# Analysis of sinking of GRP pipe string DN 1600

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***Abstract:*** *The aim of this research it was the installation of a discharge sea water pipeline DN 1600 of GRP (glass-reinforced plastic) material in Vlore Bay. The analysis of the sinking of the pipe string has been carried out by using CAESAR II software and considering different load cases. The calculation was carried out for static condition (sustained load) and transient condition (occasional load). In consideration were taken the allowable stress due to bending for pipe and allowable stress for joint subjected to freezing. From the calculation it was evident that the most critical effect on the GRP pipe has done by the position of Concentrated Force due to the presence of sledge (approx.. 2 tons), not disassembled from the string after moving in the position from the shore. In these cases, where the central crane has been maintained fixed or moved downward, the maximum bending stress on the pipe was 12.03 MPa. According to the verification carried out for pipe wall in the transient condition (occasional load) the stability was good and for the joint, was shown, in the worst scenario, a bending stress 158% of the allowable one. In this case it is possible to have the crack of the resin used for the freezing of the joint but not the failure of the system joint + pipe that can withstand without, failure, a stress 3 times greater than the allowable stress for pipe according to EN1796.*
***KEY-WORDS****: COMPOSITE MATERIAL, PIPE STRING, ALLOWABLE STRESS, INFLECTION TEST.*

## *1. Introduction*

The Cooling Water System is one of the components of the Combined Cycle Generating Power Plant that is constructed in the bay of Vlore. Its scope is to provide the Plant with the required quantity of water for the necessity of cooling circuit of the engines and to return to sea, after use, the warm water.
For this reason was carried out the installation of the submarine pipelines deputed to the carriage of the water from the open sea to the plant and to their discharge.
The pipelines installed there are hereby listed:

- 1800 mm I.D., approx 1455 m long GFR Water Intake pipeline
- 1600 mm I.D., approx 1050 m long GFR Outfall Water Discharging pipeline
- 90 mm I.D., approx 1475 m long High Density Polyethylene Chlorination line.

Glass Fiber Reinforced (GFR) pipes were used for the lightness, handiness, corrosion resistance and versatility that this material is supplying, together with the reliability of the bell & spigot pipe joint system that is ensuring a high quality sealing and a high resistance to the axial forces.
For safety the two pipelines have been considered to be buried in a way that, when completely installed, their top was protected by a minimum cover of 1,5 m of soil. The trench was of sufficient dimensions to accommodate the pipelines and had appropriate slopes to prevent the surrounding material to fall into the excavation during pipeline installation.
The two lines were laid in the same trench from the connection points onshore till the Outfall point (approx. 645 m far from the shoreline); afterwards the trench have accommodated only the 1.800 mm I.D. line till the Intake structure.
The two pipelines were laid on a 30 cm layer of bedding material, to prevent damages deriving from possible idle bumps against rock or stones and to provide smoothed alignment within the allowable piping profile.
At the end of the laying of the two pipelines the trench was backfilled with the aforesaid minimum cover of material.
The same trenched configuration is foreseen on the land, from the shoreline to the connection with the plant structure.
The bell & spigot pipe joint is shown in the figure 1. There are shown the two O ring gaskets which ensure a high quality sealing and the locking device which ensure a high resistance to the axial forces.

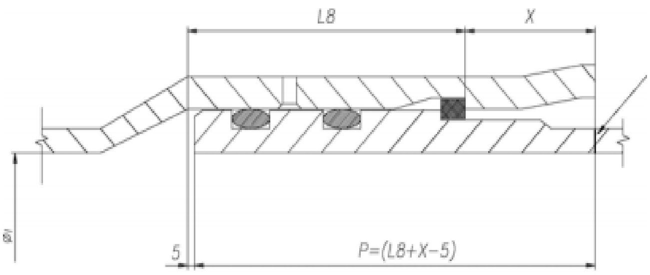


***Figure 1.*** *The bell $ spigot joint*

The methodology of the joint installation is shown in the figure 2, where is very important the alignment of the two pipes.

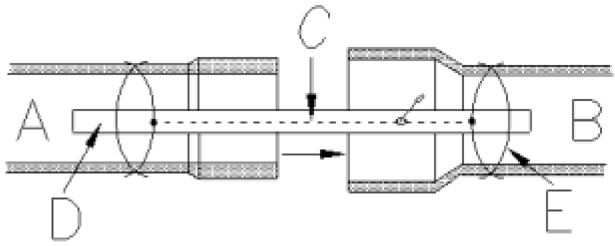


***Figure 2.*** *The methodology of the joint installation*

A - Pipe being installed
B - Installed pipe
C – Tirfor, one on each side
D – Plank on both sides to prevent pipe damage.
E – Nylon or rope

The prefabricated GFR pipes were 12 m long and the installation of the pipelines were carried out one by one by the specialized divers. After the pipelines installation, during the commissioning of the cooling water system was realized that the water didn't came in the Power Plant. After these were carried out some surveys either with divers and ROV (remote operated vehicle), which was put in the pipeline from the water intake and from the diffuser. Was found that both the lines (intake and discharge) were opened at least in one point and had nonconformities in a part of their lengths.
These break points were found uncovered, without the backfilling material and so no protected from the wave action and currents.
After this a root cause analysis was carried out and was decided to make a hydraulic design review and after this to fit a proper installation methodology.

