay vessel

The different stages of pipeline installation procedure are outlined below.

12.1.1 Mobilisation and demobilisation

The mobilisation and demobilsation procedures of the pipelay vessel shall be defined prior to installation. A checklist shall be prepared to confirm that the vessel is equipped with all the necessary material and equipment required for pipeline installation.

A mobilisation yard location will depend on the vessel provider and occupancy. A typical list of items and not limited to be included in the checklist are outlined below.

- A&R head, laydown and start-up assemblies
- Project specific rigging material
- Wet buckle contingency
- All type of anodes
- Buckle detector
- Transition joints
- Operation specific mobilisation items

12.1.2 Pipelay operation

This section includes the normal operation on-board pipeline installation vessel. The typical pipeline vessel will carry out the following operations during installation.

- **Offshore pipe loading:** Pipe loading will be performed with the aid of pipe supply vessels, which transport pipe from the spool base to pipelay vessel. Pipe joints will be loaded on board pipelay vessel from the pipe supply vessels coming alongside, using the dedicated pipe transfer cranes. On board the pipelay vessel, the joints will be transferred from the landing area either, directly into the double joint factory via the conveyor system, or to the cargo hold area for storage via an envelope hatch. The pipe joints are to be handled with care during offshore loading, to avoid coating and pipe end damage.

- **Double joint / single joint welding stations:** Single pipe joints can be transferred to the double joint station directly from a pipe supply vessel by means of either a transverse and longitudinal conveyor system, or from the pipe storage holds to the double joint factory by means of the gantry cranes and the longitudinal and transverse conveyor system. The pipe ends will be bevelled at the double joint factory. Pipe ends should be bevelled in accordance with welding requirements. The girth weld should be checked carefully at NDT stations and QC on the weld should be achieved.

- **Firing line**: In the main firing line, the joint station should be internally cleaned with pressurised air, prior to welding. The double joint ends should be pre-heated by induction coils after which, the double joint will be transferred to the line-up station and welded to the pipe string. On completion of all welding activities and arrival in the NDT station, the weld should be tested, after which the field joint coating can be applied.

- **Pipeline monitoring and control system:** A pipelay vessel specific monitoring and control system should be installed on board vessel, to fulfil the integrity of the system, pipeline, and vessel during pipelay. A detailed list of on board safety system should be outlined by pipelay operator prior to installation.

- **Field Joint station:** The field joint coating is carried out just before the barge stinger. The pipeline weld will be protected by shrink sleeves and filled with PU foam at the field joint stations.

12.1.3 Installation vessel and stinger configurations

The various stinger configurations have been identified for varying water depth along the pipeline route. It is important to minimise the number of stinger configuration changes during the installation, as it will lead to longer installation time which will impact the project cost significantly.

For the Balticconnector project, two different stinger configurations have been selected to handle pipelay close to shore and deep section.

The typical installation vessel, which can be employed and used for the S-lay operation and calculations with its main particulars and capacities are listed in Table 12-1.

Item	Unit	Value
Main particulars		
Length overall (incl. stinger)	m	236
Length overall (excl. stinger)	m	183
Length between perpendiculars	m	150
Breadth	m	26
Capacities		
Maximum tension capacity[1]	t	165
Maximum DMA tension	t	120
Maximum A&R tension (hydraulic winch)	t	125
Maximum A&R tension (electric winch)	t	225
Maximum allowable bottom tension[2]	t	80
1) Vessel is equipped with three tensioners with a capacity of 55 t each 2) Specifies the nominal dynamic bottom tension for Hs = 3m, Vc = 1kts, Vw = 30 kts		

Table 12-1 Main particulars and capacities of pipelay vessel

The stinger configurations proposed for the pipeline installation analyses in shallow and deep water sections are presented in Figure 12-2. The stinger radius of R=300m and R=160m are proposed for shallow (up to 56 m) and deep water (greater than 56 m) depth respectively.

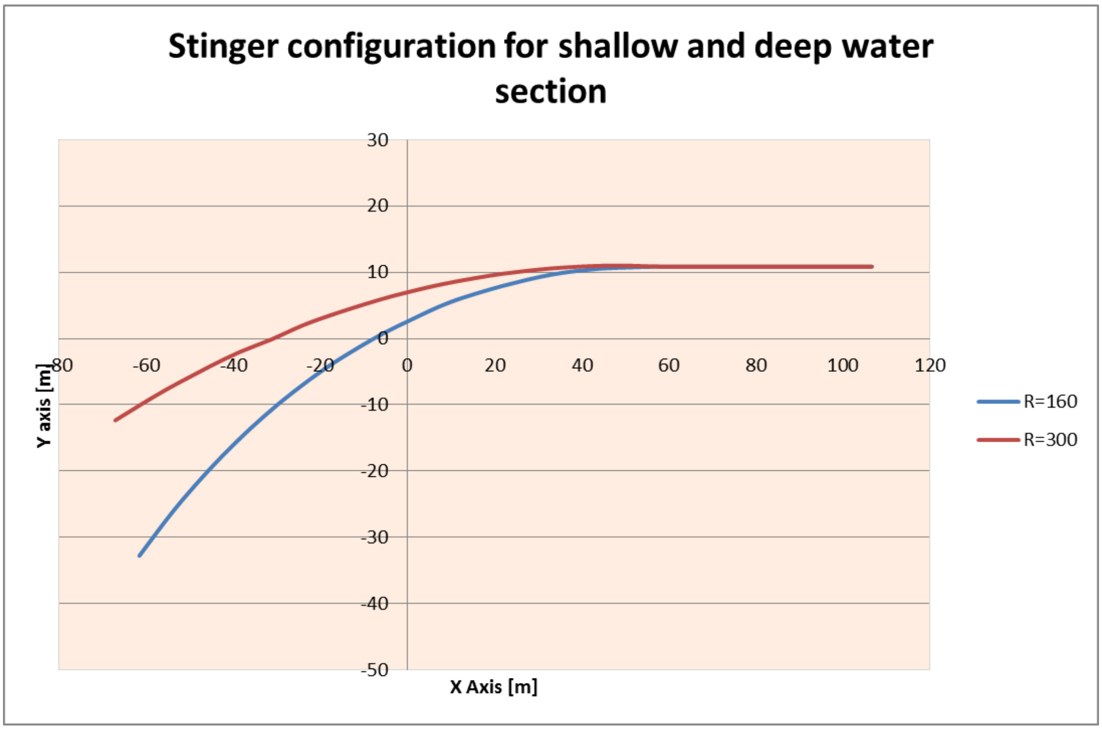


Figure 12-2 Stinger configurations for shallow and deep water section

The stinger configurations are proposed to meet the installation criteria at overbend and sagbend during installation. The stinger configuration largely depends on the pipelay vessel, stinger length, pipeline profile, and water depth. Few pipelay vessels are equipped with articulated stingers, and stinger configurations proposed in this report are not relevant for such vessels.

12.1.4 Pipelay initiation

Pipeline initiation will be done by means of pull-in operation at landfall location both in Finland and Estonia sites. The detailed pull-in operation will depend on the landfall solution at both shore locations. The pull-in operation will be initiated from vessel, where pull-in cable will be attached to pipeline via pull-in head.

Pull-in winch will be installed on the shore with the required capacity to commence the pull-in of pipeline. The details of the pulling operations and drawing can be found in landfall design section 10.

12.1.5 Normal pipelay

The normal pipelay will start when the pull-in head will be in the target box at the landfall location. The pipeline support configuration on the pipelay vessel will be optimised for the normal pipelay mode of operation. In the overbend region, the pipeline should be supported satisfactorily until it lifts off.

The pipeline route is divided into several sections in accordance with changes in water depth, soil conditions, and concrete thickness, to obtain a vessel tension schedule. Analyses are performed for critical pipe property and water depth combinations along the route.

12.1.6 Pipeline laydown

The pipeline will be laid down at the specified target position close to Estonian whore, where above water tie-in (Davit lift) method will be used to weld the pipeline laid from Finland and Estonian shores.

The laydown target box should be defined at relatively flat seabed in shallow water section, to avoid any excessive stress accumulation in the pipeline during tie-in.

The Davit lift procedure is outlined in section 12.1.8.

12.1.7 Abandonment and recovery

A procedure should be outlined for abandonment and recovery of the pipeline in case of interrupted pipelaying activities, such as severe weather conditions or a component failure within the pipelay system.

The A&R operations will either be performed by the single wire system or dual wire system depending on the environmental conditions and which pipe section is concerned (i.e. the occurring tensions).

The abandonment and recovery analyses have not been carried out in the FEED phase and should be performed during the detailed design study.

12.1.8 Above water tie-in (Davit lift)

Midline tie-in or above water tie-in (AWTI) is an operation where two laid down pipelines on the seabed are welded together after being lifted above water using vessel davits. This section will include following assessments:

- Steps for recovering the pipelines
- Steps for lowering the completed pipeline
- Offshore procedures to be followed during execution

A typical approach and procedure is outlined below and in Figure 12-3.

- Once both the pipeline ends are lowered on the seabed with an over length for the tie-in, davit lifts cable will be connected to the pre-installed clamping section on the pipelines.
- After connection, the pipelines will be slowly pulled to vessel and lined up for surface alignment. Two surfaces of the pipeline will be welded together on the side of the vessel.
- After welding the pipeline, the field joint coating will be applied, and pipeline will be lowered to the seabed as shown in the figure below. In order to avoid overstressing of the pipeline, vessel will move sideward.

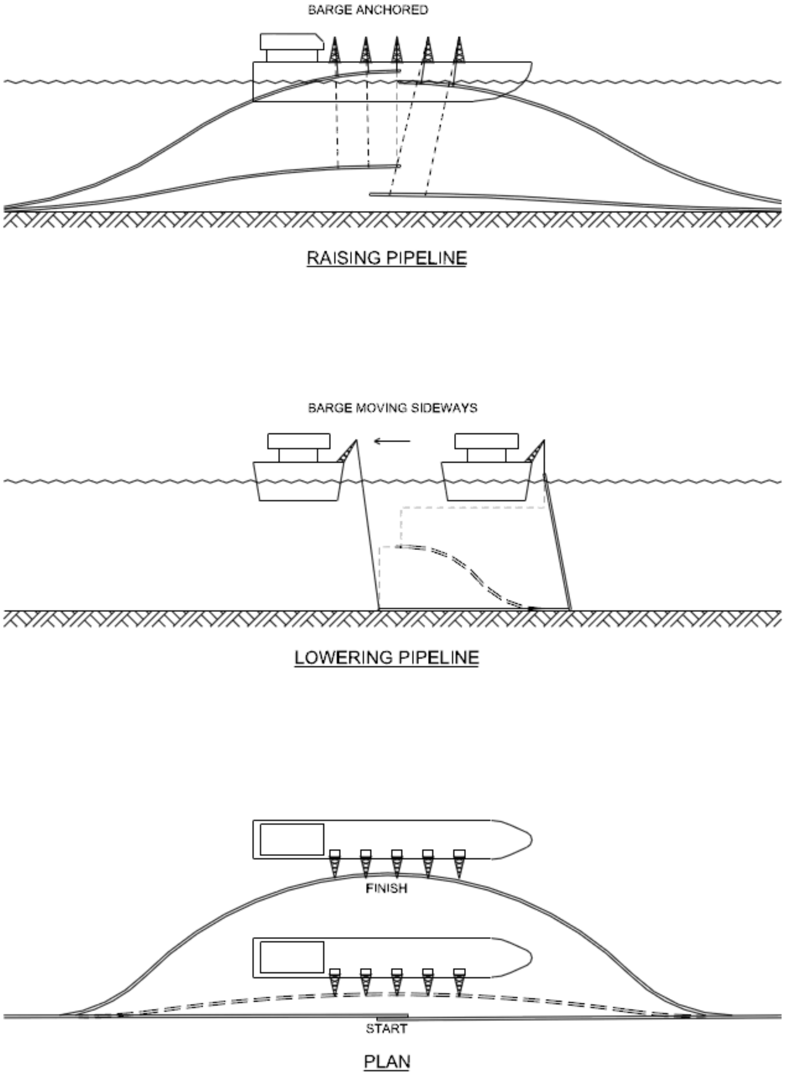


Figure 12-3 Typical davit lift connection procedure

12.2 Acceptance criteria

The installation analyses presented in this report are conducted in accordance with DNV-OS-F101 (Oct-2013), Ref. /1/.

Acceptance criteria include:

- Local buckling check (load controlled condition), Ref. /1/ Sec. 5, D600
- Simplified laying criteria, Ref. /1/, Sec. 13, G300
- Concrete crushing, Ref. /1/, Sec. 13 – G200
- No contact between pipeline and last roller
- Curve lay stability vs. planned routing
- Installation vessel capacity

12.2.1 System regions

When verifying the system integrity during installation, the following pipe regions are distinguished:

- **Overbend:** The pipeline section ranging from top of stinger to the stinger tip.
- **Stinger tip:** The pipeline section located just above the last roller on the stinger.
- **Sagbend:** The pipeline section range from the stinger tip to the seabed touch-down point.
- **Touch-down point:** The point where the pipeline first touches the seabed.

12.2.2 Local Buckling Check

The most critical limit state for the pipeline installation is normally the local buckling-combined loading, which yields the capacity of the pipeline when being exposed to the combination of bending moment, axial load and pressure (internal and external).

The Local Buckling Check (LBC) is performed for the entire pipeline from top of stinger to the point where the pipeline is resting on the seabed. The LBC is performed in accordance with DNV-OS-F101, Ref. /1/.

In the overbend region, the local buckling check is performed using a load controlled formulation considering that on a local scale the bending of the pipe between the rollers is determined by the interaction between weight and tension and hence is load controlled. The formulation takes into account the point loads from the rollers.

The LBC of the stinger tip region is also based on the load controlled condition. Since contact with the last roller on the stinger is not permitted, the loading of the stinger tip region is expected to be small compared to the overbend- and sagbend regions.

Local buckling in sagbend and touch-down regions is also evaluated by the load controlled condition.

Materials, load and safety factors applied in the local buckling check are summarised in Table 12-2.

Item	Symbol	Unit	Value	DNV-OS-F101
Pipe/Material Factors				
Material strength factor	α_U	-	0.96	Sec. 5, C306 (Normally)
Maximum fabrication factor	α_{fab}	-	0.93	Sec. 5, C307 (HFW)
Safety Factors				
Material resistance factor	γ_m	-	1.15	Sec. 5, C203 (ULS/ALS)
Safety class resistance factor	γ_{SC}	-	1.04	Sec. 5, C204 (Safety class low)
Functional load factor	γ_F	-	1.2 (ULS, System Check) 1.1 (ULS, Local Check) 1.0 (ALS)	Sec. 4, G303
Environmental load factor	γ_E	-	0.7 (ULS, System Check) 1.3 (ULS, Local Check) 1.0 (ALS)	Sec. 4, G303
Condition load factor	γ_c		0.77 (Overbend) 1.0 (Outside Overbend)	Sec. 4, G304
Strain resistance factor	γ_ε	-	2.0	Sec. 5, D609 (Safety class low)

Table 12-2 Safety factors for local buckling check

12.2.3 Simplified Laying Criteria

The Simplified Laying Criterion (SLC) is formulated in Ref. /1/, Sec. 13, G300. Criteria are given for the overbend and the sagbend regions, respectively. Note that the simplified laying criteria do not distinguish between ULS and ALS.

- Overbend

For the overbend region simplified laying criterion is given in terms of maximum allowed strain values for static and static plus dynamic loading, respectively. The simplified laying criteria for the overbend region are given in Table 12-3 for DNV 450 HFW F D (equivalent to API grade X65).

Criterion	Loading condition	Max. allowable strain
I	Static	0.250%
II	Static + Dynamic	0.305%

Table 12-3 Simplified laying criteria for X65/DNV 450 HFW FD (overbend region)

- Sagbend

In the sagbend region the simplified laying criterion is given in terms of a maximum allowed equivalent (von Mises) stress. For combined static and dynamic loads the equivalent stress, σ_{eq}, shall fulfil the criterion with all load effect factors set to unity.

$$\sigma_{eq} < 0.87 \times f_y$$

12.2.4 Concrete crushing

During pipeline installation, excessive compressive forces in the overbend region can lead to crushing of the concrete coating. A concrete crushing check will be performed in line with

Ref. /1/, Sec. 13 G200, to ensure that the mean overbend strain is below the limit at which concrete crushing first occurs.

The mean overbend strain is calculated as:

$$\varepsilon_{mean} = -\frac{D}{2R} + \varepsilon_{axial}$$

Where,

D = outer steel diameter
R = stinger radius
ε_{axial}= axial strain contribution

The mean overbend strain shall satisfy:

$$|\gamma_{cc}\varepsilon_{mean}| \geq |\varepsilon_{cc}|$$

Where,

ε_{mean} = calculated mean overbend strain
γ_{cc} = safety factor for concrete crushing
ε_{cc} = limit mean strain

The limit mean strain for the concrete coated pipe is 0.2% and the safety factor for concrete crushing is set to 1.05.

12.2.5 Curve lay stability

A curve lay stability assessment is performed to calculate the minimum stable curve lay radius that can be obtained given the lay tension from the lay operation, the submerged pipe weight and the lateral seabed friction coefficient, cf. Ref. /1/.

The minimum stable lay radius is calculated from the following expression.

$$R_{min} = \frac{Residual\ lay\ tension}{\mu_{lateral} \cdot W_{sub}} \cdot \alpha$$

Where,

α = safety factor (accounting for the uncertainties in soil capacity, friction coefficient, etc.) cf. Table 12-6
$\mu_{lateral}$ = lateral friction coefficient, cf. Table 12-6
W_{sub} = submerged weight of the pipeline section per unit length

12.3 Installation analysis methodology

An installation will be performed as conventional S-lay installation using typical installation vessel. Installation will be performed with empty pipeline, which is considered as the normal case (ULS). Pipeline flooded with seawater up to the waterline is analysed as an accidental case (ALS).

The pipeline properties used for the calculations are documented in section 3.

12.3.1 Pipeline material

The selected material for the pipeline is DNV HFW 450 FD, and the stress-strain curve used for the analyses is presented in Figure 12-4.

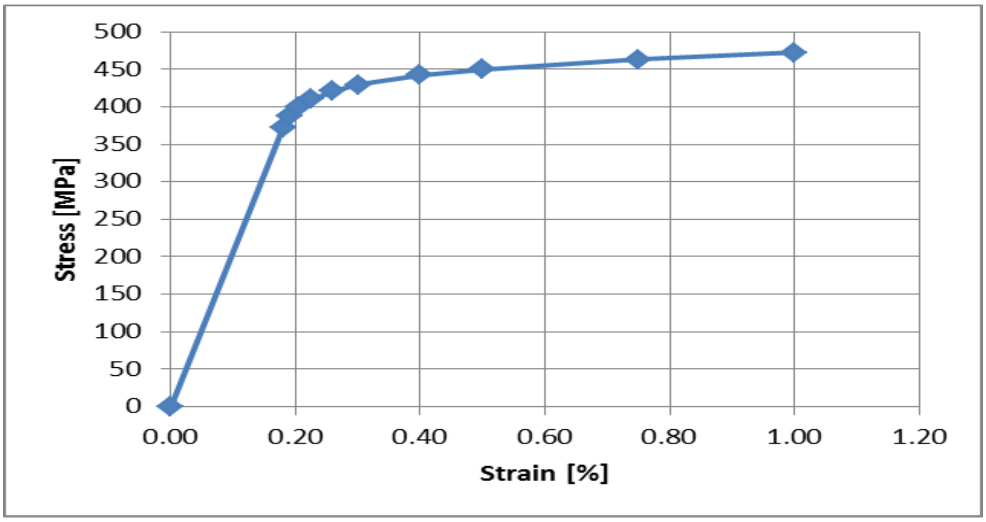


Figure 12-4 Pipeline material stress-strain curve for analyses, Ref. / /1/

12.3.2 Seabed & seabed friction

The seabed profile used for the analyses is based on the results of the route optimisation described in section 5. The various soil profiles used for the calculations are specified in pipe-soil assessment shown in section 9. The lateral seabed frictions used for the analyses are presented in Table 12-6.

12.3.3 Analyses software

The analysis is performed using OrcaFlex (v. 9.8e). OrcaFlex is a marine dynamics program developed by Orcina for static and dynamic analysis of a wide range of offshore systems. It is a fully 3D non-linear time domain finite element program using a lumped mass element.

Alongside OrcaFlex, an in-house developed spreadsheet (Excel) is used for various pre- and post-processing of data.

Figure 12-1 shows an impression of the model set up in OrcaFlex. The pipeline is modelled from a flat part of the stinger to a termination point on the seabed located after the touch-down point. The top end of the pipeline is connected to a winch used to measure the lay tension.

When the static model solution has converged, loads are extracted from OrcaFlex and post-processed using an in-house developed spreadsheet (Excel).

12.3.4 Dynamic amplification factor (DAF)

The analysis is performed as static analysis, whilst dynamic loads are estimated by multiplying static loads by Dynamic Amplification Factors (DAF) to take into account environmental loading and loading arising from installation vessel motion.

The design load effect for the installation analysis is thus expressed in the following format (Ref. /1/, Sec. 4, G302):

$$L_{Sd} = L_F \gamma_F \gamma_C + L_E \gamma_E$$

Where,

L_{Sd} = characteristic load
L_F = functional load = static load
γ_F = functional load factor
γ_C = conditional load factor
L_E = environmental load = $DAF \cdot L_F - L_F$
γ_E = environmental load factor

Table 12-4 provides a summary of dynamic amplification factors used in the analyses.

Parameter	Pipe Region	DAF
Local Buckling Check, Load Controlled Condition		
Bend moment	Overbend	1.05
Bend moment	Stinger tip	1.40
Bend moment	Sagbend	1.20
Simplified Laying Criteria		
Strain	Overbend	1.15
Eqv. Stress	Sagbend	1.15
General		
Axial force		1.25
Connector Clamp		
Bend moment		1.15

Table 12-4 Dynamic amplification factors applied in analysis

12.4 Installation results

The installation analyses are performed for various water depths for all pipeline cross sections. The results section will consist of pipeline top and bottom tensions, pipeline capacity check, allowable curve radius, loads from davit lift tie-in.

As the pipeline has different CWC requirements along the pipeline, the pipeline is divided into two sections. The details of the pipeline sections are depicted in Table 12-5.

Pipeline cross sections for Installation analyses									
Pipeline profile			Internal liner		Anti-corrosion coating		Concrete weight coating		
Pipeline section name	OD	WT	Thickness	Density	Thickness	Density	Thickness	Density	Strain Limit
	[mm]	[mm]	[mm]	[kg/m]	[mm]	[kg/m]	[mm]	[kg/m]	ε_{CC} [%]
BC_low_45	508.0	12.7	0.1	1500.0	3.5	930.0	45.0	3400.0	0.2
BC_low_55	508.0	12.7	0.1	1500.0	3.5	930.0	55.0	3400.0	0.2
BC_high_80	508.0	12.7	0.1	1500.0	3.5	930.0	85.0	3400.0	0.2

Table 12-5 Pipeline profile for preliminary pipeline installation

Further on, various installation cases have been considered to include the pipeline profile change, soil condition, and water depth. The installation cases considered for the analyses are depicted in Table 12-6. The stinger configurations have been selected as R_160 and R_300, i.e. with stinger radius of 160m and 300m respectively. The detailed stinger configurations are presented in section 12.1.3.

Pipeline installation cases	Pipeline Profile	Selected WD for calculation	Friction coeff.	Safety Factor (α)	Stinger radius configuration
BCP0	BC_low_55	30.0	0.6	1.2	R_300
BCP1	BC_low_55	40.0	0.6	1.2	R_300
BCP2	BC_low_45	30.0	0.6	1.2	R_300
BCP3	BC_low_45	40.0	0.6	1.2	R_300
BCP4	BC_high_80	30.0	0.6	1.2	R_300
BCP5	BC_high_80	56.0	0.6	1.2	R_160
BCP6	BC_low_45	52.0	0.2	1.2	R_160
BCP7	BC_low_45	70.0	0.2	1.2	R_160
BCP8	BC_low_45	80.0	0.6	1.2	R_160
BCP9	BC_low_45	100.0	0.2	1.2	R_160

Table 12-6 Analyses cases and stinger configuration

12.4.1 Pipeline tension

The top and bottom tension on the pipeline are calculated from the analyses for all the cases. The results are depicted in Table 12-7 and Table 12-8 for empty and flooded cases, respectively.

The pipeline integrity has also been evaluated during the installation for all acceptance criteria, and it has been found to be within the acceptable range. The stresses, strains and utilisation ratios for empty and flooded cases are depicted in Table 12-7 and Table 12-8, respectively.

