REPUBLIC OF LEBANON

COUNCIL FOR DEVELOPMENT AND RECONSTRUCTION

EXPANSION AND UPGRADE OF AL-GHADIR WASTEWATER TREATMENT PLANT PROJECT – PHASE I: DESIGN, BUILD AND OPERATE, INCLUDING TWO NEW SEA OUTFALLS AND INSPECTION & ASSESSMENT OF EXISTING SEA OUTFALLS

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Part 5 – Other Information

Book 2/2

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DAR AL HANDASAH NAZIH TALEB & PARTNERS
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SECTION XIV – REFERENCE DESIGN REPORT FOR JNAH SEA OUTFALL

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LEBANESE REPUBLIC

COUNCIL FOR DEVELOPMENT AND RECONSTRUCTION

BEIRUT - LEBANON

CONSULTANCY SERVICES

FOR

Beirut City Services Upgrading Project
Phase III - Lot 4

Sea Outfall Design Report



June 2018

KHATIB & ALAMI

CONSOLIDATED ENGINEERING COMPANY S.A.L.

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List of Abbreviations

FC	Fecal Coliforms
NF	Near Field
FF	Far Field
OD	Outside Diameter
ND	Nominal Diameter
ID	Internal Diameter
PN	Nominal Pressure
HDPE	High Density Polyethylene
SDR	
PE	Polyethylene
FS	Factor of Safety
PS	Pumping Station
MSL	Mean Sea Level

EXECUTIVE SUMMARY

The Carlton – Ghadir Collector Project calls for the construction of two major lifting stations – one near the Grand Café next to the Moevenpick Hotel and Resort (PS1) and the other near at the Sultan Ibrahim roundabout, to pump the sewage to the Ghadir wastewater Treatment Plant (PS2). PS2 shall also receive the sewage coming from the Rihab - Sabra area.

In order to ensure the proper operation of the project, especially during power outages, the Sultan Ibrahim lifting stations need to be equipped with an overflow line.

Thus, in order to minimize the environmental impact of the discharged effluent through dilution of the overflow line, it was decided to design it as a complete sea outfall.

The overflow was thus routed through an onshore part, starting at the lift station overflow manhole and composed of a 135m long ND900 GRP pipe, routed under the roundabout pavement, connected to a 120m long OD800mm HDPE pipe running inside the northern cell of the existing twin cell stormwater culvert flowing into the pressurized connection manhole constructed at the shoreline. The onshore part is then relayed by an HDPE offshore part, of OD 1400mm, conveying the effluents to the foreseen diffusing location, at around -17m depth. The length of the offshore outfall part had to be extended to 908.6m, including a 200m long diffuser in order to meet both the minimum discharging depth criterion of 15m and the minimum initial dilution criterion of 1:50. The selected pipeline material was PE80 SDR26 for the main pipeline and PE100 SDR26 for the diffuser including risers and ports. The minimum pressure rating of the pipeline was set at PN6. The diffuser ports are to be fitted with wide bill duckbill valves.

The sea outfall diameter had to be increased to OD1400mm in order accommodate a peak water flow of 1.5m³/s to minimize the head losses through the marine section of the pipeline and thus allow for gravity flow under ordinary weather conditions. The CorHyd Software, a computer code for the calculation of flow characteristics in multiport diffuser constructions, has been used to compute the head losses in the marine section of the outfall as well as to optimize the diffuser's design in order to obtain a homogeneous discharge distribution along the diffuser, minimize the total head, achieve operational stability and prevent sedimentation or ambient water intrusion in the diffuser under varying inflow and ambient conditions.

1. Introduction

1.1 Project description

The Carlton – Ghadir Collector Project calls for the construction of two major lifting stations – one near the Grand Café next to the Moevenpick Hotel and Resort (PS1) and the other near at the Sultan Ibrahim roundabout, to pump the sewage to the Ghadir Wastewater Treatment Plant (PS2). In addition to the a/m sewage, and since the scope of work includes the separation of sewer and storm water pipes at Rihab –Sabra area, PS2 shall also receive the sewage coming from the Rihab - Sabra area.
In order to ensure the proper operation of the project, especially during power outages, the Sultan Ibrahim lifting stations need to be equipped with an overflow line.
Thus, in order to minimize the environmental impact of the discharged effluent through dilution of the overflow line, it was decided to design it as a complete sea outfall.
The Sultan Ibrahim outfall is divided into two sections: an onshore section followed by an offshore section.

1.2 Scope of the Report

This present document is a design report whose purpose is to define the main characteristics of the outfall, through a hydraulic and marine design that includes the following:

1.2.1 Design Parameters and Data

i. Environmental existing data such as wind velocity and direction, temperature and salinity.
ii. Met-ocean data such as the characteristics of waves, tides and currents.
iii. Geotechnical investigation

1.2.2 Calculation notes

i. Hydraulic design of the Outfall using Corhyd dedicated software
ii. Diffusion modeling using Cormix 10.0GTD dedicated software
iii. On-bottom stability and maritime protection design of the Outfall

2. Design Overview

The optimum location, dimension, length and direction of the sea outfall pipe will be determined in order to reach, at the point of initial mixing as well as in the general area, the best dilution rate, therefore, attending the minimum pollutant effect.
The whole layout will take into consideration the following:

i. Peak flow of waste water
ii. Waves and currents action on pollutants spreading
iii. Characteristics of sea water in diffusion area
iv. Pipe stability and protection

2.1 Regulations and Standards

In the absence of relevant National Acts of Parliament or Lebanese Government legislation, regulations or standard related to the design of sea outfalls, the following documents will be used as guidelines through the design steps:

2.1.1 Environmental Regulations

i. **Decision number 8/1 of Ministry Of Environment (MEO)** related to the characteristics and standards of air pollutants and liquid waste generated by Waste Water Treatment Plants published in Official Journal number 10 dated 1/3/2001
ii. **Decision number 52/1 of Ministry Of Environment (MEO)** related to the setting up of the characteristics and rates related to the air, water and land pollution published in Official Journal number 45 dated 12/9/1996
iii. **Guidelines on sewage treatment and disposal for the Mediterranean region** (United Nations Environment Program Mediterranean Action Plan, MEDPOL) published in Athens,2004
iv. **Standard Practice Document (SPD11: Sea Outfalls)** publication of the Republic of Lebanon, Council for Development and Reconstruction, Commission of the European Communities, Directorate-General, External Economic Relations Delegation, Ministry of Hydraulic and Electric Resources and Ministry of Environment.

2.1.2 Met-ocean Data

i. **Climatic Atlas of Lebanon, Part 1**, Precipitation - Temperature - Atmospheric Pressure - Clouds, Reprint 2 published by the Directorate of Civil Aviation (named hereafter DCA) dated 1977 under the Ministry of Public Works and Transportation (named hereafter MPWT).
ii. **Climatic Atlas of Lebanon, Part 2**, Relative humidity, published by the DCA
iii. **Climatic Atlas of Lebanon, Part 3**, Winds, published by the DCA
iv. **Volume 2 Technical Specifications**, of the Tender Documents : Project (ref 1407) WASTE WATER DISPOSAL INTO THE SEA (SEA OUTFALL) IN MANARA AREA BEIRUT and contract (Ref 18426) Construction of sea outfall.
v. **Sea outfall of the Sour water treatment plant** (Stability Report): Document obtained from KCC.
vi. **Potential Touristic Development of the Ain El Mreisseh Coast:** Document obtained from BTUTP.
vii. **Summerland Resort Project – Beirut, Lebanon – Facilities Studies – Water Sport:** Document obtained from KCC – including HR Wallingford report (results from numerical wave modelling using the Met Office European wave model). at two points located offshore of the site at coordinates:
- Point 1: 34.00N 35.13E
- Point 2: 33.75N 35.13E

2.1.3 Marine Structures standards

i. **Construction Industry Research and Information Association CIRIA**, Manual on the use of rock in coastal and shoreline engineering, Special publication 83, 1991, Center for Civil Engineering Research and Codes Report 154.
ii. **Coastal Engineering Manual CEM 1110-2-1100** dated June 2006.
iii. **DNV, Rules for Submarine Pipeline Systems** dated 1981, Reprint 1993.
iv. **DNV, Recommended practice, RP-F109, on-bottom stability design of submarine pipelines** dated October 2007.
v. **BS 6349, Code of practice for maritime structures, Part 1**, General criteria
vi. **BS 6349, Code of practice for maritime structures, Part 2**, Design of quay walls, jetties and dolphins.
vii. **BS 6349, Code of practice for maritime structures, Part 7**, Guide for the design and construction of breakwaters.

viii. **Pipe life Norge AS, Technical Catalogue** for Submarine Installations of Polyethylene Pipes.

2.2 Design Criteria

The design of the outfall shall be according to the following criteria:

i. Hydrodynamic stability, checked by using 50 years return period $H_{1/50}$ wave height.
ii. Dilution criteria: The diffuser will be designed to provide a minimum average initial dilution of 50 – based on similar projects recently executed by CDR (Sour WWTP sea outfall and Manara sea outfall)

Additional design criteria include:

- *The marine section of the pipeline shall be buried within a dredged trench with a minimum cover depth of 1.0 m below the crown of the pipe until sea water depth of 13m.*
- *Discharge shall be at a minimum 15m depth below sea water elevation.*
- *Pipeline, riser, crosshead and discharge nozzle sizes shall be selected in the design to achieve good flow balance between individual ports. All diffuser ports shall be at a nominal height of 0.8 m above the seabed.*
- *The outfall structures and pipeline shall have a design life of 50 years.*
- *Pipe and all related devices, joints, valves ... of ISO PN nominal pressure greater than 1.5 times the functioning pressure (including transitional phenomena) for a 1m3/s discharge pipe to accept at least a 0.5 bar depression.*

Possible effects of the various environmental actions are to be taken into account to the extent to the situation considered.

3. Environment

Data supplied by generally recognized consultants will normally be accepted as a basis for design.
The various environmental factors will be described by characteristics parameters based on statistical data or long term observations. If sufficient data directly applicable for the Sultan Ibrahim location are not available, reasonable conservative assumptions shall be used.
Statistical data shall be utilized in describing environmental parameters of a random nature (e.g. waves, wind).
Environmental data can be summarized as follows:

i. Pipeline Route :
   - Location
   - Route Survey
   - Seabed Properties

ii. Environmental Conditions:
   - Tide
   - Wind
   - Waves
   - Current
   - Air and Sea Temperatures

iii. Effluents data
   - Flow rate
   - Effluent characteristics

3.1 Pipeline Route

3.1.1 Location

The pipeline route is usually selected taking into account several factors that may either directly or indirectly damage the pipe or be affected by a possible pipe rupture. These are:

i. *Population density*
ii. *Vicinity of dwellings*
iii. *Ship traffic*
iv. *Fishing activity*
v. *Offshore operations*
vi. *Unstable seabed*

Known future developments in the vicinity of the pipeline route are to be taken into consideration.
The Sultan Ibrahimsea outfall includes 2 main parts:

- An onshore part, starting at the lift station overflow manhole and composed of two sections: a first section, 135m long DN900 GRP pipe, routed under the roundabout pavement, and a second section, 120m long 800mm HDPE pipe running inside the northern cell of the existing twin cell stormwater culvert. This part ends at the shoreline pressurized connection manhole.

- An offshore part, conveying the effluents flowing into the pressurized manhole to the foreseen diffusing location, at around -17mdepth. The length of the offshore outfall part is 908.6m, including a 200m long diffuser. Invert levels of pipe, distances, (X,Y) coordinates and slopes are shown in the following table.

Table 1- Invert levels of pipe, distances, coordinates and slopes

Designation	X	Y	Z	Cumulative Distance (m)	Slope (%)
A	-339287.36	-31561.77	-4.0	0	-
B	-339387.36	-31561.77	-5.493	100	1.49
C	-339487.36	-31561.77	-6.987	200	1.49
D	-339587.36	-31561.77	-8.48	300	1.49
E	-339687.36	-31561.77	-10.083	400	1.49
F	-339787.36	-31561.77	-12.126	500	2.04
G	-339887.36	-31561.77	-14.169	600	2.04
H	-339995.96	-31561.77	-16.387	708.6	2.04
Hbis	-340112.83	-31561.77	-18.775	825.47	2.04
I	-340195.97	-31561.77	-21.207	908.61	2.93

Where H & I are respectively the beginning and the end of the diffuser section.



Figure 1-Aerial view of Sultan Ibrahim area

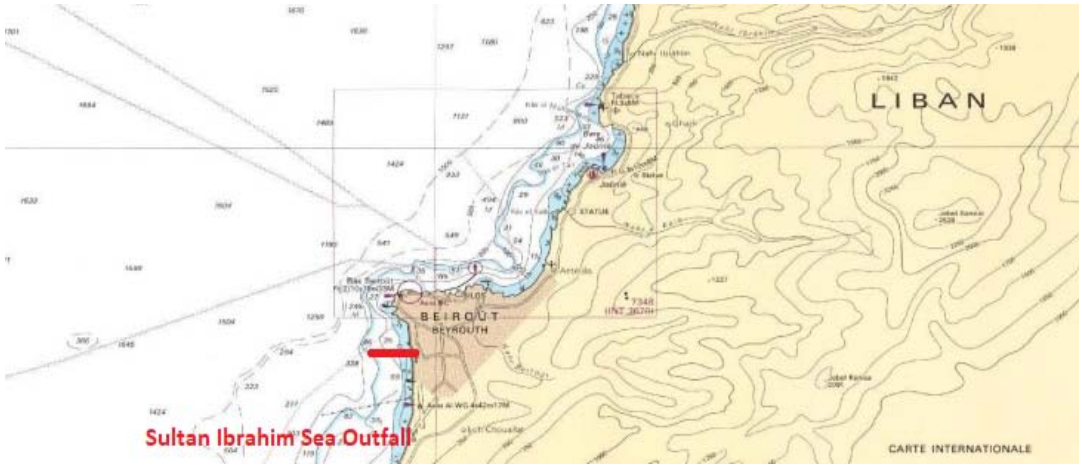


Figure 2-Location of Sultan Ibrahim Outfall on the SHOM (Service Hydrographique et Océanographique et de la Marine) plan.

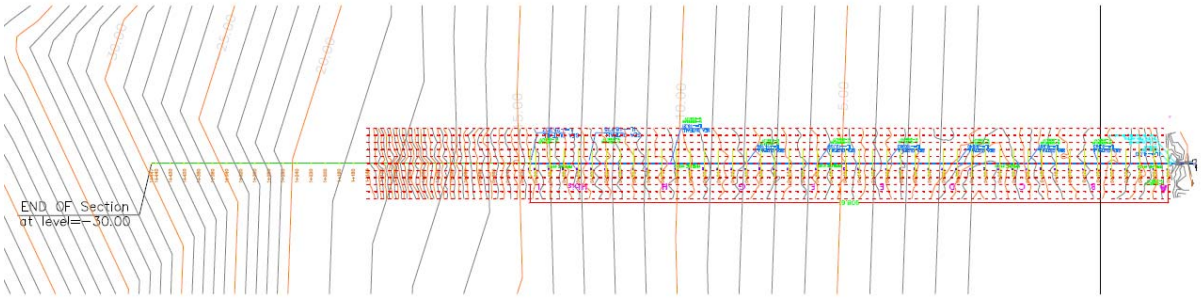


Figure 3-Plan view of the previewed onshore part connecting the Sultan Ibrahim Lifting Station to the offshore part.

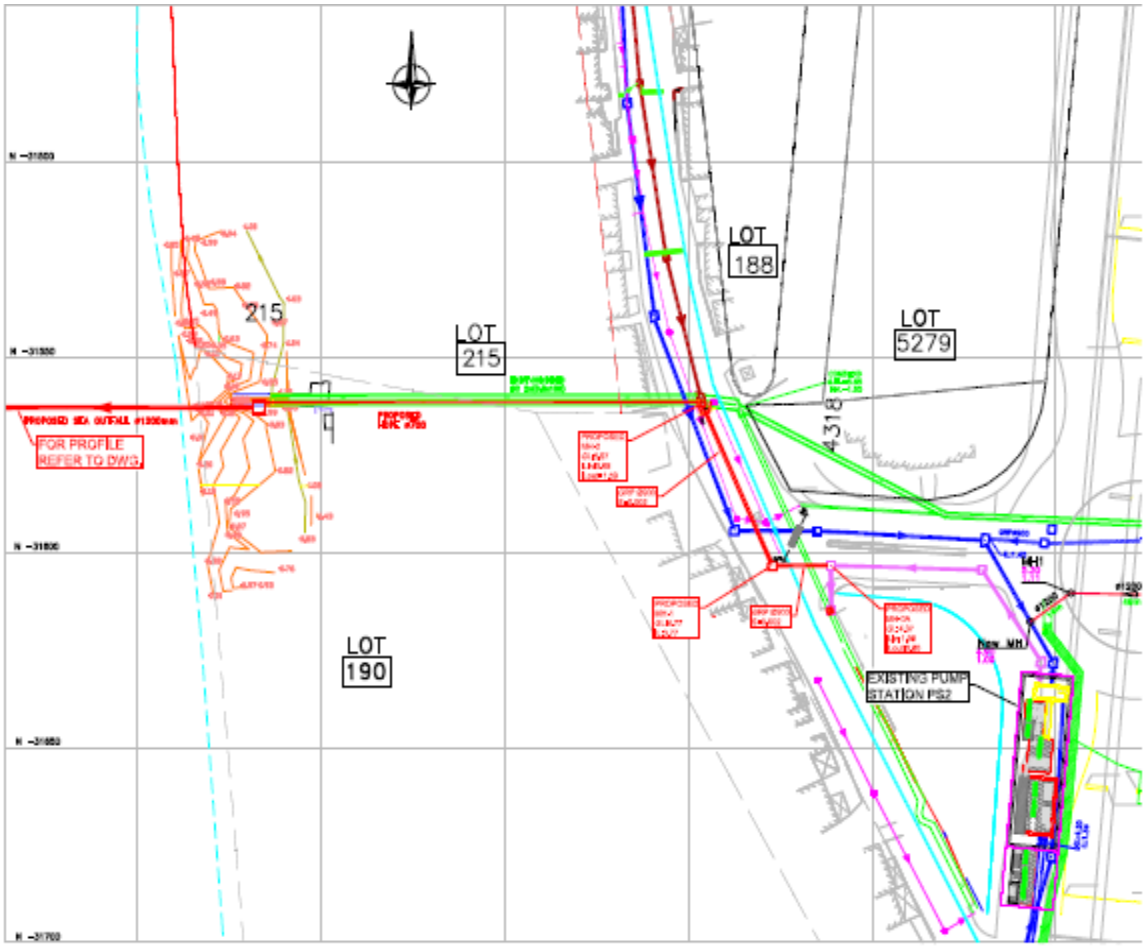


Figure 4- Plan view of the lifting station overflow connection to sea outfall

3.1.2 Route survey

A detailed route survey has been performed. It has provided sufficient data for design and construction. The route survey has covered sufficient width and been done with such accuracy to permit the safe and proper installation and operation of the pipeline.

The results of the topographic and bathymetric surveys conducted in the Sultan Ibrahim outfall area are presented under separate cover. They have shown the following:

i. Water depth range between 0m and -40m over 1150m (counted from the shoreline)
ii. Existing sea bed slope varies from shallow water to deep water zone as subsequent:
- from 0.0m to -5.52m : 2.76% over 200m
- from -5.52m to -6.29m : 0.77% over 100m
- from -6.62m to -5.66m : -2.40% over 40m
- from -5.66m to -7.27m : 2.68% over 60m
- from -7.27m to -21.76m : 2.78% over 520m

Figure 5, after the CNRSL CANA Research Project, shows that the depth of 60m is reached at a distance of around 2.2Km from the shoreline.

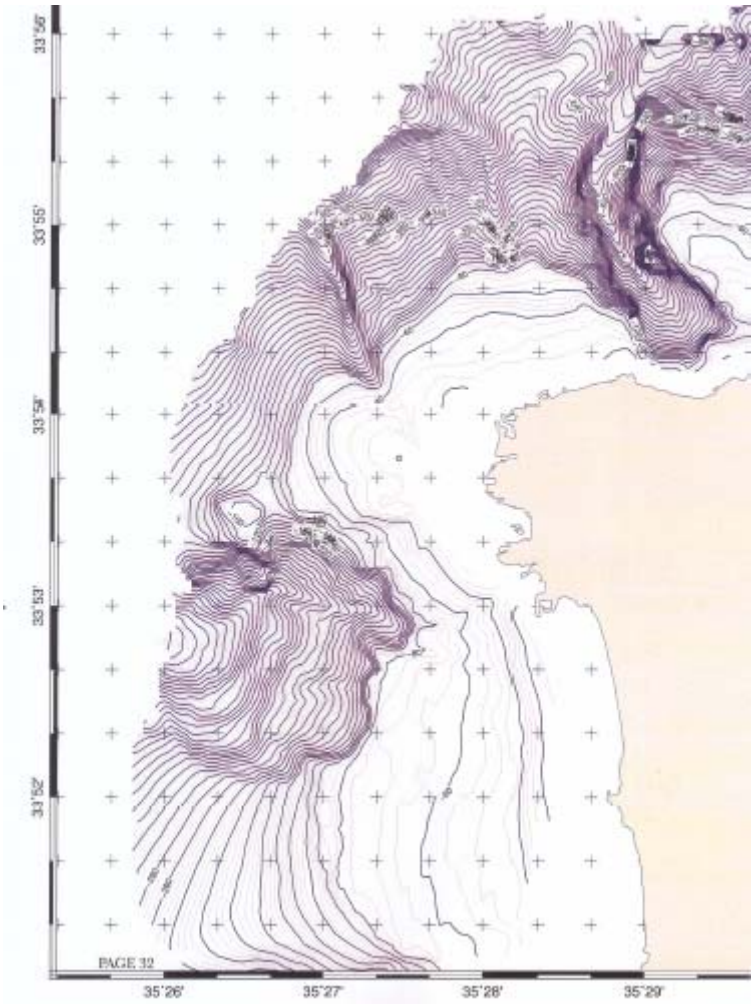


Figure 5-The CNRSL CANA Research Project

Notes:

- Bench marks were set on site
- Tidal variation was monitored during the bathymetric survey period.
- Topographic and Bathymetric surveys are tied to 0.00NGL = +0.40CD (Refer to paragraph 3.2.1 for abbreviations and tidal values)

3.1.3 Seabed Properties

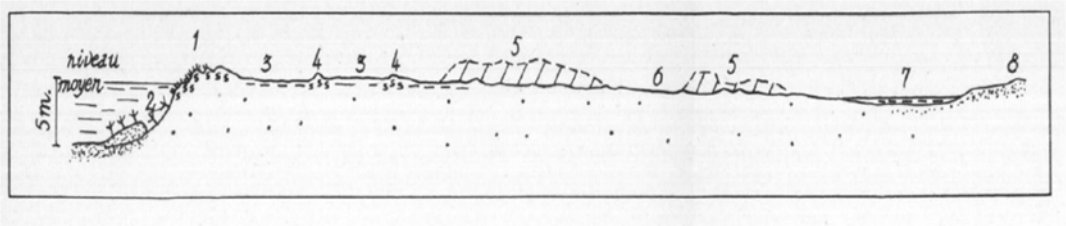


Figure 6-Typical Cross section of a Sandstone platform at Jnah

From the spearing (lançage) campaign carried out on site between stations 0+045 and 0+200, it was found that the seabed substratum is rocky (Sandstone) in nature, covered with Sand of variable thickness (3 to 5m).

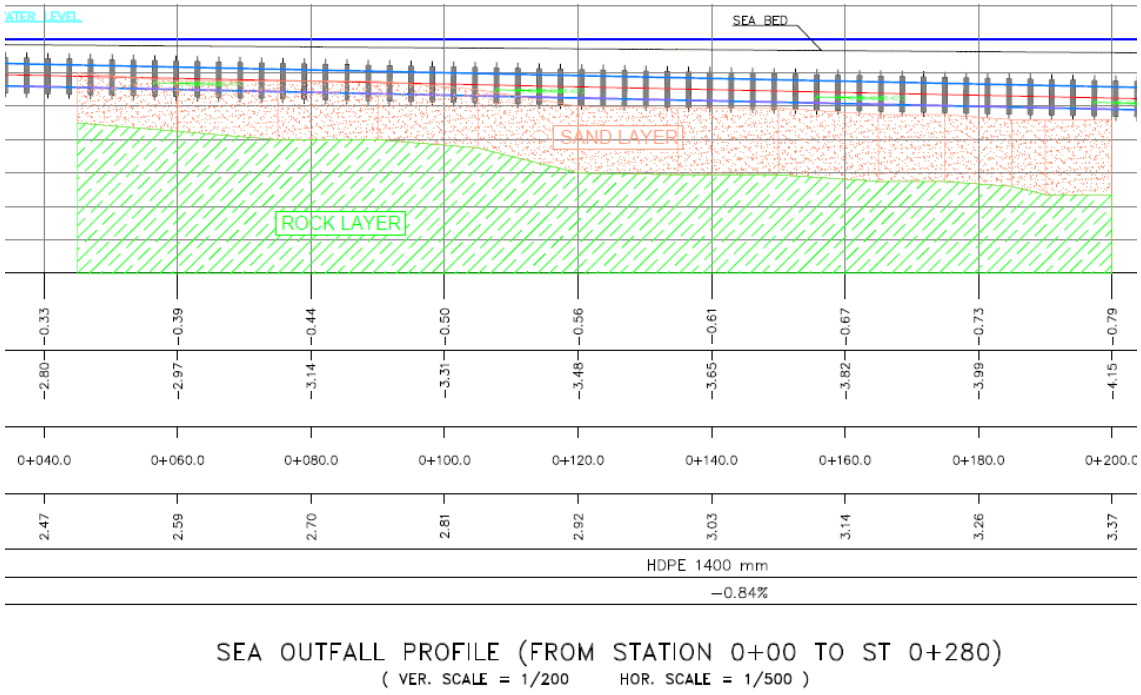


Figure 6bis Soil profile between Stations 0+045 and 0+200

This implies hard rock and consequently an insignificant magnitude of settlement under loading.

3.2 Environmental Conditions

Possible effects of the various environmental actions were considered in the present design report .

3.3 Tide

Tides are small along the Lebanese coastline. Their currents are weak and usually dominated by wind generated surface currents.
The tide bar along Beirut coast can be summarized as follows (referring to SHOM Nautical Chart nº 7255 - Marée carte SHOM en CD, 0mNGL=0.4m CD)

Table 2 – Tide bar along Beirut coast

	Beirut Chart Datum
Mean Spring High Water	+0.4m
Mean Neap High Water	+0.3m
Mean Neap Low Water	+0.2m
Mean Spring Low High Water	+0.1m

Table 3 – Tide bar along Beirut coast

With NGL : Nivellement Général du Liban
CD : Chart Datum

Referring to SHOM Nautical Chart, the Chart Datum CD is 0.40m below NGL along Beirut coast.
As seawater warms up, it expands, increasing the volume of the global ocean and producing thermostatic sea level rise.
According to HR Wallingford report (results from numerical wave modelling using the Met Office European wave model) at two points located offshore of the site at coordinates (Point 1: 34.00N 35.13E and Point 2: 33.75N 35.13E), the extreme water levels are as follows:

Table 4- Extreme water levels at Beirut Airport Coast

Beirut Airport	
Return Period	NGL (Above MWL)
1 year	+0.25m
10 years	+0.35m
50 years	+0.45m
100 years	+0.55m

With MWL : Mean Water Level

HR Wallingford specifies a 0.20m provision for sea water level rise is to be considered. The following table shows the return period of the extreme Water level rise:

Table 5-Extreme water levels at Beirut Airport Coast reflecting the 0.20m provision specified in HR Wallingford report

Beirut Airport	
Return Period	NGL
Mean Sea Level	+0.20m
1 year	+0.45m
10 years	+0.55m
50 years	+0.65m
100 years	+0.75m

3.3.1 Wind

With reference to the **Climatic Atlas of Lebanon**, Part 1, winds velocities and direction monitoring at Beirut Airport (1957 to 1964) proves the following ascendant wind directions and velocities:

- **i.** The dominant wind direction is ESE with a maximum speed reaching 5m/s; follows the directions W, SW, WSW and SE with the same wind velocity. These winds are encountered almost all year around.
- **ii.** A lesser dominant wind is SW direction with 10m/s maximum speed. This direction is mostly distributed from March to August. WSW and SSW follow with the same speed.
- **iii.** Stronger winds are less dominant and occur with W direction mostly and can blow with SW, W, SSW, S, SSE,SE and E directions. Their speed can reach 15m/s and can occur from November to May.
- **iv.** The most critical winds (heavy storms) can occur in SW, WSW and W with a velocity higher than 16m/s (can reach 20 to 30m/s). These winds occur generally from October to March.

The above was further corroborated with data available on the **WINDFINDER Website**, the wind celerity and directions are represented with the following rose:

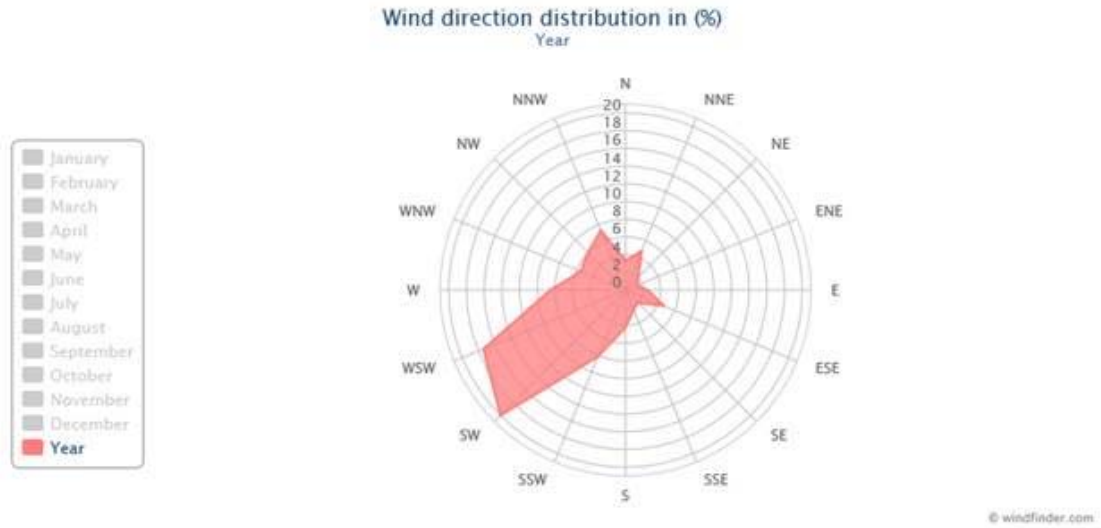


Figure 7-Wind Rose

3.3.2 Waves

Regarding waves height and directions, 2 buoys installed at two points located offshore of the site (Point 1: 34.00N 35.13E & Point 2: 33.75N 35.13E) - HR Wallingford report (results from numerical wave modeling using the Met Office European wave model), (**Figure 8**), show the following values:



Figure 8-Buoys location

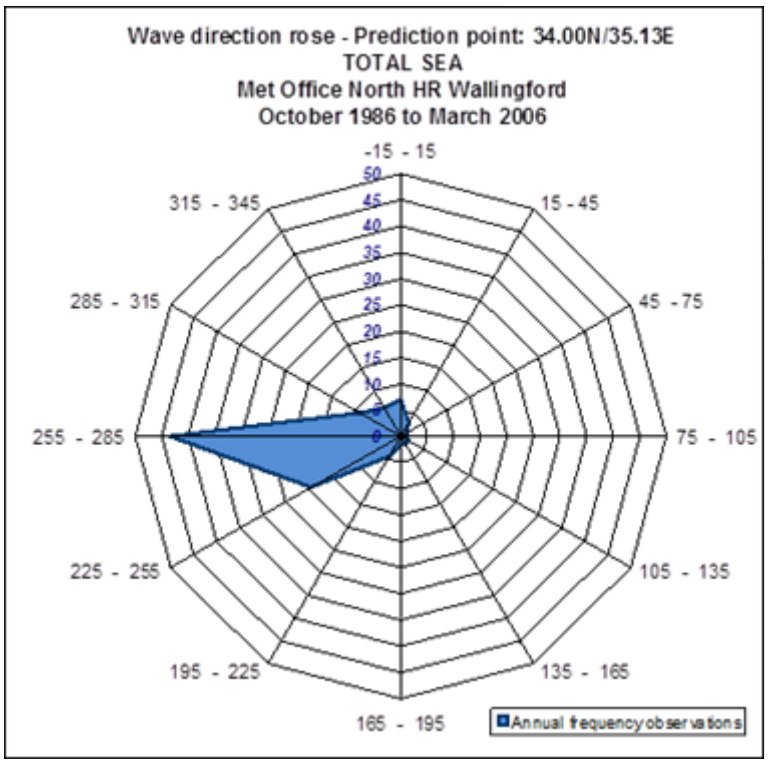


Figure 9- Wave direction Rose – North

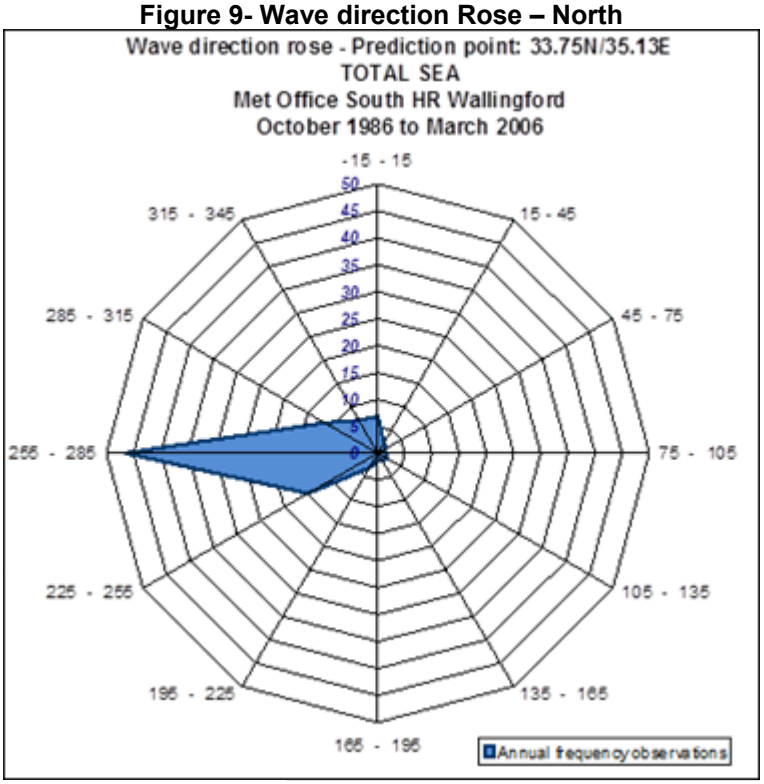


Figure 10-Wave direction Rose – South

Knowing that most of the storms come from the West and the South West, only the statistics for the Point 2 were considered hereafter.