## *2. Root cause analysis*

The first inspection was carried out with divers in the point where the water intake pipeline was broken. It was a sequence of events that leads to the failure of the pipe. These events are summarized as follows:

- Misalignment and successive displacement of the piping joint in the suction line (with failure of one connection)
- Further to continuous suction of SWMP (for several days during the commissioning activities), the displacement of this joint caused the partial uncovering of the surrounding areas of the pipeline.
- Some of the fill around the pipe was consequently drawn into the pipe leaving a void.
- Finally, the very rough and adverse sea conditions caused the displacement of the joint that, consequently, was no buried anymore.

The reason of the failure was the defective installation of the offshores pipelines. Anyway was decided to make deeper surveys in all the lengths of the pipeline and was found that both the pipes were broken and disconnected from the pipe strings with several displacements and these sections were filled with sediments, etc.
The results of the survey are shown in the figure 3.

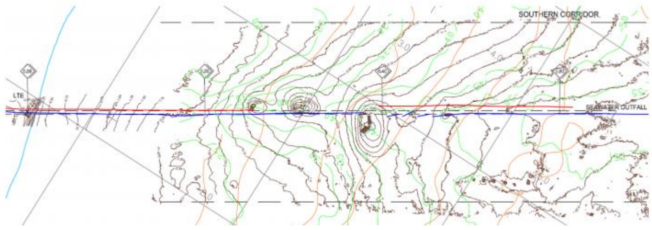


***Figure 3.*** *Detail plan view of damaged CW lines*

These results have shown that e length of 380 m of the intake pipeline and a length of 370 m of the outfall pipeline should be replaced. In order to be successfully was decided to make a hydraulic design review and to find a proper method for pipe installation.
To avoid the joint of the pipes offshore, was decided to prefabricated long strings (approximatively 100 m) onshore and then to transport and laid them on the offshore trench. To put such heavy pipeline in a shallow sea shore it was an engineering challenge and it was not easy to find the proper methodology for the pipeline installation.
Here below we will show the analysis of sinking of pipe string and the methodology used to lay such strings in the trench.

## *3. Hydraulic design review*

In the hydraulic design review were included:

1. Hydrodynamic condition

Waves, water levels, currents, temperature [1]

2. Seabed stability

Bed mobility, seasonal changes, scour [2]

3. Sediment suspension

Concentration profile, ingress rate [3]

4. Pipe hydrodynamics

Hydraulic gradient, sediment transporting capacity [4]

5. Temperature dispersion near the diffuser [5]
6. Sea state analysis
7. In service bottom stability

The results taken from the hydraulic design are shown here below:


***Figure 4.*** *Computer model of wave conditions, at SMB, intake and pipelines routes.*

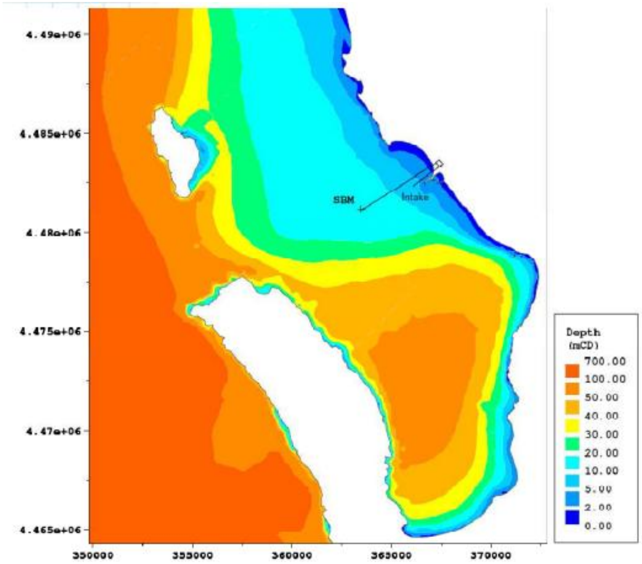


***Figure 5.*** *Bathymetry on gridded model "SWAN"*

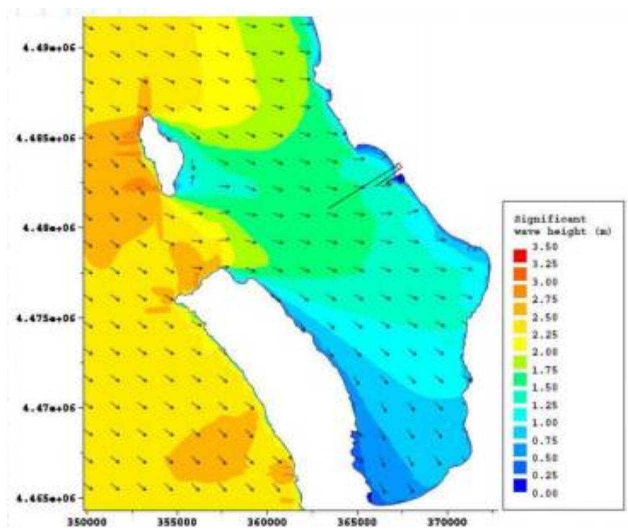


***Figure 6.*** *Transformation of offshore waves and swell plus locally-generated waves*

| Return period (years) | Offshore wave condition | | | | Wave condition at Intake | | | |
|---|---|---|---|---|---|---|---|---|
| | Wave direction (°N) | Significant wave height (m) | Mean wave period (s) | Peak wave period (s) | Significant wave height (m) | Mean wave period (s) | Peak wave period (s) | Wave direction (°N) |
| 1 | 220 | 3.5 | 6.9 | 8.5 | 1.5 | 3.7 | 8.7 | 235 |
| | 250 | 2.4 | 5.8 | 7.1 | 1.6 | 4.2 | 7.1 | 243 |
| | 270 | 2.1 | 5.4 | 6.7 | 1.4 | 3.9 | 6.4 | 251 |
| | 290 | 2.5 | 6.0 | 7.4 | 1.7 | 4.4 | 7.1 | 258 |
| | 320 | 2.7 | 6.3 | 7.7 | 1.6 | 4.1 | 7.9 | 276 |
| | 360 | 2.6 | 6.0 | 7.4 | 1.2 | 3.1 | 7.1 | 291 |
| 10 | 220 | 4.5 | 7.9 | 9.7 | 2.0 | 4.5 | 9.7 | 235 |
| | 250 | 3.6 | 7.0 | 8.6 | 2.5 | 5.2 | 8.7 | 241 |
| | 270 | 3.2 | 6.7 | 8.2 | 2.3 | 5.2 | 7.9 | 246 |
| | 290 | 3.6 | 7.2 | 8.9 | 2.5 | 5.5 | 8.7 | 251 |
| | 320 | 3.7 | 7.3 | 9 | 2.3 | 5.0 | 8.7 | 270 |
| | 350 | 3.5 | 7.0 | 8.6 | 1.8 | 4.2 | 8.7 | 284 |
| 100 | 220 | 5.6 | 8.7 | 10.7 | 2.4 | 5.0 | 10.7 | 235 |
| | 250 | 4.7 | 8.0 | 9.9 | 2.9 | 5.8 | 9.7 | 240 |
| | 270 | 4.2 | 7.6 | 9.4 | 2.9 | 5.8 | 9.7 | 243 |
| | 290 | 4.8 | 8.3 | 10.2 | 2.9 | 6.3 | 10.7 | 247 |
| | 320 | 4.7 | 8.2 | 10.1 | 2.7 | 5.6 | 10.7 | 264 |
| | 350 | 4.3 | 7.9 | 9.7 | 2.3 | 4.7 | 9.7 | 280 |