20" BC S-Lay Analysis Results (Pipe Empty)												
Item		Unit	Pipeline installation cases									
			BCP 0	BCP 1	BCP 2	BCP 3	BCP 4	BCP 5	BCP 6	BCP 7	BCP 8	BCP 9
Static Loads												
Top Tension		[kN]	312	351	203	242	650	764	384	486	549	662
Bottom Tension		[kN]	202	222	114	139	487	523	106	139	164	236
Lay-Back Distance		[m]	148	167	148	168	150	164	171	213	237	285
Stinger Tip Clearance		[m]	0.9	0.7	0.9	0.7	0.9	3.4	5.4	3.7	3.1	2.3
Max. Point Load From Rollers		[kN]	39	47.2	67.9	42.9	56.8	58.7	62.9	44.7	46.8	47.5
Max. Bend Moment	Overbend	[kNm]	517	925	507	851	545	904	837	840	839	840
	Stinger Tip	[kNm]	275	755	265	705	223	489	430	295	243	179
	Sagbend	[kNm]	750	761	764	772	672	660	570	468	423	353
Max. Compressive Strain	Overbend	[%]	0.1	0.2	0.1	0.2	0.1	0.2	0.2	0.2	0.2	0.2
	Sagbend	[%]	0.1	0.2	0.2	0.2	0.1	0.1	0.1	0.1	0.1	0.1
Max. Eqv. Stress	Sagbend	[MPa]	325	330	326	331	307	304	253	214	198	173
Local Buckling Check, Load Controlled Condition (ULS)												
Utilisation Ratio	Overbend	[]	0.20	0.68	0.19	0.56	0.24	0.65	0.53	0.54	0.54	0.54
Utilisation Ratio	Stinger Tip	[]	0.15	1.09	0.14	0.95	0.10	0.48	0.36	0.17	0.12	0.07
Utilisation Ratio	Sagbend	[]	0.76	0.79	0.79	0.81	0.62	0.62	0.45	0.32	0.27	0.22
Simplified Laying Criteria Check												
Utilisation Ratio (Criterion I)	Overbend	[]	0.45	0.78	0.43	0.71	0.51	0.81	0.72	0.73	0.73	0.74
Utilisation Ratio (Criterion II)	Overbend	[]	0.42	0.73	0.41	0.67	0.48	0.76	0.67	0.69	0.69	0.70
Utilisation Ratio	Sagbend	[]	0.99	1.01	1.00	1.01	0.94	0.93	0.77	0.66	0.60	0.53
Concrete Crushing Check												
Utilisation Ratio	Overbend	[]	0.42	0.41	0.43	0.42	0.38	0.76	0.80	0.79	0.78	0.77

Table 12-7 Results from pipeline installation for empty case

20" BC S-Lay Analysis Results (Pipe Flooded)												
Item		Unit	Pipeline installation cases									
			BCP 0	BCP 1	BCP 2	BCP 3	BCP 4	BCP 5	BCP 6	BCP 7	BCP 8	BCP 9
Static Loads												
Top Tension		[kN]	760	791	631	673	1120	1200	891	1100	1220	1460
Bottom Tension		[kN]	598	589	489	496	908	854	337	374	421	577
Lay-Back distance		[m]	148	149	168	149	168	151	161	168	209	232
Stinger Tip Clearance		[m]	0.9	0.9	0.7	0.9	0.7	0.9	3.3	5.2	3.3	2.8
Max. Point Load From Rollers		[kN]	38.1	45.9	103	41.6	94	57.5	68.1	54.5	58.4	60
Max. Bend Moment	Overbend	[kNm]	524	1090	513	1050	554	944	861	881	894	915
	Stinger Tip	[kNm]	247	830	261	814	192	511	454	336	287	223
	Sagbend	[kNm]	673	691	694	702	620	661	549	458	419	353
Max. Compressive Strain	Overbend	[%]	0.09	0.22	0.09	0.21	0.08	0.16	0.15	0.15	0.15	0.15
	Sagbend	[%]	0.12	0.13	0.13	0.13	0.10	0.11	0.09	0.07	0.06	0.04
Max. Eqv. Stress	Sagbend	[MPa]	313	320	316	319	306	321	265	235	223	204
Local Buckling Check, Load Controlled Condition (ALS)												
Utilisation Ratio	Overbend	[]	0.16	0.75	0.15	0.68	0.19	0.54	0.43	0.46	0.48	0.53
Utilisation Ratio	Stinger Tip	[]	0.09	1.00	0.10	0.96	0.06	0.40	0.31	0.18	0.14	0.09
Utilisation Ratio	Sagbend	[]	0.49	0.51	0.51	0.53	0.43	0.49	0.33	0.24	0.21	0.16
Simplified Laying Criteria Check												
Utilisation Ratio (Criterion I)	Overbend	[]	0.50	1.02	0.48	0.95	0.56	0.89	0.78	0.82	0.84	0.89
Utilisation Ratio (Criterion II)	Overbend	[]	0.47	0.96	0.45	0.89	0.53	0.84	0.74	0.77	0.80	0.84
Utilisation Ratio	Sagbend	[]	0.96	0.98	0.97	0.98	0.94	0.98	0.81	0.72	0.68	0.62
Concrete Crushing Check												
Utilisation Ratio	Overbend	[]	0.36	0.36	0.37	0.37	0.32	0.71	0.74	0.71	0.70	0.66

Table 12-8 Results from pipeline installation for flooded case

The top tensions for empty and flooded cases are plotted along the pipeline length, the various set of calculations has been carried out for various soil type, water depth and pipeline profile.

The pipeline top tensions against water depth for the BC pipeline profile with 45 mm CWC for empty and flooded condition are presented in Figure 12-5.

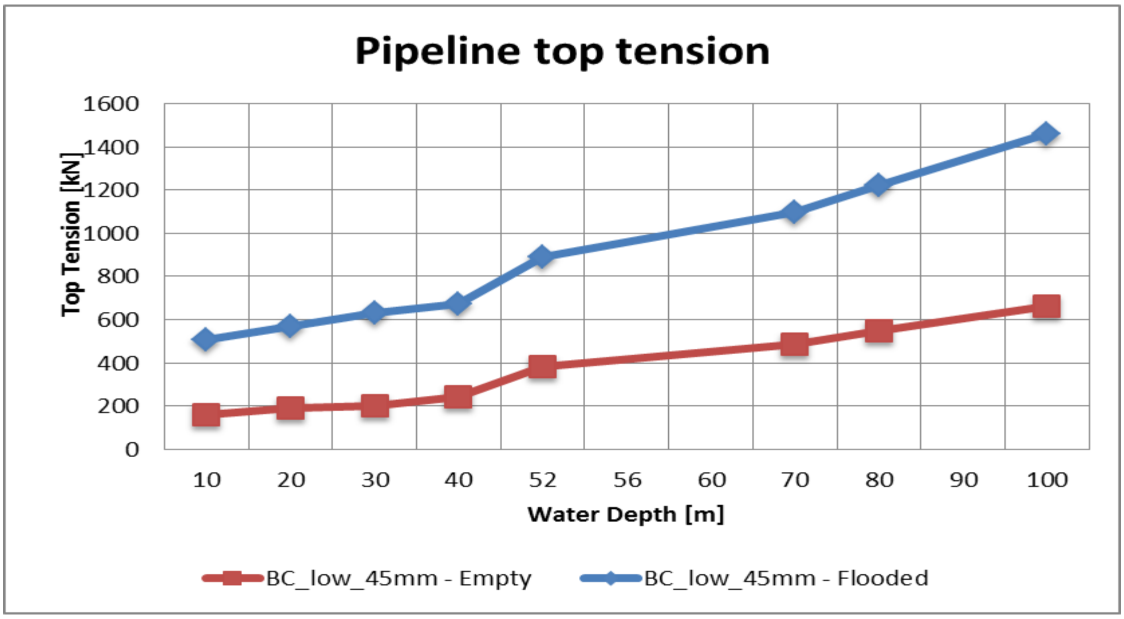


Figure 12-5 Pipeline top tension for empty and flooded for pipeline section with 45 mm CWC

The pipeline top tensions against water depth for the BC pipeline profile with 55 mm CWC for empty and flooded condition are presented in Figure 12-6.

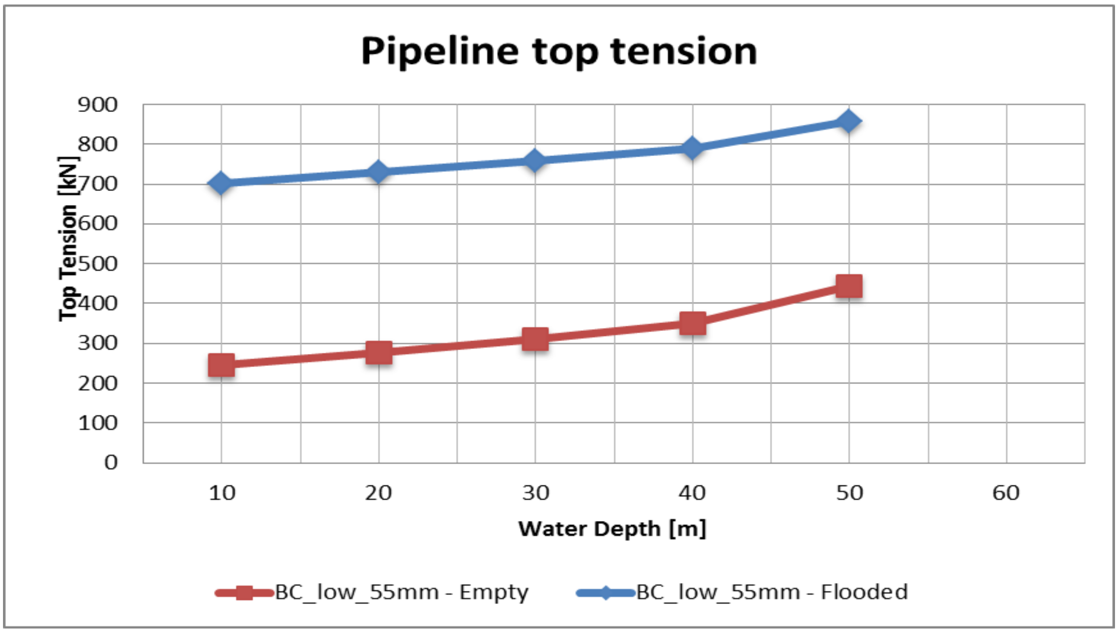


Figure 12-6 Pipeline top tension for empty and flooded case for pipeline section with 55 mm CWC

The pipeline top tensions against water depth for the BC pipeline profile with 80 mm CWC for empty and flooded condition are presented in Figure 12-7.

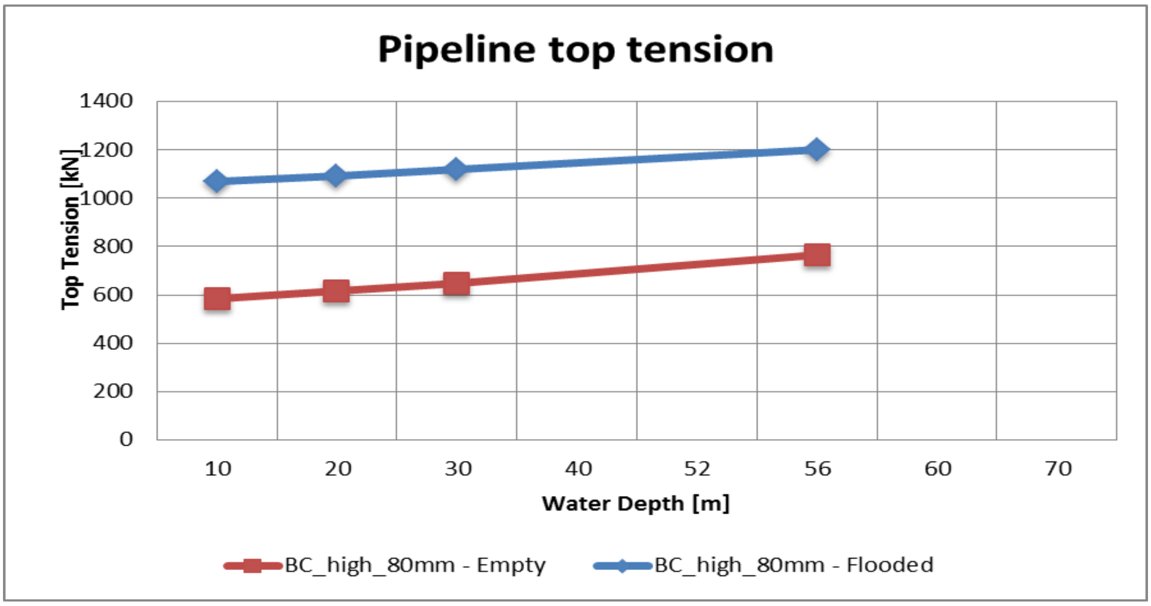


Figure 12-7 Pipeline top tension for empty and flooded case for pipeline section with 80 mm CWC

12.4.2 Allowable pipeline curve radius

The allowable pipeline curve radius is calculated based on the methodology described in section 12.2.5. The residual lay tensions for all the installation cases together with stable lay radius for empty condition are listed in Table 12-9.

20" BC S-Lay Analysis Results (Pipe Empty)											
Item		Pipeline installation cases									
	Unit	BCP0	BCP1	BCP2	BCP3	BCP4	BCP5	BCP6	BCP7	BCP8	BCP9
KP Range	-	0 - 11	11-19	72 – 80.4		19 - 26		26- 36	36 - 55		55 - 72
Water depth	[m]	30	40	30	40	30	52	56	70	80	100
Soil properties	-	Clay	Clay	Clay	Rock	Rock	Rock	Clay	Clay	Rock	Clay
Residual lay tension	[kN]	253	278	143	174	609	654	133	174	205	295
Stable lay radius	[m]	826	908	450	251	402	431	575	754	296	1280

Table 12-9 Residual lay tension and stable lay radius for empty case

Figure 12-8 summarises the minimum stable curve radius for the pipeline with CWC of 45mm. The selected stable radius for the BC pipeline is 1200 m, and pipeline curve between 96m - 100m water depth will not be stable; however, it is localised and can be settled by optimising the installation procedure at the later stage of the project.

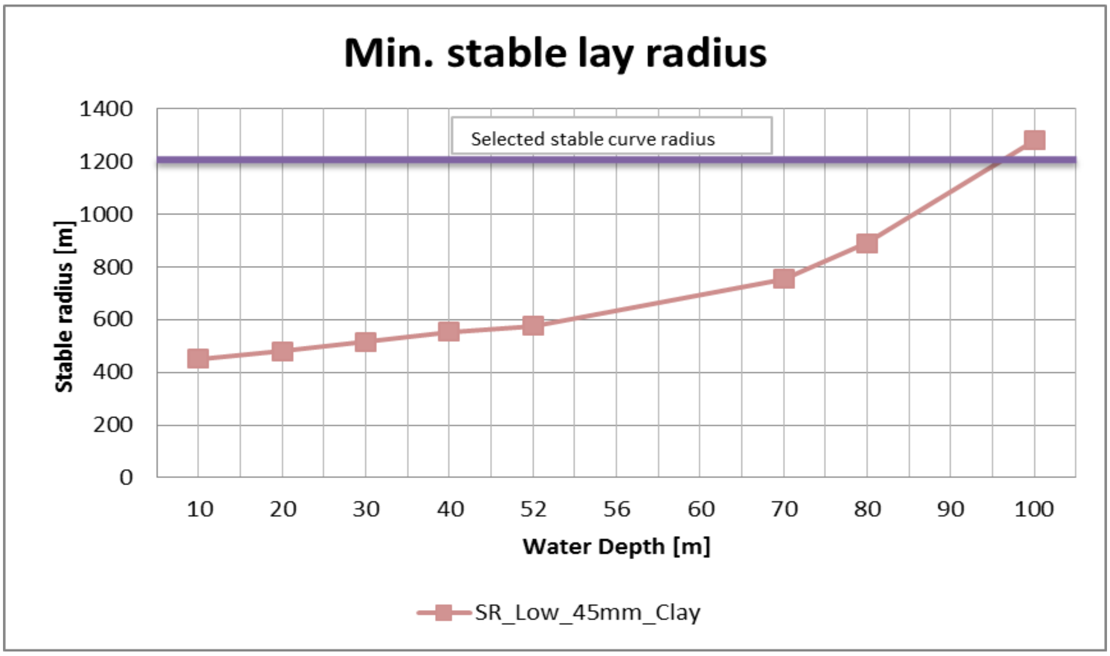


Figure 12-8 Min. stable lay radius on clay for pipeline section with 45mm CWC

12.4.3 Above water tie-in (Davit lift) results

The pipelines were laid down on the seabed with a certain overlap tolerance, which can be calculated based on the water depth and vessel winch/crane capacity. For the calculation overlap tolerance is assumed to be 4m. Total 6 winches/cranes have been utilised for the analyses. The model layout is presented in Figure 12-9. The pipeline_Finland and pipeline_Estonia profiles have been used for pipeline to Finland shore and Estonia shore, respectively.

A relatively flat seabed should be utilised for davit lift operation. The water depth assumed for this operation is 20~25m, conservatively 25m water depth is used for the calculations.

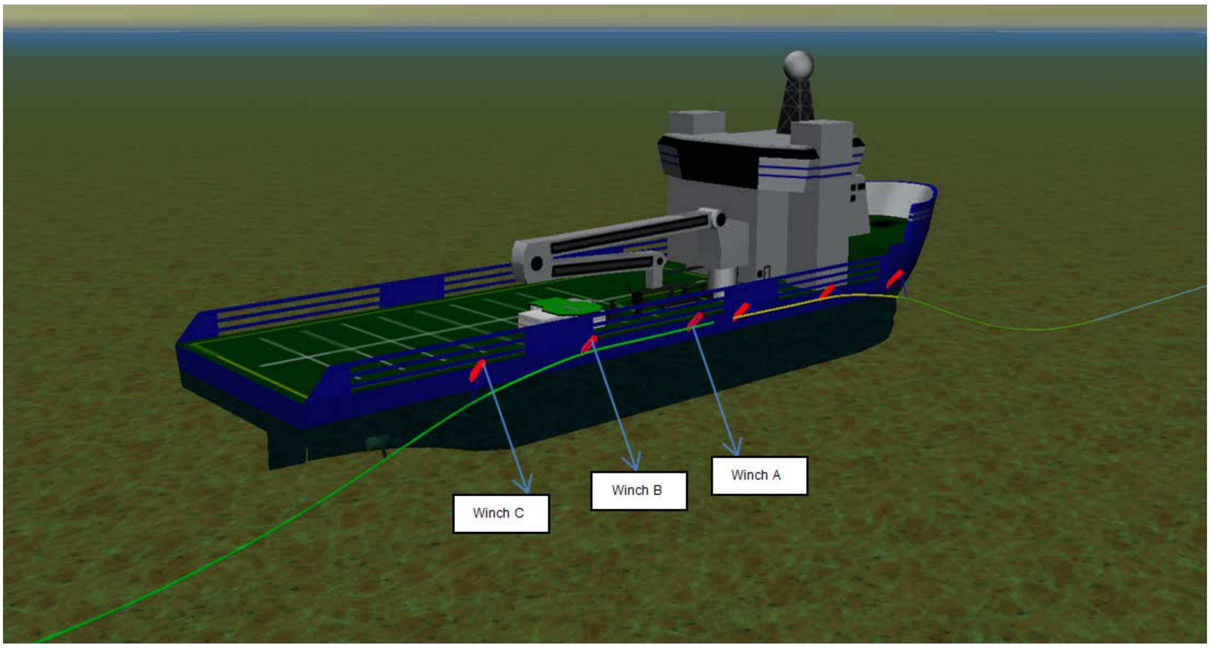


Figure 12-9 Davit lift procedure and winches arrangement

The pull-in forces are calculated on one of the pipeline profile, as both the pipeline profiles are identical and winches are arranged symmetrically. The pull-in load on 3 winches and residual tension of pipeline_Finland profile is depicted in Table 12-10. At this stage, detailed pipeline integrity has not been checked during the procedure.

Pipeline profile	Water depth [m]	Pull-in load on winch [kN]			Residual Tension [kN]
		Winch A	Winch B	Winch C	
Pipeline_Finland	25.0	196.0	490.0	345.0	658.0
Pipeline_Estonia	25.0	196.0	490.0	345.0	658.0

Table 12-10 Pull-in loads on winches and residual lay tension during the davit lift procedure

12.5 Pipelay vessel availability

There are several criteria that must be met for the installation of the Balticconnector offshore pipeline. The pipeline design has been performed with consideration of the availability of pipelay vessels to install the pipeline to ensure a cost-effective contract award philosophy. By ensuring the pipeline meets typical pipelay specifications, the contract award for the installation activities will be more competitive.

The key criteria that can limit the availability of vessels are described below.

<u>Lay method: S-lay</u>
As specified in earlier studies, the only method to install a 20" pipeline with concrete coating at the relatively shallow water depths in the Gulf of Finland is by using an S-lay vessel. There is typically a maximum size of 16" OD for reel-lay vessels and the J-lay method is only applicable to larger water depths. The towing method is feasible; however, it is only practical for shorter pipeline lengths where a fewer number of above water tie-ins would be required.

Station keeping – Dynamic positioning
Pipelay vessels ensure station keeping either through anchor positioning or dynamic positioning. In the Gulf of Finland, due to the risk of UXOs, anchor positioning of the pipelay vessel is not an option, hence the need for a dynamic positioning vessels where side thrusters allow for precise manoeuvring.

Vessel draft
To avoid the need for more than one pipelay vessel, the minimal water depth that the pipelay vessel may operate in needs to be aligned with the water depth along the entire offshore pipeline route. The route itself is designed to avoid the shallower peaks situated throughout the Finnish archipelago. At the landfall locations, the vessel draft will determine where the pipelay vessel can be positioned when performing the pipeline pull-in operation to shore.

Tensioning capacity
The maximum tensioning capacity is determined in the static installation analysis by assessing the top tension required for the installation of the heaviest pipe sections in the water depths of the optimised pipeline route. Using a specific stinger configuration suited to the relatively shallow water depths, the top tension is calculated. It conservatively includes a dynamic amplification factor of 1.5 to incorporate increases in tension due to the motion of the vessel during pipelay.

Pipeline diameter: 20” OD plus coating (max 675 mm OD)
The pipeline diameter, including all coatings, can limit the type of vessel used. The larger pipelay vessels can install pipeline up to 60” OD, whereas smaller shallow water pipelay barges may have a lesser capacity of pipe size.

Based on the above criteria, a selection of capable pipelay vessels has been identified and listed in Table 12-11.

VESSEL	VESSEL DRAFT (WORKING)	MAX QUARTERS CAPACITY (PERSONS)	TENSIONERS (No.)	TOTAL TENSIONING CAPACITY	MINIMUM DIAMETER	MAX. DIAMETER	S-LAY (CENTER LAY)	S-LAY (SIDE LAY)	PERMANENT REEL S-LAY	REMOVABLE REEL S-LAY	MINIMAL WATER DEPTH
	[m]			[tons]	[in.]	[in.]					[m]
ALLSEAS GROUP, S.A SOLITAIRE	9	420	3	1033	2	60	•				15.2
ALLSEAS GROUP, S.A AUDACIA	8	270	3	516	2	60	•				12.2
ALLSEAS GROUP, S.A LORELAY	7	230	3	162	2	28	•				10.7
CNOOC HAI YANG SHI YOU 201	8	380	2	357	6	60	•	•			14.9
EMAS AMC LEWEK CHAMPION	6	358	2	197	6	60	•				8.5
EMAS AMC LEWEK CENTURION	8	220	3	398	4	36	•				11.9
McDERMOTT DERRICK BARGE 16 (DB16)	5	184	3	134/45	2/4	10/48		•		•	6.1
OCEANIC 5000	8	398	3	236	6	60	•			•	9.1
SAIPEM FDS	8	236	3	540/736	4	20				•	15.8
SAIPEM CASTORONE	11	702	3	736/1475	8	60	•				11.5
SAIPEM FDS 2	10	325		1475/1962	4	36				•	15.8
SEA TRUCKS GROUP JASCON 18	6	400	3	590	4	60	•			•	
SEA TRUCKS GROUP JASCON 25	5	355	2	118	4	60		•		•	
SEA TRUCKS GROUP JASCON 30	5	298	3	98	4	60		•		•	
SEA TRUCKS GROUP JASCON 34	5	355	2	118	4	60		•		•	
SEA TRUCKS GROUP JASCON 35	6	400	2	393	4	60	•			•	
SUBSEA 7 POLARIS	5	263	2	112/750	4	48/24	•				7.6
SUBSEA 7 SAPURA 3000	6	330	3	240/357	6	36/20	•				9.1
SUBSEA 7 SEVEN BOREALIS	12	399	3	268	4	24/46	•				
TECHNIP GLOBAL 1200	7	264	3	369	4	60	•				13.7
TECHNIP GLOBAL 1201	7	264	3	369	4	60	•				13.7

Table 12-11 Capable pipeline vessels for Balticconnector offshore pipeline installation

13 OIMR and de-commissioning philosophy

External and internal inspections, maintenance and repair shall be performed in accordance with the requirements given in DNV-OS F101, Ref. /1/, Section 11.

13.1 External inspection

The main purpose of the external inspections is to determine the position, configuration and external condition of the pipeline, ensuring that design requirements remain fulfilled and that no damage has occurred.

The frequency of future external inspections shall be determined based on an assessment of a number of factors. However, critical sections of the pipeline system vulnerable to damage or subject to major changes in the seabed conditions, i.e. supports and/or buried sections of the pipeline, shall be inspected at shorter intervals, normally on an annual basis.

13.2 In-line inspection

In-line inspection shall be carried out by intelligent inspection pigs to confirm the integrity of the pipeline system. In particular, the in-situ wall thickness will be determined during in-line inspection, thus determining the inner and outer pipeline steel corrosion.

The purpose of determining the inner corrosion is to confirm that the gas remains non-corrosive.

The purpose of determining the outside corrosion is to ensure that the corrosion protection system, including corrosion coating and sacrificial anodes, is undamaged and working as intended.

Operational pigging inspections with intelligent pigs shall be carried out at 4-5 years intervals.

