Mechanical and hydraulical stability of the offshore composit structure

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Abstract: *The aim of this research it was the installation of an intake sea water GRP (glass-reinforced plastic) structure in Vlore Bay. A detailed study about the extreme wave condition offshore was carried out by the SWAN (Simulating Waves Nearshore) model. Then the mechanical structure stability calculation was performed with AxisVM8, a finite element modeler & solver. Conservative hypothesis were taken, such as to use the max static pressure value as per energy balance calculation on the sea waves. The program Fluid Flow was used to model the pipe line end with pipe chimney connections. The scope of the calculation was primarily to balance fluid flows in each chimney in order to match the range of 0, 2 – 0, 25 m/s velocity at each chimney entrance.*

The result of the mechanical and hydraulically stability verification for the sea water intake structure take in consideration it was positive and this structure was installed successfully.

KEYWORDS*: COMPOSITE MATERIAL, SIGNIFICANT HEIGHT OF WAVE, WAVE ENERGY, VON MISE STRESS, STRESS INTENSIFICATION FACTOR.*

1. *Introduction*

In worldwide practice nowadays the use of offshore installations with GRP composite material is in a continuous development.

The main characteristics that the engineered solution was deemed to be inclusive, where:

From the construction point of view:

2. Easiness of construction
3. Easiness of installation
4. Easiness of handling
5. They are virtually non-corrosive to seawater
6. Relatively high solidity / specific weight ratio
7. Limited sensitivity to changing temperatures
8. Their production with relatively large diameters guarantees a low flow rate.

From engineering point of view:

1. Very shallow sea depth (around 2,5 m)
2. Turbulent area in case of windstorm (sea storm) with sea water waves breaking.
3. Possibility, during sea storm to suck-in sand.
4. Possibility, during sea storm to dry a part of the structure.

2. Prerequisites and means for solving the problem

2.1 Definition of wave parameters

2.1.1 Study of wave typology in Vlora bay

The aim of this study is:

1. Determination of the nature of waves in Vlora bay
2. Computer wave modeling resulting in wave distribution, given in a graph of Hs with respect to Tm and Tp for each sector, where:

- Hs- significant wave height
- Tm- wave period
- Tp- peak wave period

2.1.2 Nearshore wave modeling

Wave propagation from offshore and wave growth due to winds in Vlore Bay were modelled using the SWAN (Simulating Waves Nearshore) wave model to predict wave condition in this area [1]. From the study was found that under extreme wave conditions for all directions and for 1, 10 and 100 year return periods, the largest significant wave height is for the 100 year period equal to 4.2m. It should be noted that the water intake structure will receive waves from all directions, which will be considered in its design.

Figure 1, show the results of the SWAN model for a 100 year return period for offshore waves.

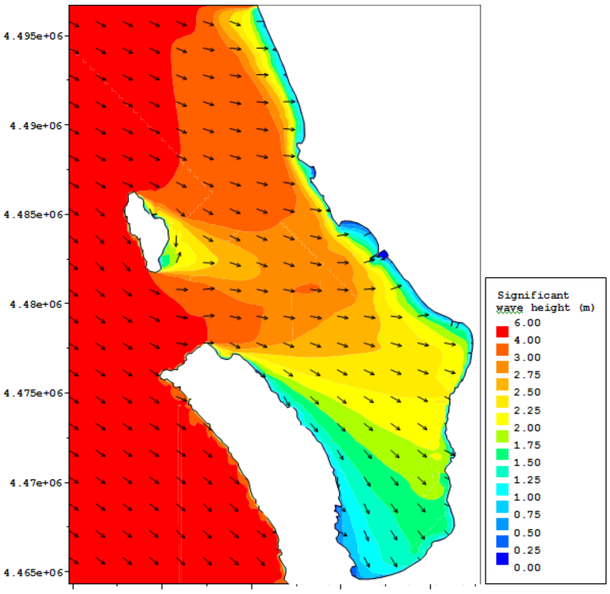

Figure 1. *SWAN model results for 100 year return period offshore wave.*

2.2 Prediction of extreme wave condition (Weibull).

There are several different methods of estimating extreme events from limited data. They are based upon the idea of fitting a standard probability distribution to the range of data which is available. The extreme wave heights are then obtained by substituting the corresponding extreme probability levels into the fitted equation [2].

