Dynamic Monitoring of GBS Terminal

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ABSTRACT: A Gravity Based Structure (GBS) is operating in the Adriatic sea, offshore Italy. The GBS contains two LNG tanks and supports a number of topside steel structures. The design was carried out for two earthquake scenarios: Operating Basis Earthquake (OBE) and Safe Shutdown Earthquake (SSE). Two types of analyses have been performed in the design phase: an analysis based on a three-step solution where the output describes seabed earthquake motions along with the impedance functions of the rigid foundation, and an analysis based on the global (one-step) method including the GBS in the computational model of the soil. The output is the time histories of earthquake motions (accelerations or displacement) at specified points on the structure or the acceleration response spectra at these points. There are three acceleration units positioned at two different elevations on the GBS, one on the East Side (top and bottom) and the other at the West Side (top). Each unit contains 3 linear accelerometers. Relating to the GBS concrete structure, this paper deals with data collection and the statistical analysis of the structural dynamic response data recorded since October 2009. Data recorded during storms and four seismic events, that took place on 20th May-3rd June, 2012 are compared to design accelerations of the GBS, computed for OBE and SSE earthquakes, and are used to determine the GBS's natural frequencies and relevant mode shapes.

1 INTRODUCTION

The following paragraphs show the assumptions that form the basis of the dynamic design of the GBS. Main characteristics, type and location of the monitoring system installed on the GBS will also be provided. In conclusion, the final two paragraphs provide results relating to the acceleration data analysis measured during the most significant seismic and sea storms events experienced to date.

2 DESIGN SEISMIC ANALYSIS

The design analyses of the GBS was carried out in several steps: to characterize the structural spectra and the interaction between soft and stiff soils and the GBS structure and for the calculation of the spectra at the base of the topside structures. The GBS has been designed considering the GBS-to-soil interactions and the static and dynamic loads due to the Operating Basis Earthquake (OBE) and to Safe Shutdown Earthquake (SSE). OBE has a return period of 475 years and will not cause any damage/interruption of normal working conditions, while the SSE with a return period of 5000 years, the safe shutdown of the plant and complete escape evacuation and rescue operations are facilitated. The analyses allow to determine the stiff and damping of soil, free-field seabed motion and Soil-Structure Interaction (SSI). Two types of analyses were performed: 1) An analysis based on the three-step solution in which the output are seabed earthquake motions along with the impedance functions of the rigid foundation, 2)

Analysis based on the global (one-step) method in which the GBS is included in the computational model of the soil. In the Type 2 analysis, the output is either the time histories of the earthquake motions (acceleration or displacement) at specified points on the structure, or the acceleration response spectra at these points.

The objective of this type of analysis is to provide the structural engineer with a simplified soil interface model and earthquake excitation so that he can undertake a SSI analysis with a refined model of the GBS and the tanks. The three-step method, was proposed by *Kausel et al. (1978)*.

The FEM analysis allows to determine the static earthquake load cases where the structure is assumed to behave linear elastically. The GBS structure is modeled with a well known program, ABAQUS, 4-noded shell elements of type S4R. A plot of the GBS model is shown in Figure 1 and of the Topside structures in Figure 2.