***Table 1.*** *Offshore and inshore extreme waves and climate*

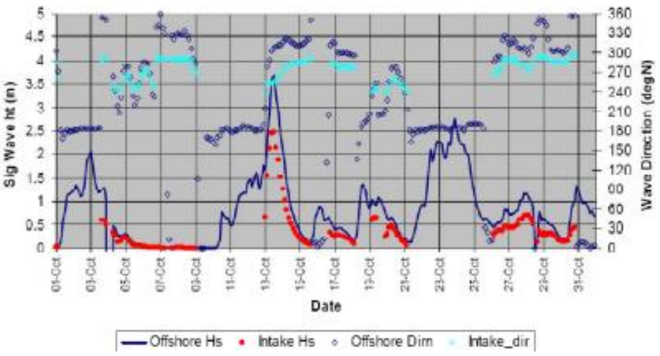


***Figure 7.*** *Offshore and inshore construction storms for one month*

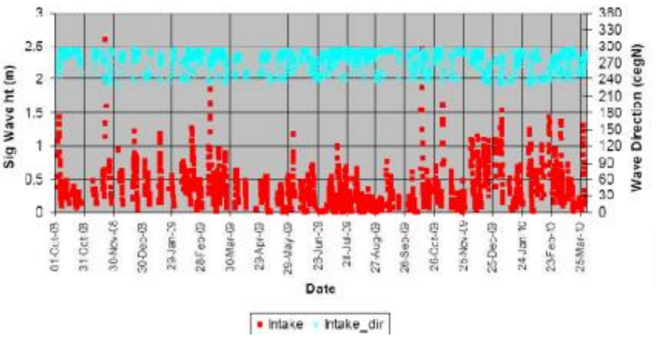


***Figure 8.*** *Offshore and inshore construction storms for two years*

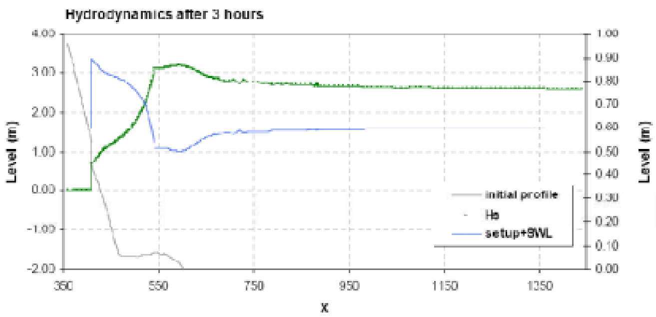


***Figure 9.*** *Waves in the surf zone*

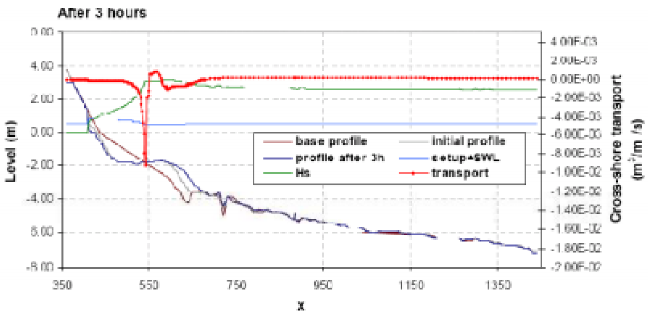


***Figure 10.*** *Bed mobility in the surf zone*

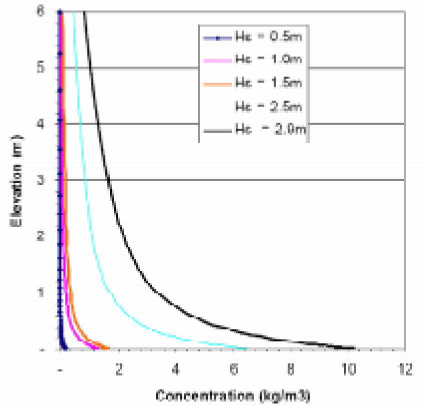


***Figure 11.*** *Sediment suspension by waves of different sizes*

| Test | Frequency | $H_s$ (m) | $T_p$ (s) | $V_r$ (m/s) | $C_m$ (kg/m$^3$) | Ingress (kg/s) | Ingress (tonne/h) |
|---|---|---|---|---|---|---|---|
| 1 | 50 d/y | 0.5 | 8.7 | 0.1 | 0.01 | 0.03 | 0.07 |
| 2 | 11 d/y | 1.0 | 8.7 | 0.1 | 0.12 | 0.4 | 1.5 |
| 3 | 3 d/y | 1.5 | 8.7 | 0.1 | 0.24 | 0.8 | 3 |
| 4 | 1:10 y | 2.5 | 8.7 | 0.1 | 1.15 | 3.8 | 14 |
| 5 | 1:100 y | 2.9 | 10.7 | 0.1 | 2.23 | 7.4 | 27 |

***Table 2.*** *Sediment suspension by waves of different sizes*

| Sand depth (m) | 0.018 | | 0.036 | | 0.09 | | 0.18 | | 0.45 | |
|---|---|---|---|---|---|---|---|---|---|---|
| | $C_v$ kg/s | i m/km | $C_v$ kg/s | i m/km | $C_v$ kg/s | i m/km | $C_v$ kg/s | i m/km | $C_v$ kg/s | i m/km |
| 0.2mm sand | | | | | | | | | | |
| Clean | 2.0 | 0.75 | 2.3 | 0.8 | 3.4 | 1.1 | 5.8 | 1.4 | 18 | 2.4 |
| Aged | 3.0 | 1.2 | 3.4 | 1.3 | 4.6 | 1.5 | 7.3 | 1.8 | 21 | 2.9 |
| 0.1mm sand | | | | | | | | | | |
| Clean | 10 | 0.7 | 13 | 0.8 | 21 | 0.9 | 38 | 1.2 | 168 | 2.2 |
| Aged | 23 | 1.1 | 27 | 1.2 | 37 | 1.4 | 62 | 1.6 | 232 | 2.7 |