Extreme value distribution:

$$P(H_s) = 1 - \exp\{-[\frac{H_s - a}{b}]^c\} \qquad (1)$$

Where H_S is the significant wave height, P = probability less than H_S, and a, b, c are parameters to be found.

Weibull scales:

$$\log\{-\log[1 - P(H_s)]\} = c[\log(H_S - a) - logb] \qquad (2)$$

$$y = \log\{-\log[1 - P(H_S)]\} \qquad (3)$$

$$x=\log (Hs-a) \qquad (4)$$

x and y are ploted on linear scales. Waves of a given return period (N years) are determined graphically from the appropriate probability. The expected highest individual wave (H_{max}) in a sequence is related to H_S by the approximate formula:

$$\frac{H_{max}}{H_S} = \left(\frac{1}{2}\ln N\right)^{\frac{1}{2}} \qquad (5)$$

Where N is the number of waves in the sequence.

2.3 The SWAN wave transformation model

SWAN is a computational spectral wave transformation model. It can be used to obtain realistic estimates of wave parameters in coastal areas, from given wind, seabed, and current conditions [3].

The SWAN models represents the waves in terms of the two dimensional wave action density spectrum N (σ, θ), even when the nonlinear phenomena dominate. The action density is equal to the energy density divided by the relative frequency:

$$N(\sigma,\theta) = \frac{E(\sigma,\theta)}{\sigma} \qquad (6)$$

Where σ is the relative frequency and θ is the wave direction. In SWAN the two-dimensional wave action density spectrum may vary in time and space. Its evolution is described by the spectral action balance equation, which for Cartesian coordinates is:

$$\frac{\partial}{\partial t}N + \frac{\partial}{\partial x}C_x N + \frac{\partial}{\partial y}C_y N + \frac{\partial}{\partial \sigma}C_\sigma N + \frac{\partial}{\partial \theta}C_\theta N = \frac{S(\sigma,\theta)}{\sigma} \qquad (7)$$

3. *Solution of the examined problem*

3.1 Design basis and basic assumptions

From the above study we found that the wave climate in Vlore Bay consist of those waves generated offshore that propagate through the west and north entrance to the bay and those generated locally within the bay by winds from the south. The largest wave of 4,5m occur from the direction sector 195 to 225°N. These waves have an associated mean wave period of 7 to 8 seconds. The 100 year return period of significant wave height is 3,5m.

The significant wave height is, by international definition, the average height – measured from wave top to bottom, of the highest 30% waves. This means that, according to probability scatter, the highest wave is normally 1,5 x significant wave. The energy of the wave, i.e the actual height when measured from average sea water level is 1,5/2 x significant wave height. For this reason we can assume by calculation that the effective highest wave, measured from the average sea water level is 2,62m (return period on 4 hours during the highest 100 year storm).

Actually, absolute wave height as indicated is that measured in the Vlore Bay when the effect of shallow sea depth is disregarded. When the sea depth is less than half of the wavelength, the wave starts to "feel" the effect of the sea bottom: its velocity decrease and finally the wave break, losing part of its energy [4]..

The energy of the wave is then lower that the correspondent maximum "static equivalent" height, according to Bernoulli equation (mass and energy conservation principle for fluids in motion). The pressure and the resultant force of the wave break than, associated to this wave height is that due to Stevino's approximation.

The value resulting from Bernoulli-Stevino, was used for mechanical design calculation purposes.

3.2 Geometry description of the structure

In order to limit the flow velocity to a value in the range of 0,2 – 0,25 m/s, seven GRF pipe "chimneys" diameter 1600 and 1800 mm (selected according to hydraulic calculation) shall be welded through lamination PN10 type on a 1800 mm diameter underground piping (the header). In order to limit weight and dimensions of the header two pieces (12m and 9m respectively) of 1800 mm ID pipe shall be connected through an 18000 mm metallic clamp. The connection of the existing pipe end, at 250 m from shoreline shall be executed via another metallic clamp.