13.3 Maintenance and repair

A recommended practice for pipeline repair is given in DNV-RP-F113, Ref. /11/.

Pipeline damage may be caused by internal and external corrosion, unstable seabed conditions, or anchors and dropped objects from the surface. The risk of damage has been assessed in the *QRA Report*, Ref. /33/.

The extent of possible damage may vary from insignificant to a fully buckled and/or ruptured pipeline.

A pipeline repair philosophy, which takes into consideration the risk of damage to the pipeline and the commercial costs of a gas transmission stop, shall be established before pipeline commissioning.

The repair philosophy shall describe the requirements for spares and availability of installation/repair vessels and equipment.

13.4 De-commissioning philosophy

It is expected that the pipeline may be left in place after the design life is exceeded.

It is considered unlikely that the pipeline will be completely removed.

The de-commissioning operations involve:

- Cleaning of the pipeline
- Survey
- Trenching and backfilling of sections not buried sufficiently
- Engineering and management

The pipeline is expected to be left in place, cleaned and filled with seawater. The pipeline ends at the landfall shall be properly secured so the pipeline will not present any danger or nuisance.

A preliminary assessment of the time that will pass before the pipeline decomposes is based on experience and extrapolation of observed corrosion rates. It is estimated that the sacrificial anodes that protect against corrosion have a realistic lifetime somewhere between 60–100 years, which depends of the final design of the anodes. These figures are based on experience. Full decomposition of the line pipe steel is, however, expected to take much longer. It is estimated to take more than 1000 years to fully decompose.

14 References

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/2/ DNV-RP-C203, Fatigue Design of Offshore Steel Structures, October 2012

/3/ DNV-RP-C205, Environmental Conditions and Environmental Loads, April 2014

/4/ DNV-RP-F102, Pipeline field joint coating and field repair of linepipe coating, May 2011

/5/ DNV-RP-F105, Free Spanning Pipelines, February 2006

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/8/ DNV-RP-F109, On-Bottom Stability Design of Submarine Pipelines, November 2011

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/32/ Balticconnector Seabed Survey, *Geophysical Survey and ROV Inspection, 2013 Survey*, MMT, Doc. No. 101501-GAS-MMT-SUR-DWG-ALIGN(001-020), Rev. A

Balticconnector FEED Documentation

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/34/ Balticconnector, *Pipeline Design Basis*, Doc. No. 30614_4-05C-00001, Rev. 02

/35/ Balticconnector, *Metocean Data Report*, Doc. No. 30614_4-05C-00003, Rev. 02

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APPENDIX I. External anti-corrosion coatings

No.	Anti-corrosion coating type	Pros	Cons
1	Three Layer Polyethylene	• Excellent adhesion and chemical resistance due to FBE layer within. • High resistance to cathodic disbondment. Does not shield cathodic protection current. • Has very good resistance to abrasion and sharp impacts. • Low propensity to absorb moisture and salts. • Ease and reasonable cost of field joint coating application. • Sa. 3 surface preparation is minimum requirement necessary to achieve desired field joint coating quality. • Lower cathodic protection requirements, i.e. much less anodes when compared to AE and FBE. • Highest shear strength achievable between coating and CWC amongst FBE and 3-LPE / 3-LPP coating systems considered with minimum 36.3 Psi (250kPa) obtainable. • Reduction in operating expenditure (OPEX) could be beneficial when compared to other systems.	• Comparatively higher capital expenditure (CAPEX) when used as an offshore coating system when compared to FBE or AE.
2	Asphalt Enamel (AE)	• Low propensity to absorb moisture and salts. • Higher impact resistance than FBE. • Comparatively lower capital expenditure (CAPEX) when used as an offshore coating system when compared to FBE. • Ease and lower cost of field joint coating application. • Sa. 3 surface preparation is minimum requirement necessary to achieve desired field joint coating quality. • Impingement concrete weight coating application with no pinholes.	• Higher cathodic protection requirements, i.e. more anodes when compared to 3-LPE / 3-LPP coating system. • Low adhesion properties. • Reasonable resistance to cathodic disbondment but lower than 3LPE. • Lowest shear strength achievable between coating and CWC amongst FBE and 3-LPE / 3-LPP coating systems considered with minimum 22 Psi (150kPa) obtainable.

Table I-1 Comparison of AE and 3LPE coating

APPENDIX II. RFO / pre-commissioning philosophy

Flooding and hydrotesting

When all construction activities (pipelay, tie-in, trenching, crossing construction and artificial backfilling) have been carried out, the final integrity of the installed pipeline is documented by hydrostatic testing. This requires that the pipeline be water-filled, using seawater pumped into the pipeline through a simple water winning arrangement that includes filtering. If the pipeline is subjected to on-bottom stability issues during the temporary phase, the pipeline can be flooding immediately after pipelaying to achieve stability.

To prevent internal corrosion of the linepipe steel, the seawater may be treated with oxygen scavengers and/or biocides. The oxygen scavenger removes the oxygen which may fuel corrosion, and the biocide prevents the growth of anaerobic bacteria.

A typical oxygen scavenger is sodium bisulphite ($NaHSO_3$), a dosage of 65mg/l (ppm) being required to for an oxygen concentration of 10ppm. A common biocide is glutaraldehyde at an active concentration of 50 – 75mg/l (ppm). As glutaraldehyde reacts with sodium bisulphite the oxygen scavenger should be given a few minutes reaction time before the biocide is added, or alternatively an over dosage must be used. Some commercially available sodium bisulphites are combined with a catalyst, which may reduce the requirement for time delay or over dosage.

An alternative biocide is sodium hydroxide (NaOH), also known as caustic soda or lye. To reach a pH of 10.3, which is lethal to most organisms, a dosage of 0.4 – 0.6l/m3 of 30% NaOH is needed. However, the use of lye will result in large amounts of precipitated carbonates and hydroxides, which may impede the function of valves, and form calcarious deposits that are not easily removed from the pipe wall.

However, as any oxygen in the seawater will quickly be consumed by negligible rust formation, and the risk of bacterial contamination is low, any treatment of the test water may be omitted, in particular if the residence time in the pipeline does not exceed 60 days.

The hydrostatic testing comprises a strength test as well as a leak test, and is carried out by pressurising the water to the specified leak test pressure, which is kept during the specified holding period. The holding period shall take into account that time is needed for temperature variation stabilisation. The holding period should not be less than 24 hours, after stabilisation has been documented. During the holding period the pressure is closely monitored, and any pressure drop which cannot be ascribed to variations in atmospheric pressure, water levels or seawater temperature signals a leak, which must then be localised. To facilitate leak detection the test water can be mixed with a powerful dye or a hydrocarbon tracer, which can be sensed by subsea leak detection equipment that is towed along the pipeline.

Due to environmental concerns, the use of dye can be minimised by mounting dye sticks at critical locations, such as tie-in points. Dye sticks or dye applied as a paint are inserted by divers just prior to tie-in operations. The dye stick can, again for environmental consent reasoning, be made of what is popularly labelled ‘invisible’ dye, which is fluorescent and visible only by a diver carrying an inspection tool.

Should a leak occur, which has been known to happen, it normally takes the form of a violent rupture, which is easily localised even if the pipeline has been trenched and backfilled. If a visual survey does not suffice to locate the failure, it is possible to launch a ‘pinger’ pig, which can be tracked acoustically until it stops at the rupture.

If there is an environmental concern of using dye, alternative means of leak detection can be adopted as listed below, DNV-RP-F302, Ref. /13/.

- Active acoustic methods
- Bio sensor methods
- Capacitance methods
- Fibre optic methods
- Methane sniffer methods
- Optical camera methods
- Passive acoustic methods
- Mass balance methods

Gauging and cleaning

It shall be documented that there are no dents in the linepipe wall, which could induce failure in the long term, or obstruct the passage of cleaning and batching pigs. For this purpose gauging and caliper pigs are propelled through the pipeline during water filling. The caliper pig is a so-called intelligent pig, equipped with sensors that measure the internal diameter at a number of points around the circumference, and it is not normally used during construction. The device is sufficiently sensitive to pick up the individual girth welds, and produces a chart showing the average bore against the distance travelled. In this way any anomaly can be located for diver inspection and cut out if necessary.

The gauging pig is normally a simple aluminium plate, which during construction activities is recovered and inspected. Since a successful gauging run is often a contractual interface, and certainly a key component in the insurance of the pipeline, the contractor will try to perform gauging as early as possible. More than one gauge plate is often propelled through the line, particularly when the installation including pre-commissioning is split between more contracts.

According to DNV-OS-F101, Ref. /1/, the diameter of the gauge plate should be 97 % of the nominal pipe ID, but a smaller plate diameter may well be typical in order to take account of weld root penetration and misalignment. The gauging pig is normally incorporated in one of the pig trains used to water fill and clean the pipeline interior, as shown in Figure II-1 below, after which the test water is displaced from the pipeline.

During and after water filling, the pipeline interior shall be cleaned. The cleaning trains include both brush pigs and swabbing pigs, the latter removing any brushes that may have broken off. The pig trains are normally propelled by the treated seawater pumped in for the purpose of the hydrotesting, but further cleaning by running brush and swabbing pigs in air may take place during and after de-watering. In Figure II-1 a typical flooding, cleaning, and gauging pig train is shown. Note that the length of the train is 900 m.

The gel slug is discharged at the receiving end, commercially available gels being environmentally sound and approved by agencies such as the UK Centre for Environment, Fisheries and Aquaculture Sciences (Cefas).

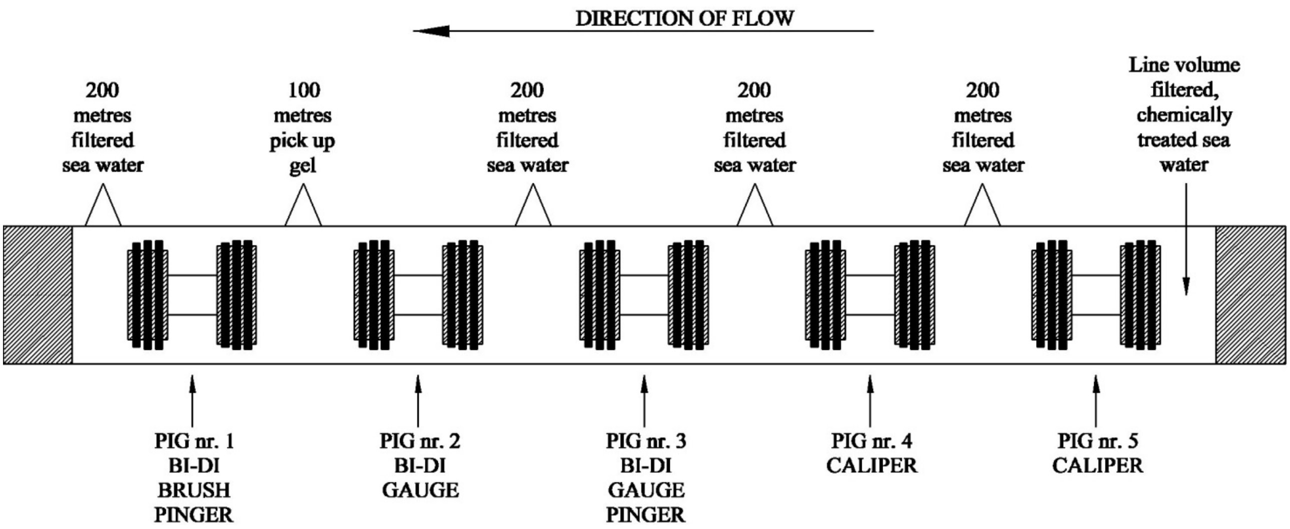


Figure II-1 Example of pig train used for flooding, cleaning and gauging

As seen in Figure II-1 the cleaning operation may be facilitated by gel-plug technology. A gel is a plastic fluid with the capability to pick up loose and loosely adhering solids. The gel slug is inserted into the pipeline, followed by an appropriately designed scraper pig. The train will consist of more scraper pigs collecting any gel slipping by the pig driving the gel. The plastic fluid will move through the pipeline in a manner known as plug flow. The central part of the slug moves as a semi-solid plug with little exchange of material with the fluid making up the annular flow region adjacent to the pipe wall, which moves at a velocity lower than the mean velocity of the total gel plug. The core of the gel in front of the mechanical pig, moving faster than the gel on the outside closer to the wall, creates a tractor action, pulling and lifting the debris-laden gel away from the front of the pig and into the gel plug. The debris, which would remain in front of the pig in a conventional operation, is thus picked up and eventually distributed throughout the length of the slug. Gels can be produced with a range of viscosities, including solid gel pigs, capable of removing wax or paraffin deposits.

De-watering and drying

The de-watering operation must be planned with a view towards the disposal of the water, particularly if it is treated with corrosion inhibitors, as dumping in coastal areas is not likely to be acceptable. Thus, for the Balticconnector a temporary outfall pipeline must be constructed so the water can be discharged at sea, after separation of solids in a settling pond. The water is discharged through a diffuser head to ensure dilution to a concentration that reduces risk to marine life. These problems can be mitigated by flooding with untreated test water, as discussed in Section 9.2 above, or using oxygen scavenger only.

Figure II-2 Typical de-watering pig

Pipeline de-watering runs are carried out by means of air-propelled pig trains during or after cleaning, see above. A typical de-watering pig is shown in Figure II-2.

As the pipeline is to be used for natural gas, complete drying is necessary as any residual water may react with the gas to form hydrates, which may obstruct the flow and impair the proper functioning of valves. The presence of water will also make any impurities of hydrogen sulphide (H2S) and carbon dioxide (CO2) highly corrosive. To dry the pipeline the following methods may be used, alone or in combination:

- Methanol (or glycol) swabbing
- Hot air drying
- Vacuum drying

In the swabbing method a batch of methanol or tri-ethylene glycol (TEG) is enclosed between pigs and propelled through the pipeline by compressed air. Residual water will be dissolved in the hygroscopic substance, leaving a film that is mostly methanol or glycol.

An alternative procedure, which combines cleaning and drying in one operation, is gel pigging, as described above. Modern gel-forming agents can produce gels from an array of liquid components. By incorporating gels based on hygroscopic fluids, such as methanol, into the cleaning train the water is removed along with the debris.

Hot air drying utilises the ability of hot air to contain a large amount of water as vapour, whereas vacuum drying relies upon the lowering of the boiling point of water at low pressures. For the 80 km Balticconnector the vacuum pumps will have to work for several days to decrease the pipeline pressure below a few millibar. To limit the time vacuum drying is often used as the last step, i.e. after most of the water has been removed by swabbing or gel pigging.

Nitrogen purging and gas filling
(only required if there is a significant duration between pre-commissioning and operation)
To prevent any internal corrosion between pre-commissioning and operation, in case the pipeline is not immediately operational, the pipeline may be filled with a non-corrosive gas, such as nitrogen. Provided the pipeline has been dried as described above, a typical nitrogen purity would be 95% (i.e. 95% N_2, 5% atmospheric gasses). However, if any free water is present the nitrogen should constitute more than 99.98% of the gas.

For a vacuum dried gas pipeline the nitrogen is simply let in, in other cases the air in the pipeline is displaced by nitrogen, a process known as purging. Liquid nitrogen is vaporised through heat exchangers and injected into the pipeline. To guarantee a low level of oxygen, the amount of injected nitrogen should be approximately twice the volume of the pipeline.

Nitrogen is introduced at the upstream end of the pipeline with a –50°C dew point or lower, at a controlled rate to prevent over-compression and subsequent re-condensation of water. Dew point control is critical, and the infill rate and controlling pressure shall be determined to ensure that at no time the dew point is above –20°C. Whilst the initial purge is performed regular monitoring of the oxygen content of the atmosphere in the vicinity of the discharge point shall take place.

Nitrogen shall be discharged and the dew point monitored until the separation pig has been received, during which time the nitrogen dew point is to be –20°C or drier at atmospheric pressure at the outlet end of the pipeline. The pipeline shall then be packed with nitrogen to a final pressure of at least 1.1 barg. A higher nitrogen overpressure may be specified to ensure that pinhole leaks will result in gas outflow rather than water ingress.

If the pipeline is really completely clean and dry, and is taken into operation within a reasonable time span (one year, say) after pre-commissioning, there is no need to fill the pipe with nitrogen or any other form of non-corrosive gas.

When completed, the pipeline is found in what would normally be the final ‘hand-over’ condition, and the installation or pre-commissioning contractor will de-mobilise. Gas filling of the pipeline takes place during commissioning of the pipeline system, including the onshore sections and the compressor station(s). The commissioning procedure, prepared by the pipeline operator, shall focus on the on-shore compressor stations, and not be limited to the activity related to the offshore pipeline section.

APPENDIX III. Wall thickness design calculations

HEADING

HFW, ZONE-2

Balticcinnector Gas pipeline
20" Gas pipeline-HFW manufacturing process
Zone-2

Open case | Save case | Delete case

OS-F101 V01-03
01.03.2009
Ramboll Oil & Gas

DNV

GEOMETRY

OD	[mm]:	508.00	t_{nom} [mm]:	12.70	t_{fab}: [mm]	0.70	t_{corr} [mm]:	0.00	f_0 [%]: 1.50	α_{gw} [-]: 0.80

MATERIAL

SMYS [MPa]:	450.0	$f_{y,temp}$ [MPa]:	0.0	E [MPa]:	2.07E+05	α_h [-]:	0.93	α_{fab} [-]: 0.93
SMTS [MPa]:	535.0	$f_{u,temp}$ [MPa]:	0.0	ν [-]:	0.3			Suppl. Req. U: ☐

LOADS

p_{design} [barg]:	80.0	@ [m]:	0.0	ρ_{design} [kg/m^3]:	65.0	γ_{inc} [-]:	1.10	Depth [m]:	0.0	Max. elevation:	99.9
p_{test} [barg]:	92.4	@ [m]:	0.0	ρ_{test} [kg/m^3]:	1009.0			ρ_{ext}: [kg/m^3]:	1009.0	Min. elevation:	0.0
p_{min} [barg]:	0.0	@ [m]:	0.0	ρ_{min} [kg/m^3]:	50.0						

WALL THICKNESS DESIGN

Calculate ☑ Burst ☑ Collapse & Propagating

$p_{syst\text{-}test}$ [barg]: 92.4
$p_{mill\text{-}test}$ [barg]: 209.0

Safety Class	Corr.:	Der.:	Pmin:	Code Check	t_{req} [mm]:	utility [-]
HIGH	☑	☑		Burst, op.:	7.35	0.548
SYSTEM TEST	☐	☑		Burst, sys. test:	6.08	0.442
HIGH	☑	☑	☐	Collapse:	8.46	0.290
LOW	☑	☑		Prop. Buckling:	11.90	0.851

LOAD INTERACTION

Calculate ☐ Load Controlled ☐ Displacement Controlled

γ_c [-]: 0

		Func.	Env.
Moment [kNm]	[kNm]:	1.00E+02	1.50E+01
Axial force [kN]	$_E$ [kN]:	1.40E+03	4.00E+02
Strain [%]	ε_E [%]:	1.24E+00	

Condition	Safety Class	Corr.:	Der.:	Pmin:	Code Check	t_{req} [mm]:	utility [-]
SYSTEM TE	HIGH	☑	☑	☐	LCC, comb. a:		
					LCC, comb. b:		
				☐	DCC, comb. a:		
					DCC, comb. b:		

END OF PAGE

APPENDIX IV. Trawl impact analysis calculations

Gasum		RAMBOLL	
Project:	**Balticconnector FEED – Offshore pipeline** Trawl impact assessment 20” gas pipeline across the Gulf of Finland	**Project no.:**	**30614_4**
Author: Checker: Approver:	SDR FARH NC	Date: Rev. No.:	2015-11-04 01

Scope:
This MathCad sheet provides the analytical solution for the trawl impact energy and as a consequence; denting of the pipeline.

The references to the equations are given at the right hand side of the equations.

Code reference:
/1/ DNV-RP-F111, Interference Between Trawl Gear and Pipelines, October 2010

/2/ DNV-RP-F107, Risk Assessment and Pipeline Protection, October 2010

Output from the calculation are the following data:
- Impact energy associated with the steel mass of the trawl board and clump weight
- Penetration depth of trawl board and clump weight into concrete coating
- Acceptance criteria

Input data

Pipe dimensions and material data

Outer diameter of steel pipe $OD = 508mm$

Nominal wall thickness $t_{nom} = 12.7mm$

Inner diameter of pipe $ID = OD - 2 \cdot t_{nom} = 482.6 \cdot mm$

Corrosion allowance $t_{corr} = 0mm$

Thickness of 3-layer PE system $t_{PE} = 3.5mm$

Thickness of concrete coating $t_{cc} = 45mm$ Minimum Concrete coating is considered for conservative estimates

Outer diameter of the pipeline $OD_{tot} = OD + 2 \cdot (t_{PE} + t_{cc}) = 605 \cdot mm$

Specified minimum yield stress $SMYS = 450MPa$

Material strength factor $\alpha_u = 0.96$ DNV-OS-F101 Table 5.6

Temperature de-rating value of the yield stress $f_{ytemp} = 0MPa$

Yield stress $f_y = (SMYS - f_{ytemp}) \cdot \alpha_u = 432 \cdot MPa$

Crushing strength of concrete coating $Y = 105 \cdot MPa$

Trawl data

Trawl board mass $m_{t\_trawl} = 3000kg$

Clump weight $m_{t\_clump} = 3000kg$

Hydrodynamic added mass for the Clump weight $m_{a\_clump} = 1350kg$

Hydrodynamic added mass co-efficient $C_a = 2.14$

Hydrodynamic added mass for the trawl board $m_{a\_trawl} = C_a \cdot m_{t\_trawl} = 6420\,kg$ DNV-RP-F111 Table 3-1

The reduction factors R_{fa} and R_{fs} is chosen from DNV-RP-F111, Figure 3-3

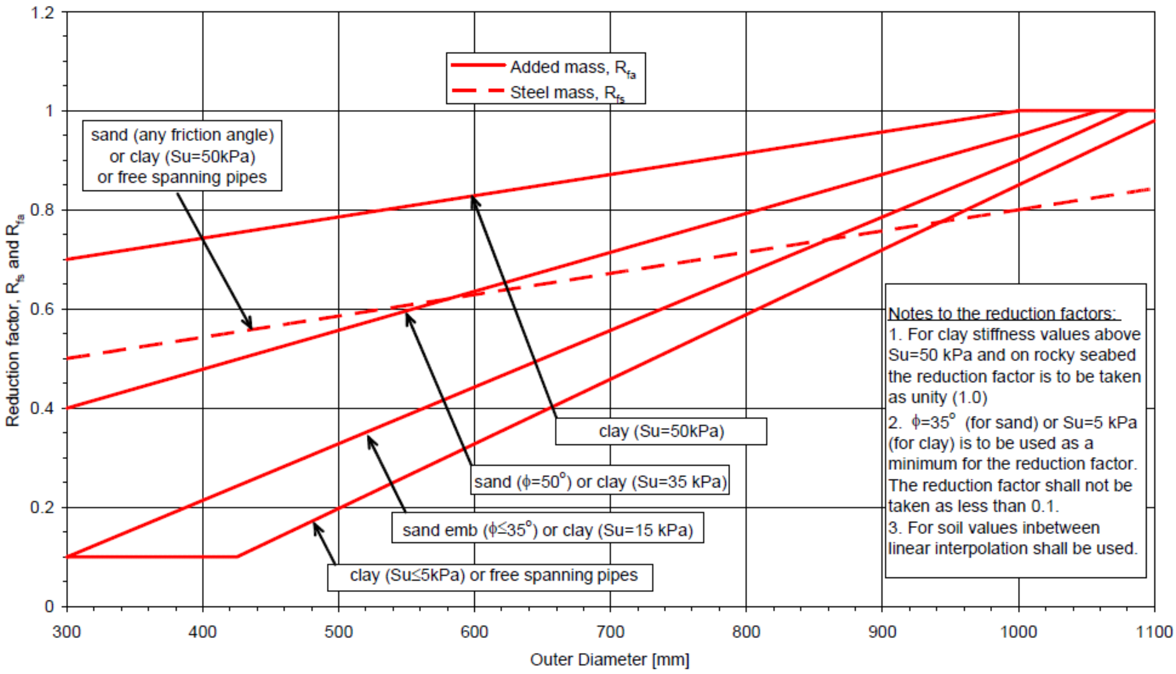


Figure 3-3
Reduction factors for concrete coated and bare steel pipes.