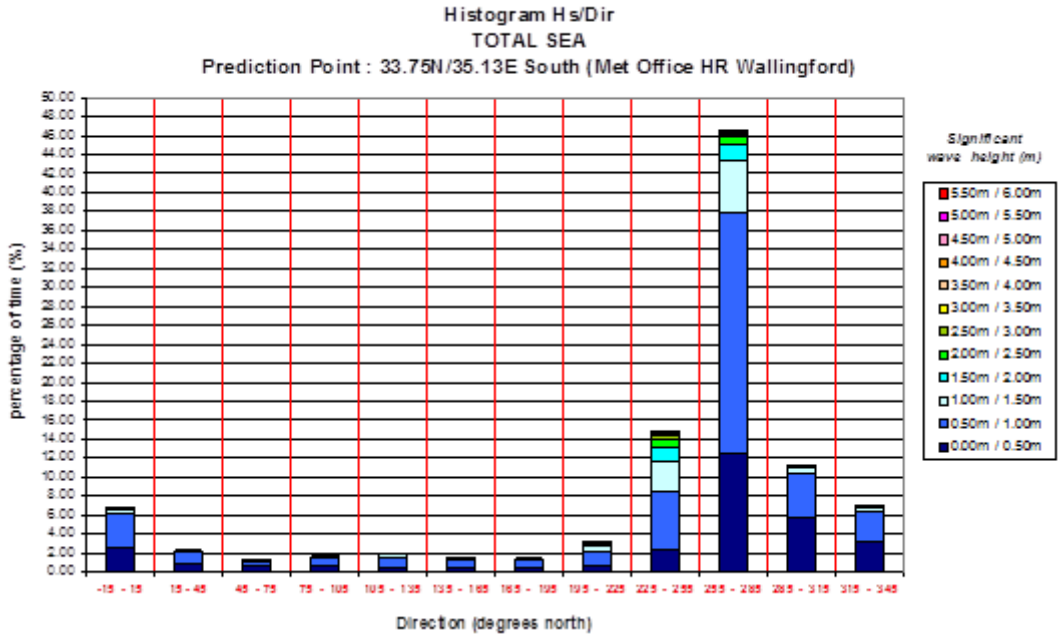


Figure 11- Wave direction Histogram – South

This was further corroborated by data from CNRSL (Prof. Nijad Kabbara, 2005) showing Polar scatter diagram of measured significant wave height H_{m0} vs. mean wave direction Θ_w for Beirut location (wave buoy).

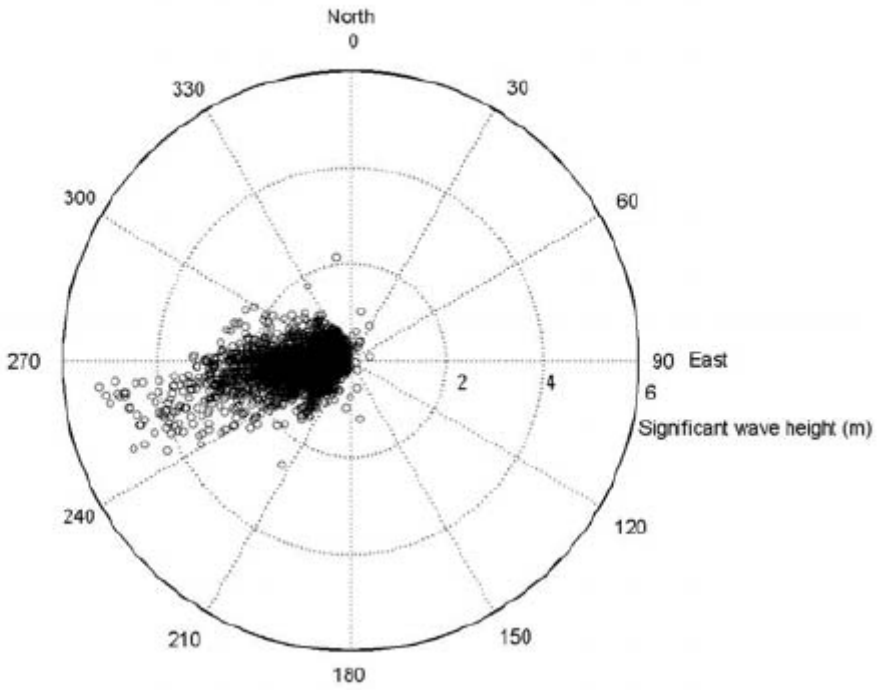


Figure 12- Polar scatter diagram of measured significant wave height Hm0 vs. mean wave direction θw for Beirut

The results of the swells of the project are summarized in the following table:

Table 6- Swells results for the projects

Return period (years)	Total Sea N225° - N255°	
	Hs(m)	Tp (s)
0.1	3.90m	8.7 to 10s
1	5.15m	11.2 to 12.5s
5	6.00m	11.2 to 12.5s
10	6.30m	11.2 to 12.5s
20	6.65m	11.2 to 12.5s
50	7.10m	12.5 to 15s
100	7.40m	12.5 to 15s

Moreover, the Results of Wind and Wave Data Analysis for the Lebanese Coastal Area carried out by the CNSRL (Dr. Nijad Kabbara, 2005) for the years 2002 and 2003 have indicated that the most intense period as regards the wave and wind conditions in the Lebanese coastal area extends from November to March. And that the maximum wave height recorded during this period was 9m and occurred in Beirut (Golf, south of Beirut). The study further indicate that the influence of swell waves is evident in Beirut – further confirming HR Wallingford's report.
In the present design report, the biggest wave has been used, i.e. in the N225˚ – N255˚ direction.

Moreover, since the most penalizing direction to the wave propagation is the one that is perpendicular to the coast, we have retained the N255˚ direction.

3.3.3 Sea Currents

In general, sea currents are created due to:

i. External forces such as winds and storms
ii. Internal forces such as difference in temperature or salinity

Hereafter, **figure 13** showing the sea surface currents in the Mediterranean Sea.

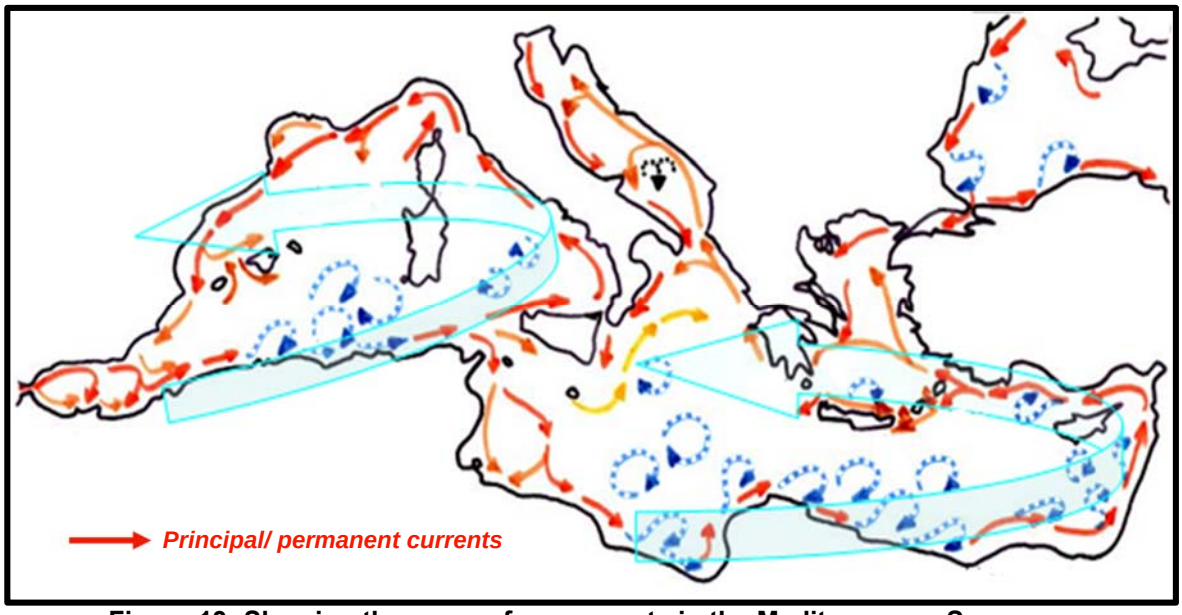


Figure 13- Showing the sea surface currents in the Mediterranean Sea

The principal / permanent surface currents along the Lebanese shore are mainly northern and can vary according to winds and other. All measurements show a velocity varying from a maximum of 0.50 m/s in February to a minimum of 0.05 m/s in August.

These currents are considered weak in comparison to waves generated currents which velocities can reach 2m/s.

3.3.4 Air and Sea Temperature

Beirut is located in the East Mediterranean region. Therefore, it is characterized by a Mediterranean climate: dry and hot summers with little or no rain at all, cool and rainy winter. Average temperature varies between 17◦C and 24◦C. Dominant winds directions are West and South-West. Various forms of data (charts, tables…) shown hereafter, and acquired from the DCA and other sources give an overview of the climate characteristics at Tyr region.

The **Climatic Zoning for Lebanon (Ministry of Public Works, 2005)**, shows temperature variations at Beirut station.

Table 7-Temperature Variations

Temperature in °C		
Station	***Beirut***	
January	*Minimum*	*4.4*
	Average	*15.3*
	Maximum	*19.4*
February	*Minimum*	*11.7*
	Average	*16.5*
	Maximum	*20.6*
March	*Minimum*	*12.2*
	Average	*17.2*
	Maximum	*21.1*
April	*Minimum*	*12.8*
	Average	*21.2*
	Maximum	*29.4*
May	*Minimum*	*17.2*
	Average	*23.0*
	Maximum	*27.2*
June	*Minimum*	*22.8*
	Average	*27.1*
	Maximum	*30.0*
July	*Minimum*	*26.6*
	Average	*28.9*
	Maximum	*31.1*
August	*Minimum*	*26.1*
	Average	*29.7*
	Maximum	*32.8*
September	*Minimum*	*20.6*
	Average	*27.8*
	Maximum	*31.7*
October	*Minimum*	*20.6*
	Average	*25.6*
	Maximum	*28.3*
November	*Minimum*	*10.6*
	Average	*20.8*
	Maximum	*28.9*
December	*Minimum*	*10.6*
	Average	*16.4*
	Maximum	*20.0*



Figure 14- Meteorological stations map



Figure 15-Annual Average Temperatures Map

3.3.5 Salinity

The salinity of the Mediterranean is uniformly high throughout the basin. Surface waters average about 38 parts per thousand except in the extreme western parts, and the salinity can approach 40 parts per thousand in the eastern Mediterranean during the summer, precisely at the Lebanese shore.

3.3.6 Average Humidity

Tabulated average humidity results extracted from the Climatic Zoning for Lebanon (Ministry of Public Works, 2005), shows the following values for Beirut:



Figure 16-Humidity Map of Lebanon

Table 8-Humidity Variations

Average Humidity in %	
Zone	***coastal***
Region	***Center***
Station	***Beirut***
January	*60.1*
February	*65.0*
March	*66.1*
April	*70.6*
May	*72.5*
June	*71.3*
July	*70.6*
August	*74.3*
September	*67.9*
October	*67.1*
November	*66.2*
December	*62.4*

3.3.7 Precipitation

The average monthly precipitation figures are shown on precipitation graph shown here below, after weather-and-climate.com:

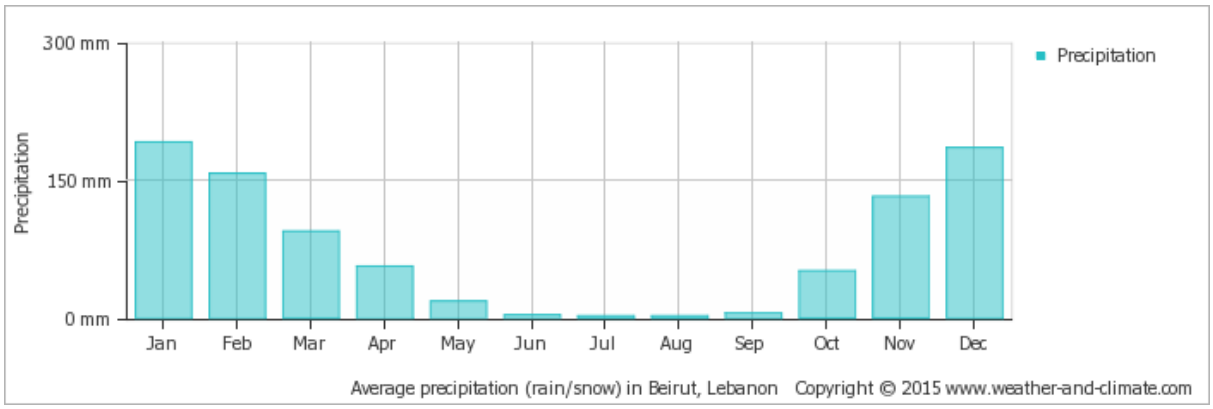


Figure 17-Average precipitation in Beirut

3.4 Effluents Data

3.4.1 Flow rate

The sea outfall studied herein shall be designed to convey the effluents coming from the overflow pipe of Sultan Ibrahim lifting station.
Design flows taken into account are summarized in the table here below:

Table 9-Expected flows

Expected flows during	***Flow rate (m^3/s)***
Peak sewage flow	*1.5*

3.4.2 Effluent Characteristics

According to tender documents, effluents characteristics are as following:

i. Density : 1000Kg/m^3
ii. Viscosity : 1.141*10^{-6} m^2/s
iii. Fecal Coliform Concentration: 10^7 FC/100ml

4. Hydraulic Design

4.1 Introduction of the Internal Hydraulic Design

Submerged multiport diffusers for waste water outfalls are designed often, considering steady flow conditions for far future scenarios. Design aims for gravity operation, lower costs for material use and the minimization of environmental impacts. Inadequate attention on the internal diffuser hydraulics also for off design conditions thereby often result in hydraulic problems like partial blockage, high head losses, uneven flow distribution, salt water intrusion and poor dilution causing higher energy demands and stronger environmental impacts.
Detailed calculations for the internal manifold hydraulics in the outfall pipes show a strong sensitivity on the representation and formulation of local losses even for relatively simple riser/port configurations. An optimization methodology yields a homogeneous discharge distribution along the diffuser, minimization of the total head and prevention of sedimentation or ambient water intrusion in the diffuser under varying inflow and ambient conditions. The final design achieves lower costs for material use and operation as well as the minimization of environmental impacts and operational stability for off-design conditions.

4.1.1 The Outfall under study and its Diffuser

The Sultan Ibrahim sea outfall comprises the:

- Construction of an onshore ND900mm GRP pipeline section of around135m length starting at the overflow of the Sultan Ibrahim lifting station and ending at the beginning of the existing twin-cell concrete storm water culvert.

- Construction of an intermediate onshore “link” section consisting of an onshore OD800mm HDPE pipeline of around 120m length running inside the northern cell of the existing twin-cell concrete storm water culvert until the shoreline pressurized connection manhole.

- Construction of an offshore OD1400mm HDPE pipeline section of around920m length starting at the shoreline pressurized connection manhole and terminated by a 192m long OD1400mm HDPE diffuser.

Moreover, the sea outfall pipeline shall be designed as High Density Polyethylene (HDPE) pipe of minimum pressure class PN 6, and the pipeline will be laid in trench in the sea bed in depths up to 13m. Furthermore, the design of the outfall shall be based on a peak water flow of 1.5m³/s and flow has to be by gravity.
It is generally considered that the port velocity should be at least 0.7 m/s, but not higher than 2 m/s for average flow. A velocity till 2.5 m/s can be considered as acceptable only for peak flow. At the maximum flow rate, the discharge flow volume from each individual diffuser port (or deviation) shall be within 10 % of the mean value for all ports.

The sea water outfall is designed to accommodate the peak sewage flow (1.5m^3/s)
In order to accommodate a wide range of flow variations without hindering the functioning of the outfall, all the diffuser ports will be equipped with duckbill type valves. In comparison with fixed (constant or invariable) port diameters the effective open area of variable area orifices (duckbill valves) changes with different discharges. Therefore, they are good for no or low discharge scenarios, where intrusion has

to be prevented. Decreasing fixed port diameter leads to a more homogeneous discharge distribution but to increased losses and total head due to higher velocities. Duckbill valves give almost homogeneous discharge profiles, due to the discharge dependent open area.

4.1.2 Verification of the Initial Dilution Criterion

Although MedPol directives call for an initial dilution of 1:100 and the Standard Practice Document (SPD11: Sea Outfalls) (publication of the Republic of Lebanon, CDR) calls in its Appendix F for an initial dilution of 1:150, the Sultan Ibrahim Outfall Project's targets an initial dilution of 1:50, adopted for two recently executed outfalls by CDR (Sour WWTP sea outfall and Manara sea outfall).

The verification of this criterion is made for the outfall at hand, under the average water level elevation at discharge location as value for the ambient water level elevation H_d =20m for the initial hypothesis of a 708.6m long outfall terminating at a depth of 15m below mean sea level, using the following formula, after - I.G. Wallis – 1977 – Water Pollution Control, Initial Dilution with Deep water Diffusers Federation:

$$S_{in} = \left(g\frac{\rho_{sea}-\rho_{eff}}{\rho_{sea}}\right)^{\frac{1}{3}}\left(\frac{D}{4}\right)\left(\frac{Q}{L}\right)^{-2/3}$$

Thus, for Qmax = 1.5m^3/s, we need to select a 200m long diffuser length to get:

50 <S_{in} = 82< 100 < 150

Thus only the 1:50 dilution criterion is met.

Should we need to meet the more stringent Standard Practice Document (SPD11: Sea Outfalls) criterion of 1:150, the sea outfall pipeline will have to be extended by some 500m. In this latter case, we get:

S_{in} = 157 > 100

Accordingly we shall base our study on this 1:50 option.

4.1.3 The CorHyd Software

CorHyd is a computer code for the calculation of flow characteristics in multiport diffuser constructions. The CorHyd computer program has been developed for the calculation of velocities, pressures, head losses and flow rates inside the diffuser pipe and, especially, at the diffuser port orifices to analyze and optimize diffuser design alternatives as well as existing diffuser configurations for different and varying discharge and ambient conditions. The calculation is based on the application of the steady continuity and work-energy equations between ambient fluid at the discharge points and the effluent inside the diffuser pipe. It implements as well all occurring losses especially when high risers, duckbill valves, multiple ports and more complex discharge configurations are applied. It includes loss calculations for complex geometries, as well as additional flow forcing due to density differences.

4.1.4 Modus Operandi of Outfalls

Waste water treatment plants commonly discharge treated effluents through outfalls into rivers or coastal waters. These plants are designed to minimize environmental impacts by reducing the pollutant concentrations of the effluent. Nevertheless, even discharges of state-of-the-art treatment plants may cause local pollution of the receiving waters, if the effluent contains persistent substances and especially if discharge flow rates are high, which is the case for most large metropolitan areas. To prevent local pollution and to protect ecologically sensitive regions, persistent substances have to be reduced directly at the source and the large discharges have to be distributed over a wider area. For the latter purpose, long outfall pipes with multiport diffuser installations are used to disperse the effluent to non-critical levels, aided by the natural pollution degradation rates of the receiving water bodies.

Also risers may carry duckbill valves, which change their effective open port area related to the pressure difference between inside and outside the valve. They avoid salt water intrusion during low flow periods and allow high discharges during peak flow periods
The flow in multiport diffusers is controlled by two boundary conditions: first, the entrance boundary (flow rate or head), and, second, the ambient/disposal boundary, where the effluent physical properties differ from the ambient fluid. Both conditions vary in time due to discharge variations (diurnal changes, storm water events and long-term changes due to increased sanitation coverage) and pressure variations, density variations, tides or waves.
An outfall design must consider both, the hydraulics occurring outside and inside a diffuser. External hydraulics affect the effluent mixing with the ambient fluid, internal hydraulics affect the flow partitioning and related pressure losses in the manifold resulting in a discharge profile along the diffuser. CorHyd covers the internal diffuser hydraulics with the following design objectives:

i. *Uniform discharge distribution along the diffuser* in order to meet dilution requirements and to prevent operational problems (e.g. intrusion of ambient water through ports with low flow). Exceptions should avoid near-shore impacts by keeping the seaward discharge higher
ii. *Minimized constructional and operational costs* using simple manifold geometries with small losses
iii. *Prevention of off-design operational problems*in order to avoid particle deposition and salt water intrusion during low flow or no-flow periods

CorHyd calculates velocities, pressures, head losses and flow rates inside the diffuser pipe and especially at the diffuser port orifices. It is used to analyze, predict and monitor the discharge behavior of the planned diffuser under different boundary conditions. The combination with CORMIX will provide a direct linkage to subsequent waste plume modeling and mixing zone analysis.

4.1.5 Manifold processes

Pipe hydraulics are characterized by continuous pressure losses due to wall friction and by local pressure losses due to geometrical changes. Manifold hydraulics (i.e. diffusers) are characterized by several flow separations, where local losses depend not only on geometrical relations but furthermore on the discharge rates. The flow distribution for simple pipe configurations with uniform geometries along the diffuser depends mainly on the ratio of branching losses and manifold losses. But most of the actual diffuser geometries have more complex geometries. Diffusers often discharge fluids with higher or lower density than the receiving waters, which cause an additional buoyant forcing on the fluid flow.
Implemented losses in CorHyd include continuous losses due to friction in all pipes (feeder, diffuser, riser, and port). Local losses are considered automatically in all pipe sections, the feeder pipe, the diffuser manifold and the attached port-riser branches.
Furthermore additional local losses may be added manually if necessary:

i. **Local Feeder losses**
- Inlet loss at headwork
- Horizontal and vertical bends
- Contractions/expansions along the feeder pipe
- Flow separation, if several diffusers are mounted on one feeder

ii. **Diffuser manifold losses**

Implemented local losses along a streamline along the diffuser pipe centerline passing the branch pipes are:

- The division of flow for the diffuser pipe passing a riser
- Horizontal or vertical bends
- Contractions / expansions along the diffuser pipe

iii. Port-riser branch losses

Implemented local losses along a streamline going from a diffuser centerline into the riser, then into the port and the discharging jet are:

- The division of flow from the diffuser pipe into a riser
- Optional: bends or additional losses in the riser
- The transition or division of flow from riser to port(s)
- Optional: additional losses in the port or at the orifice
- Optional: contraction of jet
- Optional: duckbill valves at the port orifices

4.1.6 Local loss formulations

Local losses are due to geometrical differences between one cross-sectional area of a pipe and the adjacent one (i.e. expansions, contractions, or bends) or the inlet or end of a pipe (orifice).
These changes may lead to flow detachment processes, reverse currents in dead zones, locally increased accelerations or decelerations, increased turbulence, which all cause energy losses, in closed pipe systems compensated by pressure losses.

i. Friction losses

Continuous pressure losses due to friction along the walls or boundary layers in a pipeline are calculated.

ii. Major assumptions

a. Steady flow
CorHyd assumes slowly and uniformly changing boundary parameters

b. Single phase pressure pipe
CorHyd assumes the whole pipeline as flowing full under all conditions and especially at the minimum flow rate and minimum tide. It is assumed that air entrance at the inlet (shore crossing chamber) is avoided by keeping the top pipe invert under the minimum sea level. Stratified flows due to intruded salt water cannot be analyzed in CorHyd.

iii. Geometrical assumptions

CorHyd considers round pipes. The angle between riser and diffuser axis is assumed to be nearly 90°.
The formulations for local losses applied in CorHyd assume reasonable high Reynolds numbers (above 104) and reasonable geometrical distance (above 3 times the diameter) between geometrical changes to avoid interaction of losses.
CorHyd furthermore allows applying Duckbill valves also on simple diffuser systems.

4.1.7 Design Criteria

For the outfall at hand, the calculation is done using the average water level elevation at discharge location as value for the ambient water level elevation H_d =20m (average at discharge) for the scheme described above (i.e. 708.6m long outfall completed by a 200m long diffuser).

From the data made available to us (after Touma Engineering commissioned by RELK & P for CDR PROJECT 1345 - SOUR COASTAL AREA WASTEWATER PROJECTCONTRACT 2211 – CONSTRUCTION OF SEA OUTFALL - TENDER DOCUMENTS -Volume 3 - Appendix B - Marine Survey Report), we have selected for the sea-water density, ρ_0 = 1026kg/m^3
Effluent density ρ_e and viscosity ν generally do not change significantly. Often used values for municipal waste water is ρ_e = 996-1000 kg/m^3 and ν = 1.141x10^{-6} m²/s (Tripoli WWTP sea outfall). The selected value herein is 1000 kg/m^3.
The design flow rate Q_d was taken as the peak sewage flow rate, 1.5m^3/s
The total energy head (energy in dimensions of length) is usually computed as the sum of the pipe elevation + pressure head + velocity head + losses. Thus, the minimum and maximum velocity head values are included in the minimum and maximum sums, respectively.
CorHyd output shows both the relative total head and the absolute total head. The calculated relative total head is the head required above the ambient water level H_d.
The calculated head is the amount of head that is required at the headwork to operate the system. If the headwork already provide this head, then no additional pumping is necessary under normal weather conditions.
As mentioned in paragraph 4.1.1. above, the outfall has been designed for the maximum total head available at the pressurized seashore connection manhole: 2.04m coming from:

Table 10-Headlosses in onshore sections

	Friction losses in on shore part of seaoutfall (Normal)								
cu. m/	cu. m/	gpm	l/sec	length	dia	velocity	loss	cof.	total Fric.
day	hr			m	mm	m/sec	%	c	loss m
129,600	5400	23809.52	1500.00	129.87	900	2.36	0.40	140	0.52
129,600	5400	23809.52	1500.00	120	738.46	3.50	1.06	140	1.27
									1.79
Population served=	720,000								

Table 11-Available head for the offshore section

Net available head at the lift station overflow	3.83m
Headloss in the On Shore section of the Outfall	-1.79m
Maximum total head available at the connection manhole	**2.04m**

Total head: The necessary total Head or the final flow is in the desired order of magnitude 2.04m during normal operation. However, when adverse conditions prevail, we need to add to this figure 2.5m for swell (waves) and then multiply by a factor of safety equal to 1.25 i.e. H_{max} = 5.65m vs. 2.04m net available head at the pressurized chamber at the shoreline. Thus pumping will be needed in such case.

4.1.8 Pumping needs during adverse conditions

The power to be delivered by the pump can be computed using the following formula:
P = (Q x H_{net} x 3600) / (367 x R)
Where R is the efficiency of the pumps taken equal to 80%
In our case, the net required head is equal to 5.65 – 2.04 # 3.65m
Hence the required power is on the order of 70kW

4.1.9 Feeder and Diffuser

The main outfall pipe consists of the feeder pipe, which conveys the effluent to the discharge location and the diffuser pipe, which disperses the effluent in the ambient water body. The input of both, feeder

and diffuser pipe sections is done via the start and end point coordinates xs, ys, zs, the diameter Dd and the roughness ks,d.
To reduce the input parameters the pipeline is schematized with pipe sections. The number of used sections is Nd.
Section limits are locations where either bends or diameter changes or roughness changes occur (cf. Coordinates table at the beginning of this report). The fittings can be characterized by the radius R (typical R = 3Dd) of a bend if bends between sections occur.
As much sections as necessary to represent the general position of the pipeline are used. The sections can be chosen independently of the port/riser configurations. The feeder diameter design is constraint by a maximum diameter to allow scouring of sediments during low flow periods. The near future design discharge Qnf (daily maximum) should therefore result in feeder velocities $v_{f,nf} > 0.5 m/s$
(DIN EN 1671, ATV-DVWK-A 110 (2001) and ATV-DVWK-A 116 (2005)). However to ensure that the diffuser is cleared from sediments, this limit is raised herein to 0.7 m/s in accordance with recommendations issued by operators worldwide.
This corresponds to a maximum feeder pipe diameter of

$$D_{d,max} = 1.35 \text{ x } Q\text{^}0.5 = 1.65 m$$

Here, a 1.4m diameter feeder pipe is selected (OD1400 SDR26).
The feeder velocity for the far future design flow rate Qff and the same diameter results then in

$$v_{f,ff} = {v_{f,nf} \times Q_{ff}}/{Q_{nf}}$$

Generally flow rates do not more than double or triple during the lifetime of an outfall. CorHyd output in our case has shown that on the long run (far field) feeder velocities should range between 1 to 2m/s, what is clearly acceptable in terms of operational viewpoints considering the related energy losses and in accordance with the project's technical specifications.
In the same fashion as the feeder, the diffuser pipe is constraint by a maximum scouring diameter. Theoretically this would result in a diffuser with different pipe segments as much as risers, and, under the assumption of a homogeneous discharge distribution, each with a maximal diameter of

$$D_{d,max,i} = \left({8Q_{nf} \times i}/{N \times \pi}\right)^{0.5}$$

where N is the total number of risers
iis the observed pipe section before the ith riser starting counting offshore.
Although best for sediment removal this solution, also called tapering generally is too expensive to install (about 20% more expensive than single diameter diffuser) and maintain (i.e. cleaning).
Thus, in our case the diameter of the diffuser was maintained constant at 1.4m.
Then from the diffusion study (CORMIX data from the first successful mixing calculations) that has been carried out simultaneously for the Sultan Ibrahim Sea Outfall, a number of risers N = 20 was found to be adequate to meet the initial dilution criteria.
A design rule that is often mentioned in literature (Grace 1978), recommends to keep the ratio between the cumulative port areas $\sum_{K=1}^{N} A_{p,k}$ downstream a diffuser pipe cross section area Ad,N smaller than one, with the explication that "it is impossible to make a diffuser flow full if the aggregate jet area exceeds the pipe cross-section area, since that would mean that the average velocity of discharge would have to be less than the velocity of flow in the pipe" (Fischer et al. 1979, p.419). A further suggestion taken from Fischer et al. (1979, p.419) resumes that the best ratio is usually between 1/3 and 2/3as:

$$1/3 < \sum_{k=1}^{i} (A_{p,k} | A_{d,i}) < 2/3$$

However, applying this rule leads to a port diameter of 200mm, which in turn leads to a considerable increase in the total head needed for gravity operations (more than 1 additional meters). Thus this rule of thumb was abandoned and the Dd,max,i formula was retained.
This choice led to confirm the initially selected port diameter equal to

$$D_{d,max,i} = \left(8Q_{nf} \times i / N \times \pi\right)^{0.5} = 0.23m$$

Accordingly a 280mm diameter pipe was first selected - OD 280mm (SDR 26). This diameter is larger than the minimum port size of 70 to 100 mm for primary treatment plants (just screening and settling tank). This gives a total area in accordance with Fischer et al. statement.
Regarding the riser diameter Dr, it has to allow for riser velocities, which are bigger than the diffuser velocities, but smaller than the port velocities to allow for constant flow acceleration. Accordingly, the same diameter of OD 280mm (SDR 26) has been selected in our case.

4.1.10 Sea Water Intrusion

Under low-discharge conditions, diffuser is confronted especially with issues of scouring and/or intrusion of seawater. Seawater intrusion can seldom be avoided for all discharges. Duckbill valves and small diameter pipes prevent those problems, but lead to additional pumping costs or higher headwork storage buildings.
Intrusion can be prevented if the port densimetric Froude number is bigger than 1:

$$F_p = V_p \Big/ \left(\Delta\rho / \rho g D_p\right)^{0.5} > 1 \quad \text{(Wilkinson, 1988)}$$

where Vp denotes the port exit velocity
and Dp the port diameter,

Resulting in a critical port velocity $V_{p,crit} = \left(\Delta\rho / \rho g D_p\right)^{0.5}$

4.1.11 CorHyd Output

All feeder/diffuser/riser/ port/ flow data above have been plugged into the CorHyd software.
The output, presented here below, shows the following:
Pipe velocities
Diffuser, port and jet velocities $V_d < V_p < V_j$ are in between reasonable ranges. The following operable velocities were obtained:

i. $0.5m/s < 0.53m/s < V_d < 1.16m/s < 5m/s$ for all flow increments above or equal to 0.7m3/s,
ii. $0.5m/s < 1.1m/s < V_j < 1.9m/s < 12m/s$ and higher than Vp for all flow increments above or equal to 0.7m3/s,
iii. $0.5m/s < 0.55m/s < V_p < 1.5m/s < 2.5m/s$ for all flow increments above or equal to 0.7m3/s,

Total head: The necessary total Head or the final flow is in the desired order of magnitude 2.04m vs. 2.04m net available head at the pressurized manhole (Point A).

4.1.12 Flow Distribution

The deviation plots show, for all flow magnitudes, that the flow distribution lies in between reasonable limits $q_{min} = {}^{-0.1 \times q_i}/_N < q_i < {}^{0.1 \times q_i}/_N = q_{max}$ for all port/riser configurations for 0.8m3/s flows and above.
Furthermore, based on the defined and verified diffuser configuration and dimensions, external hydraulics with modified diffuser had been checked and the set dilution criterion was verified.

4.1.13 Pipe's Characteristics

According to DIN 8074/8075 and ISO 4427/1167, the HDPE pipe is OD1400mm PE80 SDR26, PN6.3, leading to ID1292.3mm.
The HDPE Diffuser is OD1400mm PE100 SDR26, PN6.3, leading to ID1292.3mm.
The risers and ports are OD280mm PE100 SDR26, PN6.3, leading to ID258.5mm.
The proposed Duckbill valves are 10inches type MZ_1916WB - WideBill (5 psi).