The pipe shall be buried to -1,5 m from the sea bottom, while the pipe chimneys, 3 m long each, shall extend 1,5 m from the sea bottom, 1 m from average sea water level.

As per study carried out, it's clear that actual wave height around 250 m from shore line shall be in the range of 1 m or less. Once the pipe connections are verified according to mechanical stability the most problematic phenomenon to be take in consideration is that, on a shallow sea depth, the effect of turbulence of wave break may be a cause of sand entrance in the pipeline. Thus 1,5 m elevation from sea bottom line is deemed suitable, in the range of velocity selected, to limit the risk sand entrance in the pipeline during normal sea condition but may not be suitable in case of severe sea storm, thus causing the CCPP to stop its operation. Further action may be taken afterward to limit this phenomenon, with small modification to the present design.

3.3 Velocity and pressure drop calculation

The program Fluid Flow release 3 was used to model the pipe line end with pipe chimney connections. The scope of the calculation was primarily to balance fluid flows in each chimney in order to match the range of 0,2-0,25 m/s velocity at each chimney entrance, thus increasing the pressure drop where necessary, through pipe chimney orifice, that shall be performed on 1800 mm common header.

3.4 Process calculation results

In the figure 2 is given the simplified geometry of the structure.

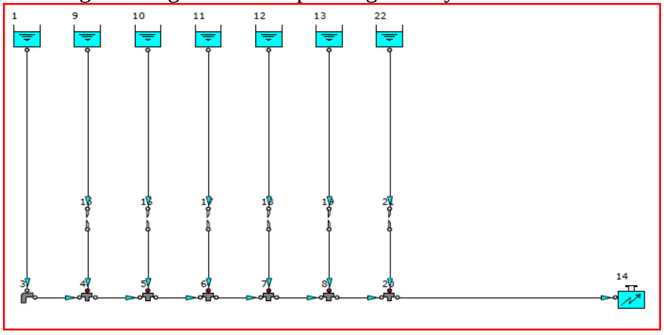

Figure 2. *The simplified geometry*

The data for the tables below are taken directly from the program.

Table 1. *Flow data in pipe stacks*

Unique name	Length (m)	ID (mm)	User number	Flow (m^3/h)	Pressure loss (Pa)	Velocity (m/s)
Pipe 4	3	1600	-14	1608	0,7	0,22
Pipe 7	3	1600	-13	1477	0,6	0,2
Pipe 6	3	1600	-10	1695	0,7	0,23
Pipe 5	3	1600	-8	1813	0,8	0,25
Pipe 3	3	1800	-5	1871	0,5	0,2
Pipe 2	3	1800	-4	1993	0,6	0,22
Pipe 1	3,1	1800	-3	1540	0,4	0,17

Table 2. *Flow data in orifice*

Unique name	Elevation (m)	Orifice size (mm)	User number	Flow (m^3/h)	Pressure loss (Pa)	Corner tap loss (Pa)
Pipe 2	3	1600	15	1993	6,4	0,22
Pipe 3	3	1200	16	1870	118,8	0,2
Pipe 4	3	950	17	1608	287,9	0,23
Pipe 5	3	900	18	1813	497,8	0,25
Pipe 6	3	800	19	1695	807,5	0,2
Pipe 7	3	700	21	1173	0,6	0,22

3.5 Mechanical stability calculation

Based upon input data as settled in 3.1, a computational finite element model was established both for 1600 mm and for 1800 mm pipe stack. Conservative hypothesis were taken, such as to use the max static pressure value as per energy balance calculation on the sea waves [5]. The pipe stack has been modeled as if rigidly connected to a fixed header (the underground pipe header) capable to withstand all transmitted forces. This approach is considered suitable for the purpose of mechanical verification, since appropriate SIF shall be applied to allowable pipe stresses to take into account the “Tee” branch connection geometry, also based upon the fact that the mechanical characteristics of the pipe stack and pipe header are homogeneous.