<u>Note: Reduction factors are conservatively take as 1</u>

Reduction factor for added mass $R_{fa} = 1$ DNV-RP-F111 Section 3.4.2

Reduction factor depending on the outer pipe diameter for steel mass $R_{fs} = 1$ DNV-RP-F111 Section 3.4.2

Span height correction factor $C_h = 1$ DNV-RP-F111 For polyvalent trawl board with a max.span height, Conservative assumed as 1

Trawl board velocity $V_{t.} = 2\frac{m}{s}$

Clump weight velocity $V_{c.} = 2\frac{m}{s}$

Lateral bending stiffness of the board $k_b = 1 \cdot 10^7 \frac{N}{m}$ DNV-RP-F111, Table 3-1

CALCULATIONS

- **Trawl Impact**

Impact energy associated with the steel mass of the trawl board	$E_s = R_{fs} \cdot \frac{1}{2} \cdot m_{t\_trawl} \cdot (C_h \cdot V_{t.})^2 = 6 \cdot kJ$	DNV-RP-F111 Equation 3.1
The impact force and energy associated with the hydrodynamic added mass of the trawl board	$F_b = C_h \cdot V_{t.} \cdot \sqrt{m_{a\_trawl} \cdot k_b} = 506.75 \cdot kN$	DNV-RP-F111 Equation 3.2 and Equation 3.3

$$E_a = \text{if}\left[R_{fa} \cdot \frac{2 \cdot F_b^{\,3}}{75 \cdot f_y^{\,2} \cdot (t_{nom} - t_{corr})^3} \le \frac{1}{2} \cdot m_{a\_trawl} \cdot (C_h \cdot V_{t.})^2, R_{fa} \cdot \frac{2 \cdot F_b^{\,3}}{75 \cdot f_y^{\,2} \cdot (t_{nom} - t_{corr})^3}, \frac{1}{2} \cdot m_{a\_trawl} \cdot (C_h \cdot V_{t.})^2\right]$$

$$E_a = 9.08 \cdot kJ$$

Conservative estimate of kinetic energy absorbed by local deformations of the coating and the pipe wall	$E_{loc\_trawl} = \max(E_s, E_a) = 9.08 \cdot kJ$	DNV-RP-F111 Equation 3.6

- **Clump Impact**

Impact energy associated with the steel mass of the clump weight	$E_{loc\_clump} = R_{fs} \cdot \frac{1}{2} \cdot (m_{t\_clump} + m_{a\_clump}) \cdot (V_{c.})^2 = 8.7 \cdot kJ$	DNV-RP-F111 Equation 3.11

Energy absorption of coatings

(If the pipe is not coated then this section should be ignored)

Energy absorption of the concrete coating	$E_{concrete} = 40kJ$	DNV-RP-F107
Energy absorption in PE coating	$E_{PE} = 0kJ$	DNV-RP-F107 Table 7
Energy absorbed by field joint coating	$E_{fjc} = 15kJ$	DNV-RP-F107 Table 7 (for polymer coatings)

<u>Impact energy transmitting to the steel pipe due to impact from trawl board</u>

<u>For Concrete coating</u>

Impact energy transmitted to the bare steel pipe from the trawl board (for the concrete coating section)	$E_{total\_trawl} = E_{loc\_trawl} - E_{concrete} - E_{PE} = -30.92 \cdot kJ$

The result shows that the impact energy from the trawl board is completely absorbed by the concrete coating.

For field Joint

Impact energy transmitted to the bare steel pipe from the trawl board (for the field joint section)

$$E_{fjc\_trawl} = E_{loc\_trawl} - E_{fjc} - E_{PE} = -5.92 \cdot kJ$$

The result shows that the impact energy from the trawl board is completely absorbed by the field joint coating.

Impact energy transmitting to the steel pipe due to impact from Clump weight

For Concrete coating

Impact energy transmitted to the bare steel pipe from the clump weight (for the concrete coating section)

$$E_{total\_clump} = E_{loc\_clump} - E_{concrete} - E_{PE} = -31.3 \cdot kJ$$

The result shows that the impact energy from the clump weight is completely absorbed by the concrete coating.

For field Joint

Impact energy transmitted to the bare steel pipe from the clump weight (for the field joint section)

$$E_{fjc\_clump} = E_{loc\_clump} - E_{fjc} - E_{PE} = -6.3 \cdot kJ$$

The result shows that the impact energy from the clump weight is completely absorbed by the field joint coating.

Penetration depth of trawl board in concrete coating

Footprint width of impacting object $b_{trawl} = 20mm$ Assumed

Depth of impacting object $h_{trawl} = 3.2m$

Trawl weight penetration depth, according to Eq. 4 (DNV-RP-F107)

$$x_{0.trawl\_4} = \frac{E_{loc\_trawl}}{Y \cdot b_{trawl} \cdot h_{trawl}} = 1.35 \cdot mm$$

Trawl weight penetration depth, according to Eq. 5 (DNV-RP-F107)

$$x_{0.trawl\_5} = \left[\frac{\left(\frac{3E_{loc\_trawl}}{4 \cdot Y \cdot b_{trawl}}\right)^2}{OD_{tot}}\right]^{\frac{1}{3}} = 25.9 \cdot mm$$

Maximum Trawl weight penetration depth

$$x_{0.trawl} = \max\left(x_{0.trawl\_4}, x_{0.trawl\_5}\right)$$

Height of impacting object

$$h_{trawl} = 2 \cdot \sqrt{OD_{tot} \cdot x_{0.trawl} - x_{0.trawl}^2} = 0.24 \cdot m$$

The energy absorbed is a function of the penetrated volume and the crushing strength of the concrete.

Solving the expression for penetration depth:

Guess on penetration $x = 25mm$

Given

$$E_{loc\_trawl} = \int_0^x Y \cdot b_{trawl} \cdot 2 \cdot \sqrt{OD_{tot} \cdot z - z^2}\, dz$$

$x_{0.trawl} = \text{Find}(x)$ $\quad x_{0.trawl} = 26.13 \cdot mm$

Corresponding height of impacting object $\quad h_{trawl} = 2 \cdot \sqrt{OD_{tot} \cdot x_{0.trawl} - x_{0.trawl}^2} = 0.25m$

Penetration depth of clump weight in concrete coating

Footprint width of impacting object $\quad b_{clump} = 20mm$ $\quad$ Assumed

Depth of impacting object $\quad h_{clump} = 1.0m$

Clump weight penetration depth, according to Eq. 4 (DNV-RP-F107)

$$x_{0.clump\_4} = \frac{E_{loc\_clump}}{Y \cdot b_{clump} \cdot h_{clump}} = 4.14 \cdot mm$$

Clump weight penetration depth, according to Eq. 5 (DNV-RP-F107)

$$x_{0.clump\_5} = \left[\frac{\left(\frac{3E_{loc\_clump}}{4 \cdot Y \cdot b_{clump}} \right)^2}{OD_{tot}} \right]^{\frac{1}{3}} = 25.18 \cdot mm$$

Maximum Clump weight penetration depth $\quad x_{0.clump} = \max(x_{0.clump\_4}, x_{0.clump\_5})$

Height of impacting object $\quad h_{clump} = 2 \cdot \sqrt{OD_{tot} \cdot x_{0.clump} - x_{0.clump}^2} = 0.24 \cdot m$

The energy absorbed is a function of the penetrated volume and the crushing strength of the concrete.
Solving the expression for penetration depth:

Guess on penetration $\quad x = 25mm$

Given

$$E_{loc\_clump} = \int_0^x Y \cdot b_{clump} \cdot 2 \cdot \sqrt{OD_{tot} \cdot z - z^2}\, dz$$

$x_{0.clump} = \text{Find}(x)$ $\quad x_{0.clump} = 25.39 \cdot mm$

Corresponding height of impacting object $\quad h_{clump} = 2 \cdot \sqrt{OD_{tot} \cdot x_{0.clump} - x_{0.clump}^2} = 0.24m$

Results

- **Trawl board**

Kinetic energy from the trawl board impact	$E_{loc\_trawl} = 9.08 \cdot kJ$	
Impact energy transmitted to bare steel pipe (from concrete coating thickness)	$E_{total\_trawl} = -30.92 \cdot kJ$	if -ve total impact energy is absorbed by coating, no dent is anticipated on steel pipe
Impact energy transmitted to bare steel pipe (from field coating thickness)	$E_{fjc\_trawl} = -5.92 \cdot kJ$	
Penetration of trawl board in concrete coating	$x_{0.trawl} = 26.13 \cdot mm$	

- **Clump Impact**

Kinetic energy from the clump weight impact	$E_{loc\_clump} = 8.7 \cdot kJ$	
Impact energy transmitted to bare steel pipe (from concrete coating thickness)	$E_{total\_clump} = -31.3 \cdot kJ$	if -ve total impact energy is absorbed by coating, no dent is anticipated on steel pipe
Impact energy transmitted to bare steel pipe (from field coating thickness)	$E_{fjc\_clump} = -6.3 \cdot kJ$	
Penetration of clump weight in concrete coating	$x_{0.clump} = 25.39 \cdot mm$	

Calculation for estimating the acceptable trawl gear weight and clump weight for the allowable dent size as per DNV-RP-F111, Section 6.2, Table 6-2.

In order to estimate the acceptable trawl board and clump weight that the pipe can withstand for the given impact frequency, the impact (tow) velocity is fixed as stated above. Based on the allowable dent size as per equation 6.1 of DNV-RP-F111, the acceptable trawl board and clump weight is calculated for impact on the concrete coating and field joint coating section.

Acceptance Criteria

Impact frequency $f = 0.9$

DNV-RP-F111 Table 6-1
Impact frequency < 1 events per km per year
Refer Design basis

$$\eta(f) = \begin{cases} 0 & \text{if } f > 100 \\ 0.3 & \text{if } 1 \le f \le 100 \\ 0.7 & \text{otherwise} \end{cases}$$

DNV-RP-F111 Table 6-2

Allowable dent size simplified method

Allowable permanent indentation of the pipe shell due to Trawl board/clump weight impact

$$H_{pc\_c\_allowable} = \eta(f)\cdot 0.05\cdot OD = 17.78\cdot mm$$

DNV-OS-F111, Eq. 6.1

Guess on impact force experience by pipe shell

$$F_{sh\_allowable} = 800kN$$

Given

$$H_{pc\_c\_allowable} = \left[\frac{F_{sh\_allowable}}{5\cdot f_y\cdot (t_{nom} - t_{corr})^{\frac{3}{2}}}\right]^2 - \left[\frac{F_{sh\_allowable}\cdot\sqrt{0.005\cdot OD}}{5\cdot f_y\cdot (t_{nom} - t_{corr})^{\frac{3}{2}}}\right]$$

$$F_{sh\_allowable} = \text{Find}(F_{sh\_allowable})$$

$$F_{sh\_allowable} = 497.41\cdot kN$$

Impact energy that would be transmitted to pipe shell for the allowable permanent indentation

$$E_{total\_allowable} = \frac{F_{sh\_allowable}^{3}}{\left[\frac{75}{2}\cdot f_y^{2}\cdot (t_{nom} - t_{corr})^{3}\right]} = 8.59\cdot kJ$$

Acceptable impact energy on Concrete coating section and field joint coating

Impact energy on concrete coating section

$$E_{allow\_concrete} = E_{total\_allowable} + E_{concrete} + E_{PE}$$

$$E_{allow\_concrete} = 48.59\cdot kJ$$

Impact energy on field joint coating section

$$E_{allow\_fjc} = E_{total\_allowable} + E_{fjc} + E_{PE}$$

$$E_{allow\_fjc} = 23.59\cdot kJ$$

- **Trawl board**

Acceptable Trawl weight calculation when impacted on concrete coating section

Acceptable trawl board weight is calculated based on the minimum impact energy from the trawl board and hydrodynamic added mass

Trawl weight associated with hydrodynamic added mass

Guess $m_{t\_addedmass\_concrete} = 300kg$

Given

$$E_{allow\_concrete} = \min\left[R_{fa}\cdot\frac{2\cdot\left(C_h\cdot V_{t.}\cdot\sqrt{C_a\cdot m_{t\_addedmass\_concrete}\cdot k_b}\right)^3}{75\cdot f_y{}^2\cdot\left(t_{nom}-t_{corr}\right)^3}, \frac{1}{2}\cdot\left(C_a\cdot m_{t\_addedmass\_concrete}\right)\cdot\left(C_h\cdot V_{t.}\right)^2\right]$$

$m_{t\_addedmass\_concrete} = \mathrm{Find}\left(m_{t\_addedmass\_concrete}\right)$

$m_{t\_addedmass\_concrete} = 11351.66\cdot kg$

Trawl weight associated with trawl board weight

Guess $m_{t\_trawlmass\_concrete} = 300kg$

Given

$$E_{allow\_concrete} = R_{fs}\cdot\frac{1}{2}\cdot m_{t\_trawlmass\_concrete}\cdot\left(C_h\cdot V_{t.}\right)^2$$

$m_{t\_trawlmass\_concrete} = \mathrm{Find}\left(m_{t\_trawlmass\_concrete}\right)$

$m_{t\_trawlmass\_concrete} = 24292.55\cdot kg$

Acceptable trawl weight for allowable permanent indentation on the pipe shell when impacted on concrete coating section

$m_{t\_acceptable\_concrete} = \min\left(m_{t\_addedmass\_concrete}, m_{t\_trawlmass\_concrete}\right) = 11351.66kg$

Acceptable Trawl weight calculation when impacted on Field joint coating section

Acceptable trawl board weight is calculated based on the minimum impact energy from the trawl board and hydrodynamic added mass

Trawl weight associated with hydrodynamic added mass

Guess $m_{t\_addedmass\_fjc} = 300kg$

Given

$$E_{allow\_fjc} = \min\left[R_{fa}\cdot\frac{2\cdot\left(C_h\cdot V_{t.}\cdot\sqrt{C_a\cdot m_{t\_addedmass\_fjc}\cdot k_b}\right)^3}{75\cdot f_y{}^2\cdot\left(t_{nom}-t_{corr}\right)^3}, \frac{1}{2}\cdot\left(C_a\cdot m_{t\_addedmass\_fjc}\right)\cdot\left(C_h\cdot V_{t.}\right)^2\right]$$

$m_{t\_addedmass\_fjc} = \mathrm{Find}\left(m_{t\_addedmass\_fjc}\right)$

$m_{t\_addedmass\_fjc} = 5669.67\cdot kg$

Trawl weight associated with trawl board weight

Guess $m_{t\_trawlmass\_fjc} = 300kg$

Given

$$E_{allow\_fjc} = R_{fs}\cdot\frac{1}{2}\cdot m_{t\_trawlmass\_fjc}\cdot\left(C_h\cdot V_{t.}\right)^2$$

$m_{t\_trawlmass\_fjc} = \mathrm{Find}\left(m_{t\_trawlmass\_fjc}\right)$

$m_{t\_trawlmass\_fjc} = 11792.55\cdot kg$

Acceptable trawl weight for allowable permanent indentation on the pipe shell when impacted on field joint coating section

$m_{t\_acceptable\_fjc} = \min\left(m_{t\_addedmass\_fjc}, m_{t\_trawlmass\_fjc}\right) = 5669.67kg$

- **Clump weight**

Acceptable Clump weight calculation when impacted on concrete coating section

Acceptable Clump weight is calculated based on the impact energy from the Clump weight plus hydrodynamic added mass

Ratio of hydrodynamic added mass and mass of the clump weight $r = \frac{m_{a\_clump}}{m_{t\_clump}} = 0.45$

Guess $m_{c\_clumpmass\_concrete} = 300kg$

Given

$$E_{allow\_concrete} = R_{fs} \cdot \frac{1}{2} \cdot \left(m_{c\_clumpmass\_concrete} + r \cdot m_{c\_clumpmass\_concrete}\right) \cdot \left(V_{c.}\right)^2$$

$$m_{c\_clumpmass\_concrete} = \mathrm{Find}\left(m_{c\_clumpmass\_concrete}\right)$$

$$m_{c\_clumpmass\_concrete} = 16753.48 \cdot kg$$

Acceptable Clump weight for allowable permanent indentation on the pipe shell when impacted on concrete coating section

$$m_{c\_acceptable\_concrete} = m_{c\_clumpmass\_concrete} = 16753.48kg$$

Acceptable Clump weight calculation when impacted on concrete coating section

Acceptable Clump weight is calculated based on the impact energy from the Clump weight plus hydrodynamic added mass

Guess $m_{c\_clumpmass\_fjc} = 300kg$

Given

$$E_{allow\_fjc} = R_{fs} \cdot \frac{1}{2} \cdot \left(m_{c\_clumpmass\_fjc} + r \cdot m_{c\_clumpmass\_fjc}\right) \cdot \left(V_{c.}\right)^2$$

$$m_{c\_clumpmass\_fjc} = \mathrm{Find}\left(m_{c\_clumpmass\_fjc}\right)$$

$$m_{c\_clumpmass\_fjc} = 8132.79 \cdot kg$$

Acceptable Clump weight for allowable permanent indentation on the pipe shell when impacted on field joint coating section

$$m_{c\_acceptable\_fjc} = m_{c\_clumpmass\_fjc} = 8132.79kg$$

Results

Acceptable trawl board weight $m_{t\_acceptable} = \min\left(m_{t\_acceptable\_concrete}, m_{t\_acceptable\_fjc}\right) = 5670kg$

Acceptable clump weight $m_{c\_acceptable} = \min\left(m_{c\_acceptable\_concrete}, m_{c\_acceptable\_fjc}\right) = 8133kg$

APPENDIX V. Cathodic protection design calculations

Project:	**Balticconnector FEED – Offshore pipeline**	**Project no.:**	**30614_4**
	Pipeline Cathodic Protection Design 20” gas pipeline across the Gulf of Finland (Exposed Pipeline Condition)		
Author:	SDR	Date:	2016-01-11
Checker:	FARH	Rev. No.:	01
Approver:	NC		

Scope:
This sheet calculates the cathodic protection requirement for pipeline based on
- ISO 15589-2, Dec 2012,
- DNV-RP-F103, Oct 2010,
- NORSOK M-503, May 2007.

Input section

Pipeline properties inputs:

KP from	KP to	Pipeline outer diameter D_o	Pipeline wall thickness T_{steel}	Pipeline section length $L_{section}$	Pipeline OD tolerance $T_{tolerance}$	Corrosion coating thickness T_{corr}	Concrete coating thickness T_{conc}	Insulation coating thickness T_{insu}	Corrosion coating cutback length on one side $L_{cutback}$
(km)	(km)	(mm)	(mm)	(m)	(mm)	(mm)	(mm)	(mm)	(m)
0.000	80.392	508.000	12.700	$8.039 \cdot 10^4$	0.000	3.500	45.000	0.000	0.340

$No\_pipe\_sections := rows(pipe\_properties) = 1.00$ $\quad KP\_from := pipe\_properties^{\langle 1 \rangle}$ $\quad KP\_to := pipe\_properties^{\langle 2 \rangle}$

$D_o := pipe\_properties^{\langle 3 \rangle} \, mm$ $\quad T_{steel} := pipe\_properties^{\langle 4 \rangle} \, mm$ $\quad L_{section} := pipe\_properties^{\langle 5 \rangle} \, m$

$T_{tolerance} := pipe\_properties^{\langle 6 \rangle} \, mm$ $\quad T_{corr} := pipe\_properties^{\langle 7 \rangle} \, mm$ $\quad T_{conc} := pipe\_properties^{\langle 8 \rangle} \, mm$

$T_{insu} := pipe\_properties^{\langle 9 \rangle} \, mm$ $\quad L_{cutback} := pipe\_properties^{\langle 10 \rangle} \, m$

Anode properties inputs:

KP from	KP to	Buried=1 /Exposed=2	Safety Factor	Anode Thickness	Anode Length	Anode gap	Pipeline metallic surface temp	Inner Anode surface temp	Percentage volume of inserted steel in anode
			SF	T_{anode}	L_{anode}	GAP_{anode}	$Temp_{pipeline}$	$Temp_{anode}$	$Inserts_{anode}$
(km)	(km)	-	-	(mm)	(mm)	(mm)	(°C)	(°C)	(%)
0.000	80.392	2.000	1.000	40.000	600.000	80.000	50.000	25.000	0.000

$PL_{status} := Anode\_properties^{\langle 3 \rangle}$ $SF := Anode\_properties^{\langle 4 \rangle}$ $T_{anode} := Anode\_properties^{\langle 5 \rangle} \cdot mm$

$L_{anode} := Anode\_properties^{\langle 6 \rangle} \cdot mm$ $Gap_{anode} := Anode\_properties^{\langle 7 \rangle} \cdot mm$ $Temp_{pipeline} := Anode\_properties^{\langle 8 \rangle} \cdot C$

$Temp_{anode} := Anode\_properties^{\langle 9 \rangle} \cdot C$ $Inserts_{anode} := Anode\_properties^{\langle 10 \rangle} \cdot \%$

KP from	KP to	Coating breakdown Factor (Refer Note 9)				Protective Current Density		Minimum negative potential	Design closed-circuit potential for anode material	Environmental resistvity
		Corrossion Coating		Field Joint Coating						
		Initial	Avg. yearly increase	Initial	Avg. yearly increase	Mean	Final			
		f_i	Δf	f_{ifjc}	Δf_{fjc}	i_m	i_f	E_c	E_a	ρ_{res}
(km)	(km)	-	-	-	-	(mA/m^2)	(mA/m^2)	(V)	(V)	(Ω-m)
0.000	80.392	$4.000 \cdot 10^{-3}$	$2.000 \cdot 10^{-4}$	0.000	0.000	120.000	120.000	-0.800	-1.050	1.500

$f_i := factor\_demand^{\langle 3 \rangle}$ $\Delta f := factor\_demand^{\langle 4 \rangle}$ $f_{ifjc} := factor\_demand^{\langle 5 \rangle}$ $\Delta f_{fjc} := factor\_demand^{\langle 6 \rangle}$

$i_m := factor\_demand^{\langle 7 \rangle} \cdot \frac{mA}{m^2}$ $i_f := factor\_demand^{\langle 8 \rangle} \cdot \frac{mA}{m^2}$ $E_c := factor\_demand^{\langle 9 \rangle} \cdot V$

$E_a := factor\_demand^{\langle 10 \rangle} \cdot V$ $\rho_{res} := factor\_demand^{\langle 11 \rangle} \cdot \Omega \cdot m$

General Inputs

Does protective current density shall be updated as per ISO 15589-2:2012 Sec 7.4.4 for elevated temperature

$Update\_I_{density} :=$ Yes / No

$Update\_I_{density} = \text{"Yes"}$

Design life (refer Design Basis) $T_{design} := 50 \cdot yr$

Electrical resistivity of pipe material (refer Design Basis) $\rho_{Me} := 0.2\Omega \cdot m \cdot 10^{-6}$

Pipe Joint length (refer Design Basis) $P_{jtlength} := 12.2m$

Anode utilization factor (minimum) (ISO 15589-2:2012 Sec 8.4or DNV-RP-F103 Sec 5.4.2 or M-503 Sec 5.8.2) $u := 0.80$

Density of anode material (refer Design Basis) $\rho_{anode} := 2750 \cdot \frac{kg}{m^3}$

Internal anode coating (refer Design Basis) $t_{internal} := 0.1 \cdot mm$

Design premises:

1. fi and Δf are Linepipe Coating breakdown factors (Sec. 7.5, table 3/4, ISO 15589-2:2012 or Annex 1, table A.1, DNV-RP-F103)
2. f_{ifjc} and Δf_{fjc} are Field Joint Coating breakdown factors (Sec. 7.5, table 3/4, ISO 15589-2:2012, Annex 1, table A.2, DNV-RP-F103)
3. i_m and i_f are Protective Current Density (mean and final) this varies based upon the burial status of the pipeline, Mean and Final is assumed same as per Sec. 7.4,1, Note 1, ISO 15589-2:2012. For DNV-RP-F103 the protective current density depends on fluid temperature and burial status, refer Sec. 5.2.4, table 5-1.
4. Pipeline operating with temperatures in excess of 25°C on the outside metallic surface of the pipe require an adjustment to the design current density. The design current densities shall be increased by $1mA/m^2$ for each degree Celsius of the metal/environmental above 25°C upto 100°C as per Sec. 7.4,4, Note 1, ISO 15589-2:2012. <u>Note this elevated temp is taken care in the below calculation based on the pipeline temperature (Applicable only for ISO).</u>
5. For riser section selected current densities i_m and i_f shall be 10 mA/m^2 higher than for the equivalent riser or pipeline below the splash zone, Entire riser section is assumed as splash zone.
6. E_c recommended minimum negative protection potential "Aerobic environment" (Sec. 7.2.1, table 1 in ISO 15589-2:2012 or Sec. 5.6.11, DNV-RP-F103) and E_a design closed-circuit potential of anode material anode depends on burial status of pipeline (Sec 8.3, table 5 in ISO 15589-2:2012)
7. It is to be noted that the inner anode surface temperature shall be considered in the design as this would give most conservative results. The inner anode surface temperature can be calculated considering heat loss through the pipe wall and coatings. However the user can select same temperature as that of the fluid temperature in order to be more conservative on design.
8. Environmental resistivity is taken as seawater resistivity for exposed pipeline and can be taken from Appendix-1 figure A-1 ISO 15589-2:2012 or based on experience. For buried pipeline the environmental resistivity is taken as seabed resistivity. If no data is available the value can be taken as 1.5 Ω-m.
9. If the coating breakdown factor is selected from ISO 15589-2:2012, the values given in ISO are the total coating breakdown factor including field joint coating and infill. Input has to be provided only for corrosion coating breakdown factor, and for field joint coating enter zero as breakdown factor.