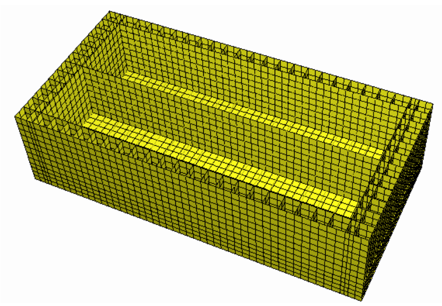

Figure 1. Mesh of GBS

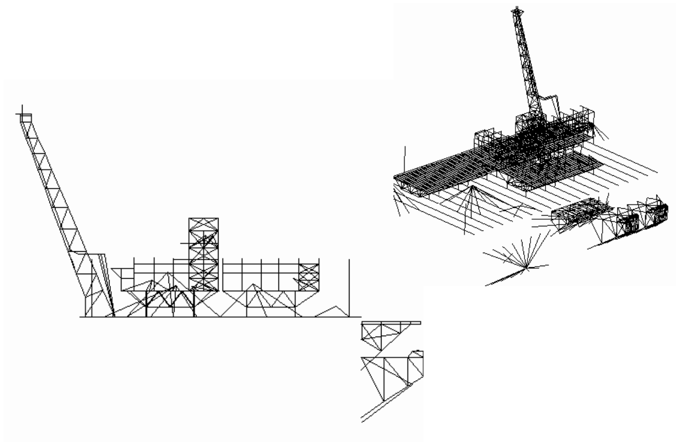

Figure 2. Mesh of Topside structures

3 FEATURES OF THE MONITORING SYSTEM

A Structural and Geotechnical Monitoring System is installed in order to monitor a GBS structural response during normal conditions and during exceptional events. For example, seismic activity, high loads from wind, waves and current. The structural and geotechnical monitoring system consists of inclinometers installed at the top of GBS, pore pressure sensors located in the soil foundation, strain gauges located at the top of LNG tanks and accelerometer at the bottom and top slabs of the GBS. The static sensors: inclinometers, pore pressure and strain gauges have acquisition frequencies of 1 Hz or less, and allow to check the GBS position, soil foundation pressure and strains on the roof of tanks. The accelerometers with acquisition frequency of 16 Hz allow to detect the dynamic response of the structure during the time variable actions and specifically the response during an earthquake and allow to determine the shapes and frequencies of vibration (see Figures 3 and 4).

4 ANALYSIS OF THE ACCELEROMETER - RAW DATA

This section provides primary results of the structural analysis of the raw accelerometer data

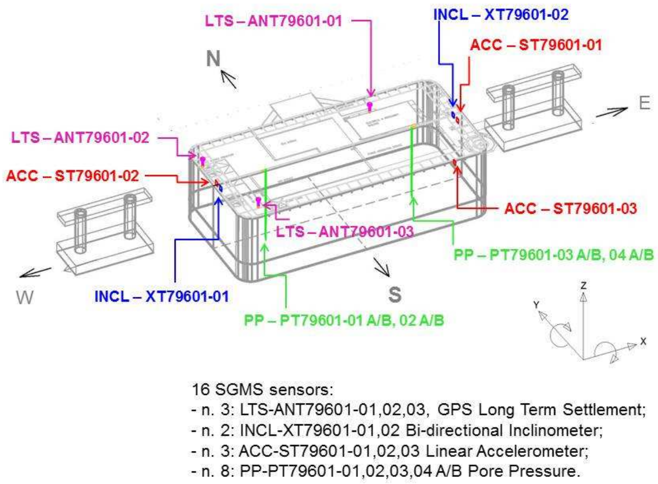

Figure 3. Position of 16 monitoring sensors.

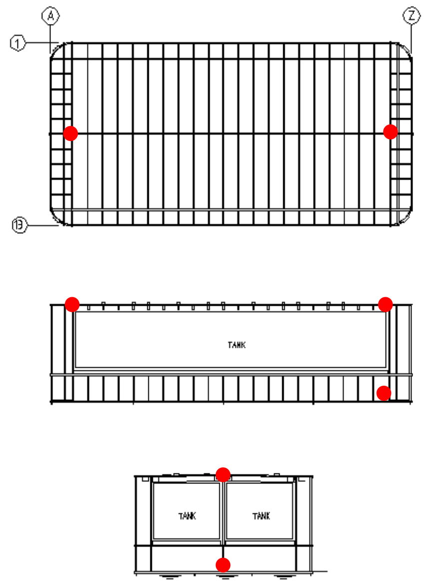

Figure 4. Accelerometer locations.

which was recorded during the seismic events occurring on May, 20th 2012 and June 3rd 2012. (see Figures 5 and 6). The analysis was focused on the validation of the raw acceleration data recorded in the *x*, *y* and *z* directions by the 3 stations and on the assessment of the GBS's structural response compared to the design values, with reference to OBE and SSE scenarios (see the following Table 1).

The acceleration stations, positioned on the bottom and top slabs of the GBS, allow to evaluate the amplifications between the two levels and to compare the monitored value to values resulting from the seismic design analysis. For each recorded seismic event in the period from 20th May – 03rd June, acceleration data was analyzed at an acquisition frequency of 16 Hz over a 20-minute-time frame starting 10 minutes before each event. As shown in Table 1, the two highest earthquake intensities were recorded on 20th May , and on 29th May .