***Table 3.*** *Transporting capacity of the pipe.*

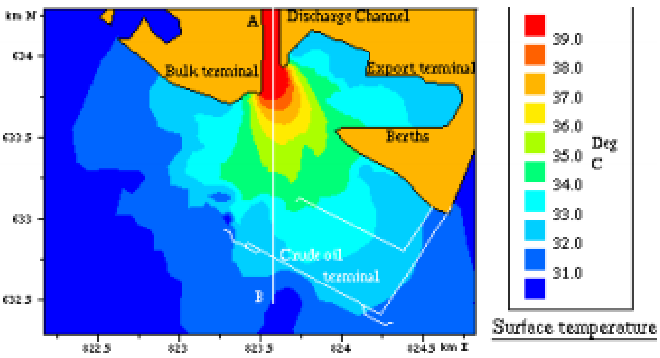


***Figure 12.*** *Surface temperature patterns*

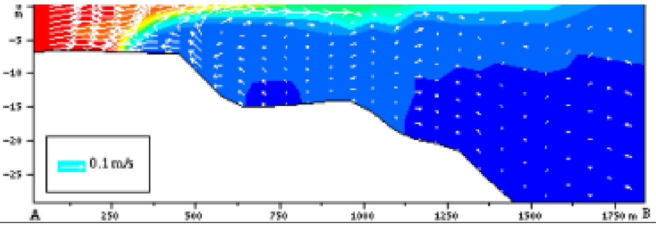


***Figure 13.*** *Vertical section of temperature and flow patterns*

### 3.1 Sea state analysis

#### 3.1.1 Bathymetry and oceanographic input data

The following input data are taken in consideration:

- Current velocity in all direction can be estimated as 0,5 m/sec.
- Significant wave height, corresponding to the position of the water outlet, with a return period of 100 years = 2,5 m.
- Wave period = 6 sec and 11 sec.

The stability analysis is based of a return period of 100 year of near-bottom environmental conditions acting on the pipe. Bothe near bottom wave induced particle velocities and near bottom current induced particle velocities are considered. The effect of the wave have been analyzed by means of the software SACS (Structural Analysis Computer System-Engineering Dynamics, Inc.) and then considering that the waves act on a direction of about 45 ° from thr pipe longitudinal axis. From the safe side, current velocity has been assumed acting in orthogonal direction to the pipe longitudinal axis with value of 0,5 m/sec nearby the water surface [6]. Reduction due to seabed interference is evaluated according to VERITEC RP E305 Code [7].

#### 3.1.2 Sea state results – waves

The velocity and acceleration, during the waves development, i.e. considering various waves position and particles depth, were calculated for wave periods T=6 sec and T=11 sec.

Should be mentioned that $U_S$ and $A_S$ are modified by a directional correction factor (= sen α where α is the direction angle of wave referred to pipeline axis; α = 45°) [8].

#### 3.1.3 Sea state results – currents

To calculate average velocity across the pipeline at the seabed, the following formula (VERITEC RP E305 Code) was applied.

$$\frac{U_D}{U_r} = \frac{1}{\ln\left(\frac{Z_r}{Z_0}+1\right)}\left\{\left[1+\frac{Z_0}{D}\right]\ln\left(\frac{D}{Z_0}+1\right)-1\right\}$$

With the assumption of:

$Z_r$ = 5 m above seabed

$U_r$ = 0,5 m/sec at el. $Z_r$

$Z_0$ is the depending on the grain size of the seabed soil.

For the specific case, i.e. gravel-cobble bed material (after seabed preparation):

$d_{50}$ = 50 mm (grain size)

$Z_0$ = 4,17 x $10^{-3}$ m (roughness).

With D = 0,8 and 0,6 m (only equivalent approx.. to half outer diameter, in reason of partial embedment), substituting in the equation:

Pipe 1600 mm:

$$\frac{U_D}{U_r} = 0{,}60$$

$$\frac{U_D}{U_r} = 0{,}5\ x\ 0{,}6 = 0{,}3\ m/sec$$

Pipe 1150 mm:

$$\frac{U_D}{U_r} = 0{,}55$$

$$\frac{U_D}{U_r} = 0{,}5\ x\ 0{,}55 = 0{,}275\ m/sec$$

### 3.2 In service bottom stability for outfall line

The "in-service bottom stability" calculation was carried out according to VERITEC RP E305 Code (1988) "On bottom stability design of submarine pipes", simplified stability analysis method.

The method gives pipes weights that form a conservative envelope of those obtained from a generalized stability analysis. This method may be used for the stability calculation where the required submerged weight is the only parameter of interest, as in our case.

The stability is given by the following expression:

$$\left(\frac{W_S}{F_W} - F_L\right)\mu \geq F_D + F_t$$

Where:

$W_S$ = submerged weight of the pipe

$F_W$ = calibration factor

μ = soil friction = 0,7 (sandy soil)

$F_L$ = lift force

$F_D$ = drag force

$F_t$ = inertia force

And therefore, the limiting value of submerged weight can be found from:

$$\mathbf{W_S \geq R}$$

Ku:

$$R = \left(\frac{(F_D + F_t) + \mu F_L}{\mu}\right) F_W$$

$$F_L = \frac{\gamma}{2g} D C_L (U_S + U_D)^2$$

$$F_D = \frac{\gamma}{2g} D C_D (U_S + U_D)(U_S + U_D)$$

$$F_t = \frac{\pi D^2 \gamma}{4g} C_M A_S$$

Where:

$C_L$ = 0,9 (lift force coefficient)

$C_D$ = 0,7 (drag force coefficient)

$C_M$ = 3,29 (inertia force coefficient)

$A_S$ = significant acceleration perpendicular to the pipeline at each phase

$U_D$ = Current velocity perpendicular to the pipeline

$U_S$ = significant near bottom wave velocity perpendicular to the pipeline

D = pipeline diameter

The relationship between $F_W$ (calibration factor) and K and M is shown in figure 14.

$$K = \frac{U_S T_U}{D}\ (Keulegan - Carpenter\ number)$$

$$M = \frac{U_D}{U_S}$$

Where:

$T_U$ = mean up-zero crossing period.