Calculations Section

No. of Pipeline Section for analysis $n := No\_pipe\_sections = 1.00$

Pipeline surface area calculation

Surface area of steel pipe to be protected $A_{steel} := \overrightarrow{\left(D_o \cdot \pi \cdot L_{section}\right)}$

Coating breakdown factor calculation

Linepipe Mean coating breakdown factor $f_c := \overrightarrow{\left(f_i + 0.5 \cdot \Delta f \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (1), DNV eqn (2) M-503 Eqn (2)

Linepipe Final coating breakdown factor $f_f := \overrightarrow{\left(f_i + \Delta f \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (2), DNV eqn (4) M-503 Eqn (3)

FJC Mean coating breakdown factor $f_{cfjc} := \overrightarrow{\left(f_{ifjc} + 0.5 \cdot \Delta f_{fjc} \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (1), DNV eqn (2) M-503 Eqn (2)

FJC Final coating breakdown factor $f_{ffjc} := \overrightarrow{\left(f_{ifjc} + \Delta f_{fjc} \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (2), DNV eqn (4) M-503 Eqn (3)

Total coating breakdown factors for line pipe with FJC

Ratio of the lengths of the cutbacks and line pipe coating $r := \overrightarrow{\frac{2 \cdot L_{cutback}}{P_{jtlength}}}$ DNV Sec 5.6.4

Total Mean coating breakdown factor $f_{ctot} := \overrightarrow{\left(f_c + r \cdot f_{cfjc}\right)}$ DNV Sec 5.6.4

Total Final coating breakdown factor $f_{ftot} := \overrightarrow{\left(f_f + r \cdot f_{ffjc}\right)}$ DNV Sec 5.6.4

Protective current calculation (Applicable only for ISO-15589-2:2012, refer section 7.4.4)

$$i_m := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} i_{mplus_i} \leftarrow i_{m_i} + \left(Temp_{pipeline_i} - 25 \cdot C\right) \cdot 0.001 \cdot \frac{A}{m^2 \cdot C} \quad \text{if } Temp_{pipeline_i} \geq 25 \cdot C \wedge Update\_I_{density} = \text{"Yes"} \\ i_{mplus_i} \leftarrow i_{m_i} \quad \text{otherwise} \end{array} \right. \\ i_{mplus} \end{array} \right.$$

$$i_f := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} i_{fplus_i} \leftarrow i_{f_i} + \left(Temp_{pipeline_i} - 25 \cdot C\right) \cdot 0.001 \cdot \frac{A}{m^2 \cdot C} \quad \text{if } Temp_{pipeline_i} \geq 25 \cdot C \wedge Update\_I_{density} = \text{"Yes"} \\ i_{fplus_i} \leftarrow i_{f_i} \quad \text{otherwise} \end{array} \right. \\ i_{fplus} \end{array} \right.$$

$I_m := \overrightarrow{\left(i_m \cdot f_{ctot} \cdot A_{steel}\right)}$ ISO eqn (A.1), DNV eqn (1)

$I_f := \overrightarrow{\left(i_f \cdot f_{ftot} \cdot A_{steel}\right)}$ ISO eqn (A.1), DNV eqn (3)

Anodes dimension and weight calculation

Internal diameter of anode

$$ID_{anode} := D_o + 2 \cdot T_{corr} + 2 \cdot T_{insu} + T_{tolerance} + 2 \cdot t_{internal}$$

Outer diameter of anode

$$OD_{anode} := ID_{anode} + 2 \cdot T_{anode}$$

Mean radius of anode

$$R_m := \frac{OD_{anode} + ID_{anode}}{4}$$

Maximum outer diameter of anode at end of life span

$$OD_{final} := OD_{anode} - 2\overrightarrow{\left(u \cdot T_{anode}\right)}$$

Maximum outer surface area of anode at end of life span

$$A_{anode} := \overrightarrow{\left[\left(OD_{final} \cdot \pi - 2 \cdot Gap_{anode}\right) \cdot L_{anode}\right]}$$

Individual weight of anode (Anode bracelet assumed to be cylindrical).
Individual weight of anodes to be optimized based on the installation vessel limitations/requirements

$$W_a := \overrightarrow{\left[\left[\frac{\pi}{4} \cdot \left(OD_{anode}{}^2 - ID_{anode}{}^2\right) - 2 \cdot Gap_{anode} \cdot T_{anode}\right] \cdot \rho_{anode} \cdot L_{anode} \cdot \left(1 - Inserts_{anode}\right)\right]}$$

Individual anode current output calculation

<u>Anode resistance (optimized) to be applied to bracelet anode:</u>

Anode resistance

$$R_{a\_sea} := 0.315 \cdot \frac{\rho_{res}}{\sqrt{A_{anode}}}$$

ISO eqn (A.8), DNV refer to ISO.

Individual anode current output at end of life, i.e. final output pr. anode

$$I_a := \frac{E_c - E_a}{R_{a\_sea}}$$

ISO eqn (A.6), DNV eqn (6)

Sacrificial anode requirement calculation

Electrochemical capacity for anode surface temperature (Section 8.3 Table 5, ISO 15589-2:2012)

$$\varepsilon_{unburied} := \left[\begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \varepsilon_{unbur_i} \leftarrow 2000 \cdot A \cdot \frac{hr}{kg} \quad \text{if } Temp_{anode_i} \le 30C \\ \varepsilon_{unbur_i} \leftarrow \left[2000 - \frac{Temp_{anode_i} - 30C}{30C} \cdot (2000 - 1500)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 30C < Temp_{anode_i} \le 60C \\ \varepsilon_{unbur_i} \leftarrow \left[1500 - \frac{Temp_{anode_i} - 60C}{20C} \cdot (1500 - 900)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 60C < Temp_{anode_i} \le 80C \\ \varepsilon_{unbur_i} \leftarrow 900 \cdot A \cdot \frac{hr}{kg} \quad \text{otherwise} \end{array} \right. \\ \varepsilon_{unbur} \end{array}\right]$$

$$\varepsilon_{buried} := \left[\begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \varepsilon_{bur_i} \leftarrow 1500 \cdot A \cdot \dfrac{hr}{kg} \quad \text{if } Temp_{anode_i} \le 30C \\ \varepsilon_{bur_i} \leftarrow \left[1500 - \dfrac{Temp_{anode_i} - 30C}{30C} \cdot (1500 - 800)\right] \cdot A \cdot \dfrac{hr}{kg} \quad \text{if } 30C < Temp_{anode_i} \le 60C \\ \varepsilon_{bur_i} \leftarrow \left[800 - \dfrac{Temp_{anode_i} - 60C}{20C} \cdot (800 - 400)\right] \cdot A \cdot \dfrac{hr}{kg} \quad \text{if } 60C < Temp_{anode_i} \le 80C \\ \varepsilon_{bur_i} \leftarrow 400 \cdot A \cdot \dfrac{hr}{kg} \quad \text{otherwise} \end{array} \right. \\ \varepsilon_{bur} \end{array}\right]$$

$$\varepsilon := \left(\begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \varepsilon_i \leftarrow \varepsilon_{buried_i} \quad \text{if } PL_{status_i} = 1 \\ \varepsilon_i \leftarrow \varepsilon_{unburied_i} \quad \text{if } PL_{status_i} = 2 \\ \varepsilon_i \leftarrow \text{"Check the pipeline burial status"} \quad \text{otherwise} \end{array} \right. \\ \varepsilon \end{array}\right)$$

ANODE MASS REQUIRED TO COVER MEAN CURRENT REQUIREMENTS: (ISO Sec. A.2 and Sec. A.7, DNV Sec. 5.4 and Sec. 5.3)

$$m_{required} := \overrightarrow{\left(I_m \cdot T_{design} \cdot \frac{1}{u \cdot \varepsilon}\right)}$$ ISO eqn (A.2), DNV eqn (5)

Required current (equal to final required current) $I_{required} := \vec{I_f}$

Calculated minimum required anodes

$$Nos\_of\_anodes := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad Nos\_of\_anodes_i \leftarrow \overrightarrow{\left(\max\left(\frac{I_{required_i} \cdot SF_i}{I_{a_i}}, \frac{m_{required_i} \cdot SF_i}{W_{a_i}}\right)\right)} \\ Nos\_of\_anodes \end{array} \right.$$

$$Criteria := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad Criteria_i \leftarrow \left| \begin{array}{l} \text{"Mass requirement"} \quad \text{if } \overrightarrow{\frac{I_{required_i} \cdot SF_i}{I_{a_i}}} < \overrightarrow{\frac{m_{required_i} \cdot SF_i}{W_{a_i}}} \\ \text{"Current requirement"} \quad \text{otherwise} \end{array} \right. \\ Criteria \end{array} \right.$$

Anode Spacing in terms of No.of Joint

$$Joint_{anode} := \text{floor}\left(\overrightarrow{\frac{L_{section}}{P_{jtlength} \cdot Nos\_of\_anodes}}\right)$$

Minimum number of anodes required per section

$$Nos := \text{Ceil}\left[\left(\frac{L_{section}}{Joint_{anode} \cdot P_{jtlength}}\right), 1.0\right] \qquad Nos = 550.00$$

Results section for mass/current demand check:

Intermediate results:

KP from	KP to	Linepipe mean coating breakdown factor	Linepipe final coating breakdown factor	FJC mean coating breakdown factor	FJC final coating breakdown factor	Ratio of the lengths of the cutbacks and line pipe coating	Total Mean coating breakdown factor	Total Final coating breakdown factor
		f_c	f_f	$f_{c\,fjc}$	f_{ffjc}	r	f_{tot}	f_{ftot}
(km)	(km)	(-)	(-)	(-)	(-)	(-)	(-)	(-)
0	80.392	0.00900	0.01400	0.00000	0.00000	0.05574	0.00900	0.01400

KP from	KP to	Area of steel pipe to be protected	Mean protective current	Final protective current	Individual anode current output at end of life	Electrochemical capacity	Current requirement criteria		Mass requirement criteria	
							Final current requirement	Final anode current output	Total required anode mass	Total provided anode mass
		A_{steel}	I_m	I_f	I_a	ε	$I_{required} \times SF$	$I_a \times Nos$	$m_{required} \times SF$	$W_a \times Nos$
(km)	(km)	(m^2)	(A)	(A)	(A)	(A*hr/kg)	(A)	(A)	(kg)	(kg)
0	80.392	128299.93	167.43141	260.44886	0.50342	2000.00	260.449	276.882	45864.76	57506.90

Main results:

KP from	KP to	Pipeline section length	Anode Thickness	Anode Length	Anode gap	Anode ID	Individual Anode Weight	Anode Spacing	No. Of Anode	Total Anode Weight	Criteria for anode spacing
		$L_{section}$	T_{anode}	L_{anode}	Gap_{anode}	ID_{anode}	W_a	$Joint_{anode}$	Nos	$Nos \times W_a$	
(km)	(km)	(m)	(mm)	(mm)	(mm)	(mm)	(kg)	(Joints)	(No's)	(kg)	(-)
0	80.392	80392	40	600	80	515.20	104.56	12	550	57506.90	Current

Total Number of anodes

$$N_{total} := \left| \begin{array}{l} \sum Nos \quad \text{if} \quad n < 1 \\ Nos \end{array} \right. = 550.00$$

Total Anode mass required without spares

$$W_{total} := \sum \overrightarrow{(Nos \cdot W_a)} = 57506.90\,kg$$

Check for attenuation calculation requirement (ISO 15589-2:2012, Sec 8.1)

$$
\text{AttenResult} := \left|
\begin{array}{l}
\text{for } i \in 1 .. n \\
\quad \left|
\begin{array}{l}
\text{Result}_i \leftarrow \text{"No attenuation check is required"} \quad \text{if } \text{Joint}_{\text{anode}_i} \cdot P_{\text{jtlength}} < 300\text{m} \\
\text{Result}_i \leftarrow \text{"Attenuation check is required"} \quad \text{otherwise}
\end{array}
\right. \\
\text{Result}
\end{array}
\right.
$$

$$\text{AttenResult} = \left(\text{"No attenuation check is required"} \right)$$

Project:	**Balticconnector FEED – Offshore pipeline** Pipeline Cathodic Protection Design 20" gas pipeline across the Gulf of Finland (Buried Pipeline Condition)	**Project no.:**	**30614_4**
Author: Checker: Approver:	SDR FARH NC	Date: Rev. No.:	2016-01-11 01

Scope:
This sheet calculates the cathodic protection requirement for pipeline based on
- ISO 15589-2, Dec 2012,
- DNV-RP-F103, Oct 2010,
- NORSOK M-503, May 2007.

Input section

Pipeline properties inputs:

KP from	KP to	Pipeline outer diameter	Pipeline wall thickness	Pipeline section length	Pipeline OD tolerance	Corrosion coating thickness	Concrete coating thickness	Insulation coating thickness	Corrosion coating cutback length on one side
		D_o	T_{steel}	$L_{section}$	$T_{tolerance}$	T_{corr}	T_{conc}	T_{insu}	$L_{cutback}$
(km)	(km)	(mm)	(mm)	(m)	(mm)	(mm)	(mm)	(mm)	(m)
0.000	80.392	508.000	12.700	$8.039 \cdot 10^4$	0.000	3.500	45.000	0.000	0.340

$No\_pipe\_sections := rows(pipe\_properties) = 1.00$ $\quad KP\_from := pipe\_properties^{\langle 1 \rangle}$ $\quad KP\_to := pipe\_properties^{\langle 2 \rangle}$

$D_o := pipe\_properties^{\langle 3 \rangle}\ mm$ $\quad T_{steel} := pipe\_properties^{\langle 4 \rangle}\ mm$ $\quad L_{section} := pipe\_properties^{\langle 5 \rangle}\ m$

$T_{tolerance} := pipe\_properties^{\langle 6 \rangle}\ mm$ $\quad T_{corr} := pipe\_properties^{\langle 7 \rangle}\ mm$ $\quad T_{conc} := pipe\_properties^{\langle 8 \rangle}\ mm$

$T_{insu} := pipe\_properties^{\langle 9 \rangle}\ mm$ $\quad L_{cutback} := pipe\_properties^{\langle 10 \rangle}\ m$

Anode properties inputs:

KP from	KP to	Buried=1 /Exposed=2	Safety Factor	Anode Thickness	Anode Length	Anode gap	Pipeline metallic surface temp	Inner Anode surface temp	Percentage volume of inserted steel in anode
			SF	T_{anode}	L_{anode}	GAP_{anode}	$Temp_{pipeline}$	$Temp_{anode}$	$Inserts_{anode}$
(km)	(km)	-	-	(mm)	(mm)	(mm)	(°C)	(°C)	(%)
0.000	80.392	1.000	1.000	40.000	600.000	80.000	50.000	50.000	0.000

$PL_{status} := Anode\_properties^{\langle 3 \rangle}$ $SF := Anode\_properties^{\langle 4 \rangle}$ $T_{anode} := Anode\_properties^{\langle 5 \rangle} \cdot mm$

$L_{anode} := Anode\_properties^{\langle 6 \rangle} \cdot mm$ $Gap_{anode} := Anode\_properties^{\langle 7 \rangle} \cdot mm$ $Temp_{pipeline} := Anode\_properties^{\langle 8 \rangle} \cdot C$

$Temp_{anode} := Anode\_properties^{\langle 9 \rangle} \cdot C$ $Inserts_{anode} := Anode\_properties^{\langle 10 \rangle} \cdot \%$

KP from	KP to	Coating breakdown Factor (Refer Note 9)				Protective Current Density		Minimum negative potential	Design closed-circuit potential for anode material	Environmental resistvity
		Corrossion Coating		Field Joint Coating						
		Initial	Avg. yearly increase	Initial	Avg. yearly increase	Mean	Final			
		f_i	Δf	f_{ifjc}	Δf_{fjc}	i_m	i_f	E_c	E_a	ρ_{res}
(km)	(km)	-	-	-	-	(mA/m²)	(mA/m²)	(V)	(V)	(Ω-m)
0.000	80.392	$4.000 \cdot 10^{-3}$	$2.000 \cdot 10^{-4}$	0.000	0.000	20.000	20.000	-0.900	-1.000	1.500

$f_i := factor\_demand^{\langle 3 \rangle}$ $\Delta f := factor\_demand^{\langle 4 \rangle}$ $f_{ifjc} := factor\_demand^{\langle 5 \rangle}$ $\Delta f_{fjc} := factor\_demand^{\langle 6 \rangle}$

$i_m := factor\_demand^{\langle 7 \rangle} \cdot \frac{mA}{m^2}$ $i_f := factor\_demand^{\langle 8 \rangle} \cdot \frac{mA}{m^2}$ $E_c := factor\_demand^{\langle 9 \rangle} \cdot V$

$E_a := factor\_demand^{\langle 10 \rangle} \cdot V$ $\rho_{res} := factor\_demand^{\langle 11 \rangle} \cdot \Omega \cdot m$

General Inputs

Does protective current density shall be updated as per ISO 15589-2:2012 Sec 7.4.4 for elevated temperature

$Update\_I_{density} :=$ Yes / No

$Update\_I_{density} = \text{"Yes"}$

Design life (refer Design Basis) $T_{design} := 50 \cdot yr$

Electrical resistivity of pipe material (refer Design Basis) $\rho_{Me} := 0.2\Omega \cdot m \cdot 10^{-6}$

Pipe Joint length (refer Design Basis) $P_{jtlength} := 12.2m$

Anode utilization factor (minimum) (ISO 15589-2:2012 Sec 8.4or DNV-RP-F103 Sec 5.4.2 or M-503 Sec 5.8.2) $u := 0.80$

Density of anode material (refer Design Basis) $\rho_{anode} := 2750 \cdot \frac{kg}{m^3}$

Internal anode coating (refer Design Basis) $t_{internal} := 0.1 \cdot mm$

Design premises:

1. fi and Δf are Linepipe Coating breakdown factors (Sec. 7.5, table 3/4, ISO 15589-2:2012 or Annex 1, table A.1, DNV-RP-F103)
2. f_{ifjc} and Δf_{fjc} are Field Joint Coating breakdown factors (Sec. 7.5, table 3/4, ISO 15589-2:2012, Annex 1, table A.2, DNV-RP-F103)
3. i_m and i_f are Protective Current Density (mean and final) this varies based upon the burial status of the pipeline, Mean and Final is assumed same as per Sec. 7.4,1, Note 1, ISO 15589-2:2012. For DNV-RP-F103 the protective current density depends on fluid temperature and burial status, refer Sec. 5.2.4, table 5-1.
4. Pipeline operating with temperatures in excess of 25°C on the outside metallic surface of the pipe require an adjustment to the design current density. The design current densities shall be increased by 1mA/m^2 for each degree Celsius of the metal/environmental above 25°C upto 100°C as per Sec. 7.4,4, Note 1, ISO 15589-2:2012. Note this elevated temp is taken care in the below calculation based on the pipeline temperature (Applicable only for ISO).
5. For riser section selected current densities i_m and i_f shall be 10 mA/m^2 higher than for the equivalent riser or pipeline below the splash zone, Entire riser section is assumed as splash zone.
6. E_c recommended minimum negative protection potential "Aerobic environment" (Sec. 7.2.1, table 1 in ISO 15589-2:2012 or Sec. 5.6.11, DNV-RP-F103) and E_a design closed-circuit potential of anode material anode depends on burial status of pipeline (Sec 8.3, table 5 in ISO 15589-2:2012)
7. It is to be noted that the inner anode surface temperature shall be considered in the design as this would give most conservative results. The inner anode surface temperature can be calculated considering heat loss through the pipe wall and coatings. However the user can select same temperature as that of the fluid temperature in order to be more conservative on design.
8. Environmental resistivity is taken as seawater resistivity for exposed pipeline and can be taken from Appendix-1 figure A-1 ISO 15589-2:2012 or based on experience. For buried pipeline the environmental resistivity is taken as seabed resistivity. If no data is available the value can be taken as 1.5 Ω-m.
9. If the coating breakdown factor is selected from ISO 15589-2:2012, the values given in ISO are the total coating breakdown factor including field joint coating and infill. Input has to be provided only for corrosion coating breakdown factor, and for field joint coating enter zero as breakdown factor.

Calculations Section

No. of Pipeline Section for analysis $n := No\_pipe\_sections = 1.00$

Pipeline surface area calculation

Surface area of steel pipe to be protected $A_{steel} := \overrightarrow{\left(D_o \cdot \pi \cdot L_{section}\right)}$

Coating breakdown factor calculation

Linepipe Mean coating breakdown factor $f_c := \overrightarrow{\left(f_i + 0.5 \cdot \Delta f \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (1), DNV eqn (2) M-503 Eqn (2)

Linepipe Final coating breakdown factor $f_f := \overrightarrow{\left(f_i + \Delta f \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (2), DNV eqn (4) M-503 Eqn (3)

FJC Mean coating breakdown factor $f_{cfjc} := \overrightarrow{\left(f_{ifjc} + 0.5 \cdot \Delta f_{fjc} \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (1), DNV eqn (2) M-503 Eqn (2)

FJC Final coating breakdown factor $f_{ffjc} := \overrightarrow{\left(f_{ifjc} + \Delta f_{fjc} \cdot T_{design} \cdot \frac{1}{yr}\right)}$ ISO eqn (2), DNV eqn (4) M-503 Eqn (3)

Total coating breakdown factors for line pipe with FJC

Ratio of the lengths of the cutbacks and line pipe coating $r := \overrightarrow{\frac{2 \cdot L_{cutback}}{P_{jtlength}}}$ DNV Sec 5.6.4

Total Mean coating breakdown factor $f_{ctot} := \overrightarrow{\left(f_c + r \cdot f_{cfjc}\right)}$ DNV Sec 5.6.4

Total Final coating breakdown factor $f_{ftot} := \overrightarrow{\left(f_f + r \cdot f_{ffjc}\right)}$ DNV Sec 5.6.4

Protective current calculation (Applicable only for ISO-15589-2:2012, refer section 7.4.4)

$$i_m := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} i_{mplus_i} \leftarrow i_{m_i} + \left(Temp_{pipeline_i} - 25 \cdot C\right) \cdot 0.001 \cdot \frac{A}{m^2 \cdot C} \quad \text{if } Temp_{pipeline_i} \geq 25 \cdot C \wedge Update\_I_{density} = \text{"Yes"} \\ i_{mplus_i} \leftarrow i_{m_i} \quad \text{otherwise} \end{array} \right. \\ i_{mplus} \end{array} \right.$$

$$i_f := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} i_{fplus_i} \leftarrow i_{f_i} + \left(Temp_{pipeline_i} - 25 \cdot C\right) \cdot 0.001 \cdot \frac{A}{m^2 \cdot C} \quad \text{if } Temp_{pipeline_i} \geq 25 \cdot C \wedge Update\_I_{density} = \text{"Yes"} \\ i_{fplus_i} \leftarrow i_{f_i} \quad \text{otherwise} \end{array} \right. \\ i_{fplus} \end{array} \right.$$

$I_m := \overrightarrow{\left(i_m \cdot f_{ctot} \cdot A_{steel}\right)}$ ISO eqn (A.1), DNV eqn (1)

$I_f := \overrightarrow{\left(i_f \cdot f_{ftot} \cdot A_{steel}\right)}$ ISO eqn (A.1), DNV eqn (3)

Anodes dimension and weight calculation

Internal diameter of anode

$$ID_{anode} := D_o + 2 \cdot T_{corr} + 2 \cdot T_{insu} + T_{tolerance} + 2 \cdot t_{internal}$$

Outer diameter of anode

$$OD_{anode} := ID_{anode} + 2 \cdot T_{anode}$$

Mean radius of anode

$$R_m := \frac{OD_{anode} + ID_{anode}}{4}$$

Maximum outer diameter of anode at end of life span

$$OD_{final} := OD_{anode} - 2\overrightarrow{\left(u \cdot T_{anode}\right)}$$

Maximum outer surface area of anode at end of life span

$$A_{anode} := \overrightarrow{\left[\left(OD_{final} \cdot \pi - 2 \cdot Gap_{anode}\right) \cdot L_{anode}\right]}$$

Individual weight of anode (Anode bracelet assumed to be cylindrical).
Individual weight of anodes to be optimized based on the installation vessel limitations/requirements

$$W_a := \overrightarrow{\left[\left[\frac{\pi}{4} \cdot \left(OD_{anode}^{\ 2} - ID_{anode}^{\ 2}\right) - 2 \cdot Gap_{anode} \cdot T_{anode}\right] \cdot \rho_{anode} \cdot L_{anode} \cdot \left(1 - Inserts_{anode}\right)\right]}$$

Individual anode current output calculation

<u>Anode resistance (optimized) to be applied to bracelet anode:</u>

Anode resistance

$$R_{a\_sea} := 0.315 \cdot \frac{\rho_{res}}{\sqrt{A_{anode}}}$$

ISO eqn (A.8), DNV refer to ISO.