The accelerations recorded at top slab level, N-S direction, show the GBS's dynamic response to seismic actions on 20th May, to have a maximum value of 0.25 m/s². The recorded data during the two seismic events (20th May-3rd June) are compared to design accelerations of the GBS, computed for OBE and SSE earthquakes using upper bound soil parameters, post-consolidated and empty LNG tanks. Maximum recorded accelerations were around 15% of the OBE scenario, well below the design limits, but with enough intensity to evaluate a GBS structural response. The maximum event of 20th May, shows in the *y* direction (North-South) an acceleration amplification of 1.7 lower than 2.1, as adopted in the design between bottom and top slabs of the GBS. The attention was directed to validate the raw acceleration data in the *x*, *y* and *z* directions at the 3 stations during the seismic periods and to compare the acceleration values to the computed ones for OBE and SSE design conditions. (see the following Table 1)

Table 1. Monitored and computed accelerations

Sensors	Recorded data (m/s2)				Design data (m/s2)	
	20 May 2012	23 May 2012	29 May 2012	03 June 2012	OBE	SSE
A1x	0,096	0,003	0,060	0,003	0,9	1,8
A2x	0,100	0,003	0,054	0,007	0,9	1,8
A3x	0,071	0,003	0,043	0,006	0,8	1,5
A1y	0,250	0,012	0,170	n.a.	1,5	3,0
A2y	0,220	0,004	0,070	0,009	1,5	3,0
A3y	0,150	0,004	0,060	0,008	0,7	1,4
A1z	0,033	0,001	0,025	0,006	1,0	2,3
A2z	0,057	0,001	0,020	0,003	1,0	2,3
A3z	0,039	0,001	0,026	0,005	0,6	1,5

The seismic design was performed in the following three phases: study of soil-structure interaction, computation of GBS and topside/sidewall structures to determine the response spectra in OBE and SSE load conditions of the main buildings, and seismic analysis of the single buildings.

The monitored accelerations of the GBS allow to identify the amplifications due to the foundation and reduce the uncertainties of the soil response during an earthquake with an intensity similar to the one which loaded the structure on 20th May. A first evaluation of the dynamic response allows us to say that the dominant frequency is lower than the design results and the amplification at the top of the GBS are less than the ones computed during the design. Less stiff and therefore with a higher natural fundamental period in the N-S direction of the GBS, suggests to reconsider the consequences of the dynamic response of the single topside or sidewall structures. The reduction of frequency and therefore of the rigidity of the structure, mainly due to the foundation system, could cause different and greater oscillations which can also be perceived at the Living Quarter level.

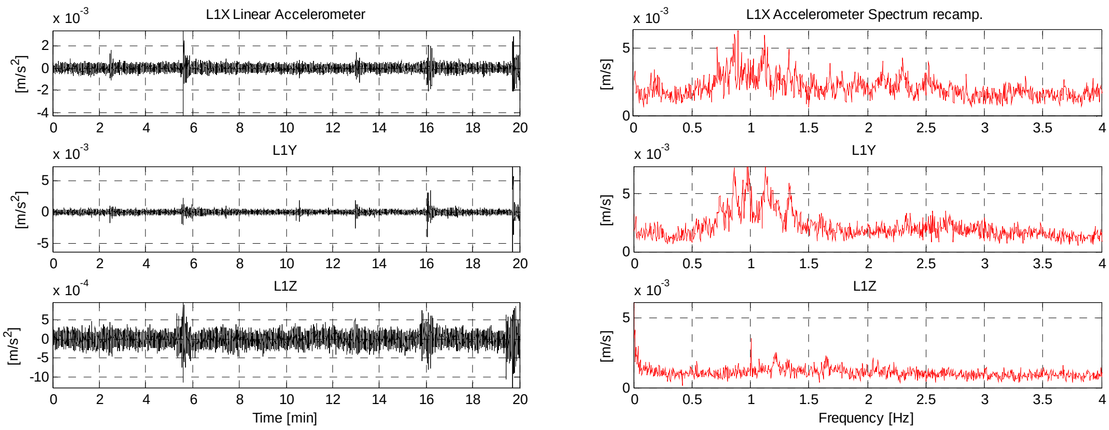

Figure 5. Accelerations and Fourier spectra Earthquake 2012/05/20.