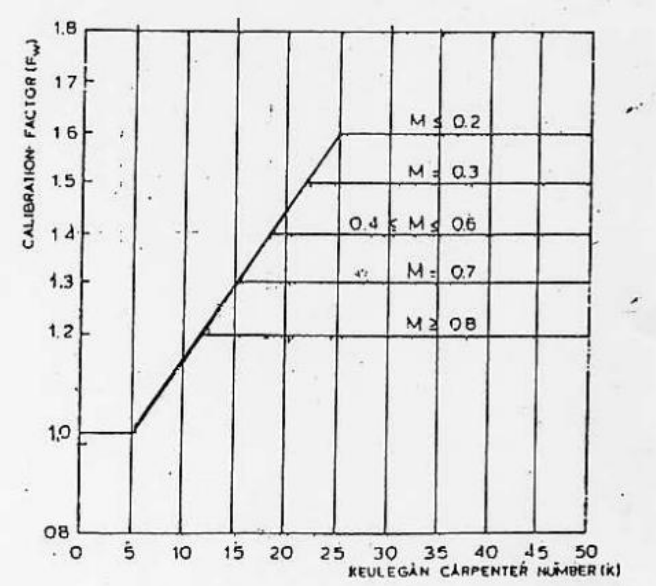


***Figure 14.*** *The relationship between $F_W$ and Keulegan-Carpenter number*

The results for the bottom stability for D = 1600 mm and T = 6 sec are shown in the table 4. The other results are not shown here due to the space that they occupy.
All the results for the bottom stability were ok and the stability of the outfall pipe was ensured.

| PHASE | (°) | 0 | 20 | 40 | 60 | 80 | 100 | 120 | 140 | 160 | 180 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| $U_s$ | (m/s) | 1,35 | 1,23 | 0,91 | 0,47 | 0,02 | 0,36 | 0,65 | 0,84 | 0,95 | 0,98 |
| $U_s$ | (m/s) | 0,95 | 0,87 | 0,64 | 0,33 | 0,01 | 0,25 | 0,46 | 0,59 | 0,67 | 0,69 |
| $A_s$ | ($m/s^2$) | 0 | 0,58 | 1,05 | 1,29 | 1,28 | 1,07 | 0,78 | 0,49 | 0,24 | 0 |
| $A_s$ | ($m/s^2$) | 0,00 | 0,41 | 0,74 | 0,91 | 0,91 | 0,76 | 0,55 | 0,35 | 0,17 | 0,00 |
| $U_d$ | (m/s) | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 | 0,3 |
| $F_l$ | (kg/m) | 60 | 52 | 34 | 15 | 4 | 12 | 22 | 30 | 36 | 37 |
| $F_d$ | (kg/m) | 46 | 40 | 26 | 12 | 3 | 9 | 17 | 24 | 28 | 29 |
| $F_t$ | (kg/m) | 0 | 71 | 129 | 158 | 157 | 131 | 96 | 60 | 29 | 0 |
| K | | 7 | 7 | 5 | 2 | 0 | 2 | 3 | 4 | 5 | 5 |
| M | | 0,3 | 0,3 | 0,5 | 0,9 | 21,2 | 1,2 | 0,7 | 0,5 | 0,4 | 0,4 |
| Fw | | 1,1 | 1,1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| R | (kg/m) | 138 | 232 | 255 | 258 | 232 | 212 | 183 | 150 | 117 | 79 |
| Ws > R | | ok | ok | ok | ok | ok | ok | ok | ok | ok | ok |

***Table 4.*** *The results for D = 1600 mm and T = 6 sec.*

## 4. *Analysis of sinking of pipe string DN 1600*

The analysis of the sinking of the pipe string has been carried out by using CAESAR II software and considering different load cases.

***Figure 15.*** *Floating pf the pipe string DN 1600*

The scheme of lifting points is indicated in the figure 16, where: L = 115 m, $L_1$ = 15 m and $L_2$ = 42,5 m.

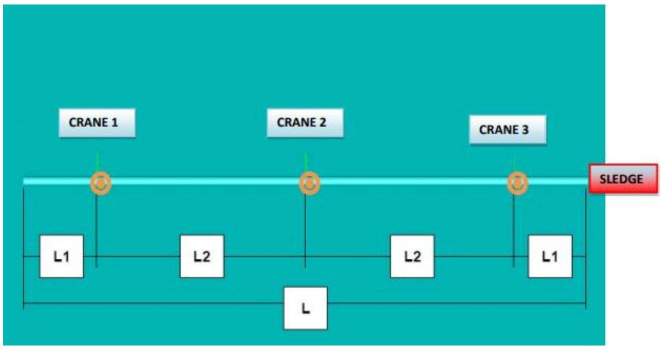


***Figure 16.*** *Scheme of lifting points*

On the left side of the string has been maintained installed during the operation a carbon steel sledge, used for towing with the weight of approximately 2 tons. This equipment was considered as a concentrated force.
The main nodes of the system are indicated below:

- Crane – 1: Node no. 34
- Crane – 2: Node no. 102
- Crane – 3: Node no. 170
- Sledge (concentrated force): Node no. 230

The following allowable stress was verified in the calculation:

Static condition (sustained load).
- Allowable stress due to bending for pipe: 9,88 MPa.
- Allowable stress for Joint subjected to freezing: 6,26 MPa.