Pipes material features
Elastic modulus in longitudinal direction: 22000 N/mm^2
Elastic modulus in transversal direction: 11000 N/mm^2
Thick of the pipe: 27 mm
Density: 1850 kg/m^3
Poisson coefficient: 0.2
Lowest allowable stress: 275 Kg/cm^2

3.5.1 Calculation

The analysis was performed with AxisVM8, a finite element modeler & solver. The action of these deformations can be relatively calculated taking account the contribution for lateral soil reaction (about 26 t/m^2) on the part of the underground pipe modelled with spring applied on shell surface. It is necessary to define the values of the maximum pressure, so that they are consistent with the intensity and wave level considered, therefore the scheme of calculation to be adopted is illustrate in the following. Considering a height wave of 2,62 m, the pressure applied to the part of pipe outside the ground (1,5 m) is:

$$\Delta P = \rho g \Delta H = 1000\frac{kg}{m^3} * 9{,}81\,\frac{m}{s^2} * 2{,}62m = 25702Pa = 25{,}7kPa \qquad (8)$$

Once the maximum stresses according to Von Mises are calculated with AxisVM8, then the appropriate Stress Intensification Factor (SIF) according to ANSI is applied in order to compare actual stress to allowable stress. The actual SIF value was calculated modelling with CAESAR II – the piping modeler and stress analysis solver program – the geometry of the piping spool.

For both 1600 mm and 1800 mm branch connection, the calculated SIF is 2,3.

4. Results and discussion

4.1Vertical pipe φ1600 mm.

The figures below show the load case geometry and the relevant mesh definition:

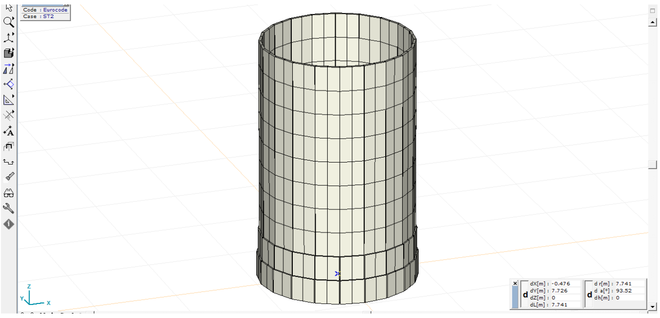

Figure 3. Vertical axial tube geometry

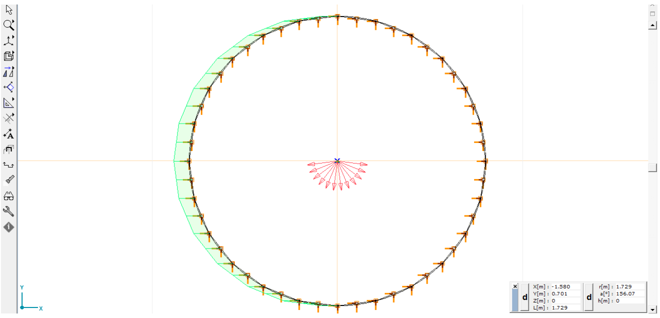

Figure 4. *Node definition and constrains*

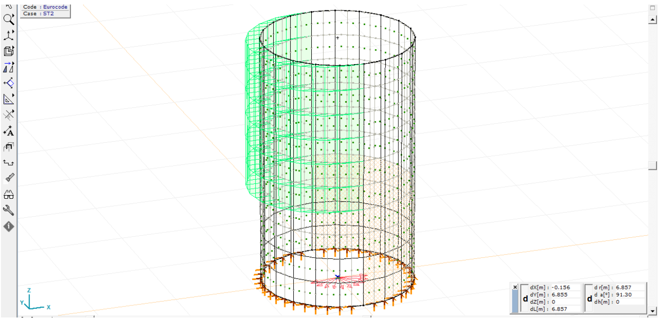

Figure 5. *Loads application on geometrical model*

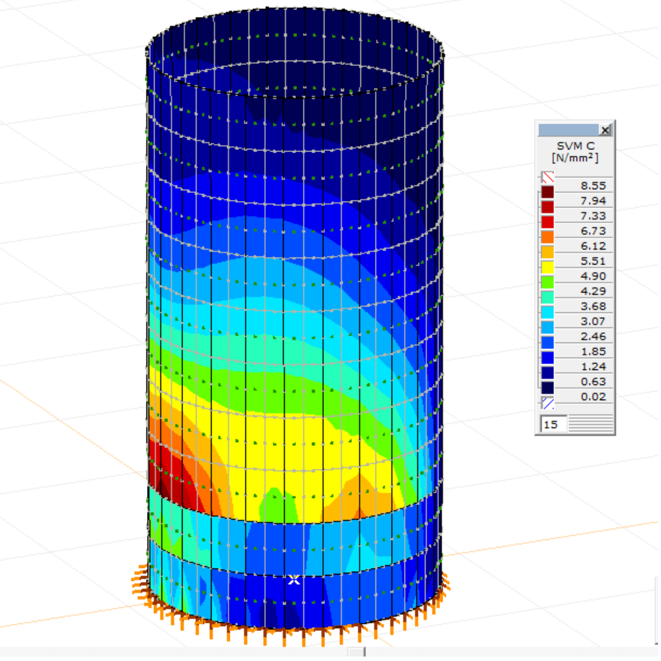

Figure 6. *Stress with Von Mises method*

Maximum stress according to Von Mises = 85,5 Kg/cm^2.
Maximum stress after SIF application = 85,5 x 2,3 = 196,7 Kg/cm^2
Allowable stress = 275 Kg/cm^2.
Safety margin 40 %.

4.2 Vertical pipe φ1800 mm.

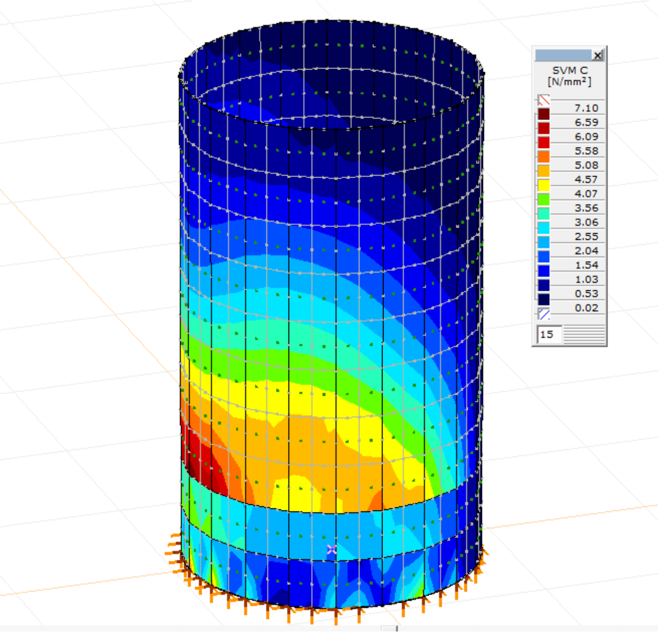

Figure 7. *Stress with Von Mises method*

Maximum stress according to Von Mises = 71 Kg/cm^2.
Maximum stress after SIF application = 71 x 2,3 = 163,3 Kg/cm^2
Allowable stress = 275 Kg/cm^2.
Safety margin 68 %.

5. Conclusion

The structure will be exposed to waves that are generated offshore that propagate through the entrance in the bay towards the site from west and north. The site will also be exposed to waves generated locally by winds from the south-east and south.

Mathematical modeling of the waves and their distribution spectra showed a satisfactory approximation of the real situation in the Vlora bay.

Hydraulic model of the structure, the flow balance having as the criterion that the suction velocity should be in accordance with the environmental directives, led to a clear concept of materialization of the structure with seven vertical tubes and a common horizontal collector.

Finite element modeling of the structure for calculating its mechanical stability, taking into account the action of the waves in the direction of the most significant height as well as the accurate calculation of their pressure from the energy balance, produced a highly reliable model for successfully assembling of the structure in an extremely complex environment of the sea.

6. References

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