4.1.14 CORHYD outputs

Summary of Results of Internal Diffuser Hydraulics simulation

CorHyd Version 5.0.2.0
Simulation run on: 20-Sep-2018

Filename:
C:\Users\Administrator\Desktop\sultan\Sultan_1400mm_24R_250mm_10inDB5b_rev1_deeper.xml
Design Case: Proposed
Site Name: Sultan Ibrahim
Prepared By: K&A
Project Date: 25-Apr-2018

<u>INPUT ambient data:</u>

Water level above datum (z = 0m) (Hd) [m] 20.00
Ambient density profile: Uniform
Ambient density (rho_am) [kg/m³] 1026.00

<u>INPUT effluent data:</u>

Density of effluent (rho_e) [kg/m³] 1000.00
Kinematic viscosity (ny) [m²/s] 1.1410E-006
Flowrates of effluent (Qd) [m³/s]
0.50 0.60 0.70 0.80 0.90 1.00 1.10 1.20 1.30 1.40 1.50

INPUT Outfall Sections:

Section Num	Length	Slope	x	y	z	Dd	Ksd [m]	R0	beta	bend[°]	risers	#'s
end	-	-	-340195.97	-31561.77	-21.21							
1	200.07	0.02	-339995.96	-31561.77	-16.39	1.292	1.00E-004	0.000	0.00	0.21	20	1-20
2	68.61	0.02	-339927.36	-31561.77	-14.99	1.292	1.00E-004	10.000	0.00	0.00	Feeder	
3	260.05	0.02	-339667.36	-31561.77	-9.67	1.292	1.00E-004	10.000	0.00	0.31	Feeder	
4	93.25	0.01	-339574.12	-31561.77	-8.28	1.292	1.00E-004	10.000	0.00	0.00	Feeder	
5	286.79	0.01	-339287.36	-31561.77	-4.00	1.292	1.00E-004				Feeder	

INPUT Risers:

Riser Num	x [m]	y [m]	z [m]	Lr [m]	TLr [m]	Dr [m]	Ksd_r [m]	ports	Lp [m]	Dp [m]	Ksd_p [m]	Orifice
1	-340194.2	-31561.8	-21.2	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
2	-340184.2	-31561.8	-20.9	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
3	-340174.2	-31561.8	-20.7	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
4	-340164.2	-31561.8	-20.4	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
5	-340154.2	-31561.8	-20.2	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
6	-340144.2	-31561.8	-20.0	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
7	-340134.2	-31561.8	-19.7	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
8	-340124.2	-31561.8	-19.5	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
9	-340114.2	-31561.8	-19.2	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
10	-340104.2	-31561.8	-19.0	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
11	-340094.2	-31561.8	-18.8	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
12	-340084.2	-31561.8	-18.5	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
13	-340074.2	-31561.8	-18.3	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
14	-340064.2	-31561.8	-18.0	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
15	-340054.2	-31561.8	-17.8	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
16	-340044.2	-31561.8	-17.6	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
17	-340034.2	-31561.8	-17.3	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
18	-340024.2	-31561.8	-17.1	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
19	-340014.2	-31561.8	-16.8	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB
20	-340004.2	-31561.8	-16.6	1.250	0.604	0.258	1.00E-004	1	1.000	0.258	1.00E-004	Duckbill MZ_1916WB

INPUT Port/Risers:

(x,y,z) Riser location
x0-x Riser location starting from diffuser end
Dp Port diameter
Dd Diffuser diameter

Riser #	x [m]	y [m]	z [m]	x0-x[m]	Dp [m]	Dd [m]
1	-340194.2	-31561.8	-21.2	0.0	0.258	1.292
2	-340184.2	-31561.8	-20.9	-10.0	0.258	1.292
3	-340174.2	-31561.8	-20.7	-20.0	0.258	1.292
4	-340164.2	-31561.8	-20.4	-30.0	0.258	1.292
5	-340154.2	-31561.8	-20.2	-40.0	0.258	1.292
6	-340144.2	-31561.8	-20.0	-50.0	0.258	1.292
7	-340134.2	-31561.8	-19.7	-60.0	0.258	1.292
8	-340124.2	-31561.8	-19.5	-70.0	0.258	1.292
9	-340114.2	-31561.8	-19.2	-80.0	0.258	1.292
10	-340104.2	-31561.8	-19.0	-90.0	0.258	1.292
11	-340094.2	-31561.8	-18.8	-100.0	0.258	1.292
12	-340084.2	-31561.8	-18.5	-110.0	0.258	1.292
13	-340074.2	-31561.8	-18.3	-120.0	0.258	1.292
14	-340064.2	-31561.8	-18.0	-130.0	0.258	1.292

15	-340054.2	-31561.8	-17.8	-140.0	0.258	1.292
16	-340044.2	-31561.8	-17.6	-150.0	0.258	1.292
17	-340034.2	-31561.8	-17.3	-160.0	0.258	1.292
18	-340024.2	-31561.8	-17.1	-170.0	0.258	1.292
19	-340014.2	-31561.8	-16.8	-179.9	0.258	1.292
20	-340004.2	-31561.8	-16.6	-189.9	0.258	1.292

BATCH Discharges and Heads:

Flow [m³/s]	Absolute Total Head [m]	Water Level Elevation [m]	Mean Jet Velocity [m/s]	Mean Effective Port Diameter D0 [m]
0.500	21.188	20.000	1.108	0.168
0.600	21.248	20.000	1.218	0.176
0.700	21.314	20.000	1.318	0.183
0.800	21.386	20.000	1.411	0.190
0.900	21.463	20.000	1.498	0.195
1.000	21.546	20.000	1.581	0.201
1.100	21.635	20.000	1.659	0.205
1.200	21.730	20.000	1.734	0.210
1.300	21.830	20.000	1.806	0.214
1.400	21.936	20.000	1.876	0.218
1.500	22.048	20.000	1.943	0.222

END OF RESULTS

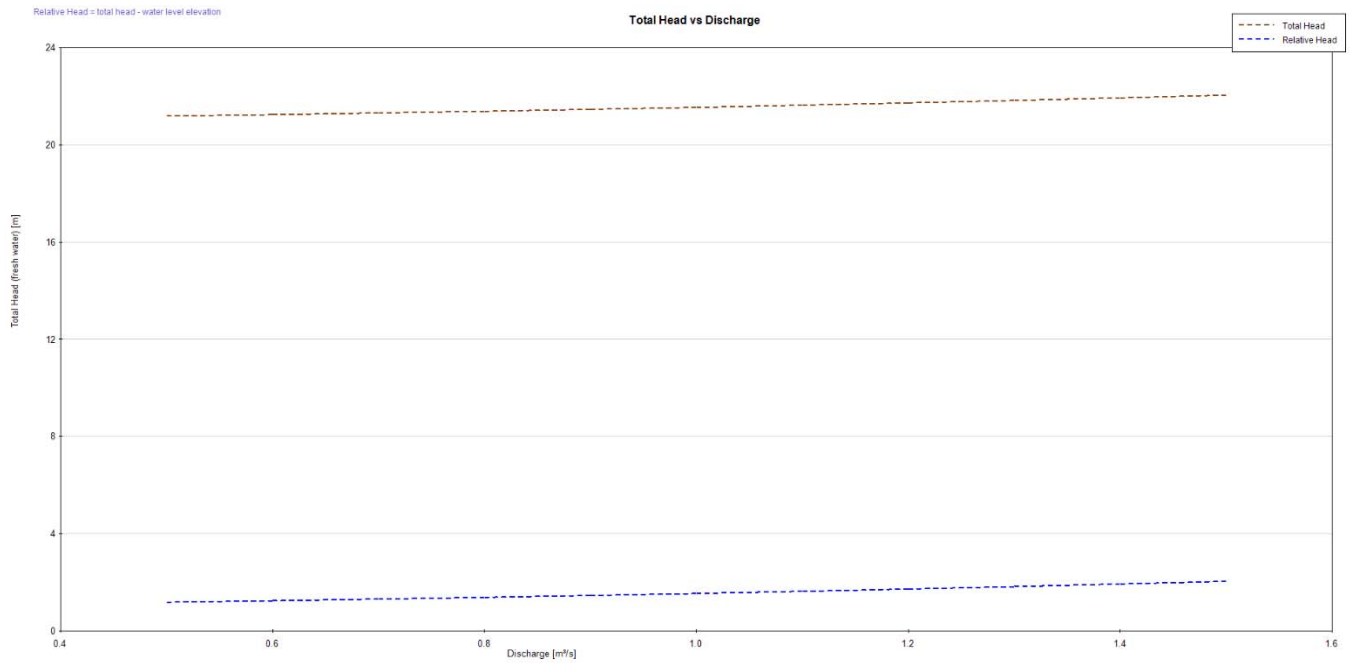


Figure 18-Total Head vs. Discharge

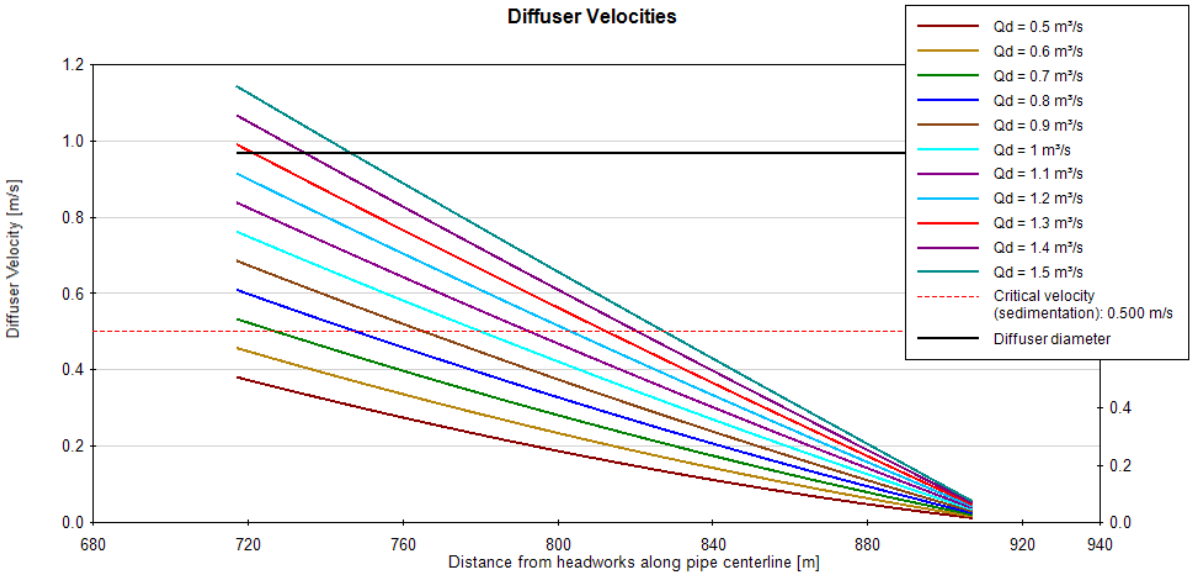


Figure 19-Diffuser velocities

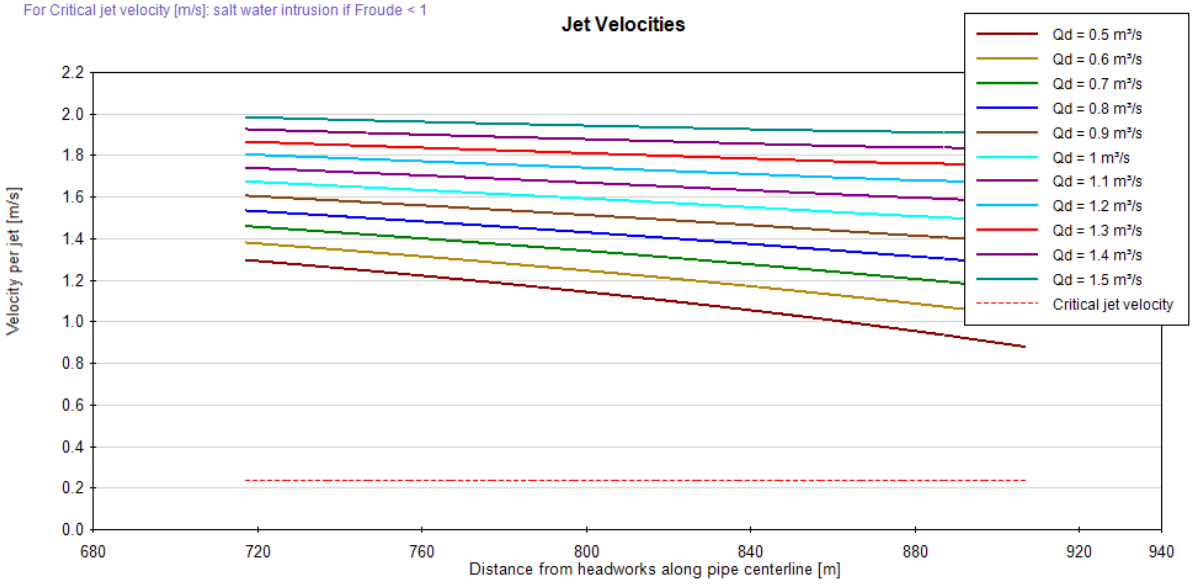


Figure 20-Jet velocities

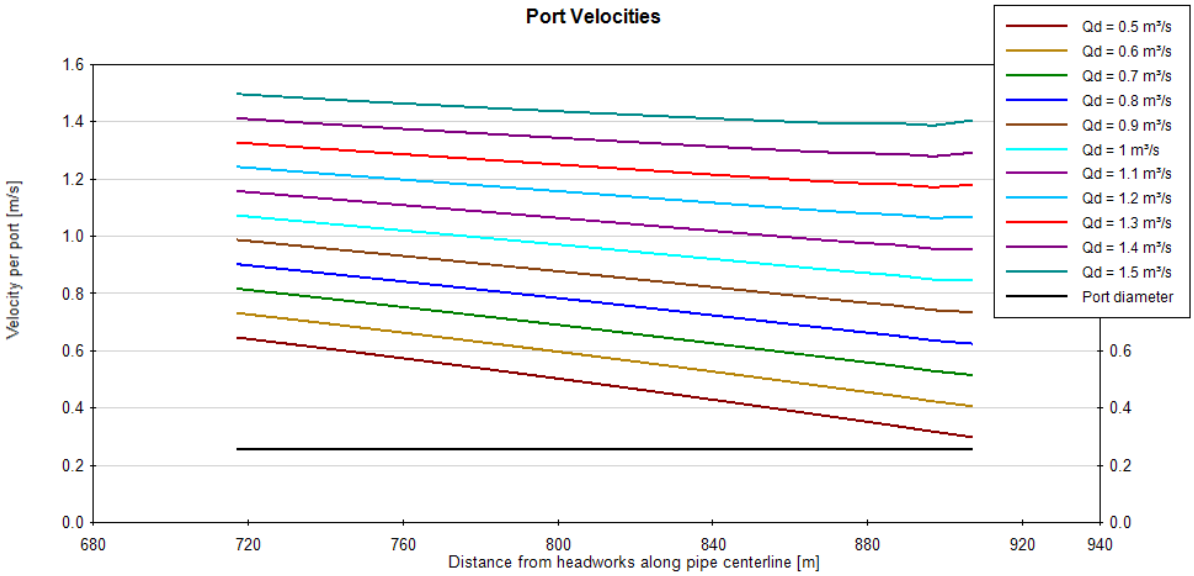


Figure 21-Port velocities

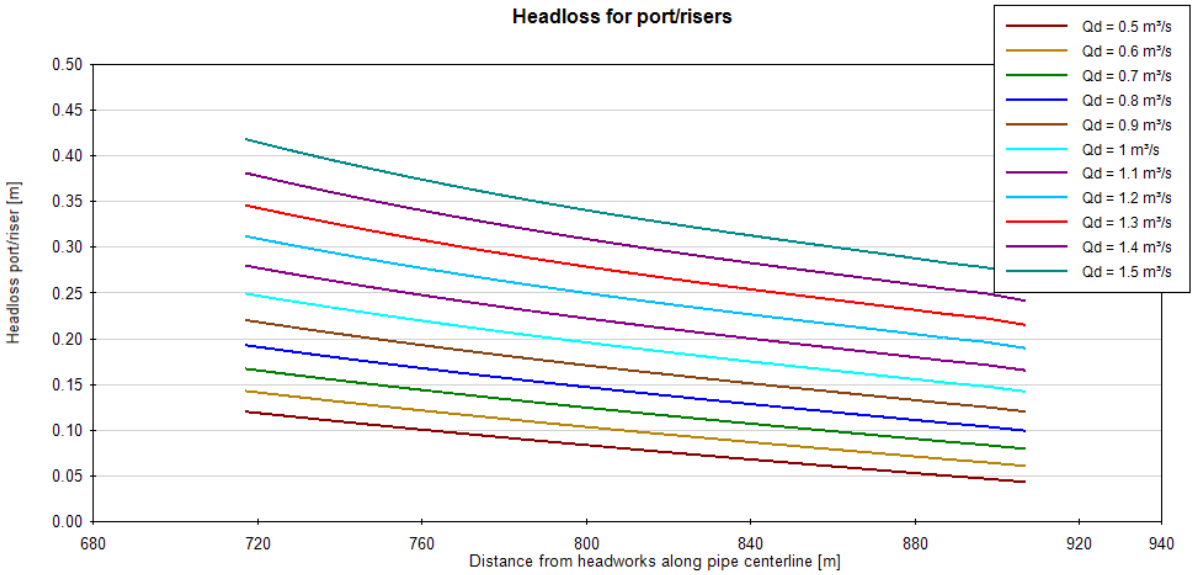


Figure 22- Head loss for port/risers

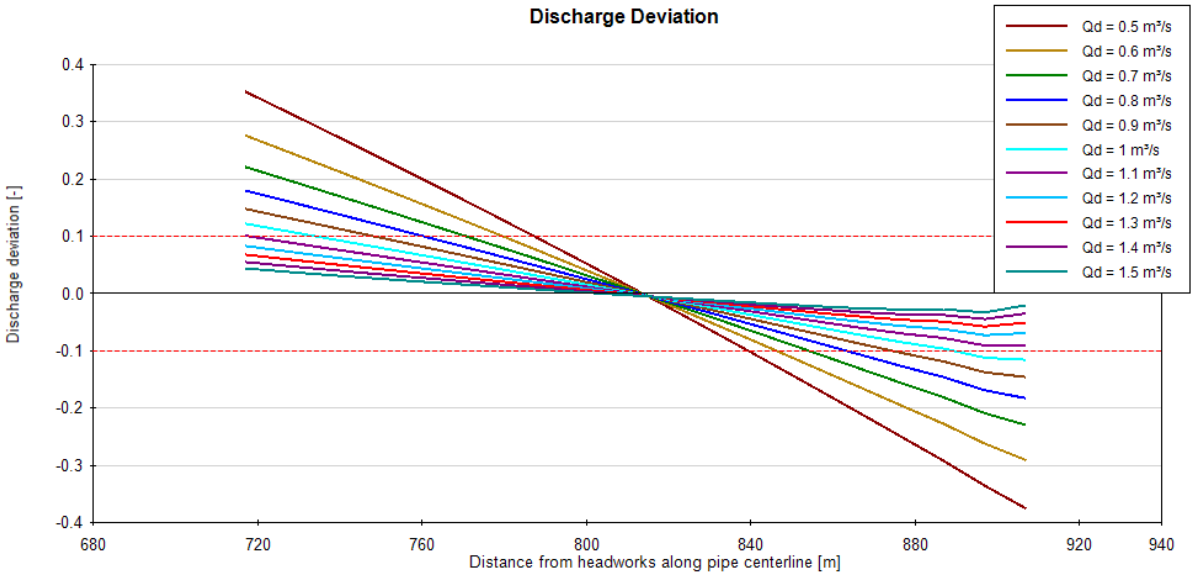


Figure 23-Discharge deviation

5. MARINE DESIGN

In order to relate permissible stresses or strains to the probability of the loading and the risks involved, these rules define two loading conditions and two design conditions.

5.1 Loading Conditions

Any part of the pipeline system is to be designed for the most unfavorable of the following loading conditions:

i. Functional loads
ii. Design environmental loads and simultaneously acting functional loads

Functional loads and design environmental loads are defined as follows:

i. Wave loads
ii. Current loads

For each of the above loading conditions and for each member or cross section to be considered, the most unfavorable relevant combination, position and direction of forces which may act simultaneously are to be used in the analysis.
All direction of wind, waves and current are to be assumed equally probable, unless statistics show clearly that wind, waves and current of the prescribed probability are different for different directions.

5.2 Design Conditions

Both loading conditions defined in the above paragraph 5.1 are to be considered for all different conditions or phases relevant to the pipeline or riser in question.
With respect to levels of permissible stresses (strains) any situation or phase is to be referred to one of the two following design conditions:

i. Pipeline system during installation
ii. Pipeline system during operation

The following basic conditions should be considered during the on-bottom stability design of submarine pipelines:

i. Environmental conditions
ii. Geotechnical conditions of seabed
iii. Topographical conditions of the seabed (e.g. slope, rocky outcrops, depressions)
iv. Bathymetric (water depth)
v. Pipe data (diameter, wall thickness, concrete coating)
vi. Location pipeline restraints (riser connections, crossings, etc.)

5.3 Return Period

The stability design is to be based on a given return period of near-bottom environmental conditions acting perpendicular to the pipe. In general, both near-bottom wave induced particle velocities and near-bottom currents will need to be considered.

The hydrodynamic stability of the outfall will be checked by using 50 years return period wave height (equal to 7.1m as stated in paragraph 3.2.3 above).

5.4 Environmental Conditions

The following environmental conditions should be evaluated at a number of positions along the length of the pipeline:

i. Waves
ii. Currents

The numbers of positions necessary to adequately define the environment are dependent upon the length of the pipeline and the variations in water depth, seabed soil and meteorological conditions. Since sufficient data on the particular area is not available, the environmental conditions used in the stability design have been based on reasonably conservative estimates based on data from other nearby locations.

Recognized methods of statistical analysis have been used to describe the random nature of the environmental conditions. Sea states are defined in terms of the significant wave height (Hs), spectral peak period (Tp) and corresponding return probability.

The form in which the wave information is available, is dependent on the amount and quality of data available for the particular location in question. This may range from a joint distribution of Hs and Tp with directional information to an omnidirectional design value for Hs with an estimated period. The design method presented in section 5.5 accepts wave input of varying degrees of sophistication.

The peak period (Tp) depends on fetch and depth limitations as well as duration of the sea states. In the absence of other information for the peak period, the following relationship is used for the upper limit:

$$T_p = \sqrt{250 H_s / g}$$

When a joint distribution of Hs and Tp is available, then the combination of Hs - Tp which gives the most extreme near-bottom conditions is selected.

The directional distribution of the current velocity may be used in the stability design.

5.5 Waves Propagation

When waves enter the shallow water from deep water, they are influenced by the bottom causing their transformation. The different processes causing wave transformation are:

-refraction,

- shoaling,

- diffraction,

- reflection,

- dissipation of energy due to bottom friction,

- percolation, and

- breaking.

While some of these processes can be easily accounted theoretically, others are not amenable to easy mathematical analysis.

5.5.1 Refraction

According to the elementary progressive (Airy) wave theory, the phase velocity C is given by the equation

$$C = \left(\left({}^{gL}/_{2\pi} \right) \times \tanh {}^{2\pi d}/_{L} \right)^{0.5}$$

This equation shows that wave celerity depends on the depth of water in which the wave propagates. If the wave celerity decreases with depth, wave length must decrease proportionately. Variation in wave celerity occurs along contours because that part of the wave in deep water is moving faster than that in shallow water. This variation causes the wave crest to bend align with the bottom contours. This bending effect is called refraction and it depends on the relation of water depth to wave length (d/L). Different bathymetric features will lead to different patterns of bending of wave crests. Examples of such refraction patterns for different bottom topographic features are shown in figure 24.
Moreover, the change of direction of an orthogonal as it passes over a slope may be approximated by Snell's law as:

$\sin \theta_2 = \frac{C_2}{C_1} \sin \theta_1$ (Refer to **figure 25**)

Where: θ_1 is the angle a wave crest makes with the bottom contours

$\theta_2$is the angle as the wave crest passes over the next bottom contour

and C_1 and C_2 are the phase velocities at the depth of the first and second contour respectively.

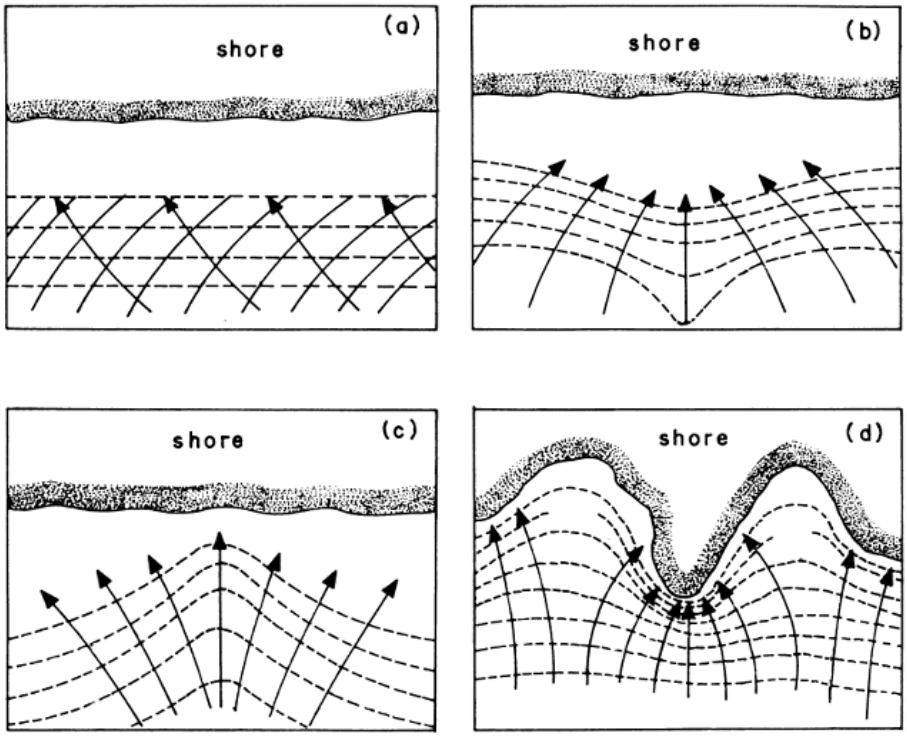


Figure 24-Typical Waves Refraction Pattern

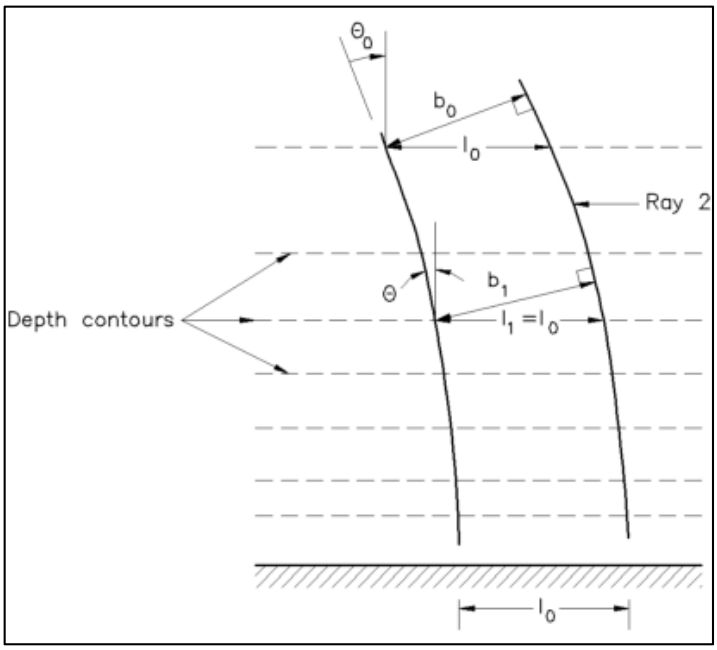


Figure 25-SNELL law along a Wave Ray

The extent of refraction is expressed in terms of the refraction coefficient (K_r) which is defined by

$$K_r = \left(\frac{b_0}{b}\right)^{0.5}$$

Where b_0 and b are the distances between the selected orthogonal in deep water and shallow water respectively.
Refraction coefficient K_r is independent of wave height.
In addition to refraction caused by variations in bathymetry, waves may be refracted by currents or any other phenomenon which causes one part of a wave to travel slower or faster than the other part. Refraction by a current occurs when waves intersect the current at an angle. In regions of tidal current or major ocean currents the effects of refraction by currents can be important.

5.5.2 Shoaling

As waves move into shallow water, the group velocity slightly increases and then decreases with decreasing water depth. Where the group velocity increases wave crests move further apart and wave heights decrease corresponding to a decrease in wave energy. Decreasing group velocity occurs for most of the near shore region so that wave crests move closer together and wave heights increase corresponding to an increase in wave energy. This process is called wave shoaling.
The variations in the height of the shoaling waves can be obtained from a consideration of the energy flux, i.e. the energy transmitted forward across a plane between two adjacent orthogonal. If the loss of energy due to bottom drag, reflection, etc. is negligible, then the energy flux is constant, such that the shallow water energy flux

$$EC_g = E_0 C_{g0}$$

It can finally derive that $K_s = \left({}^{C_0}/_{2nC}\right)^{0.5} = \frac{H}{H_0}$

K_s is called the shoaling coefficient.

5.5.3 Conclusion

The resulting wave height taking into account refraction and shoaling will be

$$H_r = H_0 K_r K_s$$

Refer to paragraph 5.5.4 for the wave height propagation calculation; two cases related to the wind sectors selected in paragraph 3.2.3 were taken into consideration.

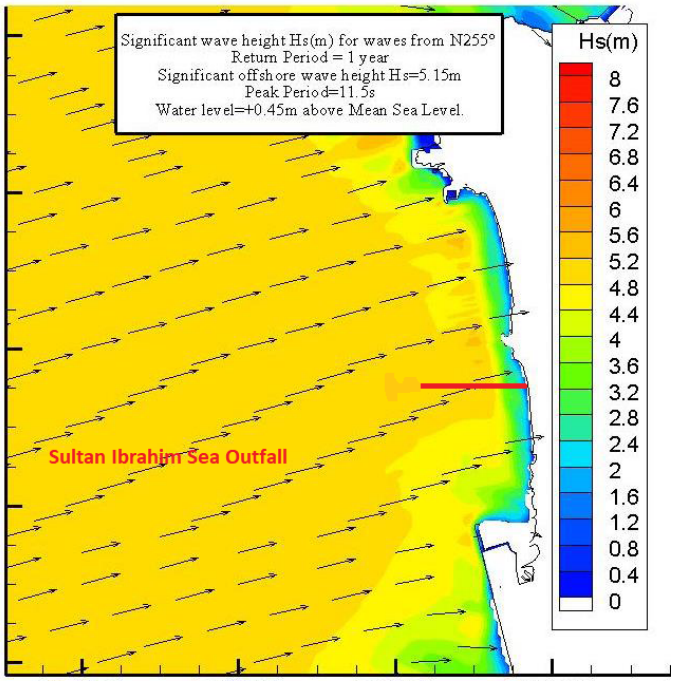


Figure 26- Significant wave height

For each return period, the breaking height has been computed in accordance with the guidelines of Part I of the Shore Protection Manual (detailed calculations are presented herein), therefore, we have the following table:

Table 12- Breaking heights vs. return period

N255 Prevailing Sector	1 year return period	50 years return period
H_{smax} (m)	5.15	7.18
d_b (m)	5.87	8.18

In the breaking zone, the pipe has to be heavily protected or covered with underwater concrete in order to be considered totally stable.

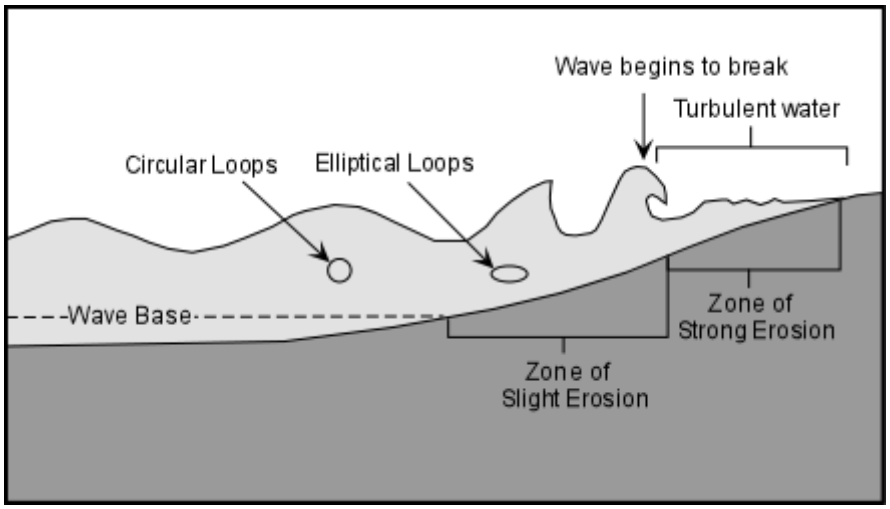


Figure 27- Schematics of breaking waves

5.5.4 Simulations for 1 and 50 years

Table 13-Wave propagation simulation for 1 year return period

Sultan Ibrahim Sea Outfall

Wave Propagation
1 year return period

Sector N225								
T	=	11.5	s	Wave Period				
H1	=	5.15	m	Maximum height of wave				
dmax	=	490	m	Water				
L_0 (longueur d'onde en eau profonde)	=	206.48	m	Deep Water wave length				
C_0 (Vitesse d' onde)	=	17.96						
C_{g0}	=	8.98		for deep water				
tanβ	=	0.03		Beach slope				
Ωb	=	1.037						
a	=	19.0083						
b	=	1.0019			db	=	5.78	Breaking wave
γb	=	0.924			Hb	=	5.34	

d	θ_0 angle of Approach											
	15											
	L1	case	Ks	Kr	C	sinΘ	Θ	H1	U	U'	D50	M50
	m						deg	m	m/s	m/s	m	t
25	165.35	transitional water	0.9246	0.9928	13.8211	0.1991	11.49	4.73	0.83	0.80	0.05	0.00
24.5	164.18	transitional water	0.9260	0.9927	13.7194	0.1977	11.41	4.73	0.85	0.82	0.05	0.00
24	162.98	transitional water	0.9274	0.9925	13.6155	0.1962	11.32	4.74	0.86	0.83	0.05	0.00
23.5	161.75	transitional water	0.9290	0.9924	13.5092	0.1946	11.23	4.75	0.88	0.85	0.05	0.00
23	160.48	transitional water	0.9307	0.9922	13.4005	0.1931	11.14	4.76	0.89	0.86	0.05	0.00
22.5	159.18	transitional water	0.9325	0.9921	13.2893	0.1915	11.04	4.76	0.91	0.88	0.05	0.00
22	157.85	transitional water	0.9343	0.9919	13.1757	0.1898	10.95	4.77	0.93	0.90	0.06	0.00
21.5	156.48	transitional water	0.9364	0.9917	13.0595	0.1882	10.85	4.78	0.95	0.92	0.06	0.00
21	155.07	transitional water	0.9385	0.9916	12.9406	0.1864	10.75	4.79	0.97	0.93	0.06	0.00
20.5	153.63	transitional water	0.9408	0.9914	12.8190	0.1847	10.65	4.80	0.99	0.95	0.06	0.00
20	152.14	transitional water	0.9432	0.9912	12.6946	0.1829	10.54	4.81	1.01	0.97	0.07	0.00
19.5	150.62	transitional water	0.9458	0.9911	12.5673	0.1811	10.44	4.83	1.03	0.99	0.07	0.00
19	149.06	transitional water	0.9485	0.9909	12.4371	0.1792	10.33	4.84	1.05	1.02	0.07	0.00
18.5	147.45	transitional water	0.9514	0.9907	12.3038	0.1773	10.22	4.85	1.07	1.04	0.08	0.00
18	145.80	transitional water	0.9544	0.9905	12.1673	0.1753	10.10	4.87	1.10	1.06	0.08	0.00
17.5	144.11	transitional water	0.9577	0.9904	12.0275	0.1733	9.98	4.88	1.12	1.09	0.08	0.00
17	142.37	transitional water	0.9611	0.9902	11.8844	0.1712	9.86	4.90	1.15	1.11	0.09	0.00
16.5	140.58	transitional water	0.9648	0.9900	11.7377	0.1691	9.74	4.92	1.18	1.14	0.09	0.00
16	138.75	transitional water	0.9687	0.9898	11.5875	0.1669	9.62	4.94	1.21	1.17	0.10	0.00
15.5	136.86	transitional water	0.9728	0.9896	11.4334	0.1647	9.49	4.96	1.24	1.20	0.10	0.00
15	134.93	transitional water	0.9771	0.9894	11.2754	0.1625	9.35	4.98	1.27	1.23	0.11	0.00
14.5	132.93	transitional water	0.9818	0.9892	11.1133	0.1601	9.22	5.00	1.30	1.26	0.11	0.00
14	130.89	transitional water	0.9867	0.9890	10.9469	0.1577	9.08	5.03	1.34	1.30	0.12	0.00
13.5	128.78	transitional water	0.9920	0.9888	10.7761	0.1553	8.94	5.05	1.38	1.33	0.13	0.01
13	126.61	transitional water	0.9976	0.9886	10.6005	0.1527	8.79	5.08	1.42	1.37	0.13	0.01
12.5	124.38	transitional water	1.0036	0.9885	10.4201	0.1501	8.64	5.11	1.46	1.41	0.14	0.01
12	122.08	transitional water	1.0100	0.9882	10.2344	0.1475	8.48	5.14	1.51	1.46	0.15	0.01
11.5	119.72	transitional water	1.0169	0.9880	10.0432	0.1447	8.32	5.17	1.56	1.51	0.16	0.01
11	117.28	transitional water	1.0242	0.9878	9.8463	0.1419	8.16	5.21	1.61	1.56	0.17	0.01
10.5	114.76	transitional water	1.0321	0.9876	9.6432	0.1389	7.99	5.25	1.67	1.61	0.18	0.02
10	112.17	transitional water	1.0407	0.9874	9.4335	0.1359	7.82	5.29	1.73	1.67	0.20	0.02
9.5	109.49	transitional water	1.0499	0.9872	9.2168	0.1328	7.63	5.34	1.80	1.74	0.21	0.03
9	106.71	transitional water	1.0599	0.9870	8.9926	0.1296	7.45	5.39	1.87	1.81	0.23	0.03
8.5	103.84	transitional water	1.0708	0.9868	8.7602	0.1262	7.25	5.44	1.96	1.89	0.25	0.04
8	100.87	transitional water	1.0827	0.9866	8.5191	0.1227	7.05	5.50	2.05	1.98	0.28	0.06
7.5	97.78	transitional water	1.0958	0.9864	8.2684	0.1191	6.85	5.10	2.78	2.69	0.51	0.35
7	94.56	transitional water	1.1102	0.9861	8.0073	0.1154	6.63	5.00	2.83	2.74	0.53	0.39
6.5	91.22	transitional water	1.1262	0.9859	7.7346	0.1114	6.40	4.90	2.89	2.79	0.55	0.44
6	87.72	transitional water	1.1440	0.9857	7.4491	0.1073	6.16	4.50	2.77	2.68	0.51	0.34
5.5	84.06	transitional water	1.1641	0.9855	7.1492	0.1030	5.92	4.10	2.65	2.56	0.46	0.26
5	80.21	transitional water	1.1870	0.9852	6.8329	0.0984	5.65	3.40	2.31	2.23	0.35	0.12
4.5	76.15	transitional water	1.2133	0.9850	6.4980	0.0936	5.37	3.10	2.23	2.15	0.33	0.09
4	71.84	transitional water	1.2440	0.9848	6.1413	0.0885	5.08	2.70	2.07	2.00	0.28	0.06
3.5	67.24	transitional water	1.2804	0.9845	5.7587	0.0830	4.76	2.40	1.97	1.90	0.26	0.04
3	62.28	transitional water	1.3245	0.9843	5.3445	0.0770	4.42	1.90	1.69	1.63	0.19	0.02
2.5	56.88	transitional water	1.3798	0.9841	4.8909	0.0705	4.04	1.40	1.38	1.34	0.13	0.01
2	50.89	shallow water	1.4521	0.9838	4.3853	0.0632	3.62	0.90	0.99	0.96	0.07	0.00