Individual anode current output at end of life, i.e. final output pr. anode

$$I_a := \frac{E_c - E_a}{R_{a\_sea}}$$

ISO eqn (A.6), DNV eqn (6)

Sacrificial anode requirement calculation

Electrochemical capacity for anode surface temperature (Section 8.3 Table 5, ISO 15589-2:2012)

$$\varepsilon_{unburied} := \left[\begin{array}{l} \text{for } i \in 1 .. n \\ \quad \varepsilon_{unbur_i} \leftarrow 2000 \cdot A \cdot \frac{hr}{kg} \quad \text{if } Temp_{anode_i} \le 30C \\ \quad \varepsilon_{unbur_i} \leftarrow \left[2000 - \frac{Temp_{anode_i} - 30C}{30C} \cdot (2000 - 1500)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 30C < Temp_{anode_i} \le 60C \\ \quad \varepsilon_{unbur_i} \leftarrow \left[1500 - \frac{Temp_{anode_i} - 60C}{20C} \cdot (1500 - 900)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 60C < Temp_{anode_i} \le 80C \\ \quad \varepsilon_{unbur_i} \leftarrow 900 \cdot A \cdot \frac{hr}{kg} \quad \text{otherwise} \\ \varepsilon_{unbur} \end{array}\right]$$

$$\varepsilon_{buried} := \left[\begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \varepsilon_{bur_i} \leftarrow 1500 \cdot A \cdot \frac{hr}{kg} \quad \text{if } Temp_{anode_i} \le 30C \\ \varepsilon_{bur_i} \leftarrow \left[1500 - \frac{Temp_{anode_i} - 30C}{30C} \cdot (1500 - 800)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 30C < Temp_{anode_i} \le 60C \\ \varepsilon_{bur_i} \leftarrow \left[800 - \frac{Temp_{anode_i} - 60C}{20C} \cdot (800 - 400)\right] \cdot A \cdot \frac{hr}{kg} \quad \text{if } 60C < Temp_{anode_i} \le 80C \\ \varepsilon_{bur_i} \leftarrow 400 \cdot A \cdot \frac{hr}{kg} \quad \text{otherwise} \end{array} \right. \\ \varepsilon_{bur} \end{array}\right]$$

$$\varepsilon := \left(\begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \varepsilon_i \leftarrow \varepsilon_{buried_i} \quad \text{if } PL_{status_i} = 1 \\ \varepsilon_i \leftarrow \varepsilon_{unburied_i} \quad \text{if } PL_{status_i} = 2 \\ \varepsilon_i \leftarrow \text{"Check the pipeline burial status"} \quad \text{otherwise} \end{array} \right. \\ \varepsilon \end{array}\right)$$

ANODE MASS REQUIRED TO COVER MEAN CURRENT REQUIREMENTS: (ISO Sec. A.2 and Sec. A.7, DNV Sec. 5.4 and Sec. 5.3)

$$m_{required} := \overrightarrow{\left(I_m \cdot T_{design} \cdot \frac{1}{u \cdot \varepsilon}\right)}$$ ISO eqn (A.2), DNV eqn (5)

Required current (equal to final required current) $I_{required} := \vec{I_f}$

Calculated minimum required anodes

$$Nos\_of\_anodes := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad Nos\_of\_anodes_i \leftarrow \overrightarrow{\left(\max\left(\frac{I_{required_i} \cdot SF_i}{I_{a_i}}, \frac{m_{required_i} \cdot SF_i}{W_{a_i}}\right)\right)} \\ Nos\_of\_anodes \end{array} \right.$$

$$Criteria := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad Criteria_i \leftarrow \left| \begin{array}{l} \text{"Mass requirement"} \quad \text{if } \overrightarrow{\frac{I_{required_i} \cdot SF_i}{I_{a_i}}} < \overrightarrow{\frac{m_{required_i} \cdot SF_i}{W_{a_i}}} \\ \text{"Current requirement"} \quad \text{otherwise} \end{array} \right. \\ Criteria \end{array} \right.$$

Anode Spacing in terms of No.of Joint

$$Joint_{anode} := \text{floor}\left(\overrightarrow{\frac{L_{section}}{P_{jtlength} \cdot Nos\_of\_anodes}}\right)$$

Minimum number of anodes required per section

$$Nos := \text{Ceil}\left[\left(\frac{L_{section}}{Joint_{anode} \cdot P_{jtlength}}\right), 1.0\right]$$ $Nos = 412.00$

Results section for mass/current demand check:

Intermediate results:

KP from	KP to	Linepipe mean coating breakdown factor	Linepipe final coating breakdown factor	FJC mean coating breakdown factor	FJC final coating breakdown factor	Ratio of the lengths of the cutbacks and line pipe coating	Total Mean coating breakdown factor	Total Final coating breakdown factor
		f_c	f_f	f_{cfjc}	f_{ffjc}	r	f_{tot}	f_{ftot}
(km)	(km)	(-)	(-)	(-)	(-)	(-)	(-)	(-)
0	80.392	0.00900	0.01400	0.00000	0.00000	0.05574	0.00900	0.01400

KP from	KP to	pipe to be protected	protective current	protective current	current output at end of life	emical capacity	Final current requirement	Final anode current output	Total required anode mass	Total provided anode mass
		A_{steel}	I_m	I_f	I_a	ε	$I_{required} \times SF$	$I_a \times Nos$	$m_{required} \times SF$	$W_a \times Nos$
(km)	(km)	(m^2)	(A)	(A)	(A)	(A*hr/kg)	(A)	(A)	(kg)	(kg)
0	80.392	128299.93	51.96147	80.82896	0.20137	1033.33	80.829	82.964	27549.47	43077.90

Main results:

KP from	KP to	Pipeline section length	Anode Thickness	Anode Length	Anode gap	Anode ID	Individual Anode Weight	Anode Spacing	No. Of Anode	Total Anode Weight	Criteria for anode spacing
		$L_{section}$	T_{anode}	L_{anode}	Gap_{anode}	ID_{anode}	W_a	$Joint_{anode}$	Nos	$Nos \times W_a$	
(km)	(km)	(m)	(mm)	(mm)	(mm)	(mm)	(kg)	(Joints)	(No's)	(kg)	(-)
0	80.392	80392	40	600	80	515.20	104.56	16	412	43077.90	Current

Total Number of anodes

$$N_{total} := \begin{cases} \sum Nos & \text{if } n < 1 \\ Nos \end{cases} = 412.00$$

Total Anode mass required without spares

$$W_{total} := \sum \overrightarrow{(Nos \cdot W_a)} = 43077.90\,kg$$

Check for attenuation calculation requirement (ISO 15589-2:2012, Sec 8.1)

$$\mathrm{AttenResult} := \left| \begin{array}{l} \text{for } i \in 1..n \\ \quad \left| \begin{array}{l} \mathrm{Result}_i \leftarrow \text{"No attenuation check is required"} \quad \text{if } \mathrm{Joint}_{\mathrm{anode}_i} \cdot P_{\mathrm{jtlength}} < 300\mathrm{m} \\ \mathrm{Result}_i \leftarrow \text{"Attenuation check is required"} \quad \text{otherwise} \end{array} \right. \\ \mathrm{Result} \end{array} \right.$$

$$\mathrm{AttenResult} = (\text{"No attenuation check is required"})$$

APPENDIX VI. Directional extreme wave and current data

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV1	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	-	-	-	-	-	-	-	-	-
	120	300	-	-	-	-	-	-	-	-	-
	150	330	0.42	0.59	0.76	3.53	3.68	3.85	-	-	-
	180	0	0.02	0.18	0.50	3.34	3.38	3.60	-	-	-
	210	30	0.34	0.50	0.59	3.47	3.60	3.68	-	-	-
	240	60	0.42	0.59	0.76	3.53	3.68	3.85	-	-	-
	270	90	-	-	-	-	-	-	-	-	-
	300	120	-	-	-	-	-	-	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		0.42	0.59	0.76	3.53	3.68	3.85	-	-	-
ENV2	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	-	-	-	-	-	-	-	-	-
	120	300	0.57	0.78	1.00	3.84	3.87	3.90	-	-	-
	150	330	0.57	0.78	1.00	3.84	3.87	3.90	-	-	-
	180	0	0.03	0.21	0.34	1.84	3.63	3.75	-	-	-
	210	30	-	-	-	-	-	-	-	-	-
	240	60	0.19	0.24	0.26	3.59	3.67	3.69	-	-	-
	270	90	0.34	0.55	0.66	3.75	3.83	3.86	-	-	-
	300	120	0.06	0.24	0.39	2.91	3.67	3.78	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		0.57	0.78	1.00	3.84	3.87	3.90	-	-	-
ENV3	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	0.81	1.09	1.40	4.44	4.50	4.55	-	-	-
	120	300	0.80	0.96	1.11	4.44	4.50	4.55	-	-	-
	150	330	0.81	1.09	1.38	4.44	4.50	4.55	-	-	-
	180	0	0.15	0.34	0.54	3.84	4.19	4.33	-	-	-
	210	30	-	-	-	-	-	-	-	-	-
	240	60	-	-	-	-	-	-	-	-	-
	270	90	-	-	-	-	-	-	-	-	-
	300	120	0.43	0.47	0.48	4.27	4.29	4.30	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		0.81	1.09	1.40	4.44	4.50	4.55	-	-	-

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV4	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	0.08	0.18	0.30	2.92	4.10	4.65	-	-	-
	120	300	1.11	1.49	1.90	5.21	5.60	5.70	-	-	-
	150	330	1.02	1.35	1.68	5.52	5.63	5.70	-	-	-
	180	0	0.44	0.67	0.88	4.04	5.20	5.70	-	-	-
	210	30	-	-	-	-	-	-	-	-	-
	240	60	-	-	-	-	-	-	-	-	-
	270	90	-	-	-	-	-	-	-	-	-
	300	120	1.11	1.49	1.90	5.52	5.63	5.70	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		1.11	1.49	1.90	5.52	5.63	5.70	-	-	-
ENV5	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	0.61	1.40	2.27	3.83	4.82	5.69	-	-	-
	120	300	1.41	1.85	2.36	5.39	5.60	5.69	-	-	-
	150	330	1.41	1.85	2.36	5.46	5.60	5.69	-	-	-
	180	0	0.43	0.83	1.27	4.92	5.21	5.69	-	-	-
	210	30	-	-	-	-	-	-	-	-	-
	240	60	-	-	-	-	-	-	-	-	-
	270	90	-	-	-	-	-	-	-	-	-
	300	120	-	-	-	-	-	-	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		1.41	1.85	2.36	5.46	5.60	5.69	-	-	-
ENV6	0	180	-	-	-	-	-	-	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	-	-	-	-	-	-	-	-	-
	90	270	1.19	1.62	2.00	4.58	5.06	5.47	-	-	-
	120	300	1.72	2.37	3.08	6.42	7.78	9.87	-	-	-
	150	330	1.72	2.37	3.08	6.24	6.40	6.48	-	-	-
	180	0	1.41	1.63	1.84	6.42	6.61	6.67	-	-	-
	210	30	0.87	1.66	2.64	5.66	7.78	9.87	-	-	-
	240	60	-	-	-	-	-	-	-	-	-
	270	90	-	-	-	-	-	-	-	-	-
	300	120	-	-	-	-	-	-	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		1.72	2.37	3.08	6.42	7.78	9.87	-	-	-

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV7	0	180	0.14	0.28	0.31	4.40	4.59	4.64	-	-	-
	30	210	-	-	-	-	-	-	-	-	-
	60	240	0.36	0.58	0.71	4.71	5.02	5.20	-	-	-
	90	270	1.90	2.57	3.30	6.91	7.44	7.84	-	-	-
	120	300	1.90	2.57	3.30	7.06	8.28	9.80	-	-	-
	150	330	1.90	2.57	3.30	6.38	6.98	7.42	-	-	-
	180	0	1.70	1.85	1.99	7.06	7.53	7.72	-	-	-
	210	30	1.09	1.41	1.74	6.07	6.29	6.47	-	-	-
	240	60	0.37	0.62	0.74	4.72	5.07	5.24	-	-	-
	270	90	0.08	0.83	1.53	4.30	5.38	6.44	-	-	-
	300	120	0.09	0.86	1.62	4.32	5.42	6.59	-	-	-
	330	150	-	-	-	-	-	-	-	-	-
	Omni-direction		1.90	2.57	3.30	7.06	8.28	9.80	-	-	-
ENV8	0	180	0.60	0.94	1.18	4.76	5.44	5.83	0.18	0.22	0.25
	30	210	0.68	0.87	1.00	4.93	5.30	5.54	0.22	0.26	0.29
	60	240	1.07	1.54	1.90	5.98	7.50	8.73	0.27	0.37	0.48
	90	270	2.14	3.00	3.87	7.74	8.83	9.76	0.15	0.25	0.30
	120	300	1.94	2.98	4.02	7.20	9.14	9.90	0.09	0.12	0.14
	150	330	1.79	2.53	3.14	5.51	6.09	6.45	0.10	0.13	0.16
	180	0	3.07	3.87	4.64	6.95	7.63	8.22	0.15	0.19	0.22
	210	30	3.07	3.87	4.64	8.29	9.14	9.90	0.26	0.32	0.38
	240	60	2.65	3.26	3.84	7.83	8.14	8.38	0.27	0.37	0.48
	270	90	1.05	1.41	1.71	5.62	6.19	6.62	0.13	0.17	0.21
	300	120	0.78	1.04	1.25	5.13	5.61	5.95	0.06	0.08	0.10
	330	150	0.60	0.92	1.12	4.75	5.39	5.74	0.10	0.14	0.19
	Omni-direction		3.07	3.87	4.64	8.29	9.14	9.90	0.27	0.37	0.48
ENV9	0	180	0.61	1.02	1.27	4.69	5.54	5.99	0.04	0.05	0.07
	30	210	0.76	1.04	1.22	5.04	5.59	5.90	0.05	0.07	0.09
	60	240	1.24	1.75	2.14	5.94	6.70	7.22	0.17	0.22	0.26
	90	270	2.54	3.30	4.03	7.61	8.51	9.29	0.34	0.47	0.60
	120	300	2.25	3.54	4.80	6.46	8.67	10.45	0.04	0.05	0.06
	150	330	2.12	2.85	3.40	5.73	6.27	6.60	0.04	0.05	0.06
	180	0	2.60	3.48	4.14	6.39	7.20	7.75	0.03	0.03	0.04
	210	30	3.48	4.47	5.45	8.59	9.62	10.45	0.07	0.10	0.14
	240	60	3.48	4.02	4.49	8.24	8.44	8.59	0.22	0.29	0.36
	270	90	1.20	1.36	1.44	6.00	6.13	6.21	0.34	0.47	0.60
	300	120	0.40	0.68	0.84	4.14	4.87	5.21	0.07	0.08	0.10
	330	150	0.63	1.04	1.31	4.74	5.60	6.04	0.04	0.06	0.08
	Omni-direction		3.48	4.47	5.45	8.70	9.62	10.45	0.34	0.47	0.60

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV10	0	180	0.81	1.12	1.34	5.11	5.73	6.10	0.11	0.17	0.21
	30	210	0.99	1.27	1.44	5.48	5.99	6.26	0.20	0.27	0.33
	60	240	1.61	2.41	3.14	6.69	6.89	6.96	0.26	0.39	0.50
	90	270	3.25	4.67	6.13	8.20	9.63	10.44	0.13	0.16	0.18
	120	300	1.90	2.75	3.53	5.82	6.58	7.11	0.08	0.10	0.12
	150	330	1.78	2.52	3.06	5.35	5.98	6.35	0.08	0.15	0.19
	180	0	2.65	3.54	4.20	6.41	7.20	7.74	0.07	0.12	0.15
	210	30	3.76	4.97	6.16	8.38	9.58	10.44	0.18	0.23	0.27
	240	60	3.76	4.97	6.28	8.75	9.63	10.39	0.26	0.39	0.53
	270	90	1.97	2.47	2.89	7.45	7.85	8.10	0.08	0.11	0.12
	300	120	1.13	1.48	1.78	5.75	6.31	6.74	0.12	0.18	0.25
	330	150	0.93	1.26	1.49	5.37	5.97	6.33	0.12	0.20	0.29
	Omni-direction		3.76	4.97	6.28	8.75	9.63	10.44	0.26	0.39	0.53
ENV11	0	180	0.86	1.19	1.43	5.16	5.82	6.22	0.17	0.22	0.25
	30	210	1.30	1.78	2.12	4.74	6.62	8.78	0.23	0.32	0.41
	60	240	1.64	2.48	3.30	6.54	7.56	8.32	0.23	0.32	0.41
	90	270	3.37	4.73	6.08	8.18	9.42	10.30	0.10	0.13	0.15
	120	300	1.98	2.84	3.62	5.87	6.60	7.10	0.11	0.15	0.19
	150	330	1.63	2.26	2.69	5.12	5.71	6.05	0.12	0.15	0.19
	180	0	2.64	3.78	4.67	6.34	7.34	8.03	0.11	0.12	0.13
	210	30	3.82	4.87	5.88	8.01	8.61	9.11	0.20	0.23	0.26
	240	60	3.87	5.12	6.37	8.77	9.61	10.30	0.23	0.32	0.41
	270	90	2.24	2.97	3.67	7.85	8.70	9.40	0.15	0.20	0.23
	300	120	0.96	1.48	1.73	5.38	6.31	6.67	0.08	0.11	0.14
	330	150	0.35	0.69	0.87	3.64	4.75	5.19	0.11	0.19	0.25
	Omni-direction		3.87	5.12	6.37	8.77	9.61	10.30	0.23	0.32	0.41
ENV12	0	180	0.88	1.31	1.58	3.60	4.20	4.90	0.21	0.29	0.38
	30	210	1.76	2.39	2.85	4.69	4.96	5.11	0.21	0.29	0.35
	60	240	2.30	3.77	5.33	6.99	8.11	8.92	0.11	0.14	0.16
	90	270	3.72	5.61	7.53	8.37	9.90	10.67	0.08	0.10	0.12
	120	300	1.73	2.23	2.58	5.40	5.90	6.21	0.09	0.13	0.18
	150	330	1.60	2.27	2.76	4.88	5.51	5.89	0.12	0.16	0.19
	180	0	2.54	3.70	4.56	6.07	7.09	7.76	0.20	0.27	0.32
	210	30	4.03	5.73	7.53	8.52	9.67	10.67	0.19	0.22	0.26
	240	60	4.14	5.73	7.53	9.00	9.90	10.67	0.10	0.12	0.14
	270	90	3.57	5.51	7.53	8.88	9.90	10.67	0.10	0.14	0.16
	300	120	1.68	2.27	2.66	5.52	5.96	6.18	0.07	0.09	0.11
	330	150	0.55	0.85	0.94	3.34	3.70	3.79	0.13	0.20	0.25
	Omni-direction		4.14	5.73	7.53	9.00	9.90	10.67	0.21	0.29	0.38

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV13	0	180	1.12	1.46	1.67	3.98	4.45	4.78	0.10	0.15	0.20
	30	210	1.93	2.68	3.27	4.95	5.11	5.17	0.06	0.09	0.12
	60	240	2.98	4.38	5.75	7.77	8.96	9.90	0.18	0.26	0.35
	90	270	3.47	5.23	7.08	8.13	9.61	10.75	0.16	0.24	0.33
	120	300	1.54	1.98	2.29	5.05	5.48	5.74	0.06	0.08	0.10
	150	330	1.55	2.21	2.68	4.86	5.56	5.98	0.07	0.09	0.12
	180	0	2.11	3.29	4.18	5.54	6.64	7.38	0.07	0.10	0.12
	210	30	3.63	4.89	6.07	8.15	9.00	9.65	0.07	0.10	0.12
	240	60	4.12	5.84	7.76	9.00	9.96	10.75	0.18	0.26	0.35
	270	90	3.92	5.84	7.76	8.96	9.89	10.54	0.17	0.21	0.25
	300	120	1.95	2.43	2.75	5.70	6.53	7.20	0.08	0.15	0.19
	330	150	1.14	1.68	2.04	4.16	4.67	4.95	0.02	0.02	0.02
	Omni-direction		4.12	5.84	7.76	9.00	9.96	10.75	0.18	0.26	0.35
ENV14	0	180	1.88	2.72	3.37	5.15	6.00	6.61	0.15	0.27	0.36
	30	210	1.91	2.65	3.27	5.77	6.15	6.36	0.21	0.26	0.30
	60	240	3.36	4.67	5.97	8.49	9.88	10.91	0.33	0.43	0.54
	90	270	1.75	2.58	3.36	6.30	6.76	7.00	0.17	0.21	0.24
	120	300	1.26	1.76	2.13	4.36	4.62	4.74	0.08	0.10	0.11
	150	330	1.00	1.76	2.32	4.24	4.76	5.03	0.09	0.11	0.12
	180	0	1.24	1.88	2.32	4.60	5.00	5.17	0.13	0.20	0.25
	210	30	1.93	2.69	3.32	6.17	6.99	7.70	0.34	0.48	0.61
	240	60	4.19	5.98	7.85	9.26	10.05	10.58	0.33	0.40	0.47
	270	90	4.19	5.98	7.68	9.16	10.15	10.86	0.10	0.12	0.13
	300	120	2.80	3.88	4.83	7.38	8.35	9.10	0.12	0.15	0.18
	330	150	2.06	3.03	3.75	5.46	6.29	6.82	0.21	0.32	0.43
	Omni-direction		4.19	5.98	7.85	9.26	10.20	10.91	0.34	0.48	0.61
ENV15	0	180	2.06	2.75	3.26	5.52	6.15	6.55	0.11	0.15	0.19
	30	210	1.76	2.70	3.62	5.76	6.38	6.75	0.15	0.18	0.21
	60	240	3.04	4.19	5.31	8.41	9.98	10.68	0.20	0.27	0.33
	90	270	1.57	2.40	3.23	5.86	7.12	8.42	0.12	0.15	0.17
	120	300	1.11	1.62	1.94	4.10	4.43	4.63	0.06	0.08	0.09
	150	330	1.22	1.61	1.92	6.14	6.85	7.30	0.10	0.14	0.17
	180	0	0.73	1.57	2.35	4.00	4.81	5.26	0.10	0.13	0.16
	210	30	1.38	1.86	2.26	6.01	6.28	6.41	0.16	0.20	0.24
	240	60	3.66	4.31	4.89	9.14	9.45	9.67	0.20	0.27	0.34
	270	90	4.05	5.69	7.40	9.16	10.05	10.68	0.16	0.20	0.23
	300	120	2.92	3.90	4.78	7.84	8.64	9.23	0.14	0.20	0.26
	330	150	2.04	3.04	3.79	5.53	6.54	7.25	0.10	0.14	0.17
	Omni-direction		4.05	5.69	7.40	9.20	10.05	10.68	0.20	0.27	0.34

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV16	0	180	2.05	2.80	3.35	5.66	6.28	6.66	0.17	0.22	0.28
	30	210	2.03	3.19	4.38	6.09	6.80	7.25	0.20	0.25	0.29
	60	240	2.78	3.71	4.58	8.34	9.95	10.58	0.29	0.39	0.49
	90	270	1.42	2.02	2.56	4.70	4.99	5.21	0.21	0.29	0.39
	120	300	1.11	1.59	1.89	3.91	4.13	4.25	0.14	0.20	0.26
	150	330	1.10	1.50	1.81	5.93	6.70	7.18	0.15	0.23	0.31
	180	0	1.57	2.03	2.43	6.82	7.47	7.93	0.18	0.25	0.33
	210	30	1.12	1.36	1.51	5.42	5.81	6.03	0.21	0.27	0.32
	240	60	3.18	3.89	4.56	8.90	9.25	9.50	0.29	0.39	0.49
	270	90	3.94	5.55	7.27	9.10	9.95	10.58	0.24	0.35	0.46
	300	120	3.11	4.27	5.33	8.11	9.09	9.83	0.17	0.25	0.34
	330	150	2.07	3.04	3.76	5.69	6.68	7.35	0.15	0.21	0.27
	Omni-direction		3.94	5.55	7.27	9.13	9.95	10.58	0.29	0.39	0.49
ENV17	0	180	2.14	3.10	3.89	5.86	6.72	7.30	0.12	0.15	0.18
	30	210	1.83	2.72	3.59	5.96	6.61	7.01	0.17	0.23	0.28
	60	240	2.69	3.77	4.82	8.52	9.94	10.62	0.17	0.23	0.29
	90	270	1.11	1.39	1.60	5.13	5.61	5.94	0.09	0.13	0.16
	120	300	0.98	1.45	1.75	5.67	6.65	7.12	0.04	0.05	0.06
	150	330	0.95	1.30	1.56	5.60	6.38	6.84	0.08	0.13	0.18
	180	0	1.36	1.80	2.19	6.49	7.19	7.68	0.12	0.18	0.25
	210	30	0.95	1.22	1.39	5.67	5.89	5.98	0.17	0.22	0.26
	240	60	2.59	3.00	3.36	7.93	8.29	8.57	0.17	0.23	0.28
	270	90	3.82	5.48	7.34	9.07	9.94	10.62	0.11	0.16	0.20
	300	120	3.24	4.35	5.39	8.26	9.11	9.75	0.09	0.13	0.17
	330	150	2.22	3.12	3.80	6.00	7.01	7.74	0.09	0.12	0.16
	Omni-direction		3.82	5.48	7.34	9.07	9.94	10.62	0.17	0.23	0.29
ENV18	0	180	1.95	2.92	3.78	5.83	6.73	7.36	0.13	0.16	0.19
	30	210	1.81	2.33	2.83	7.03	9.02	10.41	0.10	0.14	0.17
	60	240	1.19	1.88	2.54	4.77	6.21	8.36	0.10	0.12	0.13
	90	270	0.34	0.68	0.90	3.94	5.17	5.70	0.10	0.13	0.16
	120	300	0.34	0.52	0.61	3.95	4.68	4.94	0.11	0.14	0.17
	150	330	0.34	0.67	0.87	3.93	5.12	5.65	0.11	0.13	0.15
	180	0	-	-	-	-	-	-	0.08	0.09	0.10
	210	30	-	-	-	-	-	-	0.09	0.12	0.14
	240	60	0.11	0.48	0.81	2.41	4.51	5.51	0.11	0.15	0.18
	270	90	2.23	2.53	2.80	8.38	8.87	9.14	0.10	0.13	0.15
	300	120	2.81	4.21	5.73	8.38	9.50	10.41	0.10	0.13	0.16
	330	150	2.51	3.65	4.66	6.82	8.33	9.64	0.13	0.16	0.19
	Omni-direction		2.81	4.21	5.73	8.38	9.50	10.41	0.13	0.16	0.19

Met. point	Wave dir. [deg]	Current dir. [deg]	Hs,1-year [m]	Hs,10-year [m]	Hs,100-year [m]	Tp,1-year [s]	Tp,10-year [s]	Tp,100-year [s]	Uc,1-year [m/s]	Uc,10-year [m/s]	Uc,100-year [m/s]
ENV19	0	180	1.09	1.38	1.63	6.47	6.78	7.01	0.06	0.10	0.14
	30	210	-	-	-	-	-	-	0.05	0.09	0.13
	60	240	-	-	-	-	-	-	0.05	0.06	0.07
	90	270	-	-	-	-	-	-	0.05	0.06	0.07
	120	300	-	-	-	-	-	-	0.14	0.20	0.26
	150	330	-	-	-	-	-	-	0.22	0.33	0.45
	180	0	-	-	-	-	-	-	0.04	0.06	0.07
	210	30	-	-	-	-	-	-	0.02	0.03	0.04
	240	60	-	-	-	-	-	-	0.03	0.04	0.05
	270	90	-	-	-	-	-	-	0.04	0.05	0.05
	300	120	0.04	0.08	0.13	4.17	4.49	4.69	0.11	0.15	0.19
	330	150	0.76	0.94	1.13	5.16	5.40	5.63	0.24	0.34	0.43
	Omni-direction		1.09	1.38	1.63	6.47	6.78	7.01	0.22	0.33	0.45

Notes:
1. Extreme wave height in some direction for metocean point ENV1 to ENV7 and ENV18 to ENV19 were not able to quantify due the very low wave events. At these directions a nominal wave height is considered in the analysis. The values for 1-year RP, 10-year RP and 100-year RP wave events are 0.01m, 0.012m and 0.015m are considered respectively.
2. Directional extreme current values for the metocean point ENV1 to ENV7 were not able to quantify due the very low current events, for these points the Omni-directional current of ENV8 is considered in the all the directional for the metocean point ENV1 to ENV7 and is applied perpendicular to the pipeline.