Transient condition (occasional load)
- Allowable stress due to bending for pipe: 9,88x1,5 = 14,82 MPa
- Allowable stress for Joint subjected to freezing 6,26x1,5 = 9,39 MPa.

Should be noted that with the string prefabrication onshore, the joints between pipes became stiff, losing their flexibility for angular movements, due to reason of the transport and installation.
The calculation has been carried out considering six calculation schemes. Here we will show the calculation only for the scheme no.1, but obviously the calculation were carried out for all six schemes and the results will be reported here below.
Calculation no. 1.

| | | VERTICAL MOVEMENTS | | |
|---|---|---|---|---|
| LOAD CASE | DISPLACEMENTS | CRANE 1 (mm) | CRANE 2 (mm) | CRANE 3 (mm) |
| LC 1 | D1 | -500 | 0 | 500 |
| LC 2 | D2 | -500 | 0 | 750 |
| LC 3 | D3 | -500 | 0 | 1000 |
| LC 4 | D4 | -500 | 0 | 1250 |
| LC 5 | D5 | -500 | 0 | 1500 |

***Table 5.*** *Scheme of calculation no. 1*

For calculation no. 1, we have obtained the following maximum stress for different load cases:

| LOAD CASE | DISPLACEMENTS | MAXIMUM STRESS (MPa) | NODE |
|---|---|---|---|
| LC 1 | D1 | 12.03 | 170 |
| LC 2 | D2 | 12.03 | 170 |
| LC 3 | D3 | 12.03 | 170 |
| LC 4 | D4 | 12.03 | 170 |
| LC 5 | D5 | 12.03 | 170 |

***Table 6.*** *Maximum bending stress for calculation no. 1*

The maximum stress and deflection were found for all five load cases; LC1 to LC5, but here, due to space reason, we will show just for the load case LC1.

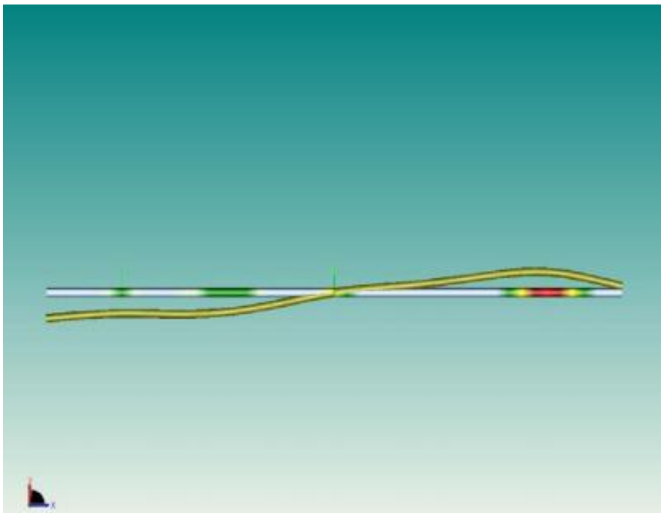

***Figure 17.*** *Deflection and maximum stress C1-LC1*

4.1 Bending and tightness test

Bending and tightness test was carried out, in order to verify the mechanical resistance of the GRF pipe to the stresses induced by the launching methodology and at the same time the hydraulic seal of the joint. The maximum bending moment, which occurs during the launch in the "S" configuration, is: $M_b$ = 618'000 N m. The volumes of water inserted in the pair of pipes were determined by means of a liter counter. During this test several bending moments were realized, using one support-support configuration with joint locked in the center as shown in the figure 18, with a span L = 22 m.

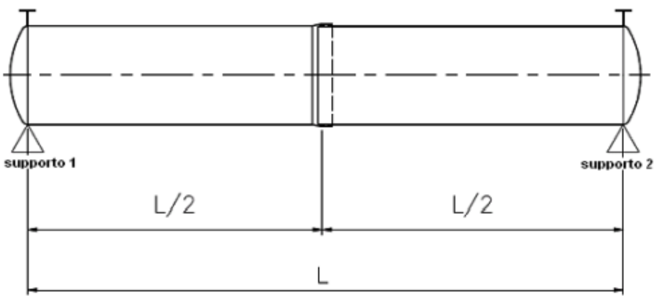


***Figure 17.*** *Bending test configuration*

In the end of the test procedure, which lasted three days, the sample was filled to the maximum. In this condition the sample remained for about 1 hour. The arrow corresponding to the center for this load condition was f = 352 mm.