Table 14- Wave propagation simulation for 50 years return period

Sultan Ibrahim Sea Outfall

Wave Propagation
50 year return period

Sector N225								
T	=	13	s	Wave Period				
H1	=	7.1	m	Maximum height of wave				
dmax	=	490	m	Water				
L_0 (longueur d'onde en eau profonde)	=	263.86	m	Deep Water wave length				
C_0 (Vitesse d' onde)	=	20.30						
C_{g0}	=	10.15		for deep water				
tanβ	=	0.03		Beach slope				
Ωb	=	1.011						
a	=	19.0083						
b	=	1.0019			db	=	7.81	Breaking wave
γb	=	0.920			Hb	=	7.18	

d	θ_0 angle of Approach											
	15											
	L1	case	Ks	Kr	C	sinθ	θ	H50	U	U'	D50	M50
	m						deg	m	m/s	m/s	m	t
25	192.73	transitional water	0.9454	0.9911	14.2250	0.1813	10.45	6.65	1.25	1.21	0.10	0.00
24.5	191.17	transitional water	0.9475	0.9909	14.1104	0.1798	10.37	6.67	1.27	1.23	0.11	0.00
24	189.59	transitional water	0.9497	0.9908	13.9937	0.1784	10.28	6.68	1.29	1.25	0.11	0.00
23.5	187.97	transitional water	0.9520	0.9907	13.8749	0.1768	10.19	6.70	1.32	1.27	0.11	0.00
23	186.31	transitional water	0.9545	0.9905	13.7538	0.1753	10.10	6.71	1.34	1.29	0.12	0.00
22.5	184.62	transitional water	0.9570	0.9904	13.6305	0.1737	10.01	6.73	1.36	1.32	0.12	0.00
22	182.90	transitional water	0.9596	0.9903	13.5049	0.1721	9.92	6.75	1.39	1.34	0.13	0.01
21.5	181.14	transitional water	0.9624	0.9901	13.3769	0.1705	9.82	6.77	1.41	1.37	0.13	0.01
21	179.34	transitional water	0.9653	0.9900	13.2464	0.1688	9.72	6.78	1.44	1.39	0.14	0.01
20.5	177.51	transitional water	0.9683	0.9898	13.1134	0.1671	9.63	6.81	1.47	1.42	0.14	0.01
20	175.63	transitional water	0.9715	0.9897	12.9777	0.1654	9.53	6.83	1.50	1.45	0.15	0.01
19.5	173.71	transitional water	0.9748	0.9895	12.8394	0.1636	9.42	6.85	1.53	1.48	0.15	0.01
19	171.76	transitional water	0.9783	0.9894	12.6983	0.1618	9.32	6.87	1.56	1.51	0.16	0.01
18.5	169.76	transitional water	0.9820	0.9892	12.5544	0.1600	9.21	6.90	1.59	1.54	0.17	0.01
18	167.71	transitional water	0.9859	0.9891	12.4074	0.1581	9.10	6.92	1.63	1.57	0.18	0.01
17.5	165.62	transitional water	0.9899	0.9889	12.2574	0.1562	8.99	6.95	1.66	1.61	0.18	0.02
17	163.48	transitional water	0.9942	0.9888	12.1042	0.1543	8.88	6.98	1.70	1.64	0.19	0.02
16.5	161.30	transitional water	0.9986	0.9886	11.9477	0.1523	8.76	7.01	1.74	1.68	0.20	0.02
16	159.06	transitional water	1.0033	0.9885	11.7878	0.1502	8.65	7.04	1.78	1.72	0.21	0.02
15.5	156.78	transitional water	1.0083	0.9883	11.6242	0.1482	8.52	7.08	1.83	1.76	0.22	0.03
15	154.44	transitional water	1.0135	0.9881	11.4569	0.1460	8.40	7.11	1.87	1.81	0.23	0.03
14.5	152.04	transitional water	1.0191	0.9880	11.2858	0.1438	8.27	7.15	1.92	1.86	0.24	0.04
14	149.59	transitional water	1.0249	0.9878	11.1105	0.1416	8.14	7.19	1.97	1.91	0.26	0.04
13.5	147.07	transitional water	1.0311	0.9877	10.9309	0.1393	8.01	7.23	2.03	1.96	0.27	0.05
13	144.50	transitional water	1.0377	0.9875	10.7468	0.1370	7.88	7.28	2.09	2.01	0.29	0.06
12.5	141.86	transitional water	1.0446	0.9873	10.5580	0.1346	7.74	7.32	2.15	2.07	0.31	0.07
12	139.14	transitional water	1.0520	0.9872	10.3642	0.1321	7.59	7.37	2.21	2.14	0.32	0.09
11.5	136.36	transitional water	1.0599	0.9870	10.1651	0.1296	7.45	7.43	2.29	2.21	0.35	0.11
11	133.50	transitional water	1.0684	0.9868	9.9604	0.1269	7.30	7.49	2.36	2.28	0.37	0.13
10.5	130.56	transitional water	1.0774	0.9867	9.7497	0.1243	7.14	7.55	2.45	2.36	0.40	0.16
10	127.53	transitional water	1.0871	0.9865	9.5326	0.1215	6.98	7.61	2.54	2.45	0.43	0.20
9.5	124.42	transitional water	1.0976	0.9863	9.3088	0.1186	6.82	7.69	2.63	2.54	0.46	0.25
9	121.20	transitional water	1.1089	0.9861	9.0775	0.1157	6.65	7.76	2.74	2.65	0.50	0.32
8.5	117.88	transitional water	1.1211	0.9860	8.8384	0.1126	6.47	7.85	2.86	2.76	0.54	0.41
8	114.45	transitional water	1.1345	0.9858	8.5907	0.1095	6.29	7.94	2.99	2.89	0.59	0.54
7.5	110.89	transitional water	1.1491	0.9856	8.3336	0.1062	6.10	7.10	3.92	3.79	1.02	2.73
7	107.21	transitional water	1.1651	0.9854	8.0662	0.1028	5.90	7.00	4.01	3.87	1.06	3.14
6.5	103.37	transitional water	1.1828	0.9853	7.7875	0.0993	5.70	6.90	4.11	3.97	1.12	3.65
6	99.37	transitional water	1.2026	0.9851	7.4962	0.0955	5.49	6.50	4.04	3.91	1.08	3.30
5.5	95.19	transitional water	1.2247	0.9849	7.1907	0.0916	5.26	6.10	3.97	3.84	1.05	2.97
5	90.81	transitional water	1.2499	0.9847	6.8691	0.0875	5.03	5.40	3.70	3.57	0.91	1.93
4.5	86.19	transitional water	1.2787	0.9845	6.5290	0.0832	4.78	5.10	3.69	3.56	0.90	1.91
4	81.29	transitional water	1.3121	0.9844	6.1674	0.0786	4.51	4.70	3.62	3.49	0.87	1.69
3.5	76.07	transitional water	1.3517	0.9842	5.7802	0.0737	4.23	4.30	3.54	3.42	0.83	1.50
3	70.45	transitional water	1.3997	0.9840	5.3617	0.0683	3.92	3.90	3.48	3.36	0.80	1.34
2.5	64.35	shallow water	1.4598	0.9838	4.9040	0.0625	3.59	3.40	3.36	3.24	0.75	1.08
2	57.55	shallow water	1.5376	0.9836	4.3948	0.0560	3.21	2.90	3.20	3.10	0.68	0.82

5.5.5 Broken Waves

In order to determine the broken wave height beyond the depth of breaking, we have used the latest model developed by MIT in 2005.
The seabed slope, tanß = 0.03, is within the range of application of the a/m model.
According to MIT's, the broken wave height can be evaluated from the computed liner wave height that would have continued as if no breaking had occurred, as follows:
$H^* = H \times K_b$
Where K_b is proportionality constant dependent mainly on the sea bottom slope:
$K_b = 2.5 \times \Upsilon_s^2 \tan ß / (\Upsilon_s^2 - \Upsilon_r^2)$ where Υ_r is a constant fitted from experimental data and Υ_s is curve fitted with respect to the bottom slope.
This gives K_b = 0.08 and thus the corresponding broken wave height is obtained for the considered return period.
Moreover, the obtained values have been corroborated with those obtained from the model output shown in paragraph 5.5.3
The values have been updated in the appended tables as well.

From the combinations of the findings above, the wave height is extracted along the outfall, leading to the following table:

Table 15-Wave height extracted along the outfall

Cumulative distance from origin to beginning of section (m)	Cumulative distance from origin to end of section(m)	Minimal depth in section (m)	Wave incidence (°N)	Hs for a 1 year return period (m)	Hs for a 50 years return period (m)
0	100	0.5	255	2.9	4
100	200	3.16	255	3.8	5.3
200	300	5.52	255	4.4	6.1
300	400	5.57	255	4.7	6.6
400	500	7.27	255	5.1	7.1
500	600	10.08	255	5.1	7.1
600	708.6	12.41	255	5.1	7.1
708.6	908.6	16.39	255	5.1	7.1

6. Stability of the pipeline

The following is the study for the stability of the pipeline.

6.1 Characteristics of the pipeline

The retained pipeline has the following characteristics for the offshore section:

- Material: HDPE PE100 SDR26 PN6.3 pipeline OD1400.
- Specific gravity of the 950kg/m3.
- External Diameter of Pipe: 1400 mm
- Internal Diameter of Pipe: 1292 mm
- Thickness of Pipe: 54 mm

6.2 Buoyancy of the pipe

The buoyancy of the pipe is evaluated without hydrodynamic stresses.

Table 16–Pipe physical and hydrodynamic characteristics

Pipe characteristics		
Variable	**Value**	**Unit**
Outer Diameter	1.400	m
Inner Diameter	1.292	m
Thickness	0.054	m
Pipe Density	950	Kg/m3
Hydrodynamic characteristics		
Variable	**Value**	**Unit**
Fresh water density	1000	Kg/m3
Sea water density	1026	Kg/m3
Pipe material section	0.228	m2
Empty pipe linear weight	216.223	Kg/m
Pipe inner section	1.311	m2
Linear weight of sea water in the outfall	1345.082	Kg/m
Linear weight of outfall full of sea water	1561.306	Kg/m
Outfall buoyancy per meter run	1578.604	Kg/m
Apparent linear weight of submerged outfall	-17.298	Kg/m

The apparent weight of the submerged pipeline full of sea water is negative, thus the pipeline floats. Adding 10% air in the pipeline, we have:

Table 17-Characteristics of pipeline with 10% air content

Air characteristics		
Variable	**Value**	**Unit**
Air percentage	10	%
Air density	1	Kg/m3
Empty pipe linear weight	216.223	Kg/m
Linear weight of air in the outfall	0.131	Kg/m
Linear weight of sea water in the outfall	1210.574	Kg/m
Linear weight of outfall filled with air and sea water	1426.929	Kg/m
Apparent linear weight of submerged outfall	-151.675	Kg/m

Then concrete collars are added to sink the pipeline:

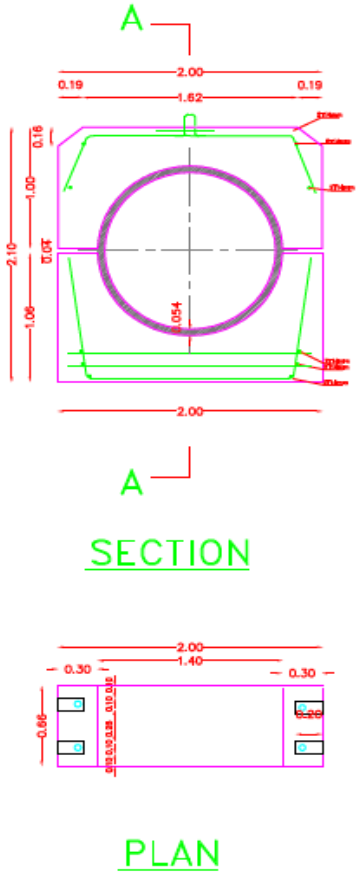


Figure 28- Concrete collar

Table 18- Concrete collars spacing to sink the pipeline filled with 10% air

Collar characteristics		
Variable	**Value**	**Unit**
Concrete density	2300	Kg/m3
Collar surface area	2.61	m2
Collar length	0.66	m
Collar volume	1.7226	m3
Collar apparent unit weight	2194.5924	Kg
Necessary weight to sink the pipeline	151.675	Kg/m
Collar minimum number per meter run	0.07	U
Collars distances on centers	14.47	m
Percentage of pipeline covered with collars	4.56	%

Thus, in order to sink the pipeline, we need one collar every 14.5m.
Moreover, in order to avoid bending the pipe and in case air is locked up inside6 the pipeline, we shall consider the case where we have 40% of air trapped inside the pipe. We have then:

Table 19-Characteristics of pipeline with 40% air content

Air characteristics		
Variable	**Value**	**Unit**
Air percentage	40	%
Air density	1	Kg/m3
Empty pipe linear weight	216.223	Kg/m
Linear weight of air in the outfall	0.524	Kg/m
Linear weight of sea water in the outfall	807.049	Kg/m
Linear weight of outfall filled with air and sea water	1023.797	Kg/m
Apparent linear weight of submerged outfall	-554.806	Kg/m

Then the collars requirement becomes:

Table 20- Concrete collars spacing to sink the pipeline filled with 40% air

Collar characteristics		
Variable	**Value**	**Unit**
Concrete density	2300	Kg/m3
Collar surface area	2.61	m2
Collar length	0.66	m
Collar volume	1.7226	m3
Collar apparent unit weight	2194.5924	Kg
Necessary weight to sink the pipeline	554.806	Kg/m
Collar minimum number per meter run	0.25	U
Collars distances on centers	3.96	m
Percentage of pipeline covered with collars	16.69	%

Thus, we need in this case to have one collar every 4m in order to sink the pipe.

Furthermore, in order to sink the pipe, 50% of air filling is usually enough. Thus, selecting such a percentage, leads to the following:

Table 21-Characteristics of pipeline with 50% air content

Air characteristics		
Variable	**Value**	**Unit**
Air percentage	50	%
Air density	1	Kg/m3
Empty pipe linear weight	216.223	Kg/m
Linear weight of air in the outfall	0.655	Kg/m
Linear weight of sea water in the outfall	672.541	Kg/m
Linear weight of outfall filled with air and sea water	889.420	Kg/m
Apparent linear weight of submerged outfall	-689.184	Kg/m

Table 22-Concrete collars spacing to sink the pipeline filled with 50% air

Collar characteristics		
Variable	**Value**	**Unit**
Concrete density	2300	Kg/m3
Collar surface area	2.61	m2
Collar length	0.66	m
Collar volume	1.7226	m3
Collar apparent unit weight	2194.5924	Kg
Necessary weight to sink the pipeline	689.184	Kg/m
Collar minimum number per meter run	0.31	U
Collars distances on centers	3.18	m
Percentage of pipeline covered with collars	20.73	%

Accordingly, we need to have a collar every 3.2m along the pipeline in order to sink the outfall.
Now that we have a collar every 3.2m along the outfall, we have to keep in mind that the pipeline has to stay afloat in order to bring it into place.
The following calculation is aimed at finding whether the pipeline floats or not:

Table 23- Verification of pipeline flotation with its final collars configuration

Pipe characteristics		
Variable	**Value**	**Unit**
Outer Diameter	1.400	m
Inner Diameter	1.292	m
Thickness	0.054	m
Pipe Density	950.000	Kg/m3
Hydrodynamic characteristics		
Variable	**Value**	**Unit**
Air density	1	Kg/m3
Sea water density	1026	Kg/m3
Pipe material section	0.228	m2
Empty pipe linear weight	216.223	Kg/m
Pipe inner section	1.311	m2
Linear weight of air inside pipeline (100% air)	1.311	Kg/m
Collar characteristics		
Variable	**Value**	**Unit**
Concrete density	2300	Kg/m3
Collar surface area	2.61	m2
Collar length per meter run of pipeline	0.207	m/m
Collar weight per meter run of pipeline	1244.209	Kg/m
Total linear weight of outfall system full of air	1461.743	Kg/m
Flotation		
Variable	**Value**	**Unit**
Submerged outfall percentage	69	%
Collar volume per meter run	0.374	m3/m
Outfall volume per meter run	1.062	m3/m
Total volume per meter run	1.435	m3/m
Weight of displaced volume of sea water	1472.204	Kg/m

Thus the pipeline floats with a moved volume of 69%.

6.3 Sections of the outfall pipeline (offshore)

In the following table and layouts, we present the components of the Outfall Pipeline on the seabed:

Table 24-Sea outfall sections

Section	Linear	Depth (relative to mean sea level)
Section AB	L=100m	Elevation(A) =-4m Elevation(B) =-5.49m
Section BC	L=100m	Elevation(B) =-5.49m Elevation(C) =-6.98m
Section CD	L=100m	Elevation(C) =-6.98m Elevation(D) =-8.481m
Section DE	L=100m	Elevation(D) =-8.48m Elevation(E) =-10.08m
Section EF	L=100m	Elevation(E) =-10.08m Elevation(F) =-12.12m
Section FG	L=100m	Elevation(F) =-12.12 Elevation(G) =-14.16m
Section GH	L=108.6m	Elevation(F) =-14.16m Elevation(G) =-16.38m
Section FG	L=200m	Elevation(F) =-16.38m Elevation(G) =-21.20m

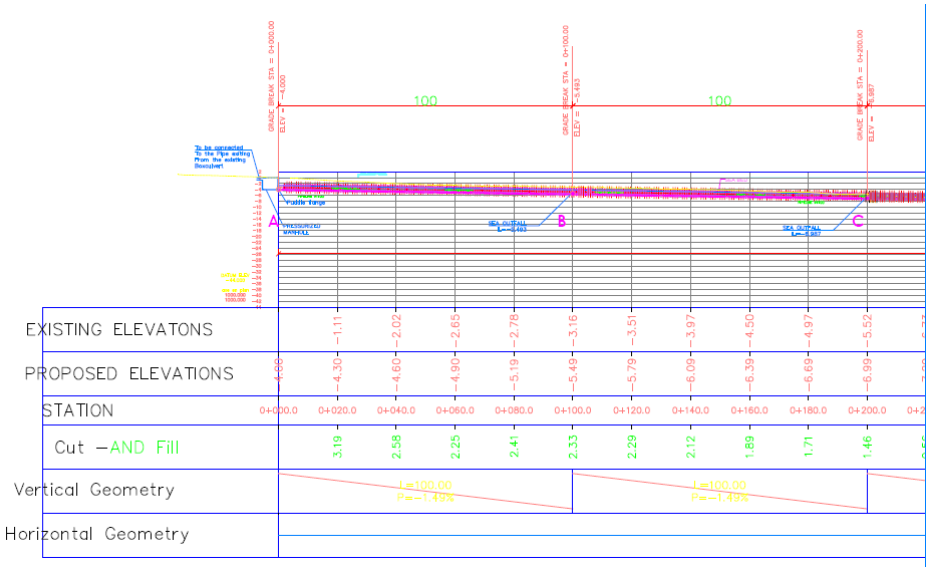


Figure 29- Layout of the section A to C

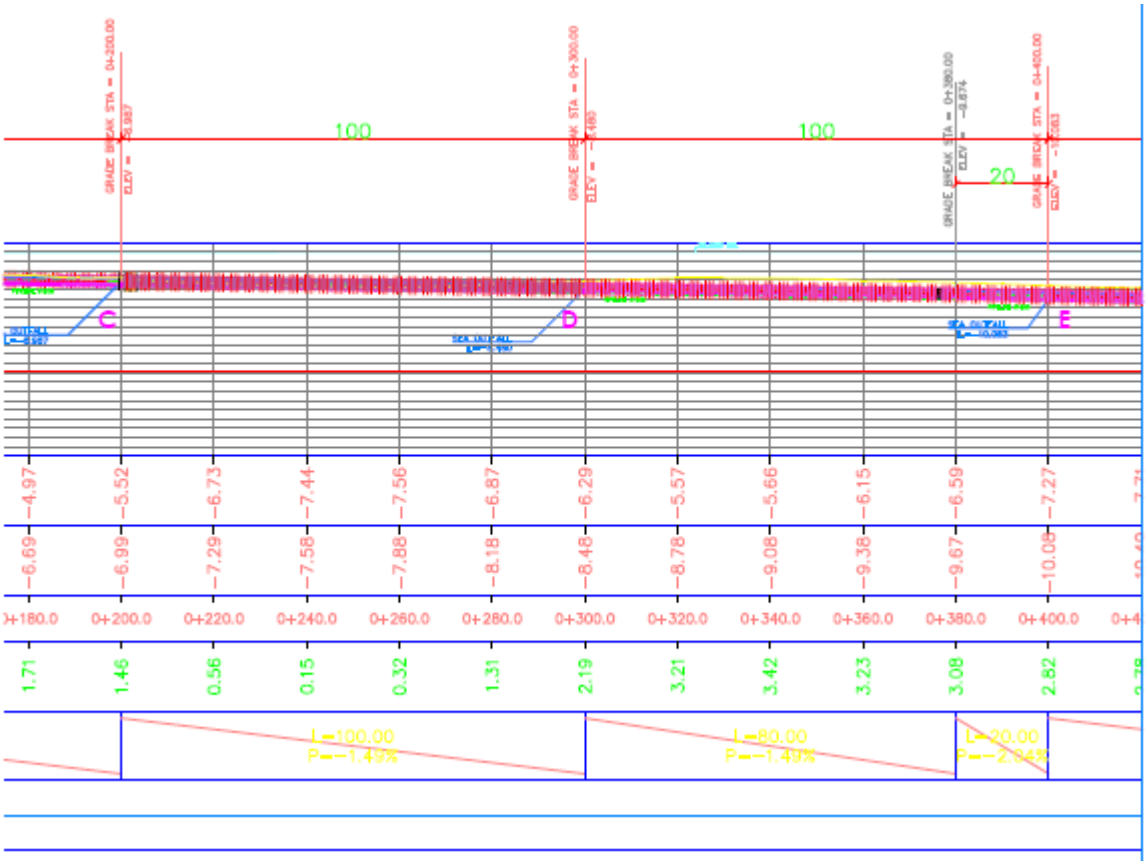


Figure 30-Layout of the section C to E

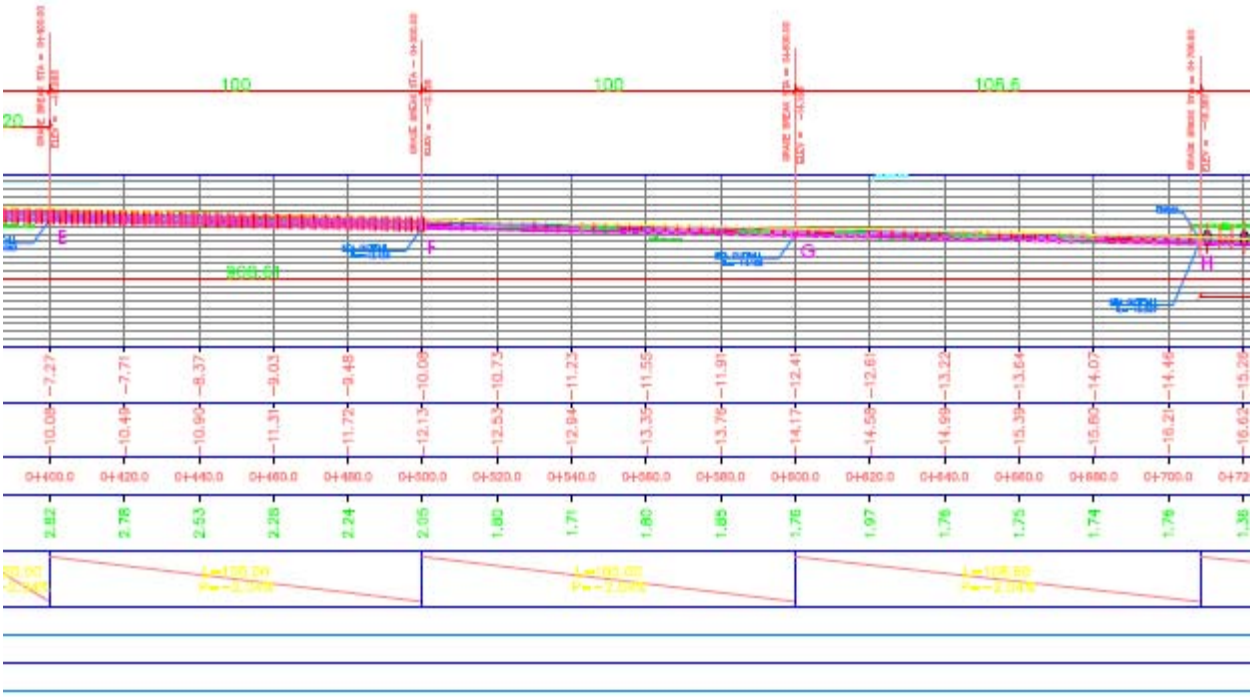


Figure 31- Layout of the section E to H

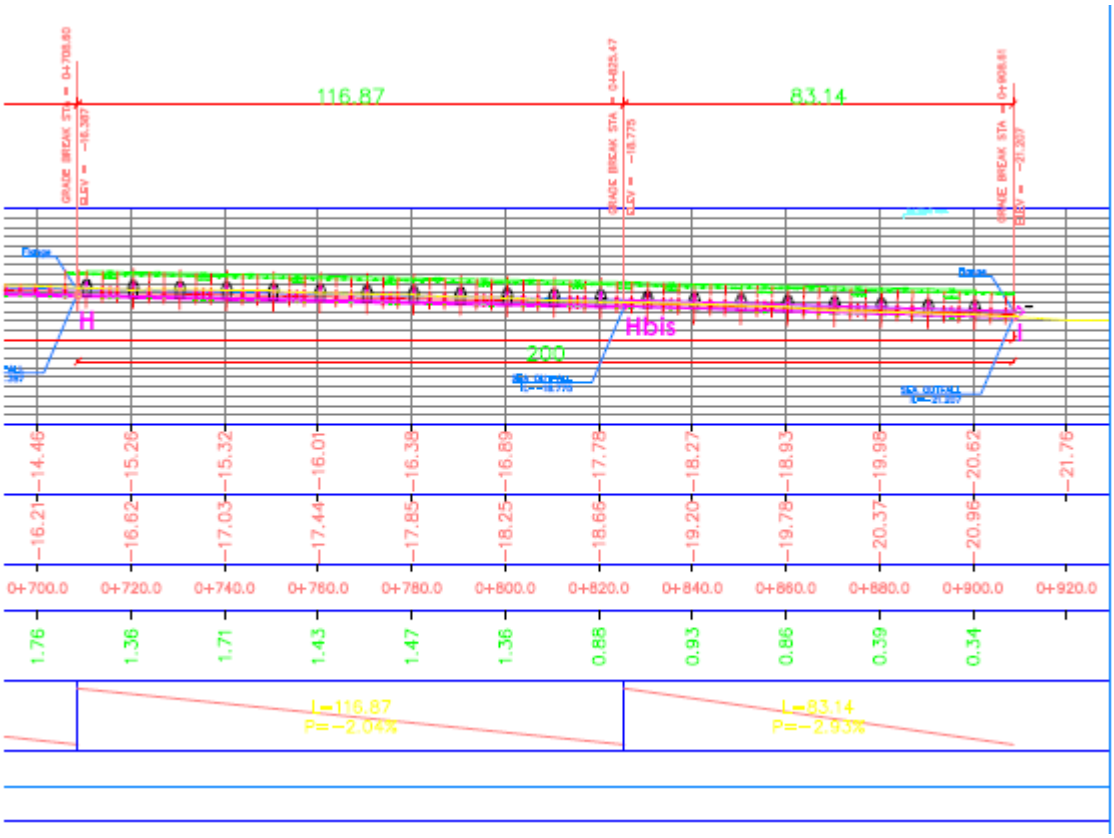


Figure 32- Layout of the section H to I

6.4 Marine currents

Tide currents are considered negligible herein.
Other currents of influence are driven by strong winds.
As a safe assumption providing a security factor, we will retain a 45cm/s velocity (1% of an extreme wind speed, i.e. 160km/h) in the normal direction of the pipe.

Further research indicate possible shore currents of 0.8m/s magnitude (mainly long shore) till the H50 rounded breaking depth of 8.2m. This is in line with latest research advancing the following formula:

V= (5Π/16fc) ϒb (gHb)1/2 tanß sinθ = 0.7m/s
Where fc is a friction factor taken here equal to 0.6

6.5 Design of the marine part of the pipeline

The sea outfall pipeline will have to be stable with respect to hydrodynamics constraints such as extreme waves. According to the state of the art of designing pipes over sea bottom, it is recommended to bury them in the surf zone, i.e. where hydrodynamics efforts are very strong due to wave breaking impact.
Outside the surf zone, the pipe will be stressed by wave orbital currents and bottom currents that are usually not strong enough to make a burying necessary, however, as mentioned previously, the pipelines shall be laid in a trench up to a water depth of 13m.

6.6 Limitation of the surf zone

The surf zone, where important efforts may occur, is delimited around the depth where the highest wave of the design storm is expected to break. We shall retain, as requested in the project's specifications, the 50 year return period event, whose characteristics are as follows:

Table 25-Design wave characteristics

Design wave characteristics (50years)	
Significant height H_S offshore	H_s=7.1m
Direction	N255°
Peak period	T_P=13s
Significant height near the surfzone	H_{50}=5.46m
Highest wave near the surfzone	H_b = 7.18m
Water depth at breaking point	db = 8.18m

The water depth at breaking has to be considered at the lowest level to include the greatest surface from the shoreline. With Hb=7.18m and a lowest level at -0.3NGL, the level of the surf zone is -8.5m. Considering that long shore currents are induced by breaking waves around this depth, we recommend that the upper part of the pipe is protected until bottom at level -10m NGL.

6.7 Pipe Ballasting

The present discharge facility works are composed of pipes, ballast, concrete and diffusers. They are all subjected to the action of currents and waves. They undergo efforts which have to be evaluated in order to properly dimension the structures that are needed to ensure not only the external stability of the outfall pipeline (pull, sliding, tilting), but also its internal stability (deformation, structural damage, rupture).

Given the interdependence between the characteristic dimensions of the various elements and the wavelength of the swell to be considered, efforts are evaluated with asymptotic formulae that stand out:

- the inertia force proportional to the immersed volume and the acceleration of the fluid,
- the drag force proportional to the square of the fluid velocity,
- the lift efforts

In our case, as stated previously, and since the sea bed consists of very hard rock, the pipeline shall be protected essentially by concrete shells or mattresses.

6.7.1 Pipe fastening by jumpers: «capping»

6.7.1.1 Sinking ballast

A common pipeline fastening system is done by install jumpers that take the shape of brackets holding down the pipeline. Jumpers are usually made of reinforced concrete and are placed on the pipeline to ensure sea bottom hold – this is especially needed in our case since we have an HDPE outfall that is highly buoyant. Concrete jumpers are usually installed using a crane that is guided by divers.

6.7.1.2 Additional ballast and pipe protection

The sinking “S” shape of an HDPE pipeline is directly dependent on the intrinsic properties of the pipe, its degree of buoyancy and a balance of the water level in the pipe. Thus, it is impossible to consider that the sinking ballast, consisting of the evenly spaced concrete collars “wrapped around” the pipeline, can assume the global ballasting of the pipeline on the seabed. Therefore, additional ballast is placed on the pipeline, once it is in place, to protect the latter from the effects of breaking waves.

The pipeline at hand may thus be fixed and protected by rocks, ballast mattresses, underwater mass concrete and jumpers. This technique is called “capping” and is carried out with the help of a crane installed on a floating barge.

In order to avoid causing structural damage to the pipeline, a transition layer is usually placed between the capping and the pipeline. Of course, when rock cover is placed, sharp rocks are carted away.

In line with the recommendations of CETMEF and OSPAR, we shall limit the height of the protection in order to minimize turbulences and suction effects.

It is worthwhile mentioning here that concrete mattresses offer the advantage of having an overall homogeneous and coherent final structure that is stable, protective and not dune-shaped.

6.8 Sultan Ibrahim Outfall

In the local context of Sultan Ibrahim - Jnah, the need for adequate protection is exacerbated by the following:

- A seabed promoting orbital velocities,
- A very “active” area,
- A very exposed site (even with frequent swell) which requires the completion date between immersion of the pipe and its final ballasting to be as short as possible,
- A quite fragile pipe under those stresses (Punching, angularity materials, bending moment, ovalisation, ...)

6.9 Hydrodynamic stresses on the pipeline

As the pipe size is small compared to the wavelength, we consider that the pipe does not create additional waves. The hydrodynamic efforts on the pipe are mainly due to the vortices induced

by currents in the normal direction with respect to the pipe axis. The orbital currents are calculated for the same wave conditions as for the limitation of the surf zone, i.e. the 1 in 50 year return period.