Table VI-1 Metocean data as provided in *Metocean Data Report*, Ref. /35/

APPENDIX VII. On-bottom stability design calculations

PIPE WEIGHT CALCULATION

Configuration			1	2	3	4	5	
INPUT		*KP start*						
		KP end						
		Length						
		Description	**55 mm (3400 kg/m^3)**	**80 mm (3400 kg/m^3)**	**45 mm (3400kg/m^3)**	**115 mm (3400 kg/m^3)**	**195 mm (3400 kg/m^3)**	
Diameter	Steel OD	[mm]	508.0	508.0	508.0	508.0	508.0	
Thicknesses	Steel	[mm]	12.7	12.7	12.7	12.7	12.7	
	Corrosion coating		3.5	3.5	3.5	3.5	3.5	
	Concrete		55	80	45	115	195	
Densities	Steel	[kg/m³]	7850	7850	7850	7850	7850	
	Corrosion coating		930	930	930	930	930	
	Concrete		3400	3400	3400	3400	3400	
	Air		0	0	0	0	0	
	Seawater		1005	1005	1005	1005	1005	
	Content product min.		65	65	65	65	65	
	Content product max.		65	65	65	65	65	
	Water absorption Weight %	%	0	0	0	0	0	

Output								
Various	Total OD	[mm]	625.0	675.0	605.0	745.0	905.0	
	Unit weight (empty pipe in air)	[N/m]	4858.1	6560.9	4213.7	9164.8	16080.6	
	Buoyancy	[N/m]	3024.7	3528.0	2834.2	4297.7	6341.9	
Submerged unit weight and (SG)	Air	[N/m] [-]	1833.4 (1.61)	3032.8 (1.86)	1379.5 (1.49)	4867.1 (2.13)	9738.6 (2.54)	
	Seawater		3636.8 (2.20)	4836.3 (2.37)	3182.9 (2.12)	6670.5 (2.55)	11542.1 (2.82)	
	Content product min.		1950.0 (1.64)	3149.5 (1.89)	1496.1 (1.53)	4983.7 (2.16)	9855.3 (2.55)	
	Content product max.		1950.0 (1.64)	3149.5 (1.89)	1496.1 (1.53)	4983.7 (2.16)	9855.3 (2.55)	

	STABLELINES **On-Bottom Stability of Submarine Pipelines** v1.4-01	*Programmed by DNV Deep Water Technology* *Contact:* *Software.Support@DNV.com*

Output file name	84-6 - 8.2	**Project:** Balticconnector FEED	**Date:** Oct-2015	**Calculations by**	**SDR**
Input file name		**References:**		**Verified by**	FARH
Output path					
Input path					

Pipeline data / Soil interaction / Environmental Parameters

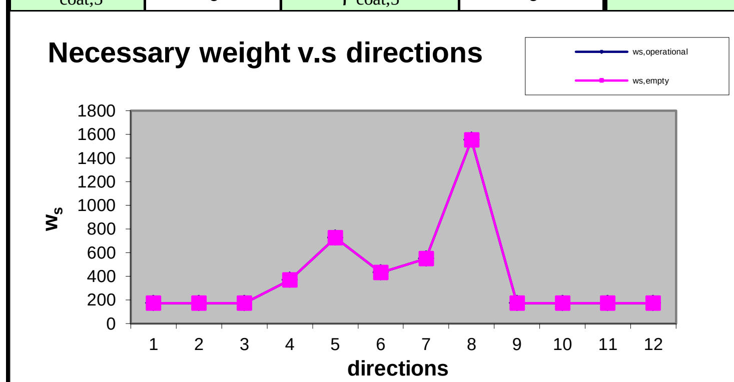
Pipeline data				Soil interaction		Environmental Parameters	
		Pipe		Clay		s	8
θ_{pipe} [deg]	138.293	ρ_{water} [kg/m^3]	1005	z_0 [m]	5.00E-06	z_r [m]	1.5
D_s [m]	0.5080	ρ_{steel} [kg/m^3]	7850	z_t [m]	-	d [m]	17.595
t_{steel} [m]	0.0127	ρ_{conc} [kg/m^3]	3400	z_p [m]	-	θ [deg]	-
t_{conc} [m]	0.0488	ρ_{mgrt} [kg/m^3]	0	$r_{tot,y}$	-	γ	1
t_{mgrt} [m]	0.0000	ρ_{cont} [kg/m^3]	65	$r_{tot,z}$	-	T_{storm} [hrs]	3
l_{FJC} [m]	0.3400	ρ_{FJC} [kg/m^3]	1000	$r_{perm,z}$	-	Stability Criterion	
Coating data				μ	0.20	10D Displacement	
Coating thickness [m]		Coating Density [kg/m^3]		γ_s [N/m^3]	10000	τ	1187
$t_{coat,1}$	0.0035	$\rho_{coat,1}$	930	γ_s [N/m^3]	11837.4	$s_{g,operation}$	1.53
$t_{coat,2}$	0	$\rho_{coat,2}$	0	s_u [N/m^2]	4000	$\gamma_{SC, empty}$	-
$t_{coat,3}$	0	$\rho_{coat,3}$	0	G_c	0.55	$\gamma_{SC, operation}$	-
$t_{coat,4}$	0	$\rho_{coat,4}$	0	z_p/D	-	y/D	10.00
$t_{coat,5}$	0	$\rho_{coat,5}$	0				

Return Period Values for Wave and Current

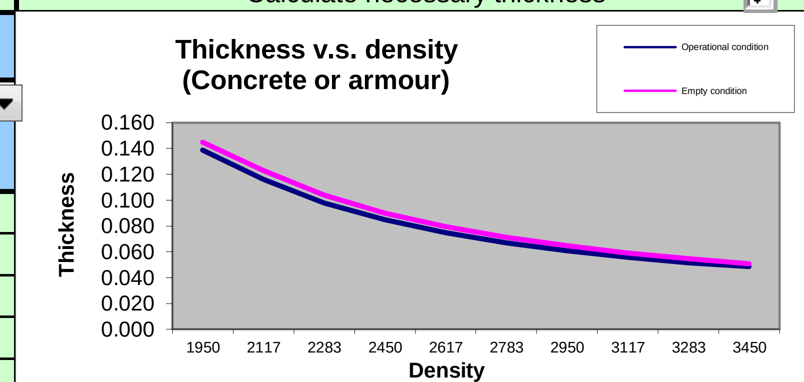
Boundary layer correction for current ☑ | Number of directional combinations considered: 12

Wave dir. [deg]	Current dir. [deg]	$H_{s,1-year}$ [m]	$H_{s,10-year}$ [m]	$H_{s,100-year}$ [m]	$T_{p,1-year}$ [s]	$T_{p,10-year}$ [s]	$T_{p,100-year}$ [s]	$U_{c,1-year}$ [m/s]	$U_{c,10-year}$ [m/s]	$U_{c,100-year}$ [m/s]
0	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
30	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
60	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
90	228.293	1.19	1.62	2.00	4.58	5.06	5.47	0.27	0.37	0.48
120	228.293	1.72	2.37	3.08	6.42	7.78	9.87	0.27	0.37	0.48
150	228.293	1.72	2.37	3.08	6.24	6.40	6.48	0.27	0.37	0.48
180	228.293	1.41	1.63	1.84	6.42	6.61	6.67	0.27	0.37	0.48
210	228.293	0.87	1.66	2.64	5.66	7.78	9.87	0.27	0.37	0.48
240	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
270	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
300	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48
330	228.293	0.01	0.01	0.02	1.81	1.91	2.04	0.27	0.37	0.48

Initial penetration for water-filled pipe ☑ | Initial penetration for water-filled pipe ☑ | Calculate necessary thickness ☑



Design condition for empty pipe		Design condition for pipe in operation	
10-year and 100-year RPV Combination		10-year and 100-year RPV Combination	
Results - Empty pipe		**Results - Pipe in operation**	
w_s [N/m]	1436	w_s [N/m]	1552
$w_{s,required}$ [N/m]	1552	$w_{s,required}$ [N/m]	1552
t_{conc} [m]	0.052	t_{conc} [m]	0.049
$t_{steel,added}$ [m]	0.000	$t_{steel,added}$ [m]	0.000
$w_{s,vertical}$ [N/m]	291	$w_{s,vertical}$ [N/m]	291



APPENDIX VIII. Geotechnical stability calculations

Introduction

The memorandum presents the general input, the methodology as well as the results for the verification of the geotechnical stability of the rock berms used for Balticconnector FEED study. For the verification, a limit equilibrium analysis program has been used. Calculations are performed for three (3) pre-lay supports required due to the Local Buckling criteria (LBC) and two (2) crossings of the existing Nord Stream (NS) pipelines.

The pre-lay supports are chosen as representatives to provide an estimate of the total Subsea Rock Installation (SRI) required for all local buckling supports. An overview of the locations requiring SRI due to local buckling can be found in the main report.

Locations, loads, and general dimensions are based on bottom roughness evaluations.

Applied Design Codes and Partial Factors

In general, the standard *EN ISO 19901-4 Part 4, 2003*, has been utilised as a basis for the calculations. In addition, load and material factors have been implemented as shown in the table below:

Item	Load Factor γ_l	Material Factor γ_M
Pipeline	1.3	-
Embankment	1.1 for main berm, 1.0 for counterfill	1.25
Soil Material	-	1.5 for soft clay 1.25 for other

Table VIII-1 Load and material factors

Soil Properties

Two soil investigation reports are currently available for the project:

- MMT Final Geotechnical Report, Balticconnector, Marine Survey 2006, Marine Mätteknik AB, Ref. /21/
- Marine Survey Report, Balticconnector Seabed Survey, Geophysical Survey and ROV Inspection, Gulf of Finland, October-December 2013, Issue for Use February 2014, Doc. No.: 101501-GAS-MMT-SUR-REP-SURVEYRE, Ref. /32/

Only for the soft clays, strength information was provided in the reports, and the recommended values were used in the calculations, including the increase over depth.

No information on the firm clay is available. As such, a reasonable estimation for the properties was prepared, but these shall be defined in the actual design of the project when more soil investigations are available. Based on the actual un-drained shear strength, the calculation results may change.

For rockfill, commonly accepted properties based on experience were used.

Material and Load Partial Factors are applied in the calculations. Refer to table below for used properties.

Item	Soft clay	Firm Clay	Rock Fill
Undrained Shear Strength cu [kPa]	4.0 + 1.5/m	20 1)	-
Friction Angle [Deg]	-	-	40
Saturated Unit Weight [kN/m^3]	14	14 1)	19.4
Note: 1) Estimated parameter, since no value is provided by lab reports			

Table VIII-2 Soil properties, characteristic values

The value for undrained shear strength of the clays includes reduction factors for anisotropy and rate effects of 0.8 and 0.75, respectively.

Modelling

Analysis utilizes SLIDE 6.0, a commercially available program from RocScience Inc. of Ontario, Canada. Calculations are performed as limit equilibrium and the traditional Bishop Simplified Method of Slices. It is noted that other methods, such as GLE/Morgenstern-Price lead to similar results.

Because of limited soil investigation results, transverse sections currently assumes horizontal seabed surface and soil layers. These should be checked in final design, but at this stage the assumption is considered justified.

Based on the input, longitudinal sections of support vary in height, but soil layers are assumed to be horizontal.

In the analyses for the longitudinal sections, the effective pipeline load applied at the rock support are decreased compared to what is stated in the bottom roughness report, due to a pressure distribution of the load through the rock fill.

Rock penetration into the soft soil at location #6 is expected to occur, and is included in the model by substituting the top soil with rock fill.

For each section of the rock berm analysed, the critical local and global failure have been identified, and the resulting factor of safety is presented. For overview of local vs. global failure, see Figure VIII-1.

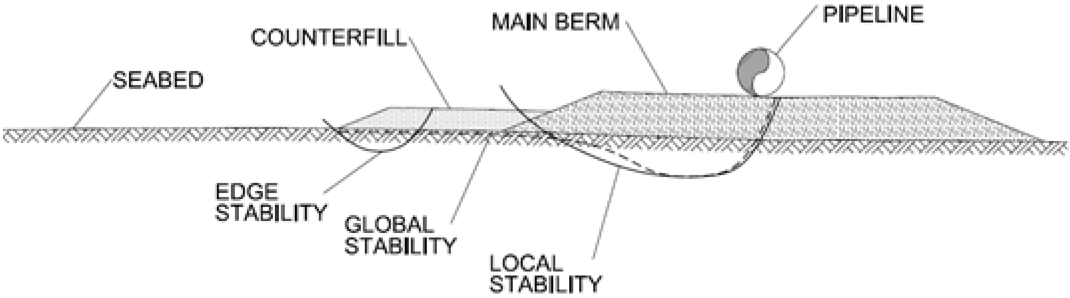


Figure VIII-2 Principal sketch, failure modes

Since the height of the counterfill required is found not to be more than 1 m, the edge stability is not analysed.

Installations of supports and pipeline

Support heights are defined according to the provided inputs from the bottom roughness evaluations. In addition, a 0.4 m rock overdump is considered for the main berm, to take installation tolerances of the rock fill into account.

The general width of all pre-lay supports is 11 m. Due to installation tolerances, the pipelines may be as close as 1m to the support embankment edge.

For the post-lay required at the Nord Stream crossing, the rock fill is installed relative to the pipeline. The analyses are hence performed with the pipeline located at centre of the rock support.

Pre-Lay Supports

Soil conditions

Three locations of pre-lay supports were provided. Ground conditions vary at the locations based on the seabed geology information:

- #6: Soft Clay over Firm Clay over Bedrock
- #15: Firm Clay over Bedrock
- #21: Sand Veneer over Bedrock

The Table below presents the locations of the pre-lay supports with estimated thickness of soil layers:

Location		Soft clay (over)	Firm Clay (over)	Sand Veneer	Bedrock
#6	KP17.816 - KP17.826	6m	8m	-	Rock defines the bottom of the calculation models
	KP17.855 - KP17.865	4m	8m	-	
#15	KP20.233 - KP20.248	-	2m	-	
	KP20.278 - KP20.288	-	2m	-	
#21	KP24.411 - KP24.421	-	-	0.5…1.0	
Note: Thickness of soil layers is estimated based on geophysical data					

Table VIII-3 Estimated Soil Layers and Thickness

Load input

The expected governing scenario is ULS for the water filled pipeline or during pressure testing. Hence, the highest loads from these are used for the analyses.

The loads were averaged over the support area. The table below presents the governing load case and the average load.

Location		Governing case	Average Load (kN/m)
#6	KP17.816 - KP17.826	Water-filling	9.4
	KP17.855 - KP17.865	Water-filling	9.9
#15	KP20.233 - KP20.248	Water-filling	6.3
	KP20.278 - KP20.288	System Pressure	13.6
#21	KP24.411 - KP24.421	System Pressure	11.4

Table VIII-4 Governing Load Cases

Dimensions of Pre-lay Support

Pre-lay support dimensions are based on Bottom Roughness analyses, which provide load input, width and height of supports. Additionally, a 0.4 m rock over-dump is applied.

Location		Height (incl.0.4m overdump)	Length (long. direction)	Width (transv. direction)
#6	KP17.816 - KP17.826	1.5…2.1m	10m	11m
	KP17.855 - KP17.865	1.15…1.5m	10m	11m
#15	KP20.233 - KP20.248	1.5…2.5m	15m	11m
	KP20.278 - KP20.288	0.9…1.4m	10m	11m
#21	KP24.411 - KP24.421	1.4…1.9m	10m	11m
Note: Length of supports does not include required slopes at the edges				

Table VIII-5 Pre-lay Support Dimensions (excluding counterfills)

Results

The following Table presents a summary of all safety factors:

Location		Section type	Safety Factor	
#6	KP17.816	Transversal	Local	1.01
		Transversal	Global	1.72
		Longitudinal	Local	1.64
		Longitudinal	Global	1.15
	KP17.826	Transversal	Local	1.10
		Transversal	Global	1.01
		Longitudinal	Local	1.43
		Longitudinal	Global	1.22
	KP17.855	Transversal	Local	1.14
		Transversal	Global	1.03
		Longitudinal	Local	1.30
		Longitudinal	Global	1.35
	KP17.865	Transversal	Local	1.19
		Transversal	Global	1.20
		Longitudinal	Local	1.76
		Longitudinal	Global	1.57
#15	KP20.233	Transversal	Local	1.75
		Transversal	Global	2.00
		Longitudinal	Local	1.86
		Longitudinal	Global	2.56
	KP20.248	Transversal	Local	1.63
		Transversal	Global	2.04
		Longitudinal	Local	1.71
		Longitudinal	Global	2.05
	KP20.278	Transversal	Local	1.47

Location		Section type	Safety Factor	
		Transversal	Global	1.40
		Longitudinal	Local	1.74
		Longitudinal	Global	3.40
	KP20.288	Transversal	Local	1.32
		Transversal	Global	1.45
		Longitudinal	Local	1.77
		Longitudinal	Global	3.31
#21	KP24.411	Transversal	Global	1.51
	KP24.411	Transversal	Global	1.43

Table VIII-6 Safety Factor Summary

It is observed that all safety factors are greater than 1.0, and therefore acceptable.

As seen in the Table, in some cased safety factors are above 2 for #15; however, while the counterfill size can be in principle optimised, it is noted that shear strength for the firm clay is estimated, and no measured shear strength is currently available. As such, if shear strength is lower in reality, safety factors will be reduced. However, during detailed design, when more knowledge about the soil conditions is available, design should be evaluated to verify the stability.

Based on the calculations the following counterfill dimensions are estimated:

Location		Counterfill		Assumed Rock Penetration Depth (m)
		Height (m)	Length (m)	
#6	KP17.816	1	14…15	0.5
	KP17.826	1	18…19	0.5
	KP17.855	1	14…15	0.5
	KP17.865	0.75	12…13	0.5
#15	KP20.233	0.5	-	-
	KP20.248	0.5	-	-
	KP20.278	0.5	4…5	-
	KP20.288	0.75	4…5	-
#21	KP24.411	-	-	-
	KP24.421	-	-	-

Table VIII-7 Estimated pre-lay embankment counterfill dimensions

The results indicate that safety factors are sufficient for the preliminary dimensioned counterfills and pre-lay support heights using limit equilibrium analyses.

<u>Recommendations</u>
It is noted that soil data is limited. For final pipeline design, additional soil investigations are recommended to prepare a safe design and dimensions for the required counterfills and pre-lay supports. In addition, in some cases, advanced finite element analysis is recommended to be performed to take into account deformation behaviour of the soil.

It is also noted that variations in shear strength and loadings may change the presented results.

Nord Stream Crossing

General

Both Nord Stream pipelines will be crossed by the Balticconnector pipeline. As such, stability of both pre-lay and post-lay supports are checked.

Carpet design is considered for the crossings, which consists of rock fill from pipeline touchdown to touchdown, forming a separation between the Balticconnector and the Nord Stream pipeline.

Soil conditions

Due to proximity of both crossings, ground conditions are similar with about 6 m of soft clay over firm clay according to seabed survey information.

Load input

The expected governing scenario is ULS for the water filled pipeline or during pressure testing. Hence, the highest loads from these are used for the analyses. The loads were averaged over the support area. The table below presents the governing load case and the average load.

Location	Governing case	Average Load (kN/m)
NS #1	Water-filling	4.0
NS #2	Water-filling	3.8
Note: Rock will be dumped to top of pipeline. As such, loads are same for pre-lays and post-lays		

Table VIII-8 Governing Load Cases

Dimensions of Pre-lay and Post-lay Supports

Pre-lay crossing dimensions are based on Bottom Roughness analyses, which provide load input and width and height of supports. Additionally, a 0.4m rock overdump is applied.

Location		Height (incl.0.4m overdump)	Length (long. direction) 2)	Width (transv. direction)
NS #1	Pre-Lay	0.7…1.4m	57m	11m
	Post-Lay 1)	1.7…2.4m	100m	11m
NS #2	Pre-Lay	0.3…1m	57m	11m
	Post-Lay 1)	1.7…2.4m	80m	11m
Notes: 1) For simplicity, for the post-lay calculations, the pre-lay section have been utilised, and the additional weight of post-lay is added as loads. 2) Post-lay length is estimated base on bottom roughness section from touchdown to touchdown. Length should be confirmed.				

Table VIII-9 Estimated Crossing Dimensions (excluding counterfills)

Results

Calculations were performed for longitudinal and transverse sections. Because of the symmetric shape of the supports, only one side has been calculated in longitudinal direction. In transverse direction, two (2) sections have been chosen: (1) at the start/end of the support with pre-lay height of about 0.3m (excluding rock overdump), and (2) at the highest point of the support of about 1m (excluding rock overdump). Start and end of each pre-lay are considered symmetrical and therefore only one section was calculated.

As mentioned, for simplicity, for post-lays the same sections have been utilised, and the additional weight of rock material was added as an additional load. Refer to the Table below for results. It is observed that all safety factor are greater than 1.0, and therefore acceptable.

Location			Section type	Safety Factor	
NS #1	Pre-lay	KP42.147, KP42.204	Transversal	Local	1.07
			Transversal	Global	1.12
			Longitudinal	Local	1.19
		KP42.175	Transversal	Local	1.24
			Transversal	Global	1.07
	Post-lay	KP42.147, KP42.204	Transversal	Local	1.23, 1.96
			Transversal	Global	1.14
			Longitudinal	Local	1.30
		KP42.175	Transversal	Local	1.43, 1.40
			Transversal	Global	2.21
NS #2	Pre-lay	KP43.064, KP43.121	Transversal	Local	1.20
			Transversal	Global	1.40
			Longitudinal	Local	1.18
		KP43.092	Transversal	Local	1.33
			Transversal	Global	1.17
	Post-lay	KP43.064, KP43.121	Transversal	Local	1.32, 1.88
			Transversal	Global	1.16
			Longitudinal	Local	1.49
		KP43.092	Transversal	Local	1.64, 1.55
			Transversal	Global	1.20
Note: For Post-lays local failure is checked at two locations					

Table VIII-10 Safety Factor Summary

Based on the calculations the following counterfill dimensions are estimated:

Location			Counterfill 2)		Assumed Rock Penetration Depth (m)
			Height (m)	Length (m)	
NS #1	Pre-Lay	KP42.147, KP42.204	0.3	6…8	0.5
		KP42.175	0.5	6…8	0.5
	Post-Lay 1)	KP42.147, KP42.204	0.5	6…8	0.5
		KP42.175	0.9	12…14	0.5
NS #2	Pre-Lay	KP43.064, KP43.121	0.3	6…8	0.5
		KP43.092	0.5	6…8	0.5
	Post-Lay 1)	KP43.064, KP43.121	0.5	6…8	0.5
		KP43.092	0.9	12…15	0.5

Notes:
1) Note that on the results sheets, the pipeline load is hidden behind the distributed loads for the post-lay weights.
2) Counterfills of post-lays are increased compared to pre-lays because of additional loads.

Table VIII-11 Estimated counterfill dimensions

Recommendations

It is observed that the area around the Nord Stream pipeline will be subject to additional loads from the supports of the Balticconnector pipeline. As such, displacements (settlements) may be expected. During detailed design, it is therefore recommended to perform advanced finite element analyses to check the effects of the pre-lay and post-lay loads on the pipeline.

Additional soil investigations are recommended to prepare a safe design and dimensions for the required counterfills and pre-lay supports.

It is also noted that variations in shear strength and loadings may change the presented results.