***Figure 18.*** *The sample of the bending test*

As a result a pair of full-size tubes were tested with the load to which the pipe will be subjected during the launching phases from the pontoon. The result of the test was very satisfactory, as the deformations even after about 70 hours were less than those theoretically expected and, at the same time, the joint passed the infra-O ring seal test. Subsequently it was decided to subject the pair of tubes to maximum stress achievable with the test configuration. Even in this configuration, the deformations found were lower than that foreseen by the theoretical calculation.

In the table 7 and the figure 19 (bending moment Nm vs time hours), are shown the comparison between the deformations found and the theoretical ones and the results of the test.

| Step | Water volume | Support reaction | Bending moment $M_b$ | Expected arrow in the middle | Measured arrow in the middle |
|---|---|---|---|---|---|
| | litri | Kg | Nm | mm | mm |
| MO | 0 | 2585 | 142171 | 40 | 46 |
| M1(1.00) | 16746 | 10957 | 618882 | 173 | 71 |
| M2(1.33) | 23499 | 14446 | 821949 | 260 | 187 |
| M3(1.50) | 27528 | 16372 | 927000 | 272 | 210 |
| M4(2.00) | 36051 | 22023 | 1233703 | 362 | 342 |
| M5(2.16) | 44234 | 24701 | 1332271 | 423 | 352 |

***Table 7.*** *Comparison between the deformations found and the theoretical ones*

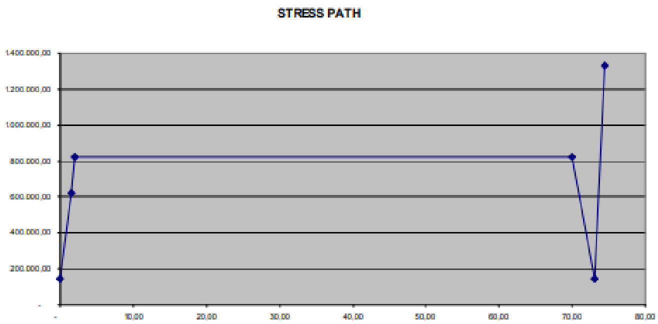


***Figure 18.*** *Stress path, bending moment vs time*

## 5. *Results and discussion*

a. From the calculations carried out, it was evident that the most critical effect on the GRP pipe was the one, by the position of the concentrated force due to the presence of sledge (approximately 2 tons), not disassembled from the string after moving the position from the shore.

b. In these cases, where the central crane was maintained fixed or was moved downward, the maximum bending stress on the pipe was 12.03 MPa.
Considering the transient (occasional) configuration valid for the floating of the string we obtained:
Transient condition (occasional load).
Pipe – Maximum bending stress 12.03 MPa < allowable

bending stress 14.82 MPa (81% of the allowable one).
Joint - – Maximum bending stress 12.03 MPa > allowable bending stress 9,39 MPa (128% of the allowable one).

c. The load cases in which the movement of central crane is upward, are more critical compared to the previous one and we obtained a maximum bending stress of 14,85 MPa for load case no. 5. The comparison with the allowable ones is the following.
Pipe – Maximum bending stress 14.85 MPa = allowable bending stress 14.82 MPa (100% of the allowable one).
Joint - – Maximum bending stress 14.85 MPa > allowable bending stress 9,39 MPa (158% of the allowable one).
d. According to the above results, the verification is OK for the pipe wall in the transient condition (occasional load) and for the joint shows, in the worst scenario, a bending stress 158% of the allowable one.
e. In this case it is possible to have the crack of the resin used for the freezing of the joint, but not the failure of the system joint + pipe, that can withstand without failure a stress 3 times greater than the allowable stress for the pipe according to EN1796.
f. When the pipe is laid on the seabed, the moment will be zero and the joint maintains its hydraulic properties ensuring the sealing through the O-Rings.
g. In order to be sure we have simulated the increase of bending moment on a Bell/Spigot joint positioned in the middle of a span of 22 m (2 pipes, as explained here above). The test was Ok and no failure of joint and pipe was observed at 2,16 times the maximum theoretical bending moment (2,16 times was due to limitation in the equipment). At the same time the hydraulic test of the joint was carried out without any leakage.

## *6. Conclusion*

After the root cause analysis a hydraulic design review was carried out including:

a. Hydrodynamic condition
Waves, water levels, currents, temperature
b. Seabed stability
Bed mobility, seasonal changes, scour
c. Sediment suspension
Concentration profile, ingress rate
d. Pipe hydrodynamics
Hydraulic gradient, sediment transporting capacity
e. Temperature spread near the diffuser
f. Sea state analysis
g. In service bottom stability

The results found were used to upgrade the methodology of the pipeline installation. A detailed analysis of sinking of pipe string was carried out using CAESAR II software and considering different load cases.
The installation of the pipelines were carried out successfully.

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