In order to evaluate the hydrodynamic stresses on the pipeline, we need to determine first the wave theory using the graph here below:

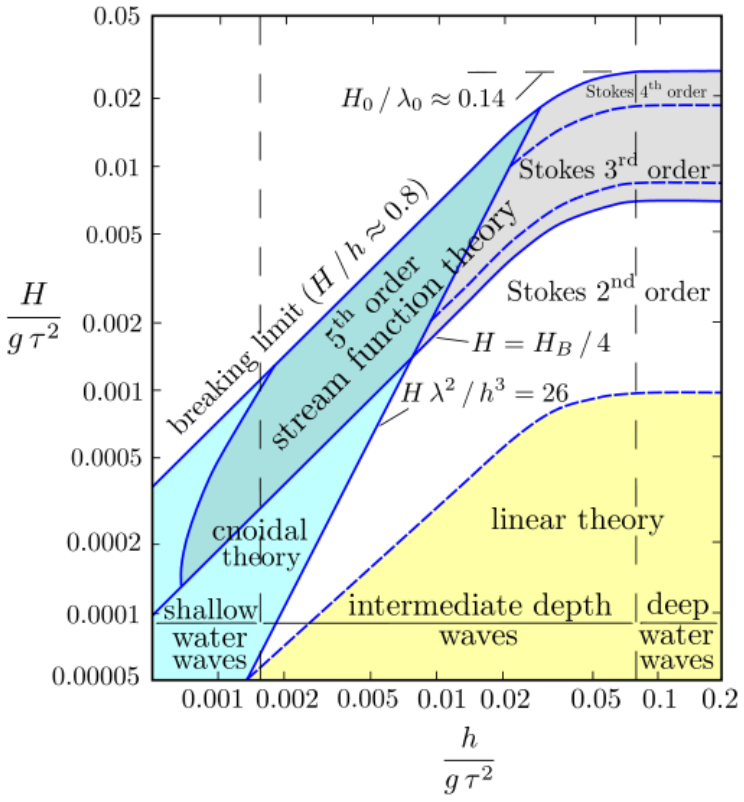


Figure 33- Wave theories graph

The wave is located in the Miche theory. With the present study's parameters, the orbital speed determined from the Miche theory is similar to that given by the linear wave theory. Hence, the latter is used to determine the orbital speeds.
The component normal to the axis of the pipe, orbital maximum speeds is obtained from the linear first order theory, based on the maximum wave height Hmax :
$U_{N\,max} = n\, U_{max} = \Pi\, Hm_{ax} \sin\theta / (T_p \sinh(2\Pi h/L)$
with:

U_{max}: maximum orbital velocity of the waves at the bottom;

U_{Nmax}: normal component of the orbital velocity; T_P: peak

wave period;

h: local depth;

θ: angle of incidence of the waves over the pipe;

n: vector normal to the axis of the pipe

L: wave length

The results are summarized in the table here below:

Table 26-Orbital velocities and accelerations

Section	Minimal depth in section (m)	Hs for 50 year return period (m)	Wave length (m)	Wave orbital velocity(m/s)	Wave orbital Acceleration (m/s^2)	Shore current (m/s)
Section AB	0.5	4	28.78	2.87	1.38	0.8
Section BC	3.16	5.3	72.29	3.48	1.67	0.8
Section CD	5.52	6.1	95.37	3.97	1.90	0.8
Section DE	5.57	6.6	95.79	3.95	1.89	0.8
Section EF	7.27	7.1	109.21	3.99	1.91	0.8
Section FG	10.08	7.1	128.02	3.57	1.71	0.45
Section GH	12.41	7.1	141.37	3.04	1.46	0.45
Section HI	16.39	7.1	160.81	2.47	1.19	0.45

Owing to the constructive dispositions that we have opted for, i.e. the outfall pipe is either in a shallow trench or partly exposed, the Sultan Ibrahim outfall is subjected to hydrodynamic stresses over its entire length.

For this purpose, the apparent weight of the outfall is computed here below:

Table 27-Apparent weight of the pipeline

Pipe characteristics		
Variable	**Value**	**Unit**
Outer Diameter	1.400	m
Inner Diameter	1.292	m
Thickness	0.054	m
Pipe Density	950.000	Kg/m3
Hydrodynamic characteristics		
Variable	**Value**	**Unit**
Fresh water density	1000	Kg/m3
Sea water density	1026	Kg/m3
Air density	1	Kg/m3
Air percentage inside pipeline during flow	25	%
Pipe material section	0.228	m2
Apparent linear weight of submerged outfall	-17.298	Kg/m
Pipe inner section	1.311	m2
Fresh water apparent weight inside the outfall	-25.564	Kg/m
Air apparent weight inside the outfall	-335.943	Kg/m
Apparent linear weight of submerged outfall	-378.805	Kg/m
Collar characteristics		
Variable	**Value**	**Unit**
Concrete density	2300	Kg/m3
Collar surface area	2.61	m2
Collar length	0.66	m
Collars distance on centers	3.20	m
Collar apparent weight per meter run of pipe	685.810	Kg/m
Apparent weight of outfall with collars	307.005	Kg/m

The apparent weight of the pipe and collar is 307Kg/m.

Estimation of the stress on the pipe
To estimate the stresses by linear meter, the Morison formula is used:

Vertical effort:
$$F_L(t) = \frac{1}{2} C_L . D_0 . \rho_{em} . u(t)^2$$

Horizontal effort: FH=FD+FM=
$$\frac{1}{2} C_D . D_0 . \rho_{em} . u(t)^2 + C_M . \rho_{em} . \frac{\pi . D_0^2}{4} . \left(\frac{du(t)}{dt} \right)$$

with:
ρem density of seawater (1026 kg/m3),
CL lift coefficient set to 0.7 for exposed and 0.37 for buried (according to Det Norske Veritas Holding AS recommended practice for on-bottom stability design of submarine pipelines)
CD drag coefficient set to 0.9 for exposed and 0.27 for buried (according to Det Norske Veritas Holding AS recommended practice for on-bottom stability design of submarine pipelines)
CM drive coefficient set to 3.29 for exposed and 2 for buried (according to Det Norske Veritas Holding AS recommended practice for on-bottom stability design of submarine pipelines)
u(t) maximum speed of current perpendicular to the pipe (m/s).

The table here below presents the efforts obtained along the pipe:

Table 28-Hydrodynamic stresses acting on pipeline sections

Hydrodynamic stresses	N/m	Kg/m
Section AB	22367	-2280
Section BC	23503	-2396
Section CD	45255	-4613
Section DE	25453	-2595
Section EF	25570	-2607
Section FG	15734	-1604
Section GH	14507	-1479
Section HI	13398	-1366

Thus the global apparent weights of the pipeline for each section are:

Global apparent weight	Kg/m
Section AB	-1972.98
Section BC	-2088.77
Section CD	-4306.14
Section DE	-2287.56
Section EF	-2299.56
Section FG	-1296.85
Section GH	-1171.74
Section HI	-1058.79

Table 29- Global apparent weight of pipeline sections

Given that the "residual weight" of stability is inferior to 0, the pipe is NOT stable. It is then necessary to add additional ballast.

We are proposing thus two alternatives to stabilize the pipeline:

A) add rock fill to stabilize the pipeline:
The properties of the proposed rocks are:

Table 30-Rocks characteristics

Rocks characteristics		
Variable	**Value**	**Unit**
Rocks density	2600	Kg/m3
Rocks density in sea water	1575	Kg/m3
Porosity	30%	
Rcks density with porosity	1102.50	m
Outfall apparent weight	-2009.862	Kg/m
Necessary rocks volume	1.8	m3/m

Thus, by adopting the following configurations, the pipeline becomes stable:

Table 31-Needed stabilizing rocks volume per pipeline section

Rocks Configuration		
Section AB	**Value**	**Unit**
Outfall apparent weight	-1972.98	Kg/m
Necessary rocks volume	1.79	m3/m
Section BC	**Value**	**Unit**
Outfall apparent weight	-2088.77	Kg/m
Necessary rocks volume	1.89	m3/m
Section CD	**Value**	**Unit**
Outfall apparent weight	-4306.14	Kg/m
Necessary rocks volume	3.91	m3/m
Section DE	**Value**	**Unit**
Outfall apparent weight	-2287.56	Kg/m
Necessary rocks volume	2.07	m3/m
Section EF	**Value**	**Unit**
Outfall apparent weight	-2299.56	Kg/m
Necessary rocks volume	2.09	m3/m
Section FG	**Value**	**Unit**
Outfall apparent weight	-1296.85	Kg/m
Necessary rocks volume	1.18	m3/m
Section GH	**Value**	**Unit**
Outfall apparent weight	-1171.74	Kg/m
Necessary rocks volume	1.06	m3/m
Section HI	**Value**	**Unit**
Outfall apparent weight	-1058.79	Kg/m
Necessary rocks volume	1.0	m3/m

By applying a factor of safety of 1.5, we get then:

Table 32-Needed stabilizing rocks volume per pipeline section after applying a Factor of safety FS=1.5

Rocks Configuration		
Section AB	**Value**	**Unit**
Necessary rocks volume	2.68	m3/m
Section BC	**Value**	**Unit**
Necessary rocks volume	2.84	m3/m
Section CD	**Value**	**Unit**
Necessary rocks volume	5.86	m3/m
Section DE	**Value**	**Unit**
Necessary rocks volume	3.11	m3/m
Section EF	**Value**	**Unit**
Necessary rocks volume	3.13	m3/m
Section FG	**Value**	**Unit**
Necessary rocks volume	1.76	m3/m
Section GH	**Value**	**Unit**
Necessary rocks volume	1.59	m3/m
Section HI	**Value**	**Unit**
Necessary rocks volume	1.44	m3/m

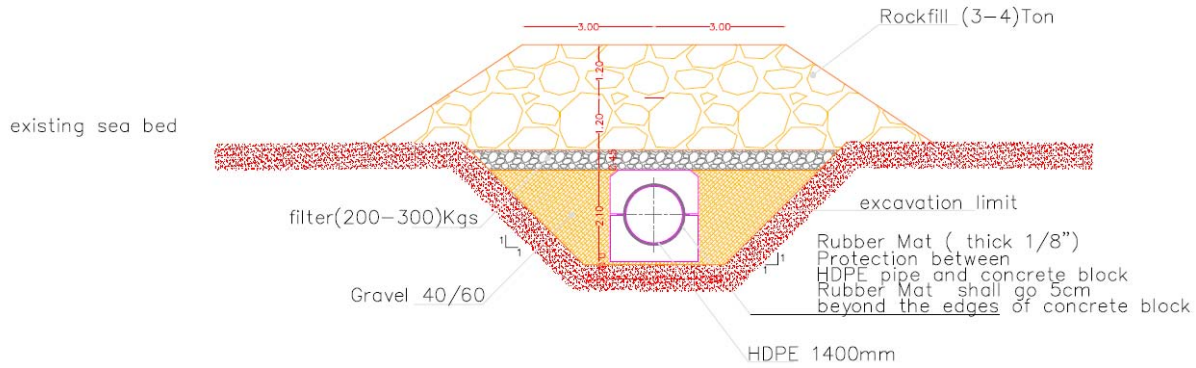


Figure 34-Typical section from A to C

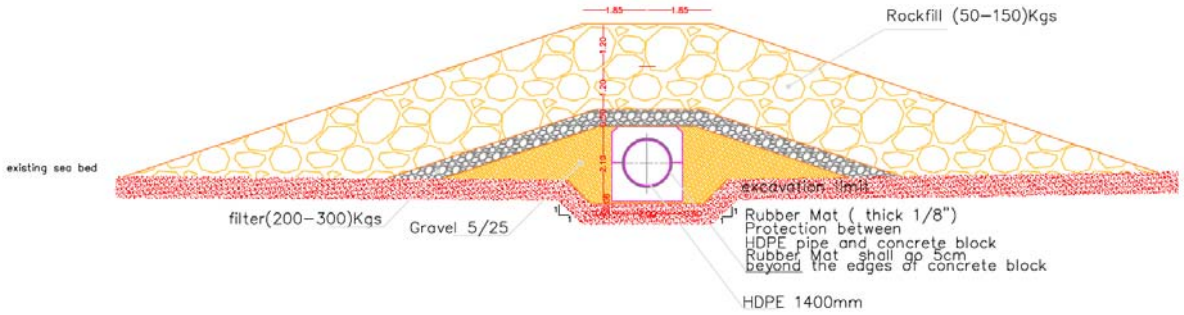


Figure 35-Typical section from C to D

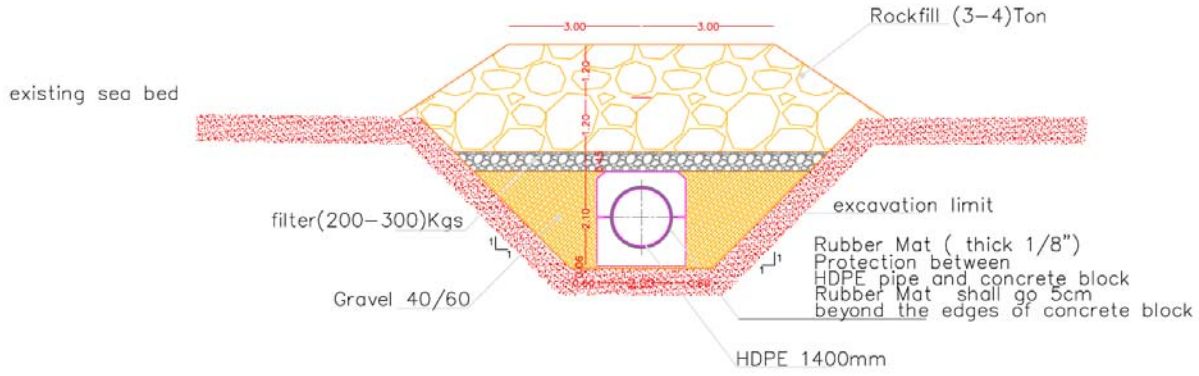


Figure 36-Typical section from D to F

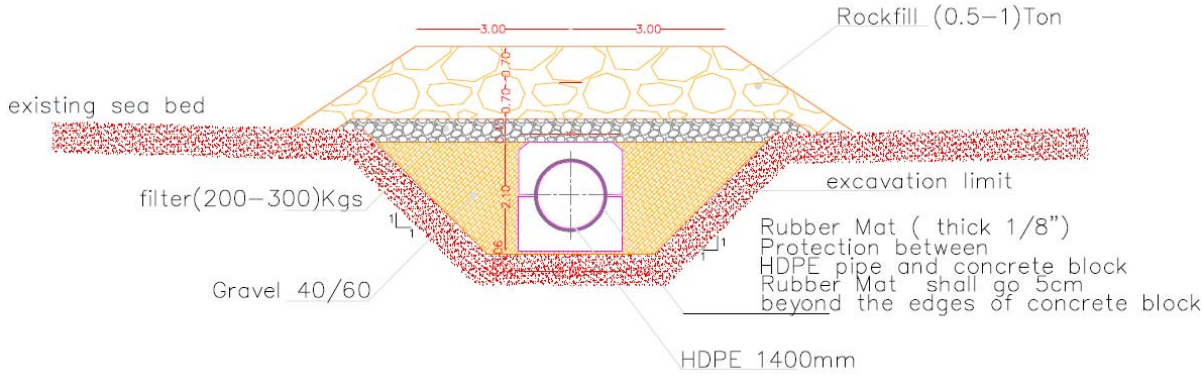


Figure 37-Typical section from F to G

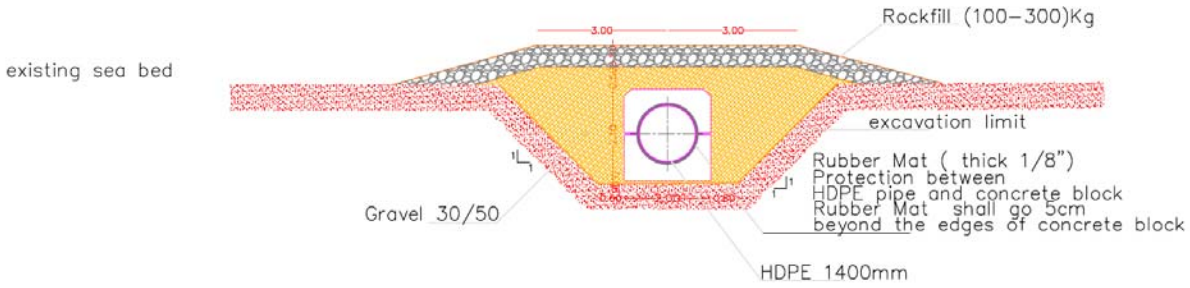


Figure 38-Typical section from G to H

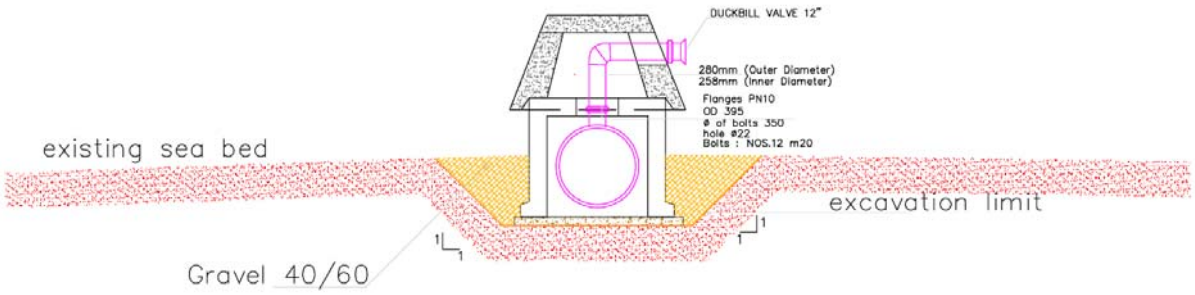


Figure 399-Typical section from H to I

B) add Jumpers and Mattresses (complemented with under water concrete) to stabilize the pipeline:
The properties of the proposed jumpers and mattresses are:

Table 333-Jumpers and Mattress characteristics

Jumper 1 characteristics		
Variable	**Value**	**Unit**
Jumper area	3.430	m2
Jumper length	1	m
Jumper weight in air	8060.5	Kg
Jumper apparent weight	4541.3	Kg
Jumper 2 characteristics		
Variable	**Value**	**Unit**
Jumper area	3.430	m2
Jumper length	0.5	m
Jumper weight in air	4030.3	Kg
Jumper apparent weight	2270.7	Kg

Mattress characteristics		
Variable	**Value**	**Unit**
Mattress volume	1.344	m3
Mattress width	0.4	m
Mattress weight in air	3158.4	Kg
Mattress apparent weight	1779.5	Kg

Thus, by adopting the following configurations, the pipeline becomes stable:

Table 34-Needed stabilizing jumpers and mattresses per pipeline section

Jumpers Configuration			
Section	**Jumper(s) Type(s)**	**Number of jumpers between two consecutive collars**	**Remarks**
AB	2	3	
BC	2	3	
CD	1(nos.2)&2(no.1)	3	One mattress on each jumper
DE	1	3	
EF	1	3	
FG	2	1	
GH	2	1	
HI	-	-	One dome structure on each riser/port

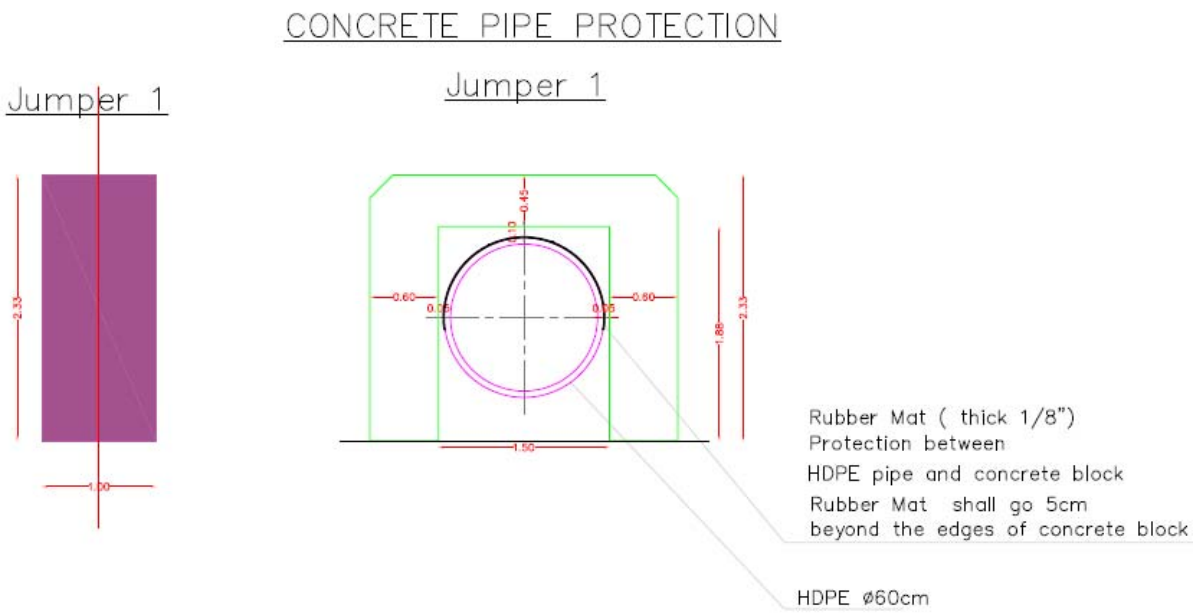


Figure 40-Jumper Type 1

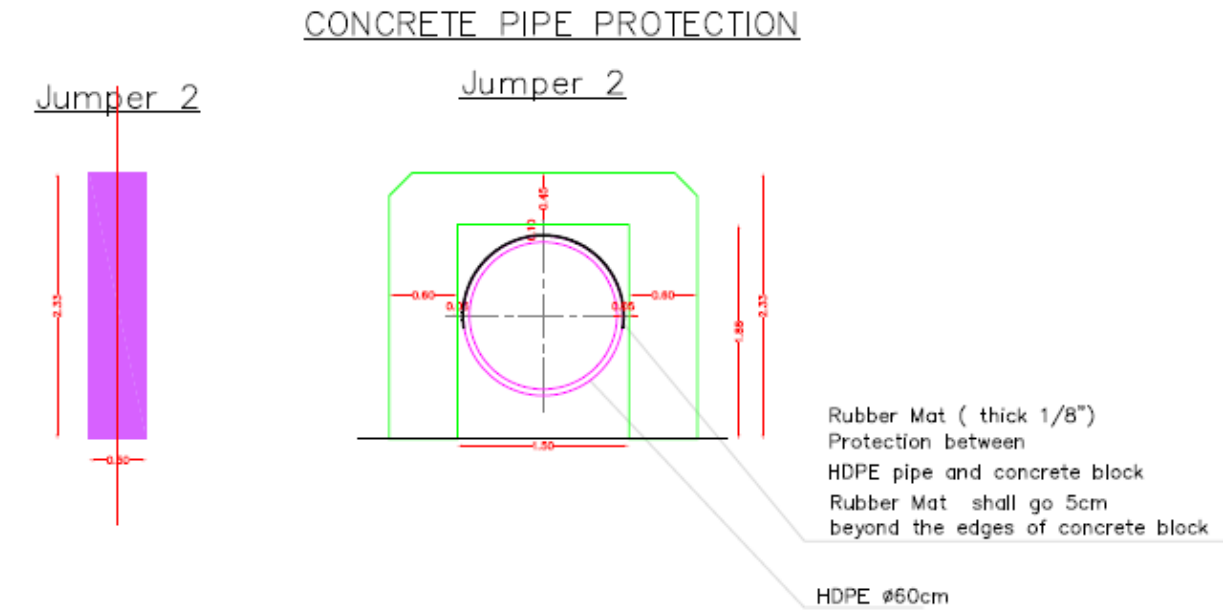


Figure 41-Jumper Type 2

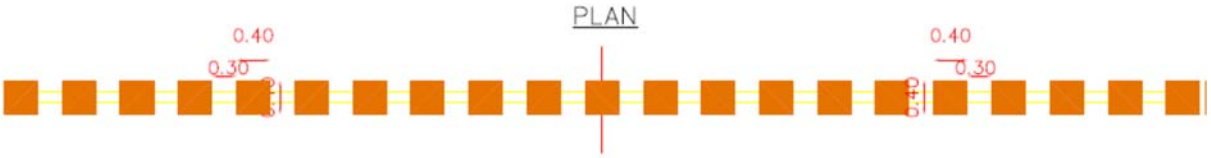


Figure 42-Mattress

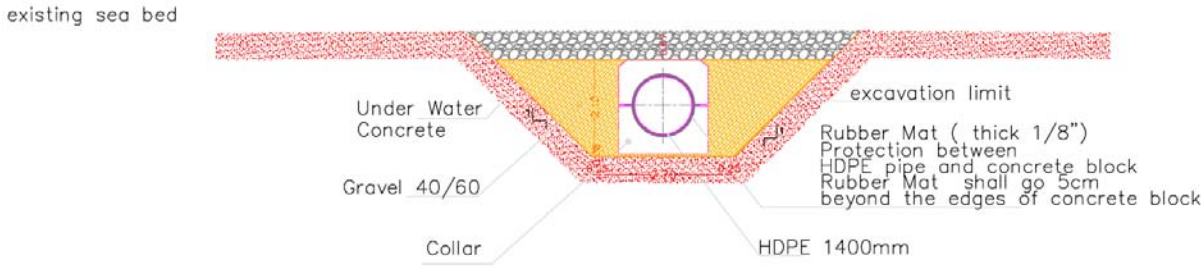


Figure 43-Typical section from A to C and from D to F – Collar Case

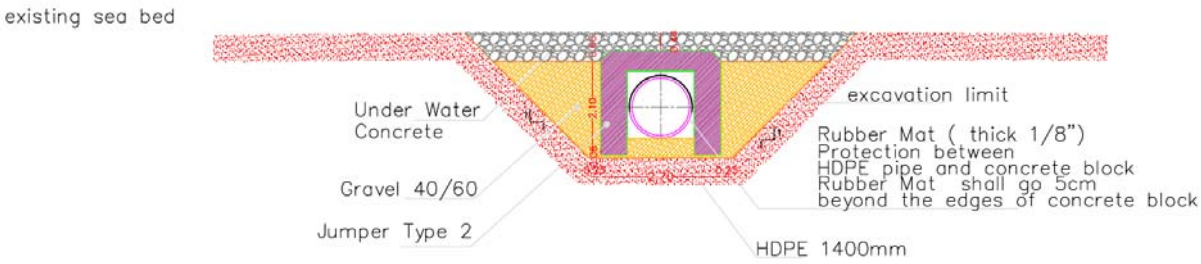


Figure 44-Typical section from A to C and from D to F – Jumper Case

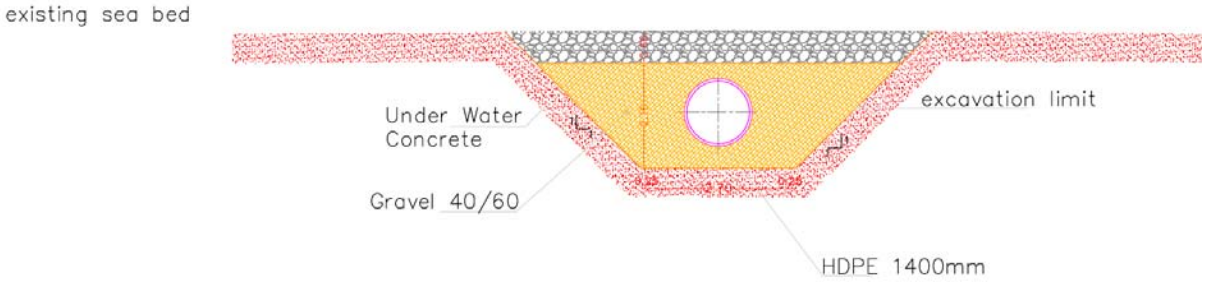


Figure 45-Typical section from A to C and from D to F – Pipe Case

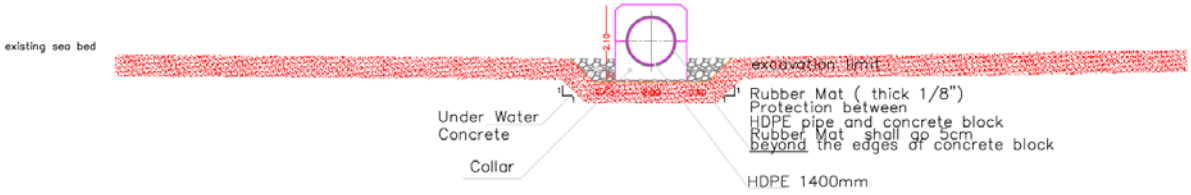


Figure 46-Typical section from C to D – Collar Case

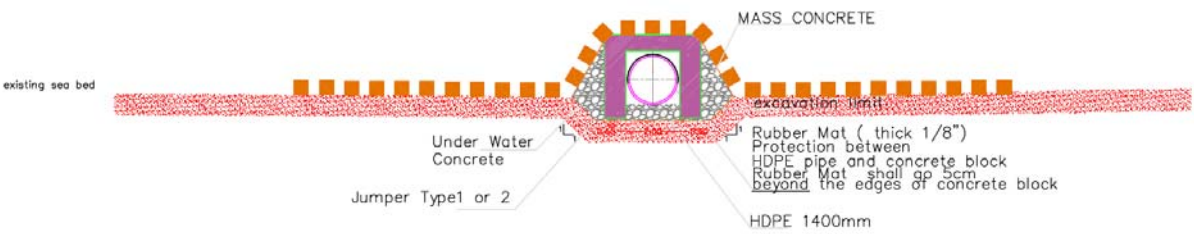


Figure 47-Typical section from C to D – Jumper Case

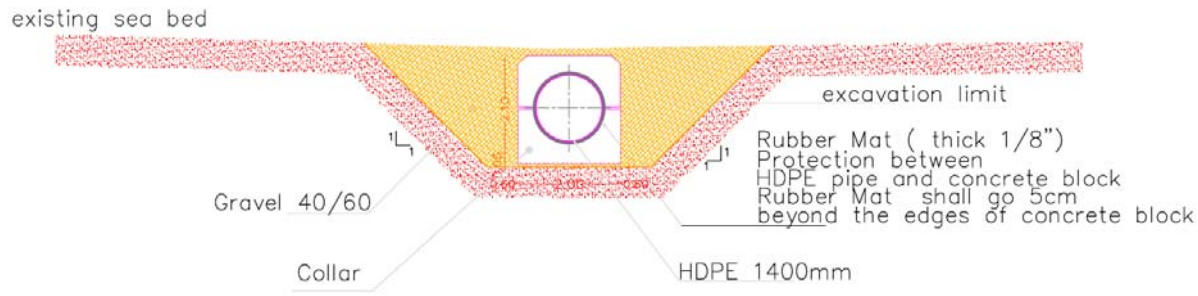


Figure 48-Typical section from F to H – Collar Case

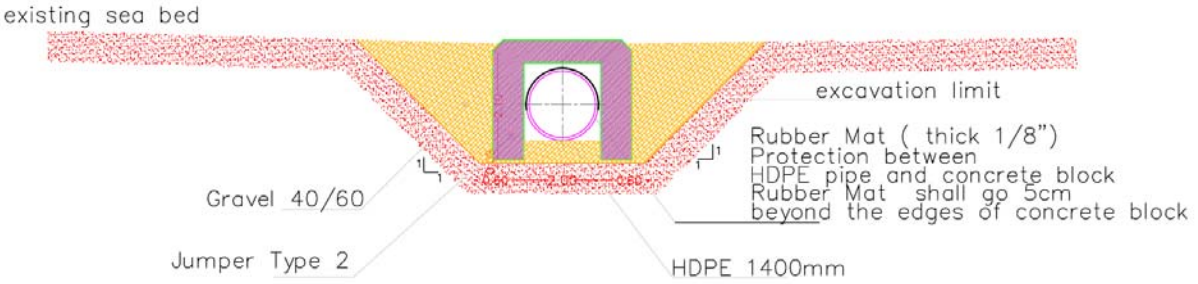


Figure 50-Typical section from F to H – Jumper Case

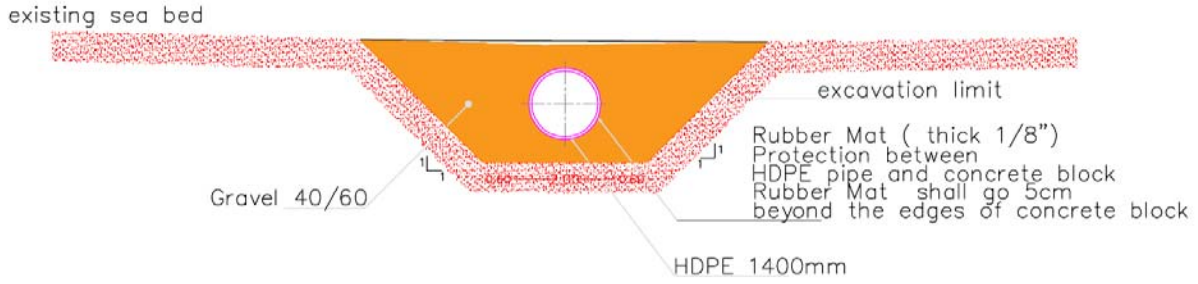


Figure 51-Typical section from F to H – Pipe Case

6.10 Diffuser protection structures

Dome-like structures have been selected for the protection of the diffuser ports and valves.

The adopted structure is composed of two elements: a lower bracket-like reinforced concrete structure with a circular opening to allow the passage of the riser element, and an upper pyramid shaped reinforced concrete element meant at protecting the port itself.

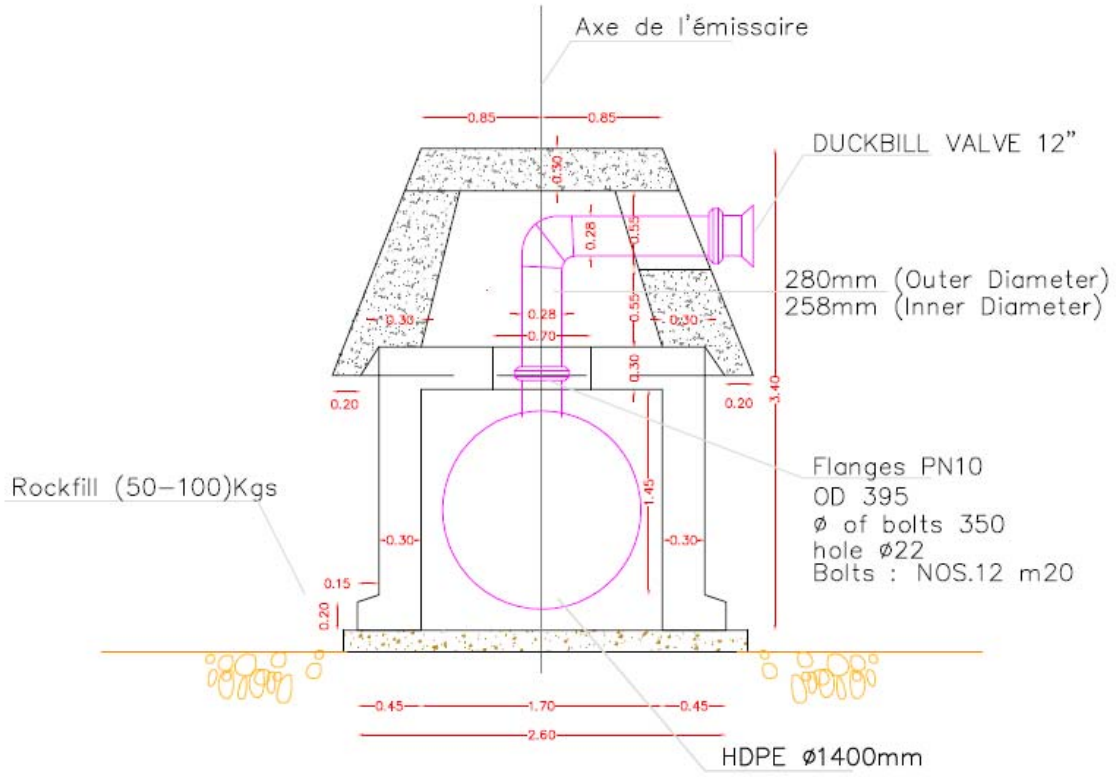


Figure 52-Diffuser protection structure

7. Design of parameters for the sinking process

The critical phase for installations of a PE-pipe is the sinking. The figure below shows the situation during sinking of a PE-pipeline.