APPENDIX IX. Global buckling and trawl pull-over analysis

/1/ Appendix – TPL

Section	KP	Water depth (m)	Span height (m)	Pull over time (s)	Horizontal force (kN)	Vertical force (kN)
1	67.50 – 80.40	20	0.1	0.380	150.6	-77.1
2	0.00 – 12.00	25	0.0	0.369	117.0	-65.4
			0.1	0.430	136.6	-69.3
			0.3	0.545	172.9	-73.4
			0.4	0.598	189.7	-74.4
			0.5	0.648	205.7	-75.0
3.1	12.00 – 19.35	30	0.0	0.397	105.0	-59.3
			0.3	0.591	156.2	-66.9
			0.4	0.649	171.6	-67.9
			0.5	0.705	186.3	-68.4
			0.7	0.808	213.6	-68.8
			0.8	0.855	226.2	-68.9
3.2 + 3.3	19.35 – 26.00	35	0.7	0.872	197.7	-63.7
			0.8	0.924	209.4	-63.8
			0.9	0.973	220.6	-63.8

Table IX-1 Considered TPL

/2/ Appendix – Model summary

Subject	Description / comments
Pipe element	PIPE288 is based upon small deflection and thin beam theory in which shear deflection is included. PIPE288 includes effect of inner and outer pressure. Effective axial force is automatically included. The element has non-linear plastic capabilities. Element length: 1 m All variations in pipe property (size, weight, etc.) along the route are included.
Seabed, pipe-soil interaction, rock cover	The seabed is flat. The pipe-soil interaction is modelled with non-linear spring elements (COMBIN39). The complete force-deflection behaviour as given in section X.X (pipe-soil interaction report) is included. Friction elements (CONTA175 and TARGE170) are placed between pipeline and seabed. These however are only activated in case of TPL when the vertical component creates a contact pressure between pipeline and seabed. Two independent elements are connected to each node, one for axial and one for lateral behaviour. Rock covered sections are modelled with the accurate pipe-rock resistance allowing axial and lateral movements inside the rock. All variations in friction properties along the route are included.
Pipeline route, model length and boundary conditions	The model is straight. For every simulation it is verified that there is a natural lock point. The landfall is modelled as fixed. Cuts along the pipeline are fixed in order to include the symmetry conditions.
Material models	The Multilinear Isotropic Hardening (MISO) model is used. This option uses the von Mises yield criterion coupled with an isotropic work hardening assumption.
Loads	Loads consist of: • Submerged weight. • Temperature and pressure loads, creating the effective axial buckling force. • TPL • Inertia effects including added mass. • Residual lay-tension No wave and current loads are applied.
Analysis procedure	1. Calculate the as-laid condition incl. lay-tension, weight and horizontal displacements. As-laid bending moments are not included directly in the model 2. Remove start-up pile. This step allows the pipeline to contract 3. Add post-lay rock cover on selected sections 4. Apply temperature and pressure loads 5. Perform TPL All load steps until operation are analysed using static procedure. Subsequent steps which are affected by inertia (GB and TPL) use transient procedure. Non-linear geometric effects are always included.

Table IX-2 Summary of 2D analysis spring model – Lateral buckling (section 1)

Subject	Description / comments
Pipe element	PIPE20 from the ANSYS library. Element includes effect of inner and outer pressure. Effective axial force is automatically included. The element has non-linear plastic capabilities. Element length: 1 m All variations in pipe property (size, weight, etc.) along the route is included.
Seabed, pipe-soil interaction, rock cover	The actual seabed topography is used. The seabed is modelled with contact elements including pipe-soil vertical stiffness behaviour and orthotropic Coulomb frictional contact. CONTA175 and TARGE170 from the ANSYS library are used to model the seabed-pipe contact pair. The seabed elements have distinct frictional characteristics with independent axial friction coefficients. For axial friction, the residual value is used in all segments. Rock covered sections are modelled with increased axial friction allowing axial movements inside the rock. Lateral displacement is restricted. Vertical uplift resistance is modelled using non-linear spring elements (COMBIN39). All variations in friction properties along the route are included.
Pipeline route, model length and boundary conditions	The route is straight in the horizontal plane and uses seabed surveys to model the seabed profile. Spool ends are modelled as free which is acceptable due to the low impact of the tie-in in order to simulate the find a conservative force profile. Cuts along the pipeline are fixed in order to include the symmetry conditions.
Material models	The Multi-linear Isotropic Hardening (MISO) model is used. This option uses the von Mises yield criterion coupled with an isotropic work hardening assumption.
Loads	Loads consist of: • Submerged weight. • Temperature and pressure loads, creating the effective axial buckling force. • Inertia effects including added mass. No wave and current loads are applied.
Analysis procedure	1. Calculate the as-laid condition incl. lay-tension, weight and horizontal displacements and bending stresses from seabed undulations are included. 2. Remove start-up pile. This step allows the pipeline to contract. 3. Add post-lay rock cover on selected section. 4. Apply temperature and pressure loads. All load steps until operation are analysed using static procedure. Subsequent steps which are affected by inertia (buckling) use transient procedure. Non-linear geometric effects are always included.

Table IX-3 Summary of 2½D (and UHB) analysis contact model – Section 2 and 3

/3/ Appendix – Hobbs force

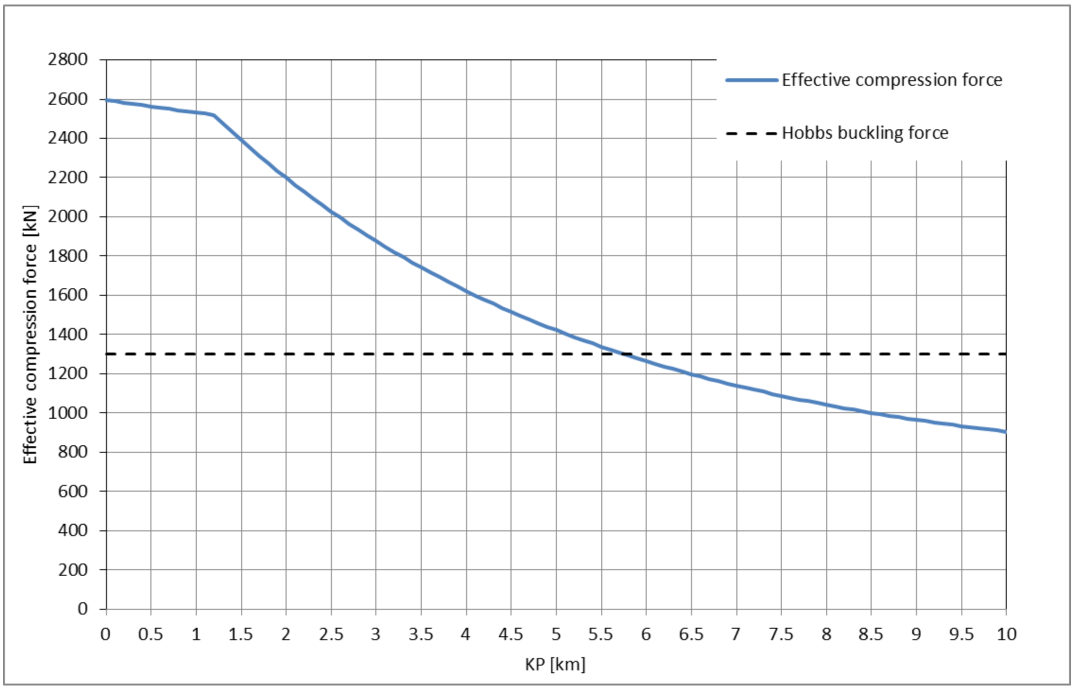


Figure IX-1 Hobbs critical buckling force – Section 1 – KP from Estonia shore

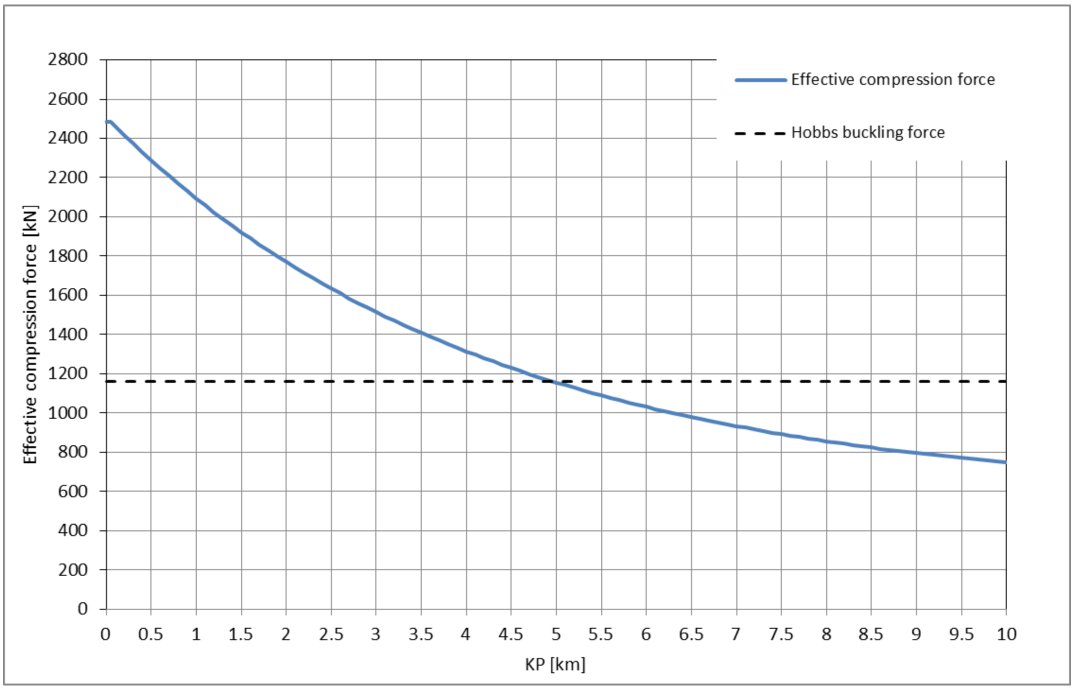


Figure IX-2 Hobbs critical buckling force – Section 2 – KP from Finland shore

APPENDIX X. Pipe-soil interaction charts

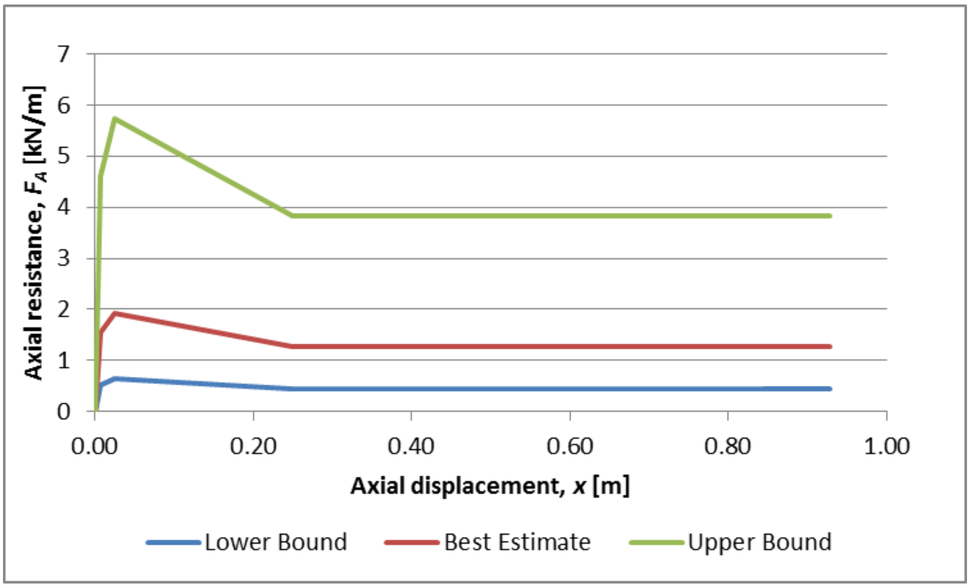


Figure X-1 Axial soil response for a pipeline resting on CLAY for Case 2

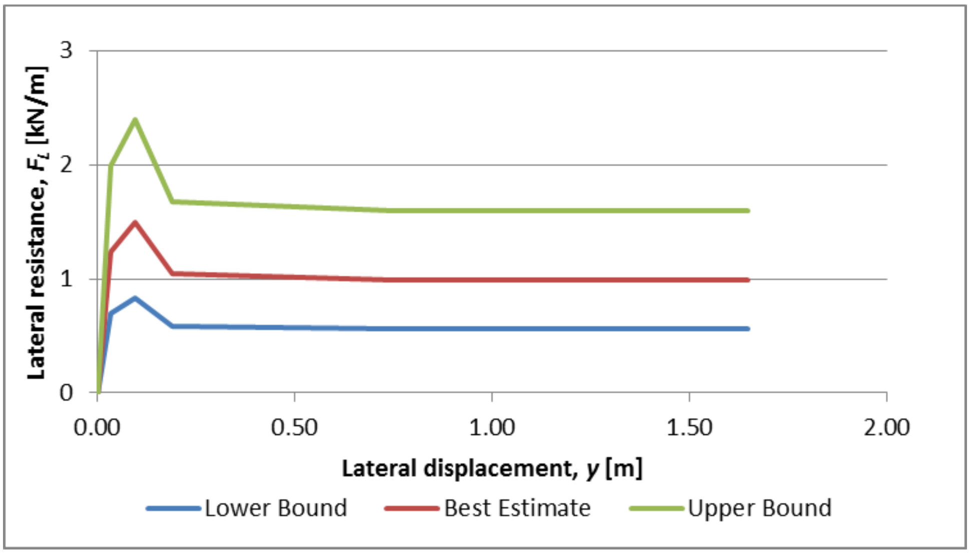


Figure X-2 Lateral soil response for a pipeline resting on CLAY for Case 2

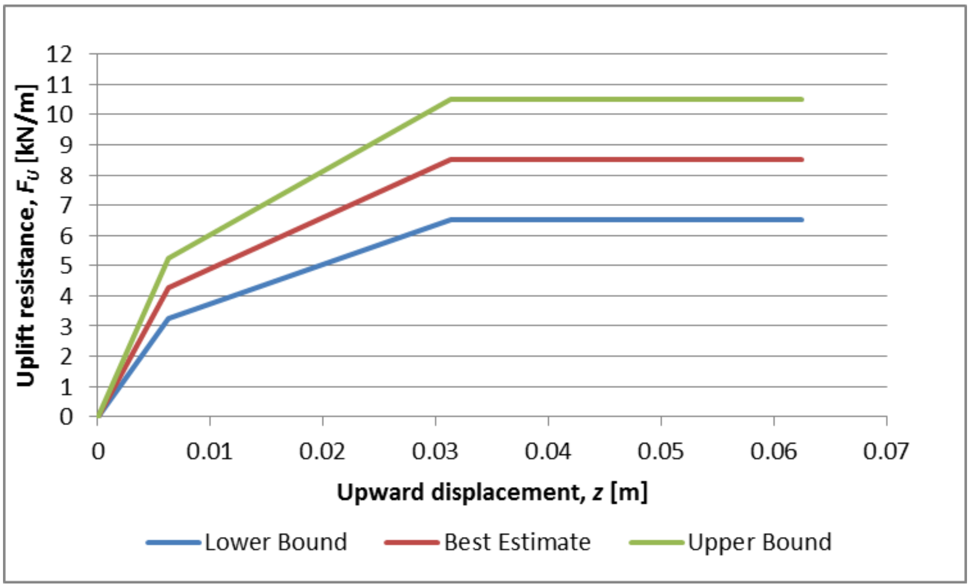


Figure X-3 Uplift resistance for a pipeline trenched and backfilled with CLAY for Case 2

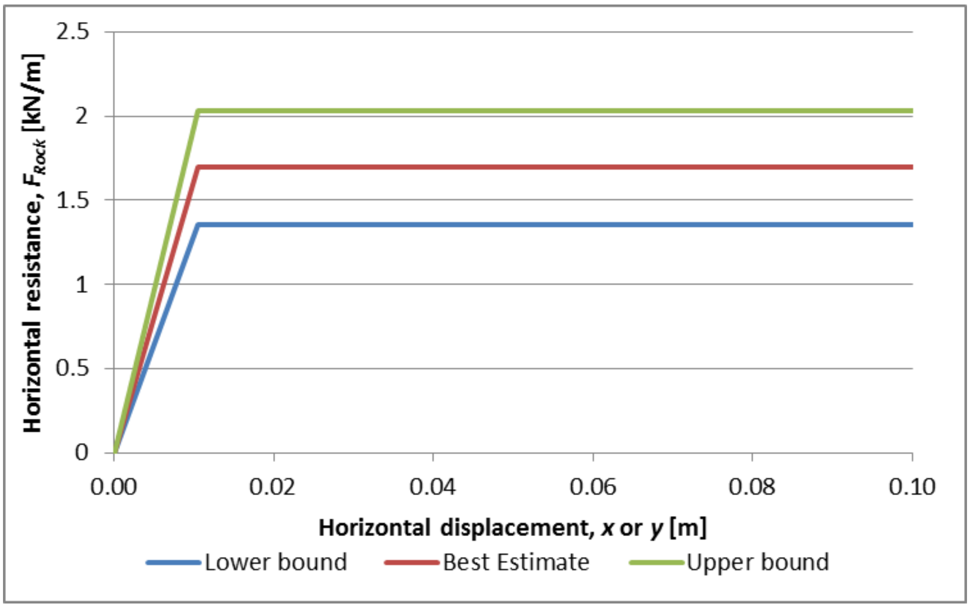


Figure X-4 Horizontal soil response for a pipeline resting on ROCK for Case 5

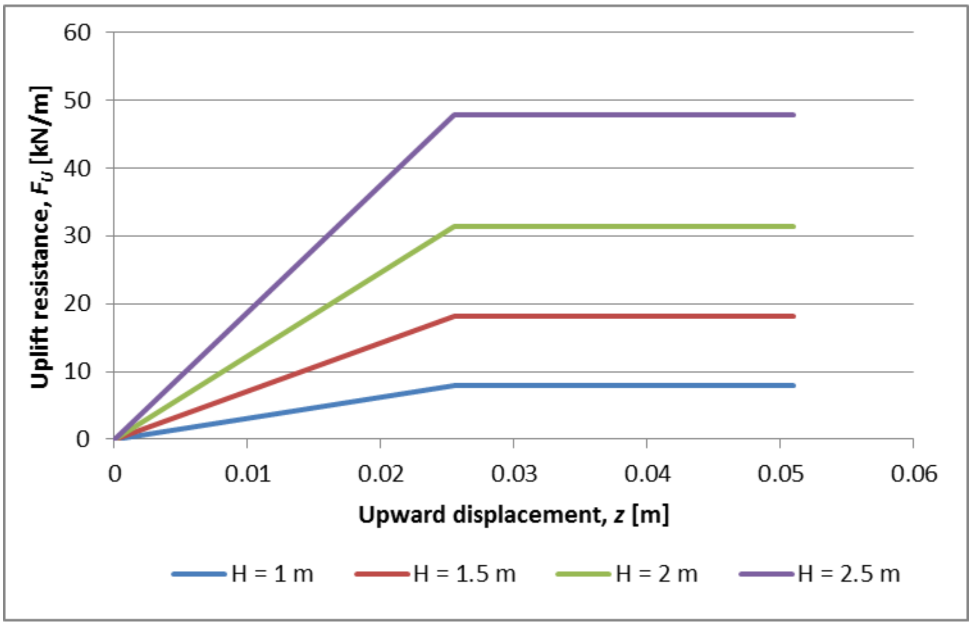


Figure X-5 Uplift resistance for a pipeline trenched and backfilled with ROCK for Case 5

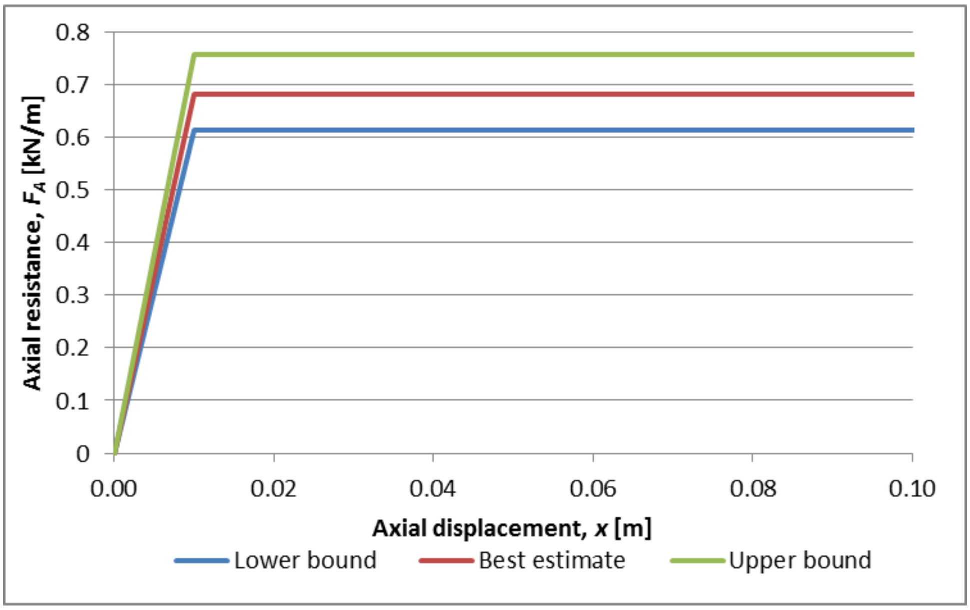


Figure X-6 Axial soil response for a pipeline resting on SAND for Case 8

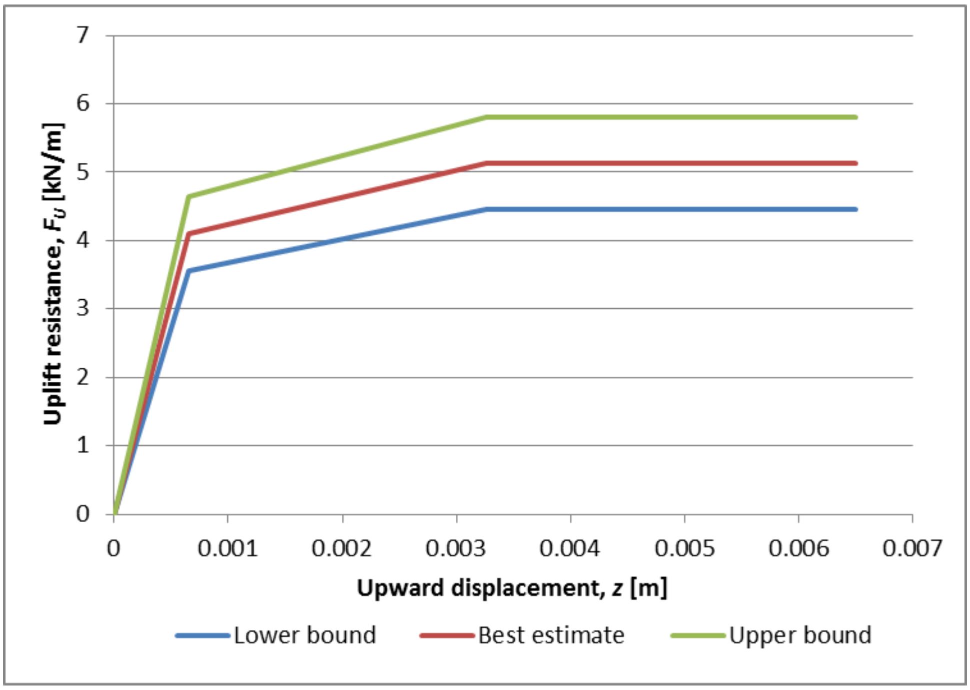


Figure X-7 Uplift resistance for a pipeline trenched and backfilled with SAND for Case 8

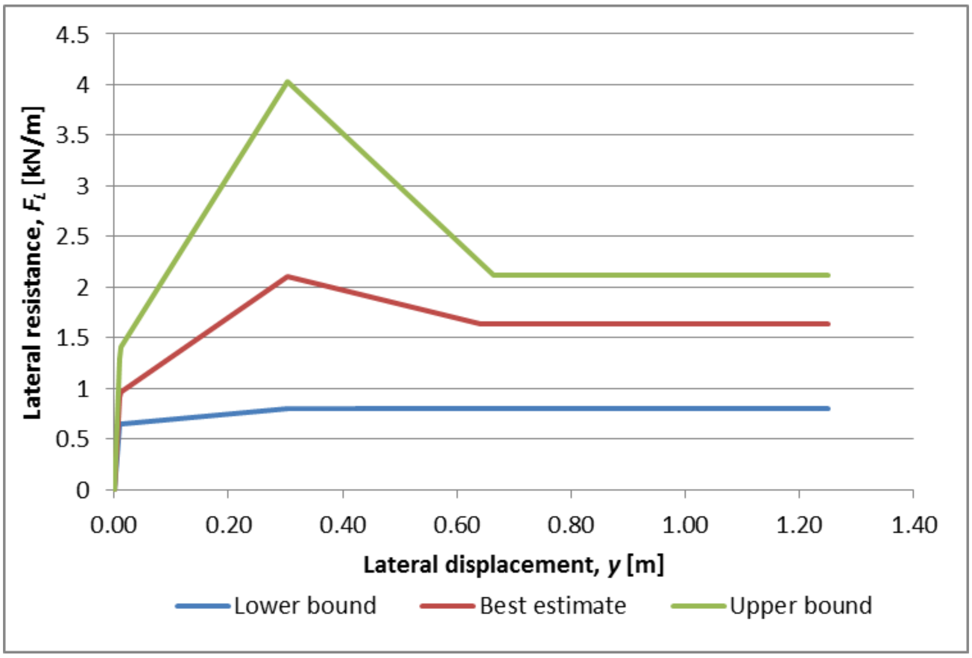


Figure X-8 Lateral soil response for a pipeline resting on SAND for Case 8