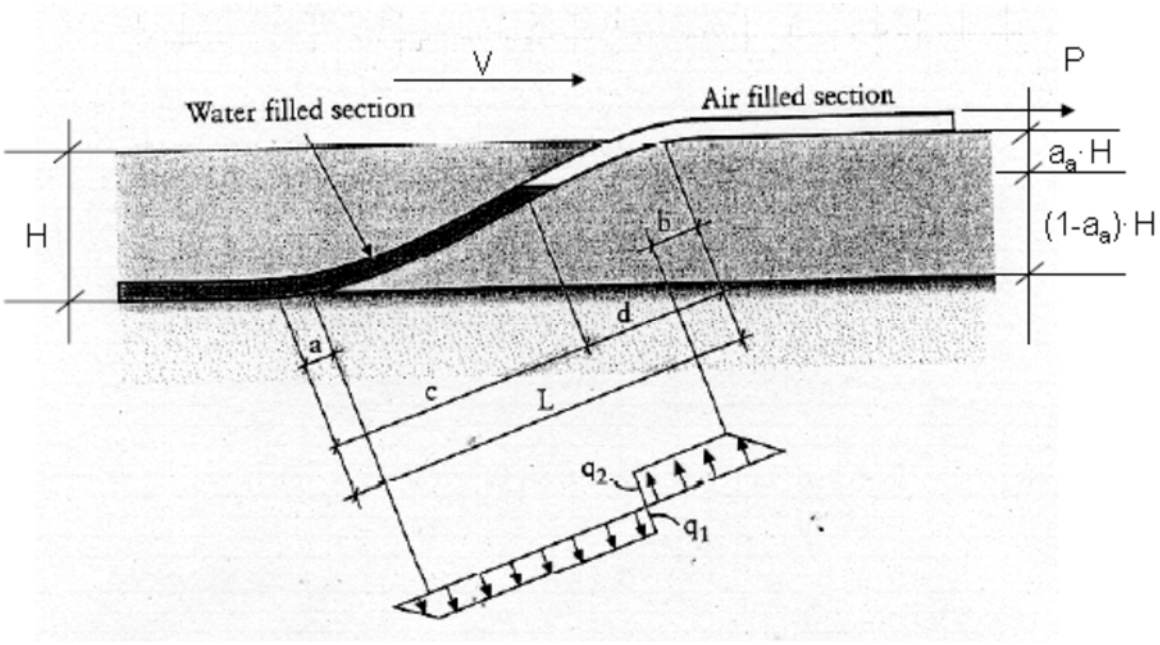


Figure 4053- Sinking of a PE pipeline

To carry out a safe installation we should consider the balance between the forces acting downward (q1)
and the forces acting upwards (q2). The downward forces are created mainly by the concrete weights on the pipeline and the upward forces are due to buoyancy of the air filled section.

To start and to continue the sinking process, the downward forces must be a little greater than the upward forces.
To control this difference is essential and the main challenge during sinking.
We must try to avoid acceleration forces on the system. This can be controlled by recording the sinking speed (v) and regulating the internal pressure (p).
If the speed is increasing, we can increase the air pressure and vice versa.
For regulation of the air pressure we apply valves and compressor.
The most critical situation for the pipeline regarding damage, is buckling at the sea surface or at the bottom due to "exceeded" buckling radius.
To secure a sufficient radius it is necessary to apply a pulling force (P) in the end of the pipe.

The following parameters are available for control and regulations during sinking :
- Air pressure (p)
- Pulling force (P)
- Sinking velocity (v)

In order for the installation to be successful from the pipeline's point of view, we need to carry out a slow sinking operation.

7.1 Internal air pressure

The internal pressure is dependent of loading from the concrete weights as calculated in the previous sections. The important parameter is the air filling rate, aa:

$$a_a = (w_{cw} + w_{pipew})/(\Pi d^2 \gamma_w/4)$$

With:

w_{cw} = weight of concrete collar in sea water
w_{pipew} = weight of pipeline in sea water per meter run of pipe
d = internal diameter of pipe
γ_w = specific gravity of sea water.

To obtain an air filled section which will balance the weight of the concrete collars, we need to apply an internal pressure (p) in the pipelines.

$p = a_a$ Hwith H being the water depth.

As it can be inferred from this formula, the internal pressure is dependent upon water depth. This means that we have to increase the pressure as the depth increases. The compressor needs to have the capability to deliver sufficient air against a pressure corresponding to the maximum depth including drop of pressure within the transmission hoses.
Knowing the longitudinal profile of the outfall pipeline, we can calculate the balancing pressure at each point. This exercise shall constitute the basis for a successful installation. To ease identification of sections during sinking, the pipeline shall be marked at fifty meters intervals.

We have computed the internal pressure for a 1400mm OD (PE80 SDR26 PN6.3) pipeline with a loading degree of concrete weights corresponding to an air filling rate of 50% for 5, 10, 15 and 20m depths:

Table 35-Pipeline internal pressure

Internal pressure	
Depth (m)	**P (bar)**
5	0.25
10	0.5
15	0.75
20	1

7.2 Pulling Force

The pulling force at the end of the pipe is applied to control the pipe's position and to increase the bending radius in the S configurations as can be seen in the figure here below:

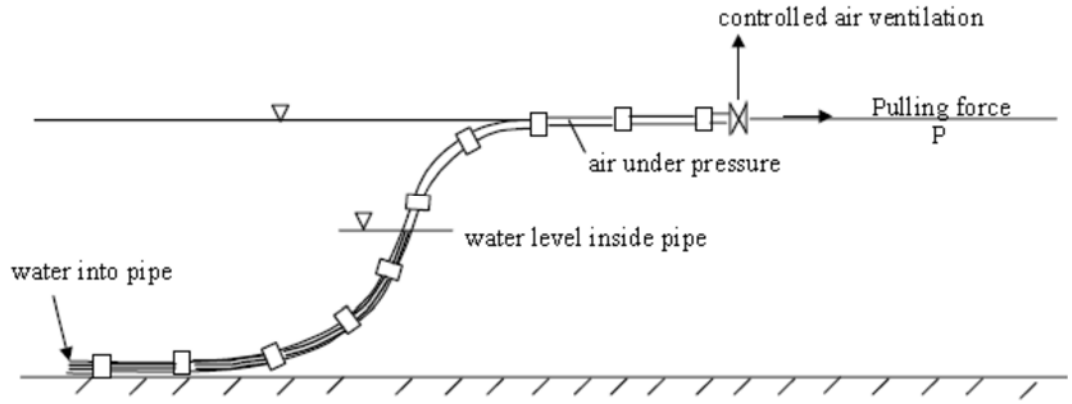


Figure 54- Sinking "S" configuration

If the loading percentage is less than 50 %, which is normally the case, the critical radius will be at the sea surface. Otherwise it will be at the seabed.

To carry out a proper calculation of the sinking process is complicated and must be done by computer programs. However, there is a simple method to find an estimate for the pulling force. This method is based on the chain link theory and is valid for deep water. The figure below shows the situation.

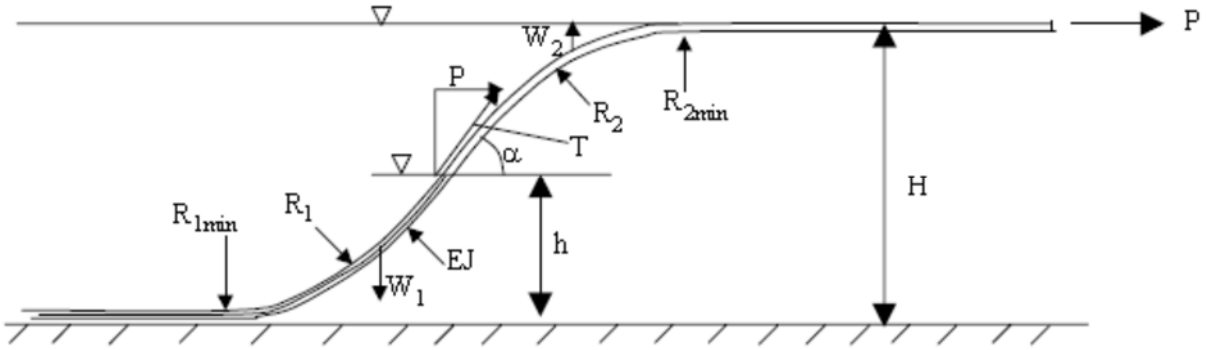


Figure 55-The chain-link theory

With:

H = depth (m)
h = internal water height (m)
w_2 = net buoyancy in air filled section (N/m)
w_1 = net weight of water filled section (N/m)
P = pulling force (N)
T= tension force in turning point (N)
α = angle between pipe axis and horizontal in turning point (º)
R_1 = bending radius in water filled section (m)

R_2 = bending radius in air filled section (m)
EJ = pipe stiffness

Moreover, the following mathematical relations can be developed:

$R_{1\,min} = P/w_1$
$R_{2\,min} = P/w_2$
$T = P + w_1 \cdot h$

$\cos\alpha = P/(P+w_1.h)$

As mentioned earlier $w_2 > w_1$ if the design air filling rate is less than 50 %. The radius R_2 at surface will be critical. In this case, the relationship between w_1 and w_2 can be approximated by:

$w_2 = w_1 (1 – a_a)/a_a$

with $a_a > 20\%$

Deep water is defined as: H > 12 which is our configuration.

In shallow water (start phase of the sinking) it is impossible to apply a force in the end before the pipe is connected to a fixed installation. When submerging the end for connecting the pipe, we have to check that the bending radius is greater than the buckling radius.

The allowable bending ratio R/D for an SDR26 pipe is equal to 34.

We have:
$R = L^2/(2H)$

With:

R = bending radius
L = submerged length of pipe or “cantilevered” section
H = connection depth

We can find the necessary pulling force using the critical radius defined above: R_{min} = 47.6m

Several computations are carried out for air fill rate increments of 5% starting at 20% and going up to 50%:

Table 36-Pipeline weight and buoyancy

air filling rate, a_a	w_1 (N/m)	w_2(N/m)
20%	2639	10556
25%	3299	9896
30%	3959	9237
35%	4618	8577

40%	5278	7917
45%	5938	7257
50%	6598	6598

Table 37-Pipeline pulling and tension forces

air filling rate, a_a	P_1 (N)	P_2 (N)	T(N)
20%	53837	215347	268128
25%	67296	201887	263740
30%	80755	188428	257703
35%	94214	174969	250017
40%	107673	161510	240682
45%	121132	148051	229696
50%	134592	134592	217062

Table 38-Stresses in pipeline

air filling rate, a_a	Tension Stress (Mpa)	Internal Stress (Mpa)	Total Stress (Mpa)
20%	1.18	0.31	1.49
25%	1.16	0.39	1.55
30%	1.13	0.47	1.60
35%	1.10	0.55	1.65
40%	1.06	0.63	1.68
45%	1.01	0.70	1.71
50%	0.95	0.78	1.73

The optimal air filling rate according to tension is at the equilibrium of net buoyancy and net water filled weight of 50%. If this rate is selected, than the minimum tension during sinking at 20m depth shall be equal to 217kN or 22t.

Forces at the surface

During the positioning of the outfall, the pipeline is subjected to hydrodynamic forces at the surface. These forces can damage the pipe despite their small magnitudes.

This section evaluates the stress in the pipe caused by such forces.

First the drag force is calculated from the waves. Then the pipe stress is computed from the previous results and the mooring spacing.

We have thus:

Table 38-Pipeline Drag force vs. mooring spacing

Hs (m)	Drag force (kN/m)	Mooring spacing (m)		
		50	100	150
0.0	0.00	0.95	0.95	0.95

0.2	0.06	1.1	1.4	1.9
0.5	0.22	1.3	2.3	4.0
0.8	0.41	1.6	3.5	6.7
1.0	0.55	1.8	4.4	8.7
1.2	0.70	2.0	5.3	10.8
1.5	0.94	2.4	6.9	14.2
1.8	1.20	2.8	8.5	17.9
2.0	1.38	3.1	9.6	20.4
2.2	1.56	3.4	10.8	23.1
2.5	1.85	3.9	12.6	27.2

With the absence of a drag force, the stress is set around 0.95MPa. This stress is induced by the pulling force set in the pipeline during the sinking process.

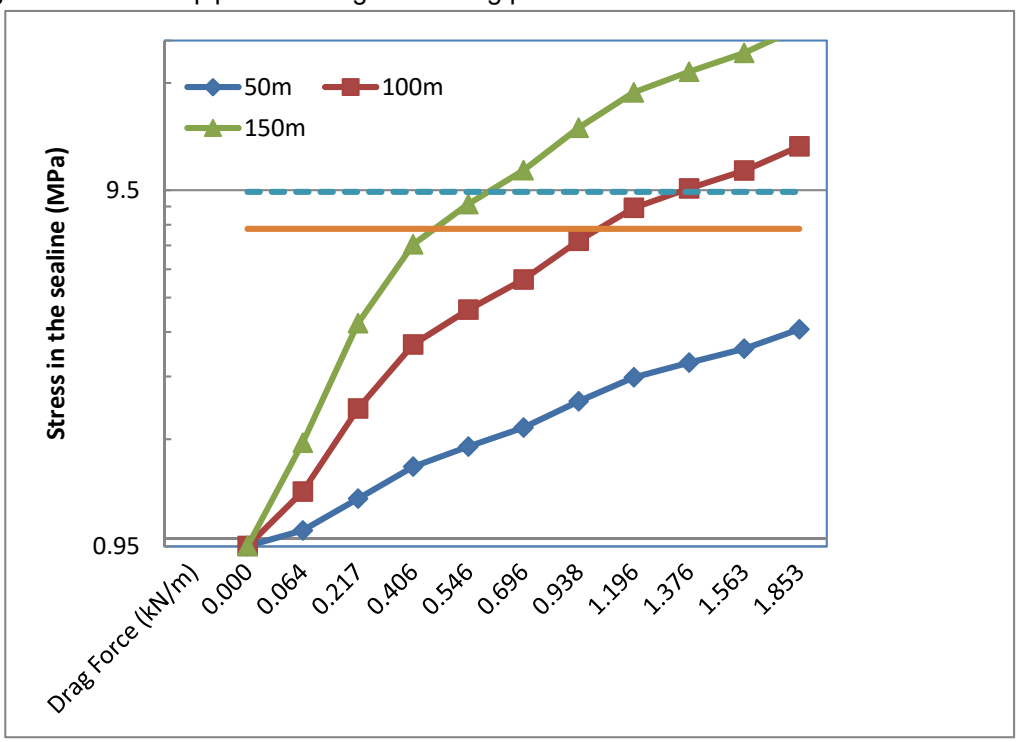


Figure 56-Stress in sea line vs mooring spacing under a tension of 217 kN

The dotted blue line represents the maximum acceptable stress in the pipe as specified by the pipe manufacturer. However, for safety reasons, we have used a reduced stress shown in orange on the graph.

By looking at the graph, we can see that for a mooring spacing of 100m , the maximum bearable drag force is around 0.95kN/m which corresponds to H_s = 1.5m.

8. Diffusion Modeling

8.1 Introduction

The unsanitary disposal of wastewaters throughout Lebanon is one of the most pressing environmental stresses facing the environment at the national scale.
Currently, the city of Beirut lacks wastewater treatment plants and openly discharges part of its untreated municipal wastewaters into the Mediterranean Sea. The principal environmental consequence related to disposal of untreated sewage is due to its potential to degrade the physical, chemical and biological characteristics of the receiving water body. The degree of degradation is highly dependent on the total volume of the wastewater being released, its characteristics, the dilution rate prior to discharge, and the characteristics of the receiving waters. The effect of wastewater on the environment is also highly dependent on the geometric installation of the discharge outfall.
Currently, the Council for Development and Reconstruction (CDR) is embarking on an effort to reduce the environmental impact of the Sultan Ibrahim lifting station overflow discharge at Saint Simon (Jnah) by funding the construction of an emergency submerged outfall along at this location to aid in diluting the final disposal of the wastewater effluent. As such, proper outfall sitting and design is essential to utilize the natural assimilative capacity of the coastal waters at this location.

8.2 Objectives

The main objective of this study is to assess the performance of the proposed submerged Sultan Ibrahim outfall by comparing the effectiveness of various discharge rates on diluting the wastewater effluent from the present existing discharge point. For this purpose, the effluent characteristics, the hydrodynamic characteristics in the area have to be defined, as well as the seawater and sediment quality. Various simulation scenarios accounting for different submerged outfall layouts shall be assessed to determine the optimal outfall structure capable of achieving the environmental and health standards that meet the guidelines recommended by the Ministry of the Environment (MOE), as well the specifications appended to the project's tender documents. A parametric sensitivity analysis shall be conducted to evaluate the effect of various input parameters on the simulated results. Finally, the most effective outfall orientation shall be identified. The general methodology to be adopted in the study is presented in the diagram hereafter.

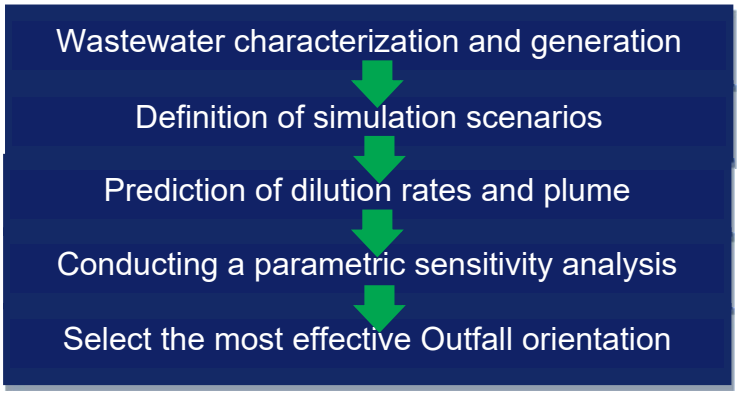


Figure 57-Study methodology

8.3 Saint Simon (Jnah) Study Characteristics

8.3.1 Ambient Data

Based on the data presented in section 3.2, we have selected herein the following data for our models:

- A pessimistic scenario with an ambient velocity of 0.05m/s, and
- An ordinary scenario under normal ambient velocity of 0.25m/s

Both scenarios have been carried out under a wind speed of 9m/s.

8.3.2 Effluents and Discharge Data

The effluent coming from the overflow of the Sultan Ibrahim lifting station will be discharged through a submerged multiport outfall that runs at an angle with the shoreline. The proposed configuration of the multiport outfall will have single port per riser. The outfall main diffuser line will have an external diameter of 1400 mm and consists of 20 risers with 1 nozzle/port per riser. Risers of 280 mm external diameter will be spaced at 8 m intervals, i.e. more than 25% of diffuser bottom elevation in order to avoid plume overlap leading to a total diffuser length of 200m. The port height (centerline) will be 1.25 m to retard bottom attachment and reduce potential adverse impacts on the benthic community in the area, while the external port diameter will be 280 mm. All ports shall be equipped with flanges in order to enable bolting on of duckbill valves. On the other hand, the design flow rate is presented in the table 40 :

Table 39-Expected flows

Expected flows during	***(m3/s)***
Peak flow rate	*1.5*

8.4 Discharge Modeling

8.4.1 Model Description and Capabilities

The latest version of CORMIX (10.0 GTD) model has been used to simulate the development of the plume resulting from the continuous discharge of wastewater effluent from the Sultan Ibrahim discharge. CORMIX is a United Stated Environmental Protection Agency (USEPA) approved simulation and decision support system that is used for assessing the environmental impacts resulting from point source discharges within their mixing zones. The model is subdivided into three separate sub-systems namely: CORMIX 1 for the analysis of submerged single port discharges, CORMIX 2 for the analysis of submerged multi-port diffuser discharges, and CORMIX 3 for the analysis of buoyant surface discharges. CORMIX is capable of simulating an effluent dilution rate and the plume trajectory from the point of release into the far-field by analyzing and combining the solutions of several flow patterns. It is a versatile model and has been applied to a broad range of environmental mixing problems from small to large discharges in a variety of receiving water bodies including small streams, large rivers, lakes, reservoirs, estuaries and coastal waters. The model is capable of classifying a given discharge into the environment into 80 generic flow classifications based on the type of plume and its expected development as a result of the discharge configuration and the ambient conditions. The following table summarizes the most encountered flow classes in each sub-system.

Classes incorporated within CORMIX

Table 41-CORMIX flow classes

Flow class	***Description***
CORMIX 1	
Classes S	*For flows trapped in a layer within linear stratification*
Classes V and H	*For positively buoyant flows in uniform density layer. Vertical or horizontal*
Classes NV, NH	*For negatively buoyant flows in uniform density layer. Vertical or horizontal*
Classes A	*For flows affected by dynamic bottom attachment*
CORMIX 2	
Classes MS	*For flows trapped in a layer within linear ambient stratification*
Classes MU	*For positively buoyant flows in a uniform density layer*
Classes MNU	*For negatively buoyant flows in a uniform density layer*
CORMIX 3	
Classes FJ	*For free jet flows without near-field shoreline interaction*
Classes SA	*For shoreline-attached discharges in a cross flow*
Classes WJ	*Wall jets/plumes from discharges parallel to shoreline*
Classes PL	*Upstream intruding plumes*

The predictive capability of CORMIX depends on how well one can characterize the input parameters. CORMIX 1 applies to better than 95 percent of submerged single-port designs, CORMIX 2 to better than 80 percent of multi-port diffusers and CORMIX 3 to better than 90 percent of buoyant surface discharges. The lack of applicability is usually the result of highly non-uniform ambient flow conditions that are prone to local re-circulating flows. Other non-applicable cases may arise due to complex discharge geometries.

8.4.2 Definition of Simulation Scenarios

The algorithm here below hereafter presents the simulation methodology that should be adopted in optimizing the proposed Sultan Ibrahim outfall to assure appropriate dilution rates that meet not only

MED POL II, but also the more stringent criteria set by the tender, seawater quality standards, which are consistent with the standards promulgated by the Lebanese Ministry of Environment (Decree 1/52).

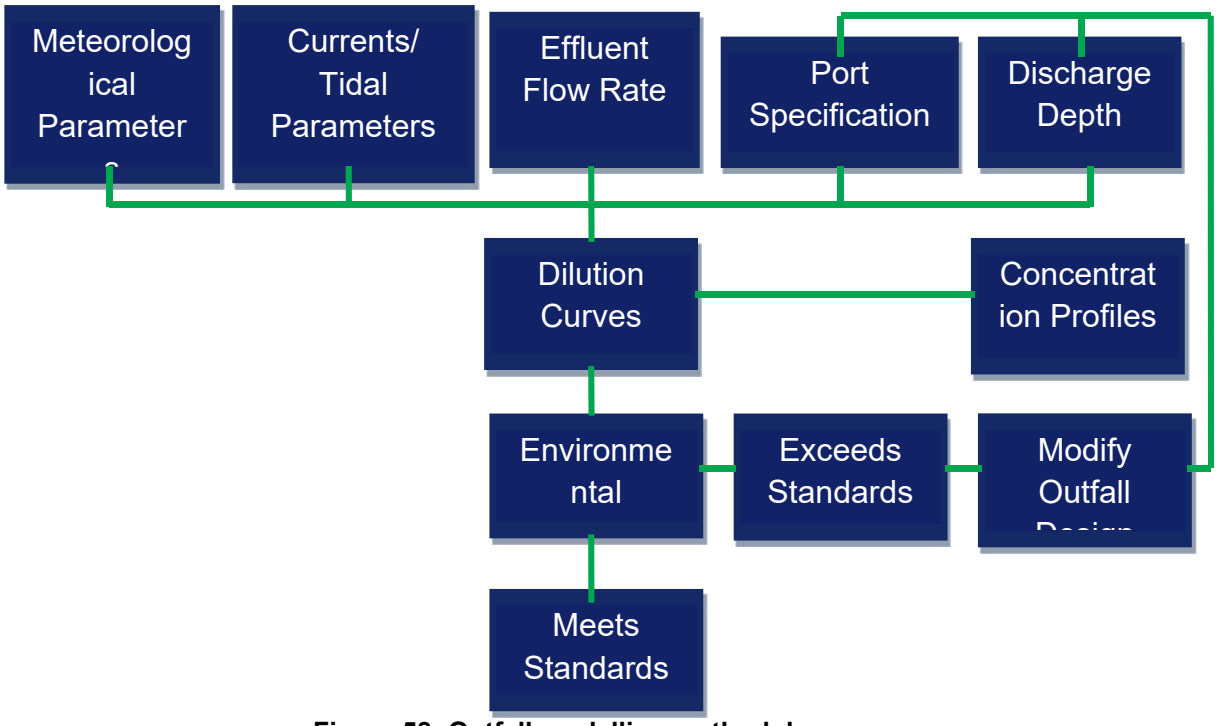


Figure 58- Outfall modelling methodology

According to the prevailing environmental standards set by the Ministry Of the Environment (MOE), the discharge depth shall not be shallower than 15m. Moreover, the initial dilution shall be of 1:100. However herein, because of the emergency nature of the overflow, a more reasonable 1:50 initial dilution criterion has been sought.
The EPA defines a mixing zone called the Zone of Initial Dilution (ZID) which, according to EPA's 301(h) regulations, extends for one water depth around the diffuser.

Based on the constraints above and the predominant current direction and starting from the shoreline, a (920m + 192m =) 1120m long line forming a 30degree angle with the principal current direction (N255) was drawn. Then, from the results of the bathymetric survey that was carried out, it was found that the corresponding depth of the diffuser end was 18.12m. Thus, as a first trial, only this geometrical configuration has been verified herein.
Both scenarios indicated above have been verified under different sewage flow rates.
Hereafter are the proposed outfall specifications:

Table 42-Outfall configuration

Outfall	***Parameter***	***Value***
Multi-port	*Number of risers*	*20*
	Riser spacing	*10 m*

	Diffuser length	*200 m*
	Port per riser	*1*
	Port diameter	*280 mm (OD)*
	Total discharge ports	*20*
	Diffuser arrangement	*Directed at N255*
	Angle between diffuser and ambient current	*30^{o} (N255 direction)*
	Nozzle orientation	*Perpendicular to pipeline - N*
	Discharge	*Horizontal discharge*

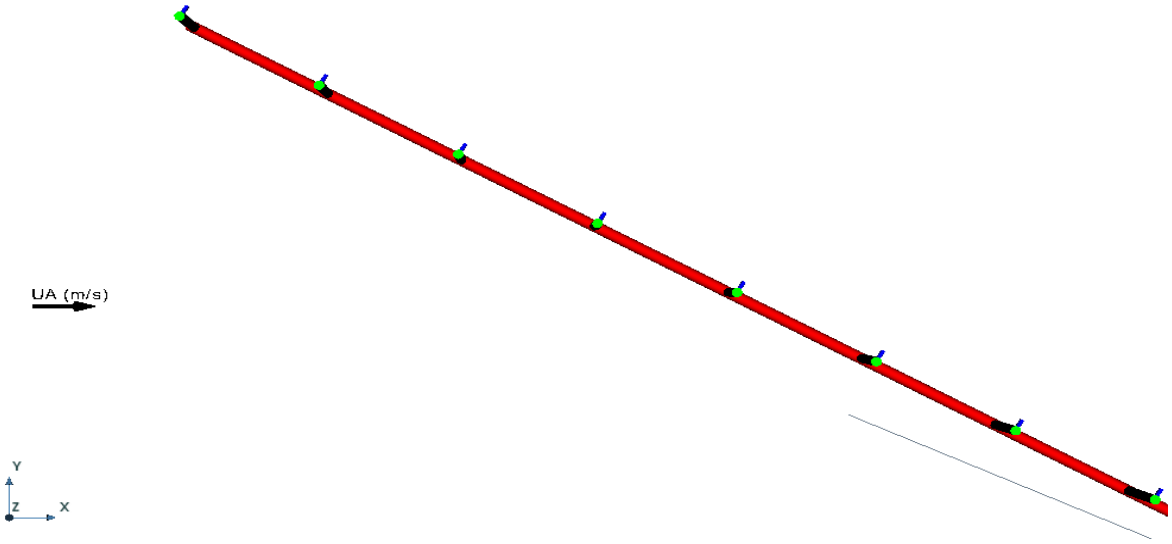


Figure 59-Outfall 3D representation

Simulated scenarios and sub-scenarios

Table 40-Adopted scenarios

Scenario	***Sub-scenario***	***Flow rate (m^3/sec)***	***Current direction***	***Current speed (cm/sec)***
S1 Multiport Discharge at a depth of 16 m	*S 1.1* *S 1.2*	*1.5* *1.5*	*N255* *N255*	*0.05* *0.25*

8.4.3 Simulation Results

The results of the optimization simulations in terms of initial dilution versus downstream distance in each sub-scenario along with the achieved dilution rates are presented hereafter.

The following table presents the required downstream distance to achieve the initial dilutions of 1:50 and 1:100.

Downstream distances to achieve 1:100 and 1:50 dilutions

Table 41- Achieved dilutions

Sub-scenario		*Downstream distance to achieve 1:50 Dilution (m)*	*Downstream distance to achieve 1:100 Dilution (m)*
Multiport Discharge depth 20m	*S1.1*	*17.8*	*124*
	S1.2	*18.1*	*73.2*

The results presented in the table above indicate that under both flow rates the selected discharge scenario considered will achieve the required initial dilution criterion of 1:50.
Higher current velocities have not been considered herein as usually bacterial concentrations decrease when the ambient current speed increases, thus retarding bottom and bank attachment that hinder dispersion. This is primarily related to the fact that the area shows a predominant current that runs parallel to the existing shoreline, curbing the early attachment of the plume to the nearby bank.
Furthermore, increasing the discharge flow rate resulted in a decrease in the achieved dilution rates as a result of the increase in the pollution load. It is important to note that, in our opinion, the initial dilution criteria has been lowered to 1:50 because of the relative shortness of the outfall (Usually achieved dilution rates are closely related to discharge depth because deeper waters enhance dilution rates by retarding the attachment of the plume to the water surface and adjacent bank).

Scenario S1.1

The first set of simulations, for scenario S1.1, shows that the initial dilution criterion is met. However, the plume slightly attaches to the shore line when the current magnitude stays very low for a relatively long period of time (nearly 4 hours) which is improbable.
The run shows that the upstream intrusion length is equal to 892.63m, the thickness in intrusion region is equal to 1.19m, the half-width at downstream end is equal to 1250.25m and that the thickness at downstream end is equal to 1.42m.In this case, the upstream intrusion is very large, exceeding 10 times the local water depth. This may be caused by a very small ambient velocity, in combination with large discharge buoyancy. The flow is hydro-dynamically classified as MU1V: submerged positively buoyant multiport diffuser discharge in uniform deep layer under weak current. The discharge configuration is hydro dynamically "stable",that is the discharge strength (measured by its momentum flux) is weak in relation to the layer depth and in relation to the stabilizing effect of the discharge buoyancy (measured by its buoyancy flux). The buoyancy effect is very strong in the present case. The effluent density is less than the surrounding ambient water density at the discharge level. Therefore, the effluent is positively buoyant and will tend to rise towards the surface. Thus, some distance the discharge buoyancy becomes the dominating factor (plume-like). The plume deflection by the ambient current is still weak. After impingement the flow spreads in all directions (more or less radially) along the layer boundary. In particular, the flow spreads some distance upstream against the ambient flow, and laterally across the ambient flow. This spreading is dominated by the strong buoyancy of the discharge. The plume predictions prior to boundary impingement and wedge formation will be acceptable, however. Plume in unbounded section contacts nearest bank at 633.24 m downstream (outside the regulatory mixing zone).
Moreover, regulatory mixing zone boundary is within the Near-Field Region, and the plume downstream distance meets or exceeds the regulatory value of 300m.
In this design case, the diffuser is located relatively close to the shore. Some lateral boundary interaction occurs at end of the near-field. This is related to a design case with a VERY LOW AMBIENT

VELOCITY. One of the options is to consider locating outfall further away from bank or shore; however the adopted length is already longer than originally planned (908.6 vs 400m).

The graphical outputs for scenario S1.1 are shown here below:

Dilution Isolines: The initial dilution criterion of 1:50 is met while the 1:100 is not.

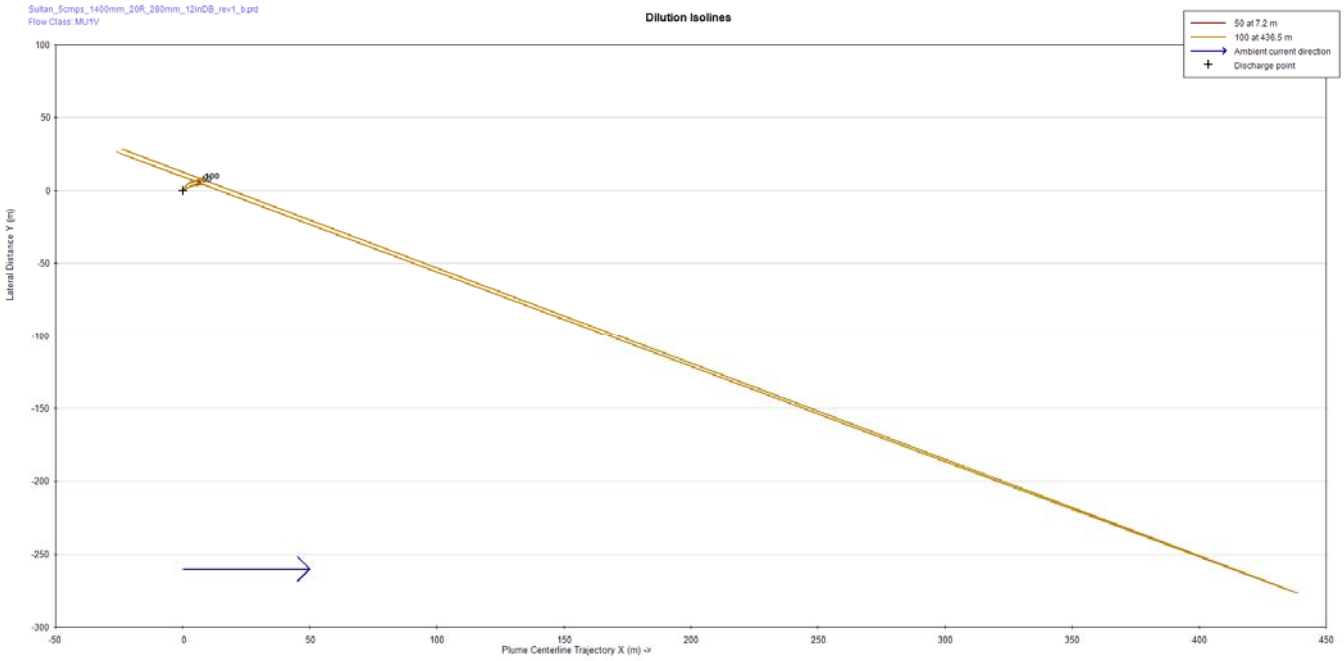


Figure 60-Dilution isolines under sub-scenario S1.1

3D View of Dilution:

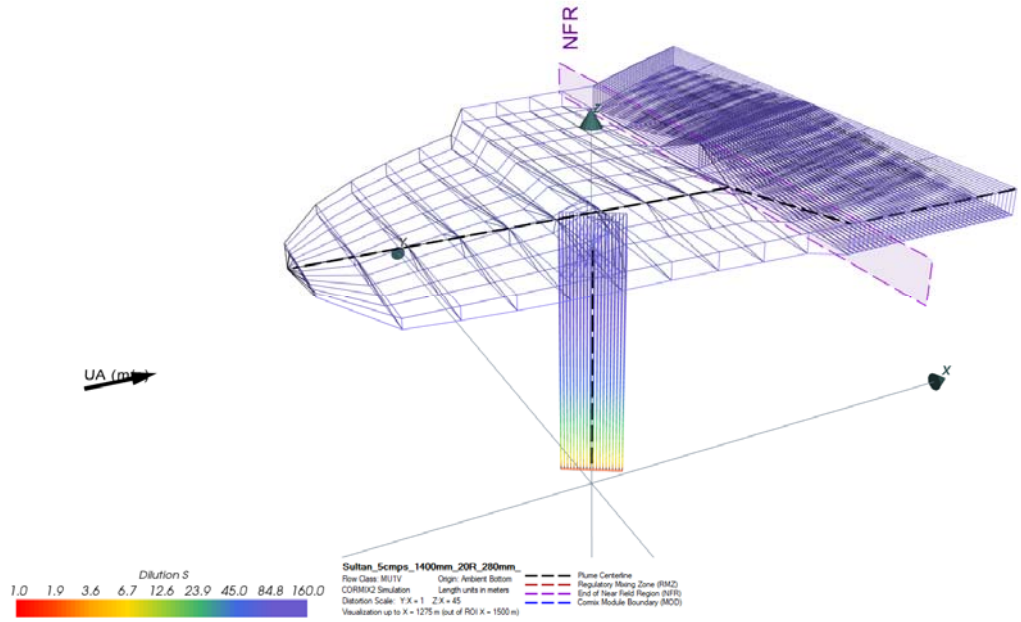


Figure 61-3D view of dilution

3D View of Discharge Excess:

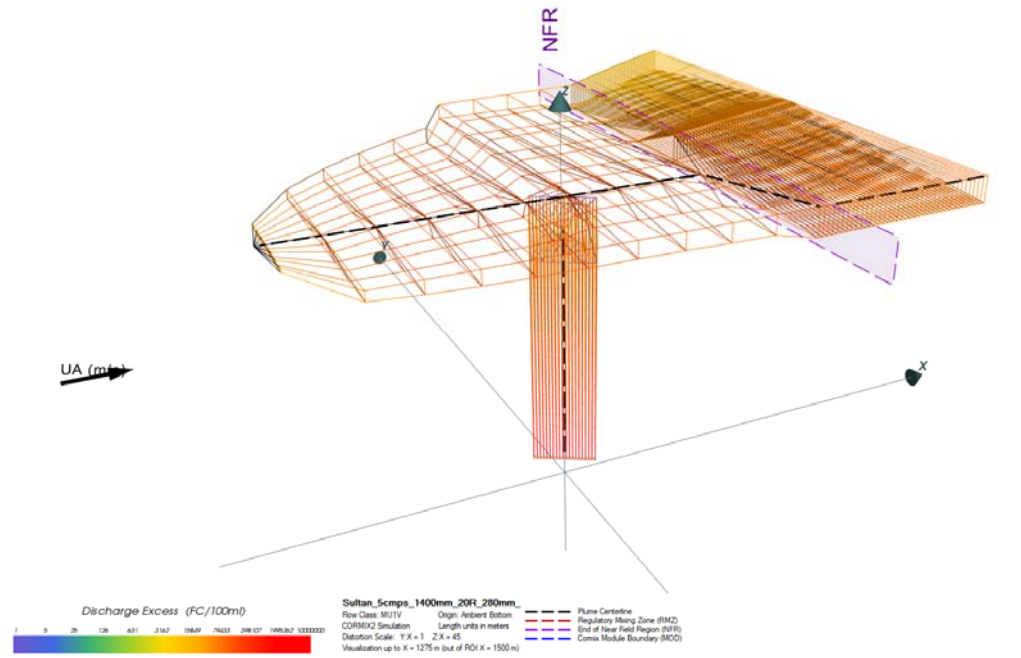


Figure 41- 3D view of discharge excess

Dilution Plan View:

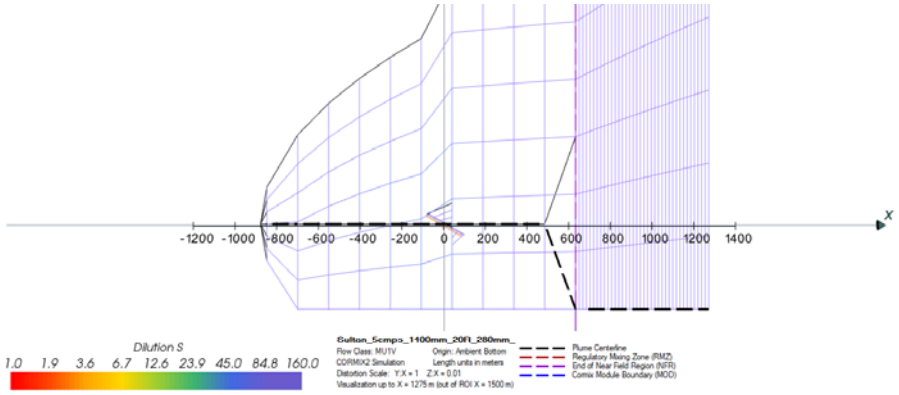


Figure 63-Plan view of dilution

Discharge Excess Plan View: part of the plumes travels backward towards the shoreline (some 892m counted from the diffuser end), but stays away from it. However, side spreading contacts the shoreline (shown in gray on the frontal view here below) North of the discharge location.

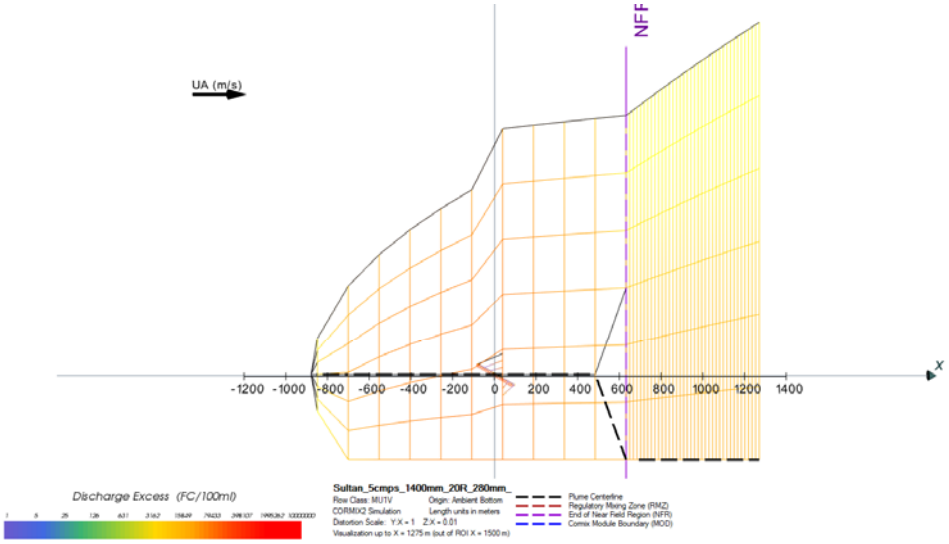


Figure 64-Plan view of discharge excess

Dilution Side View:

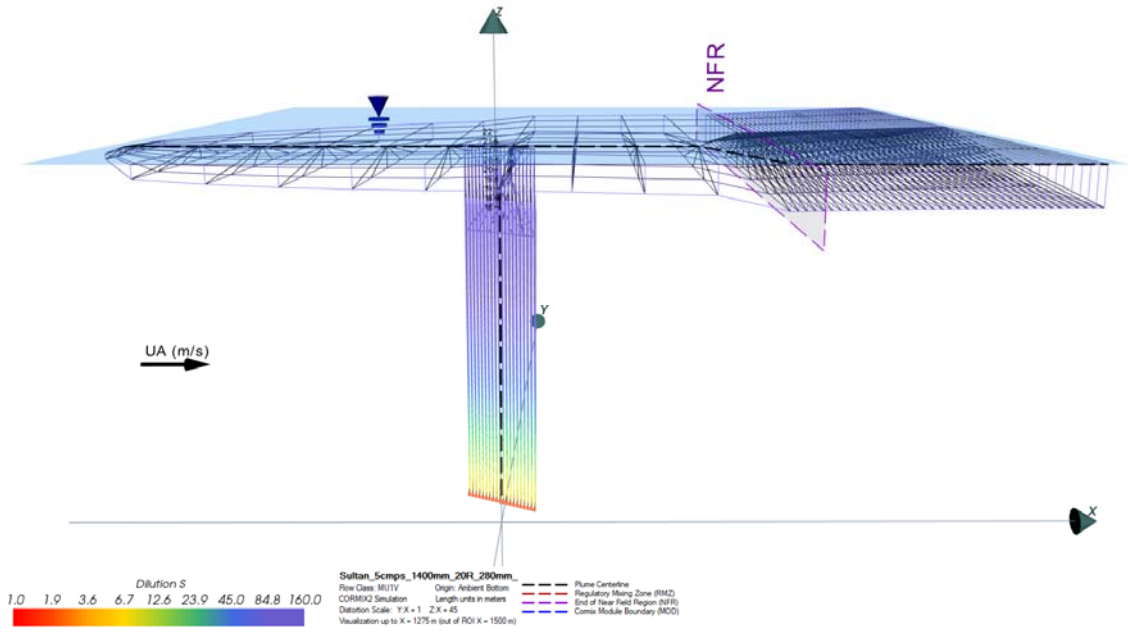


Figure 4265-Side view of dilution

Discharge Excess Side View:

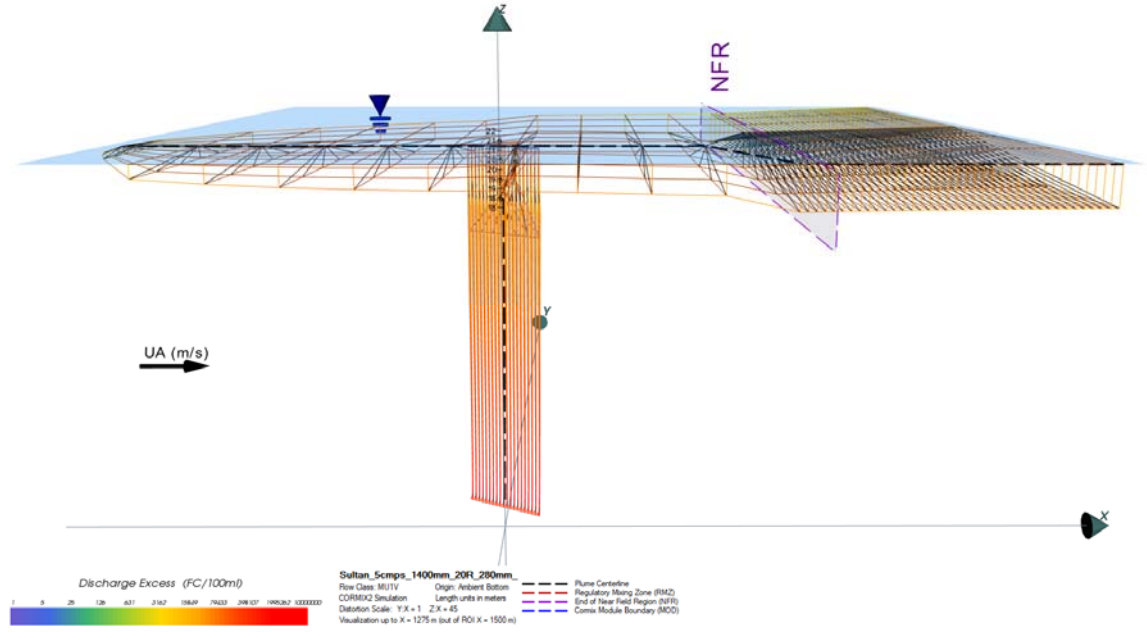


Figure 66-Side view of discharge excess

Dilution Frontal view:

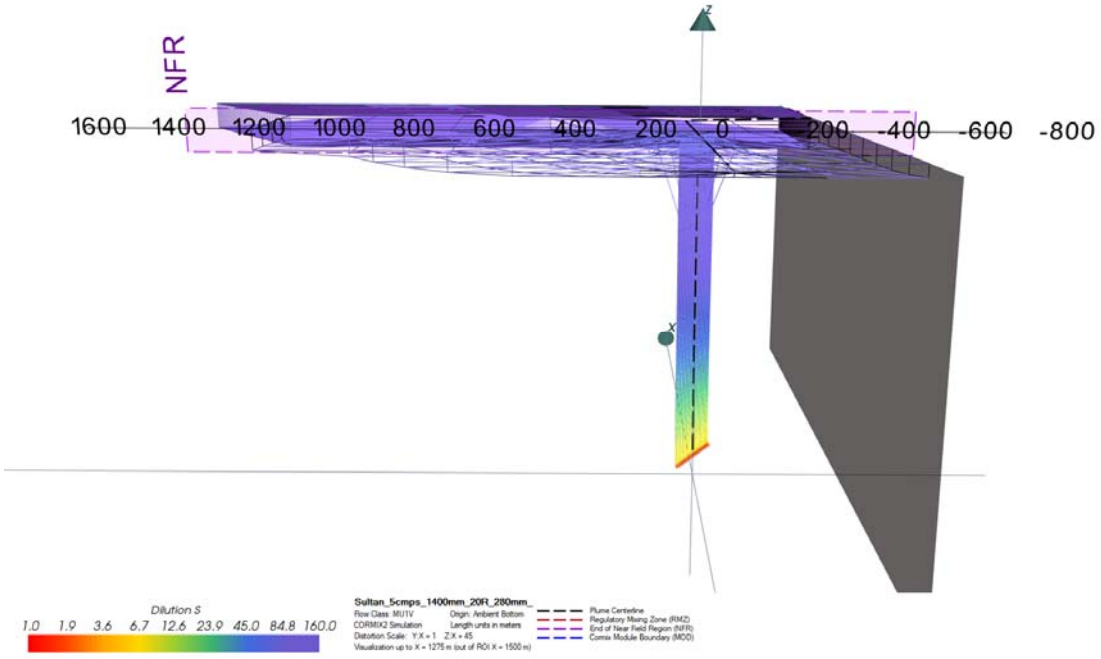


Figure 67-Frontal view of dilution

Discharge Excess Frontal view: The nearest bank is shown in grey – slight attachment.

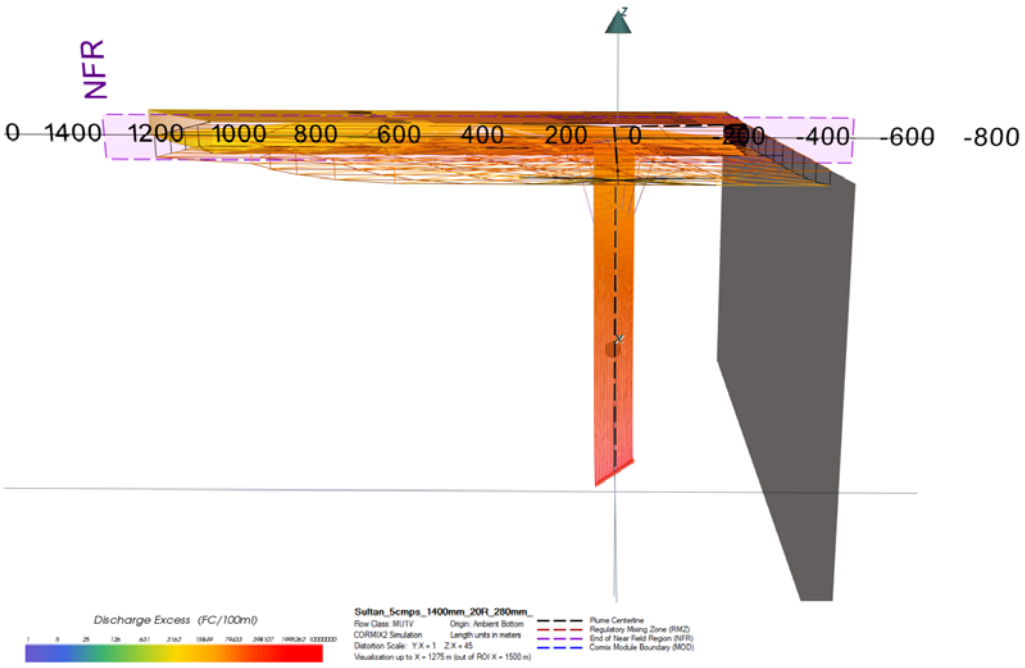


Figure 68-Frontal view of discharge excess

Scenario S1.2

The second set of simulations, for scenario S1.2, shows that the initial dilution criterion is also met. The plume in unbounded section does not contact the shore line.

The flow is hydro-dynamically classified as MU4: submerged positively buoyant multiport diffuser unstable discharge under strong current. Because of the strong ambient current the diffuser plume of this cross flowing discharge gets rapidly deflected. A near-field zone is formed that is vertically fully mixed over the entire layer depth. Full mixing is achieved at a downstream distance of about 100m.The discharge configuration is hydro-dynamically "unstable", that is the discharge strength (measured by its momentum flux) is very strong in relation to the layer depth and in relation to the stabilizing effect of the discharge buoyancy (measured by its buoyancy flux). The ambient current is very strong in the present case. The diffuser flow will experience instabilities with full vertical mixing in the near-field and there may be benthic impact of high pollutant concentrations. The effluent density is less than the surrounding ambient water density at the discharge level. Therefore, the effluent is positively buoyant and will tend to rise towards the surface. The diffuser plume is advected in the direction of the ambient flow. This plume deflection is associated with a recirculation zone at the downstream end of the plume. The plume is vertically mixed over the full layer depth in the near-field region. The plume spreads laterally along the layer boundary (surface or pycnocline) while it is being advected by the ambient current. The plume thickness may decrease during this phase.

Moreover, regulatory mixing zone boundary is within the Near-Field Region, and the plume downstream distance meets or exceeds the regulatory value of 300m.

The graphical outputs for scenario S1.2 are shown here below:

Dilution Isolines: The initial dilution criterion of 1:50 is met while the 1:100 is not.

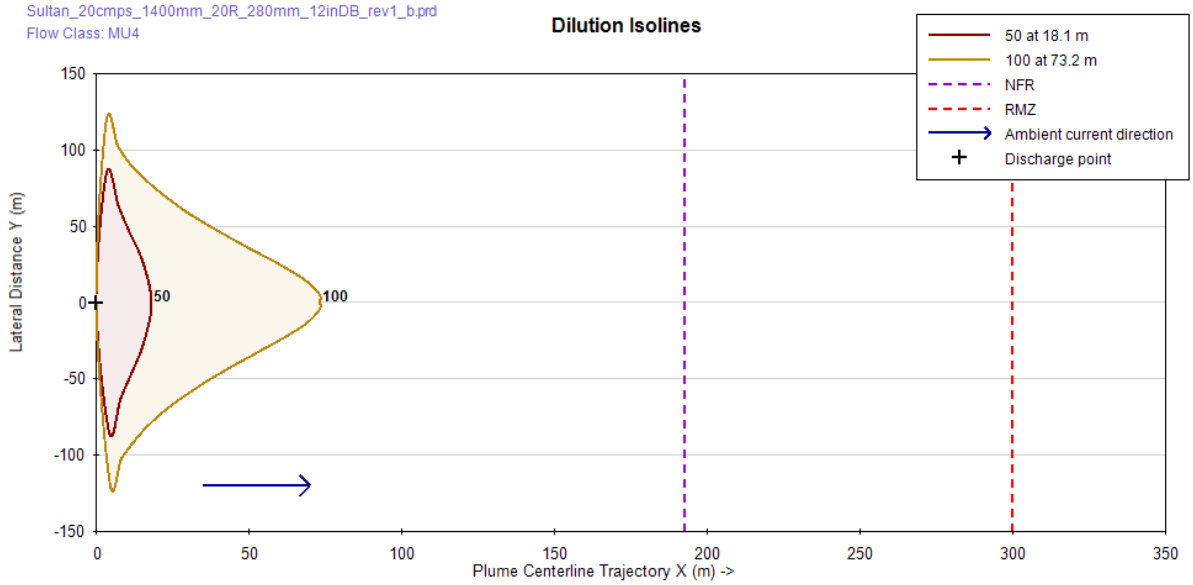


Figure 69- Dilution isolines under sub-scenario S1.2

3D View of Dilution:

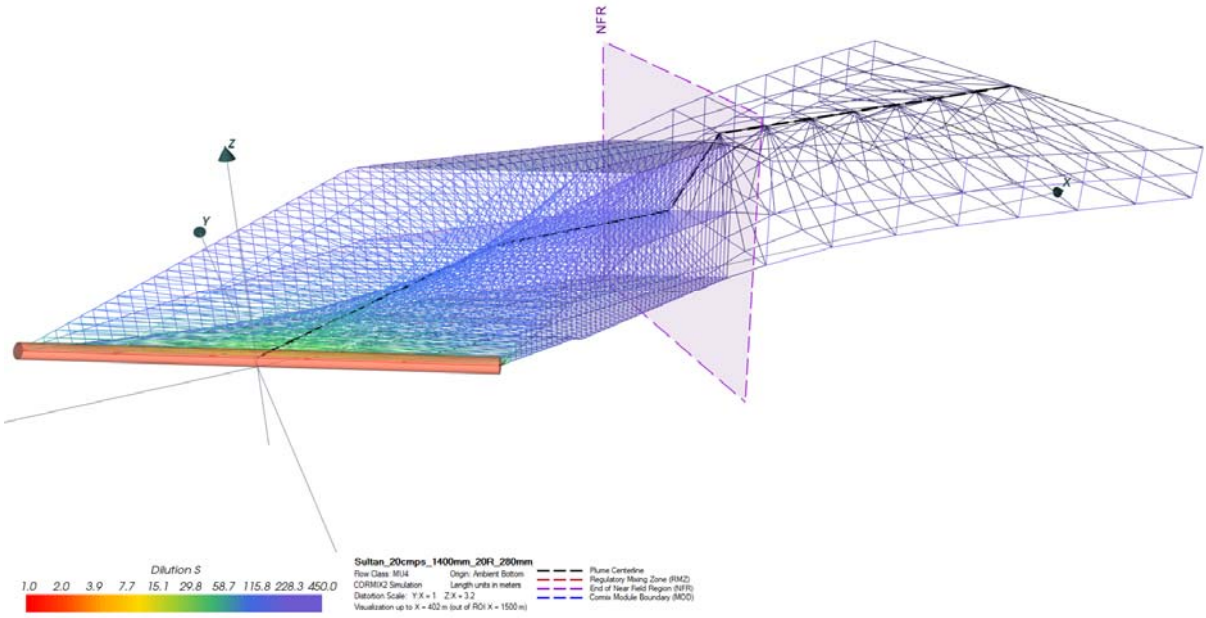


Figure 70- 3D view of dilution

3D View of Dilution from a perspective:

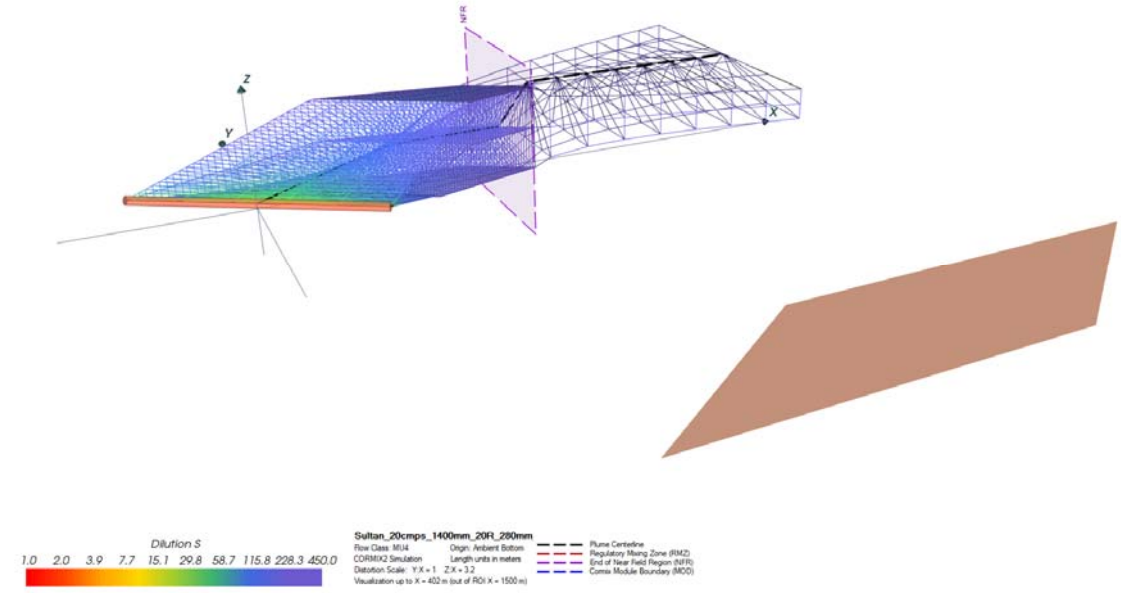


Figure 71-3D view of dilution showing the shoreline

3D View of Discharge Excess:

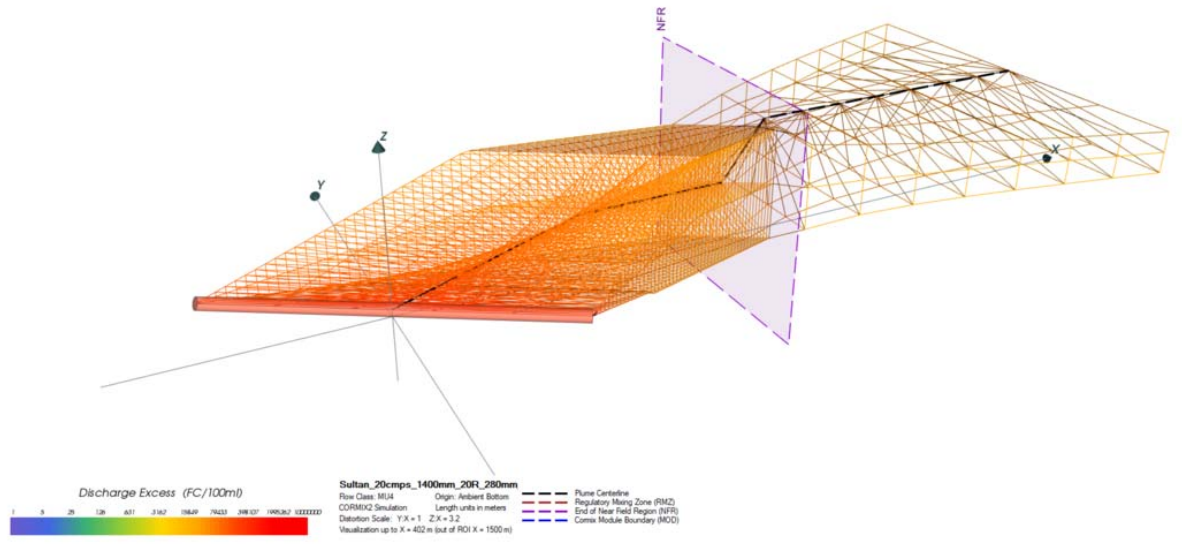


Figure 72-3D view of discharge excess

3D View of Discharge Excess from a perspective showing that the discharge stays away from the shoreline (shown in grey on the lower right):

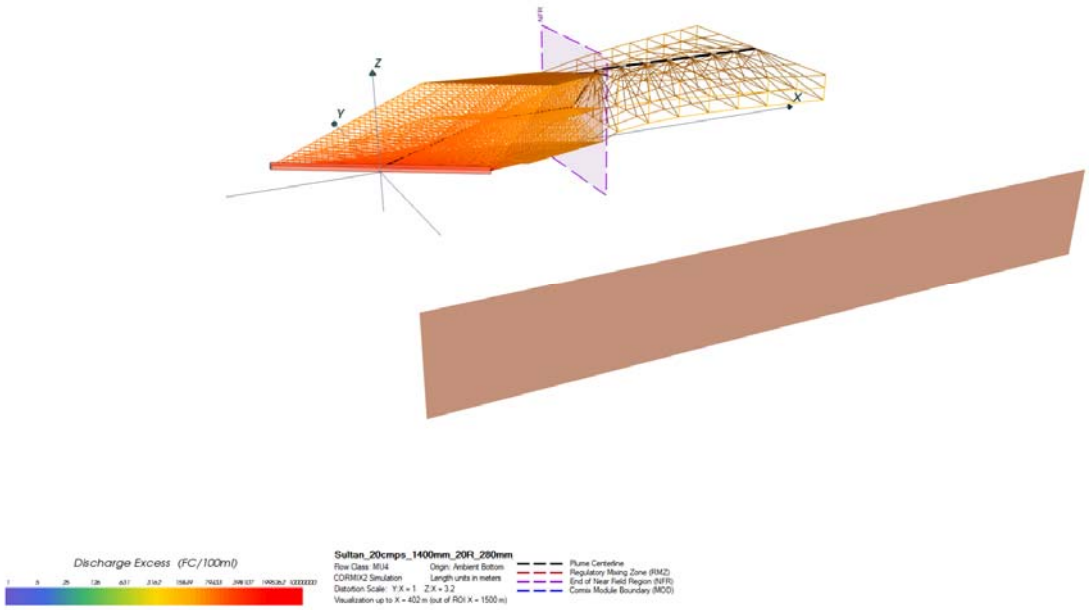


Figure 73-3D view of discharge excess showing the shoreline

Dilution Top View:

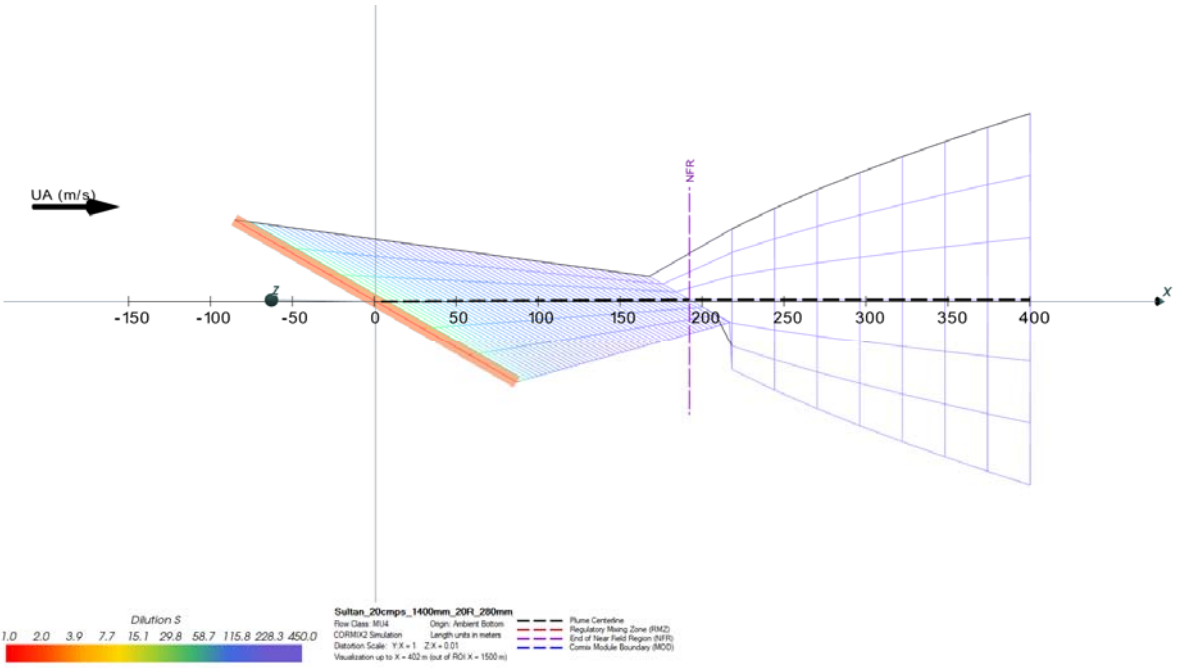


Figure 74-Plan view of dilution

Discharge Excess Top View:

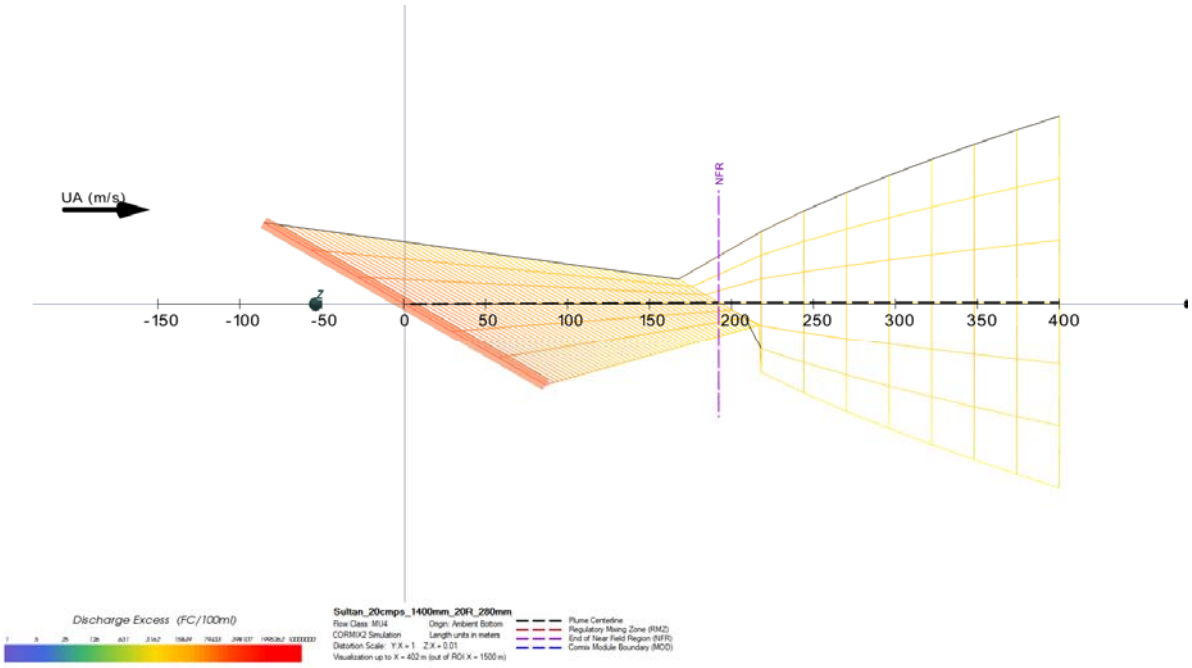


Figure 75- Plan view of discharge excess

Dilution Side View:

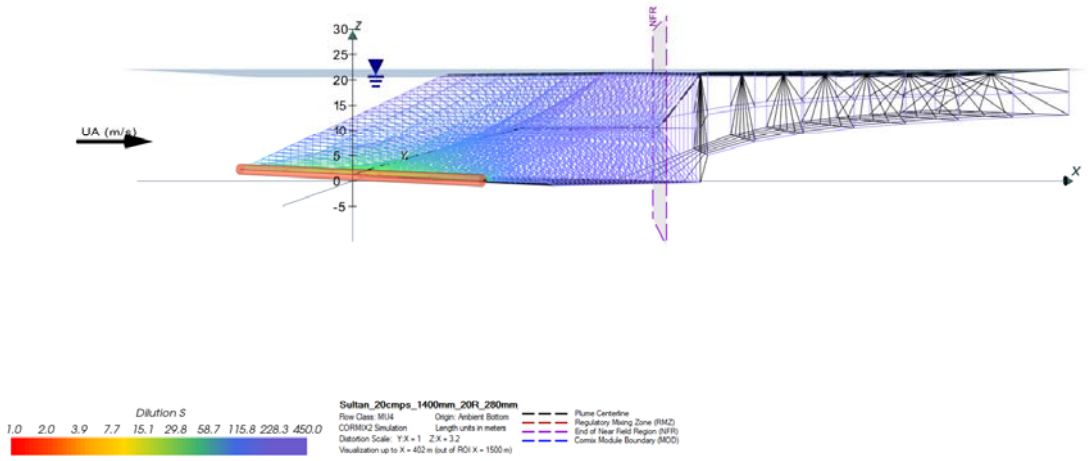


Figure 76- Side view of dilution

Discharge Excess Side View:

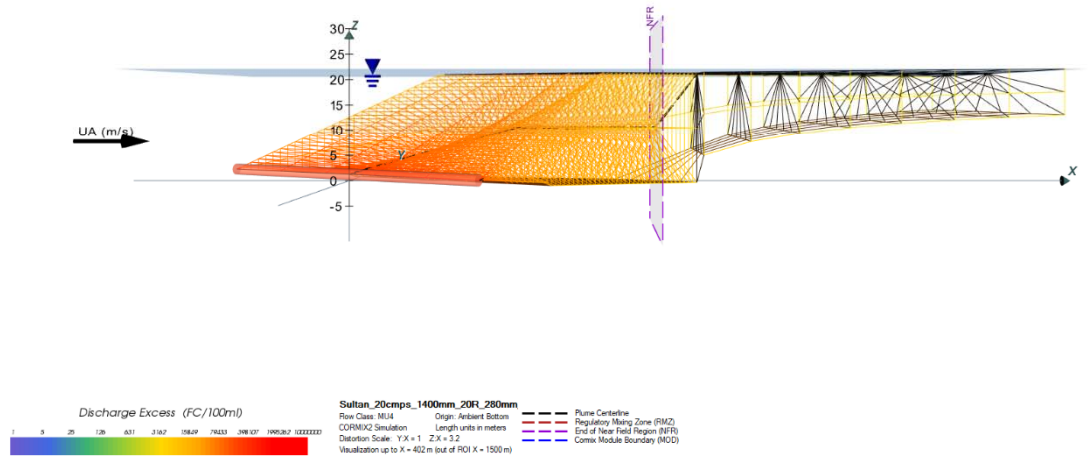


Figure 43-Side view of discharge excess

Dilution Frontal View:

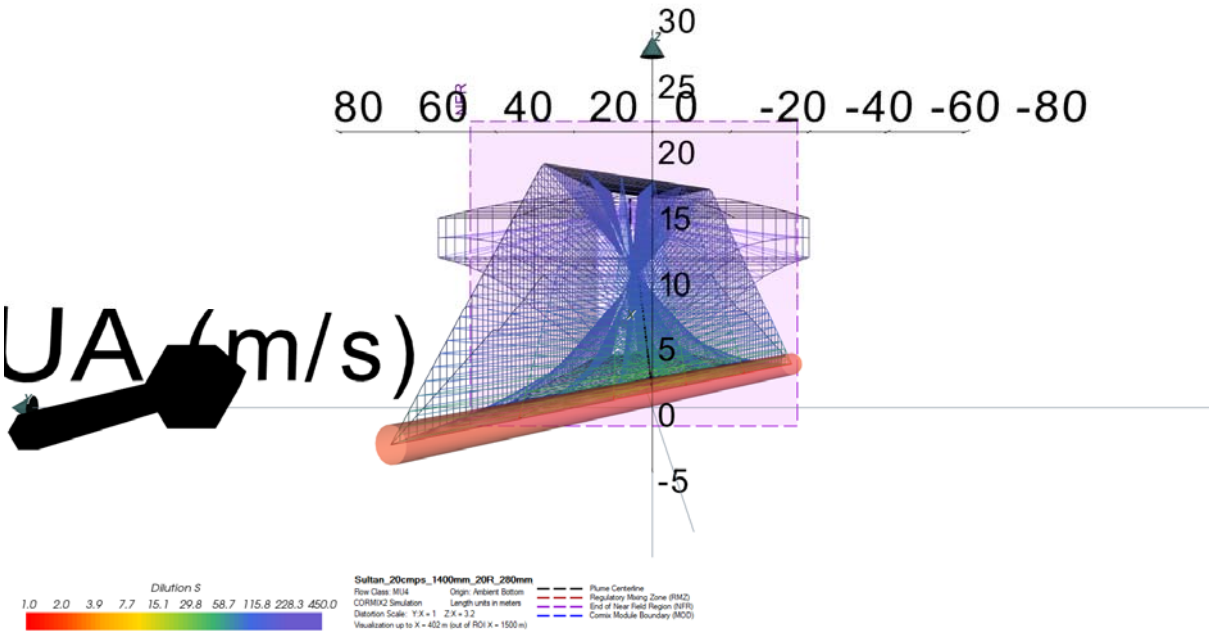


Figure 78- Frontal view of dilution

Discharge Excess Frontal View:

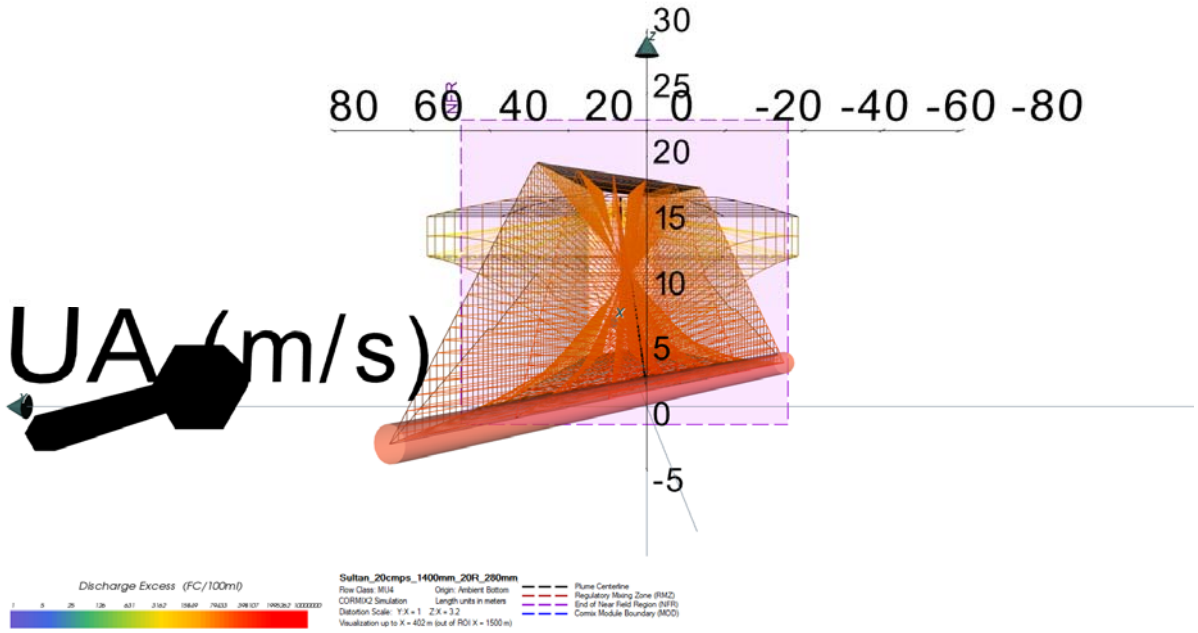


Figure 44-Frontal view of discharge excess

EXPANSION AND UPGRADE OF AL-GHADIR WASTEWATER TREATMENT PLANT PROJECT – PHASE I: DESIGN, BUILD AND OPERATE, INCLUDING TWO NEW SEA OUTFALLS AND INSPECTION & ASSESSMENT OF EXISTING SEA OUTFALLS

PART 5 – OTHER INFORMATION

BOOK 2/2

SECTION XV – REFERENCE BOQ FOR JNAH SEA OUTFALL

LEBANESE REPUBLIC

COUNCIL FOR DEVELOPMENT AND RECONSTRUCTION

BEIRUT - LEBANON

CONSULTANCY SERVICES

FOR

Beirut City Services Upgrading Project

Phase III - Lot 4

Volume 3- Bill Of Quantities



June 2018

KHATIB & ALAMI
CONSOLIDATED ENGINEERING COMPANY S.A.L.

PREAMBLE

PREAMBLE

1. The items in the Bill of Quantities do not necessarily contain full description of the required Works, and when pricing, the Contractor shall allow in his unit rates for all obligations, liabilities and services as mentioned in the Contract Documents:

   Volume I: Tendering Conditions and Procedures, Conditions of Contract;
   Volume II: Specifications;
   Volume IV: Drawings;
   and all other related Tender and Contract Documents.

2. **The methods of measurement used and which will be used for re-measurement shall be in accordance with the Method of Measurement included in the Specifications and/or as noted in the Bill of Quantities with the item description.** These methods of measurement shall be strictly adhered to notwithstanding any trade, general or local custom and shall include for necessary materials and/or plant to make complete item or system.

3. The Contractor shall, during his study of the Tender, address his queries, if any, to the Employer, who will study and clarify all such queries.

   The Contractor, shall allow in his unit rates for any shortage, error or inconsistency in the Contract Documents, which would affect the rates of his Tender, or the value of the Works.

4. The text of the Bill of Quantities shall not be altered or qualified in any way by the Tenderer. Any such alteration or qualification to the text shall be ignored and the text shall remain as printed; Moreover, such alteration or qualification is liable to result in a Tender, which is otherwise favourable, not being considered.

5. The Contractor shall satisfy himself as to the meaning of every item in the Bill of Quantities. Unless otherwise described or measured separately, the rates and prices inserted by the Contractor shall be deemed to cover all costs, expenses, risks, liabilities, obligations and services described in the Conditions of Contract and General Requirements, and all other Contract Documents, and all matters for the proper execution and completion of the Works and remedying defects therein. These rates shall allow, but not by way of limitation, for complying with all Contract Documents; for all labour and material, including samples, sampling and testing; for all temporary work of every description required; for the provision and use of all equipment and plant of every kind, whether mechanical or manual, required for the expeditious carrying out of the Works in their proper sequence; for shifting, altering and adapting such temporary work and equipment as may be required during the progress of the Works and removing at completion and making good any surfaces disturbed; for executing the requirements of the CDR Health and Safety Manual, for full customs and other import duties; for cooperation and coordination of the Works, with other Contractors; for the guarantees and bonds required; for providing all insurances and the like; for all necessary temporary services associated with these Works; for establishing and furnishing the Engineer's site office and providing the required vehicles for the Engineer's transportation; for the work in connection with measurements and the final account; for profit and for all other establishment charges and all costs of whatever nature. No claim for

additional payment will be considered for any error or misunderstanding by the Contractor in these respects maintenance costs, All works required by the Contract.

6. The unit rates and prices inserted by the Contractor shall apply throughout the Contract and to any additional work, which might be ordered by the Engineer.

7. The Contractor is expected to take all necessary precautions to avoid damaging services and where they are interrupted to reconnect them immediately. No separate payment shall be made and the rates in the Bill of Quantities shall be deemed to cover adequately all costs. The Contractor's attention is drawn to the fact that the service authorities may impose, in addition to the cost of repair, statutory penalties. Details of these requirements may be obtained from the respective authorities.

8. The Contractor shall allow in his rates for protecting all Existing and New Works against damage by whatever method deemed necessary and approved. Any work damaged before the Works are handed over is to be replaced or made good at the Contractor's expense and to the entire satisfaction of the Engineer.

9. The Quantities set-out in the Bill of Quantities are estimated quantities, and they are not to be taken as the actual and correct quantities of the Works to be executed by the Contractor in fulfillment of his obligations under the Contract.

   Notwithstanding the quantities given, all Works itemized in the Bill of Quantities will be subject to re-measurement in the unit of measurement indicated, and payment made at the corresponding unit rates on the re-measured quantities.

10. Unless otherwise stated, all measurements shall be measured net and are applicable to finished work only as completed and no allowance shall be made for wastage, working spaces, bulking, shrinkage, overlaps and the like. Unless otherwise described, the unit of measurement for each item shall be as indicated in the Bill of Quantities. Measurement shall be rounded up or down to the nearest whole number and exact half units shall be rounded up. Any thickness, which is stated in this Document, shall be the finished and specified thickness.

11. When the expression "Ditto" appears under an item in the Bill of Quantities, it shall be deemed to mean that the said item shall include the whole relevant description of the preceding item together with any description written before and/or after the said expression.

12. Certain items in the Bill of Quantities are designated "provisional" in the description column to indicate that certain items and/or quantities for the work covered by such items are the best possible estimates that may be made in advance of the execution of the work. The provision of such facilities as described in the items so designated will be used only at written direction and discretion of the Engineer. If not used either wholly or in part, the amount not used shall be deducted from the Contract price.

13. Any item of work included in the Bill of Quantities, which is not carried out, shall be omitted from the interim and final statements. Any additional item of work carried out on the Engineer's instructions shall be included in the relevant interim and final statements.

14. Contractor shall establish & install bollards on all sidewalks as per the details adopted and followed in Beirut currently.

    - Maintain locations on sidewalks for Municipal garbage containers and ensure preservation of current locations.

    - Reuse existing natural curbstones after trimming their edges and bush hammering their surfaces and ensure that only standard natural or concrete curbs are used at any one street.

    - Rehabilitation of the natural curbs via trimming their edges and bush hammering and polishing the surfaces and ensure that the available quantities are adequate and sufficient for the sidewalks of a whole street.

    - The contractor shall remove, on daily basis, the construction debris from the roads and transport them to pre-determined locations allocated in coordination with the Resident Engineer and shall obtain the approval of the land owner for using his land for this purpose prior to the removal and dumping process.

    - The Contractor should allocate a piece of land to accommodate the site offices and construction plants and equipment and shall ensure that the said plants and equipment are not left on streets.

    - Prior to commencement of any construction works on streets, the contractor shall coordinate with the traffic department to provide the required traffic diversions and detours and shall also submit a detailed work program for each road section and obtain the Resident Engineer approval prior to commencement of activities.

    - Hold intensive and close coordination with Electricite' Du Liban and other official authorities such as (Beirut Municipality, Telephone Department, Ministry of Communications etc..) prior to commencement of works in locations of electricity, lighting and telephone lines, power cables, circuit breakers and numbering of lighting columns, as well as the storm water and wastewater disposal lines and others.

- Hold intensive and close coordination with Beirut Water Authority prior to commencement of any works relevant to the water lines to obtain Authority's specifications with regards to method of installation, type of protection needed, and other works related to construction/ relocation of (water meters, valves, Hydrants, valve chambers, fittings etc..)

- Apply and obtain for all necessary permits from the relevant Authority

15. The following notations are used in the Bill of Quantities:

Linear Meter	lm
Square Meter	m^2
Cubic Meter	m^3
Number	No.
Tonne	tonne
Kilogram	kg
Lump Sum	LS
Centimeter	cm
Millimeter	mm
Reinforced Concrete	RC
Drawing	Dwg.

ENVIRONMENTAL WORKS

A. Work described as "excavate", "excavation" or "earthwork" means excavation in any type of soil encountered including rock, get out, place in temporary heaps, load, cart, disposal of surplus, return, fill and ram including working space, support of excavation and dealing with water of any nature, leveling, trimming and grading bottoms and trimming sides of excavation. The rates to be entered in the Bill of Quantities for separate earthwork of for other items that include earthwork shall be inclusive for all the operations described above and shall also include but not limited to the following:

1) diverting any existing public utilities encountered in the excavation and reinstating same to their original condition as they were before such diversion.

2) Demolishing any existing utilities that lie within the excavation limits.

3) Carefully protection from injury, supporting and restoring or replacing any damaged items, which are to be preserved in placed without being temporarily or permanently relocated.

4) Dewatering all excavations by means of pumping or bailing and or by drainage well-point system. The system or systems to be adopted for keeping excavations free from water shall be as approved by the Engineer.

5) Provision of suitable backfill where the excavated material is deemed unsuitable by the Engineer.

6) Excavation for concrete thrust blocks shall be considered as part of trench excavation.

B. Rate for reinforced concrete shall include for work and reinforcement.

C. Concrete protection to pipelines shall be measured by multiplying the net area detailed on the drawing by the actual length.

D. Earthwork in Bill for Potable Water System/Irrigation means excavation in any ground including rock and to any depth of pipeline. Earthwork shall also include extra excavation for bedding and protection and at connection points to the existing mains and for valves and their thrust blocks.

E. Fittings for water pipelines shall be of materials compatible with the pipeline used and shall at least cover the following fittings, tees, crosses, reducers, elbows (bends), all adaptors for fittings, plugs for future connections etc..

D.I.:	Ductile iron	(Pipes $\geq$ 6")
H.D.P.E.:	High Density Polyethylene	(Pipes $\leq$ 4" and for house connections)

A. Earthwork in Bills for Sewer and Stormwater Systems shall mean excavation in any ground including rock. Earthwork quantities shall be measured in cubic meters calculated by multiplying the trench width as defined on the drawings, the actual average depth excavated to formation level defined on the drawings and the length measured from center to center for manholes. Extra over earthworks for manholes shall be included with the manholes rated.

B. Reinstatement: The contractor shall be paid for reinstatement of pavement only if pavement is not part of the Roadwork. Reinstatement of pavement shall be measured in square meters as the multiplication of trench width shown on the drawings by the length of the reinstated trench. The rate for reinstatement shall allow for the additional width for side slopes of trenches plus 30 cms set back on each side.

C. Gravity pipelines shall be measured in linear meters from the outside face of one manhole to the outside face of the next manhole. Pipework inside manholes are measured with the manholes:

GRP	:	Glass reinforced plastic (storm sewers and wastewater sewers)
C.P.	:	Concrete pipe (for storm sewers only)
U.P.V.C.:		Unplasticized polyvinyl chloride (not to exceed 300 mm diameter) (wastewater sewers and house connection)

D. GRP, being a flexible pipe and acting as a linear to the surrounding soil, the contractor shall exercise special care when placing and compacting the bedding material and the selected fill material up to 30 cms above the crown of the pipe. The rate of pipeline shall allow for site measurement of vertical and horizontal deflection of each pipe laid, initially after backfilling and the long term deflection after one year.

E. Sewer house connection lines shall be measured in linear meters of actual length.

1-1-1 General Description

The project scope includes study update of the rehabilitation of roads and streets, and the rehabilitation /upgrading of the underground and surface utilities in Municipal of Beirut, taking into account the flow separation and the introduction of a new independent storm water network.

The project works were subdivided into three sequential phases (I, II, III and IV). Phases I and II were studied previously and executed, while parts of Phase III were not executed and the available study needs to be reevaluated/updated which are subject of this current project.

The scope under LOT IV covers the construction of an sewer sea outfall for the emergency sewer over flow pipe of pumping station (PS2) located at Jnah area.

1-1-2 The Works are:

1) Extend the existing(900mm Dia.) GRP overflow pipe of (PS2) pumping station located at Jnah to reach the shore line.

2) The extension necessitates to install a new sewer line consiting of :
   - (900mm Dia.) GRP pipes.
   - (800 mm Dia.) HDPE pipes.

3) Installation of (1400 mm Dia.) HDPE sea outfal pipe includiing diffusers for about 920 m length.

4) Construction of a pressurized sewer Manhole at the sea shore to connect the overflow pipe to the sea outfall.

BEIRUT CITY SERVICES UPGRADING PROJECT
PHASE III - LOT 4
BILLS OF QUANTITIES
INDEX

BILL NO.	DESCRIPTION	FROM PAGE	TO PAGE
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5	STRUCTURES AND CONCRETE WORKS	5/1	5/1
8	SEWER WORKS	8/1	8/5
10	SEA OUTFALL PIPE WORKS, EXCAVATION & ROCK PROTECTION	10/1	10/6
	SUMMARY	S/1	S/1

BILL NO. 1
GENERAL REQUIREMENTS

Item No.	Relevant Spec. Clause No.		Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	Measurement	Payment					
SECTION NO. 1 - GENERAL REQUIREMENTS							
101			Contractor's mobilisation and temporary works Unit Rate in Letters……	LS	1		
102			Bonds and insurances Unit Rate in Letters……	LS	1		
103			Additional Topographical and Marines Survey for The Sea Outfall. Unit Rate in Letters……	LS	1		
104			Additional Soil Investigation for The Sea Outfall. Unit Rate in Letters……	LS	1		
105			Shop Drawings Unit Rate in Letters……	LS	**1**		
106			As - built drawings Unit Rate in Letters……	LS	**1**		
			TOTAL AMOUNT OF BILL NO. 1 CARRIED TO SUMMARY				

BILL NO. 2
SITE WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 2- CLEARING AND GRUBBING					
2.01		**Section 2.01 - (Removal of Pavement, Structures and Obstructions)**				
2.1.1	2.02.1	Removal of existing flexible pavement layers, asphalt, base course and/or subbase course as the case may present, down to the subgrade level. Unit Rate in Letters……	m2	1,000		
2.1.2	2.02.2	Existing concrete new Jersey barriere to be removed Unit Rate in Letters……	m	50		
2.1.3	2.02.2	Demolishing of existing sewer manhole MH-D. Unit Rate in Letters……	LS	1		
		TOTAL AMOUNT OF BILL NO. 2 CARRIED TO SUMMARY				

BILL NO. 3
GRANULAR SUB BASE AND BASE COURSES

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 3 - GRANULAR SUB BASE AND BASE COURSES						
		Crushed Aggregate Base Course				
3.01	3.03.1	Crushed aggregate base course (Class B). Unit Rate in Letters……	m3	300		
		TOTAL AMOUNT OF BILL NO. 3 CARRIED TO SUMMARY				

BILL NO. 4
BITUMINOUS CONSTRUCTION

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 4 - BITUMINOUS CONSTRUCTION						
		Bituminous Prime Coat				
4.01	4.02.1	Bituminous prime coat.	m2	1,000		
		Tack Coat				
4.02	4.02.2	Bituminous tack coat.	m2	1,000		
		Hot Mix Asphalt Base Course				
4.03	4.04.1	Hot mix asphalt base course (Thickness =7cm) Unit Rate in Letters……	m3	75		
		Hot Mix Asphalt Concrete Surface Course				
4.04	4.05.1	Hot mix asphalt wearing surface (Thickness =5cm) Unit Rate in Letters……	m3	50		
		TOTAL AMOUNT OF BILL NO. 4 CARRIED TO SUMMARY				

BILL NO. 5
STRUCTURES AND CONCRETE WORKS

Item	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount (USD)
SECTION 5- CONCRETE AND STRUCTURES						
5		**CONCRETE MIXES AND TESTING**				
5.01.1	5.01.8	Concrete encasement (Calss 25Mpa, Type 5) for the HDPE pipe DN800 Unit Rate in Letters……	m^3	45		
5.01.2	5.01.8	Concrete Class 25 Mpa for the existing box culvert & connection access shaft (including new seperating walls, new top slab and all other requirement and new walls & cover slab at the discharge point at the sea side) Unit Rate in Letters……	m^3	25		
5.01.3	5.01.8	Concrete Class 25 Mpa for the reistatement of the new Jersey barriere Unit Rate in Letters……	m	50		
5.3		**STEEL REINFORCEMENT**				
5.3.1	5.03.4	High tensile steel bar reinforcement Unit Rate in Letters……	Ton	7		
		TOTAL AMOUNT OF BILL NO. 5 CARRIED TO SUMMARY				

BILL NO. 8
SEWER WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 8 - SEWER WORKS					
8.02		**PIPES AND APPURTENANCES**				
		Earthwork, not covered under roadwork for all sewer lines diameters and to any depth:				
		Earthwork. Unit Rate in Letters……	m3	400		
		Sewer lines: Provide, lay, join and test complete including plugs for future connections, couplings, gaskets, backfill bedding, all as specified, (Pipework inside manholes are measured with the manholes separately):				
8.2.1	8.02.4 B	HDPE (SDR 26) 800 mm diameter pipeline (Price shall include pipe laying inside the existing Unit Rate in Letters……	m	175		
8.2.2	8.02.4 B	GRP; 900 mm diameter pipeline (Stifness 10000). Unit Rate in Letters……	m	65		
		House connection sewer line: Provide, lay, join, and test complete including plugs for future connections, fittings, couplings, gaskets, earthwork to any depth, bedding, all as specified, (chimneys and inspection chambers measured separately):				
8.2.6	8.02.4 C	PVC 100 mm diameter. Unit Rate in Letters……	m	5		
8.2.7	8.02.4 C	PVC 150 mm diameter. Unit Rate in Letters……	m	5		
8.2.8	8.02.4 C	PVC 200 mm diameter. Unit Rate in Letters……	m	5		
8.2.9	8.02.4 C	PVC 250 mm diameter. Unit Rate in Letters……	m	5		
		TO COLLECTION				

BILL NO. 8
SEWER WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 8 - SEWER WORKS					
		Wastewater Diversion Works				
		Wastewater diversion works, complete and irrespective of the flow, before the operation of repair of existing culverts or pipes, and during the installation of new pipe or the construction of new culverts along the course of the work, and during the whole period of execution, all as directed and to the approval of the Engineer.				
8.2.10	8.02.4 B	Wastewater diversion works, irrespective of the flow. Unit Rate in Letters……	m	100		
		Cleaning of Existing Sewer Culverts or Pipes				
		Cleaning of existing sewer culverts or pipes that are to be retained complete and irrespective of the pipe diameter and type, and of the culvert size, all as specified and to the approval of the Engineer.				
8.2.11	8.02.4 B	Cleaning of existing combined double cells box culverts or Unit Rate in Letters……	m	150		
		CCTV Survey on Existing Sewer Pipes				
		CCTV system shall be capable of recording colored images. The results of cleaning and the CCTV survey shall be tabulated with video tape and colored photographs. The rate shall include cleaning of the existing sewer line and the related manholes prior to CCTV survey and a complete report on the recommendations regarding the rehabilitation or replacement of all or part of the pipes or manholes, complete all to the approval of the				
8A.2.12		CCTV survey on existing sewer culverts, or pipes that are to be retained, for any section. Unit Rate in Letters……	m	150		
		TO COLLECTION				

BILL NO. 8
SEWER WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 8 - SEWER WORKS					
8.04		**CONCRETE ENCASEMENT FOR PIPES**				
		Extra over to sewer lines provide, place and compact concrete class A protection be it concrete surround, arch, hunching or slab including formwork, reinforcement for pipes cover is less than 1.2m and where detailed to any diameter:				
8A.4.1	8.04.4	Concrete protection. Unit Rate in Letters……	m3	10		
8.05		**MANHOLES, CHAMBERS AND GULLIES**				
		Concrete manholes constructed complete including extra over earthwork in any depth, reinforcement, blinding, benching, formwork, surfaces protection, steps, Ductile Iron Cover and frame, concrete rings, hunching, pipework inside manholes, all as specified:				
8.5.1	8.05.5 A	Proposed Manhole MH-0A Unit Rate in Letters……	No.	1		
8.5.2	8.05.5 A	Proposed Manhole MH-01 Unit Rate in Letters……	No.	1		
8.5.3	8.05.5 A	Proposed Manhole MH-02 Unit Rate in Letters……	No.	1		
8.5.4	8.05.5 A	Proposed Pressurized Manhole MH-03 (Including special pressurized manhole cover) Unit Rate in Letters……	No.	1		
		Concrete house connection chamber constructed complete including earthwork, blinding, backfilling, benching, formwork, surfaces protection, ductile iron Cover and frame, concrete rings, hunching, pipework inside chamber, all as specified:				
8.5.5	8.05.5 A	Inspection chamber. Unit Rate in Letters……	No.	5		
		TO COLLECTION				

BILL NO. 8
SEWER WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 8 - SEWER WORKS					
8.06		**Sundry Works**				
8.6.1		Repair of existing sewer pipes. Unit Rate in Letters……	m	50		
8.6.2		Repair of existing manholes. Unit Rate in Letters……	No.	4		
8.6.2		Repair of existing culverts. Unit Rate in Letters……	m	75		
8.6.4		Connection to existing sewer manholes including all necessary demolition works to connect the new sewer line, supplying materials, water pumping, removal of the debris, installing and connecting sewer line, all as directed by the engineer.	No.	2		
8.6.5		Replacement of existing manholes.	No.	2		
8.6.6		Replacement of heavy duty manhole covers and frame toBS EN 124:1994.	No.	6		
8.6.7		Ditto medium duty manhole covers and frame to BS EN124:1994 standards 25 tons, 600 mm opening.	No.	2		
8.6.8		**Removal of existing pipework including excavation, backfilling, etc.., for any depth:**				
8.6.9		Pipework = 900 mm dia.	m	10		
		TO COLLECTION				

BILL NO. 8
SEWER WORKS

Item No.	Measurement & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
	SECTION 8 - SEWER WORKS					
		COLLECTION Page No. 8/1 Page No. 8/2 Page No. 8/3 Page No. 8/4 **TOTAL AMOUNT OF BILL NO. 8A CARRIED TO SUMMARY**				

BILL No. 10
SEA OUTFALL WORKS

Item No.	Measuremen & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 10-PIPE WORKS, EXCAVATION & ROCK PROTECTION						
10.1		**Pipe Works**				
10.1.1	10.02.2.4	Supply of HDPE pipes and fittings, OD1400, SDR26, PE80, Flanged ends PN10 Unit Rate in Letters……	710	m		
10.1.2	10.02.2.4	Supply of HDPE Diffuser section, OD1400, SDR26, PE100 with PN10 flanged ends and with nos. 20 OD 280 risers as per drawings flanged to PN10 Unit Rate in Letters……	200	m		
10.1.3	10.02.2.4	Supply of HDPE flanged to PN10 45 degrees bend,OD1400, SDR26, PE100, as per drawings Unit Rate in Letters……	1	No.		
10.1.4	10.02.2.4	Supply of HDPE flanged 90 degrees bend,OD280, SDR26, PE100, with port extension also flanged to PN10 as per drawings Unit Rate in Letters……	20	No.		
10.1.5		Supply of 12 inch flanged to PN10 Wide DuckBill valves (reinforced neoprene) Unit Rate in Letters……	20	No.		
		TO COLLECTION				

BILL No. 10
SEA OUTFALL WORKS

Item No.	Measuremen & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 10-PIPE WORKS, EXCAVATION & ROCK PROTECTION						
10.2		**Trench Excavation**				
10.2.1	10.06.13	Trench Excavation for OD1400 pipes off-shore (marine) From Point A to Point C (approx. 17m3/m) Unit Rate in Letters……	200	m		
10.2.1	10.06.13	Trench Excavation for OD1400 pipes off-shore (marine) From Point C to Point D (approx. 2.5m3/m) Unit Rate in Letters……	100	m		
10.2.3	10.06.13	Trench Excavation for OD1400 pipes off-shore (marine) From Point D to Point F (approx. 22m3/m) Unit Rate in Letters……	200	m		
10.2.4	10.06.13	Trench Excavation for OD1400 pipes off-shore (marine) From Point F to Point G(approx. 13m3/m) Unit Rate in Letters……	100	m		
10.2.5	10.06.14	Trench Excavation for OD1400 pipes off-shore (marine) From Point G to Point H (approx. 12m3/m) Unit Rate in Letters……	110	m		
10.2.6	10.06.14	Trench Excavation for OD1400 diffuser section off-shore (marine) From Point G to Point H (approx. 5 m3/m) Unit Rate in Letters……	200	m		
		TO COLLECTION				

BILL No. 10
SEA OUTFALL WORKS

Item No.	Measuremen & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 10-PIPE WORKS, EXCAVATION & ROCK PROTECTION						
10.3		**Gravel Bedding and Surround**				
10.3.1	10.02.4.1	Gravel Bedding and Surround and Filter from Point A to Point C (approx. 7.5m3/m of 40/60mm bedding and surround and 5m3/m of under water concrete) Unit Rate in Letters……	200	m		
10.3.2	10.02.4.1	Gravel Bedding and Surround and Filter from Point C to Point D (approx. 4m3/m of underwater concrete) Unit Rate in Letters……	100	m		
10.3.3	10.02.4.1	Gravel Bedding and Surround Point D to Point F (approx. 7.5m3/m of 40/60mm bedding and surround and 5m3/m of underwater concrete) Unit Rate in Letters……	200	m		
10.3.4	10.02.4.1	Gravel Bedding and Surround Point F to Point H (approx. 9m3/m of 40/60 mm of bedding and surround Unit Rate in Letters……	210	m		
10.3.5	10.02.4.1	Gravel Bedding and Surround Point H to Point I (approx. 2m3/m of 40/60 mm bedding and surround) Unit Rate in Letters……	200	m		
		TO COLLECTION				

BILL No. 10
SEA OUTFALL WORKS

Item No.	Measuremen & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 10-PIPE WORKS, EXCAVATION & ROCK PROTECTION						
10.4		**Laying of Pipes**				
10.4.1	10.02.2.4	Laying of OD1400 pipes off- shore (marine) Unit Rate in Letters……	710	m		
10.4.2	10.02.2.4	Laying of OD1400 diffuser section off- shore (marine) Unit Rate in Letters……	200	m		
10.5		**Reinforced concrete Works**				
10.5.1	10.02.4.1	Special reinforced concrete (C25) structures for Jumper 1. Unit Rate in Letters……	62	No.		
10.5.2	10.02.4.1	Special reinforced concrete (C25) structures for Jumper 2. Unit Rate in Letters……	602	No.		
10.5.3	10.02.4.1	Special reinforced concrete (C25) structures for matress Unit Rate in Letters……	93	No.		
10.5.4	10.02.4.1	Special reinforced concrete (C25) structures for ports protection. Unit Rate in Letters……	20	No.		
10.5.5	10.02.4.1	concrete for (C25) collars for OD1400 pipe including neoprine pads (1/8 inch thick) as per details shown on the drawings. Unit Rate in Letters……	271	No.		
10.6		**Testing and Commissioning**				
10.6.1		Testing and Commissioning of complete sea outfall system Unit Rate in Letters……	1	L.S		
		TO COLLECTION				

BILL No. 10
SEA OUTFALL WORKS

Item No.	Measuremen & Payment	Description	Unit	Qty	Unit Rate (US. $)	Amount US. $
SECTION 10-PIPE WORKS, EXCAVATION & ROCK PROTECTION						
		COLLECTION Page No. 10/1 Page No. 10/2 Page No. 10/3 Page No.10/4 **TOTAL AMOUNT OF BILL NO. 10 CARRIED TO SUMMARY**				

BEIRUT CITY SERVICES UPGRADING PROJECT
PHASE III - LOT 4
BILLS OF QUANTITIES
SUMMARY

BILL NO.	DESCRIPTION	FROM PAGE	TOTAL AMOUNT US$
1	GENERAL REQUIREMENTS	1/1	
2	EARTHWORKS	2/1	
3	GRANULAR SUB BASE AND BASE COURSES	3/1	
4	BITUMINOUS CONSTRUCTION	4/1	
5	STRUCTURES AND CONCRETE WORKS		
8	SEWER WORKS	8A/5	
10	SEA OUTFALL :PIPE WORKS, EXCAVATION & ROCK PROTECTION		
	TOTAL AMOUNT OF WORKS CARRIED TO FORM OF TENDER	**US $**	
	VAT 5.94% (Project Fund by Lebanese Gouvernment 54% & 46% by AFESD)	**US $**	
	GRAND TOTAL	**US $**	
	IN WORDS …… **……..**		