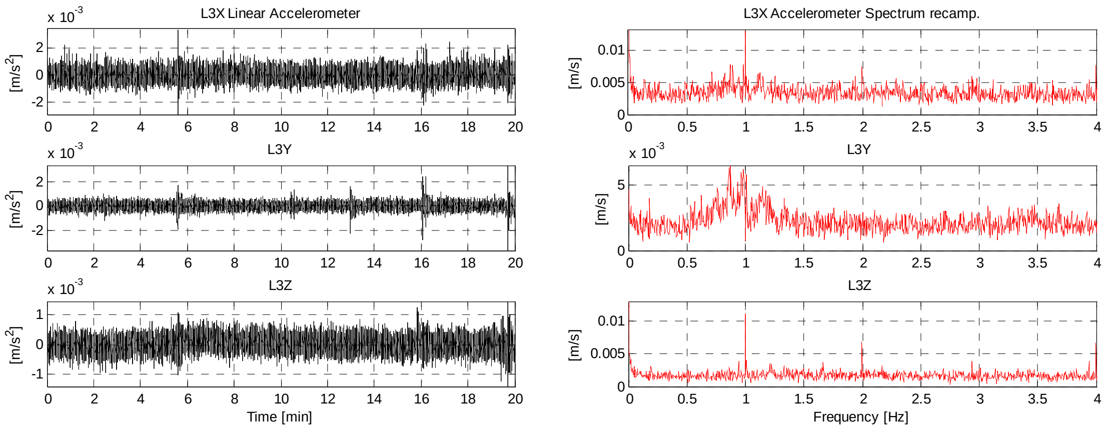

Figure 6. Accelerations and Fourier spectra Earthquake 2012/06/03.

5 DYNAMIC IDENTIFICATION AND MODE SHAPES ANALYSIS

5.1 *Stochastic systems: problem description*

Next are shown the basic steps of the Stochastic subspace identification algorithms that compute state space models from given output data as shown in Peeters and De Roeck (1999). The used form is the covariance-driven version of the algorythm.

The output $y_\kappa \in \Re^{l}$ is supposed to be generated by the unknown stochastic system of order *n*:

$$\begin{aligned} x^s_{\kappa+1} &= \mathrm{A}\, x^s_{\kappa+1} + w_\kappa \\ y_\kappa &= \mathrm{C}\, x^s_{\kappa+1} + v_\kappa \end{aligned} \tag{1}$$

with w_κ and v_κ zero mean, white vector sequences with covariance matrices given by

$$E = \left[\begin{pmatrix} w_p \\ v_p \end{pmatrix} \begin{pmatrix} w_q^T & v_q^T \end{pmatrix}\right] = \begin{pmatrix} \mathrm{Q} & \mathrm{S} \\ \mathrm{S}^{\mathrm{T}} & \mathrm{R} \end{pmatrix} \delta_{pq} \tag{2}$$

As is known, the order *n* of the system is unknown; the system matrices have to be determined $\mathrm{A} \in \Re^{nxn}$, $\mathrm{C} \in \Re^{lxn}$ up to a similarity transformation as well as $\mathrm{Q} \in \Re^{nxn}$, $\mathrm{S} \in \Re^{nxl}$, $\mathrm{R} \in \Re^{lxl}$ so that the second order statistics of the output of the model and of the given output are equal.

The main step of stochastic subspace identification problem is the projection of the row space of the future outputs into the row space of the past outputs, as shown in the work of Van Overschee and De Moor (1996).

5.2 *Sea storms and earthquakes analysis*

In this section we report the results obtained from the stochastic subspace identification problem on two seismic events (May, 20th 2012 and June 3rd 2012), whose accelerometer data have been reported in Figures 5 and 6 including four sea storms. Table 2 takes into consideration and shows the characteristics relating to the sea storms events.

Table 2. Characteristics of the sea storm events.

	WEST wavemeter		EAST wavemeter		LQ anemometer	EAST anemometer
	Significant Wave Height [m]	Peak Wave Direction [degree]	Significant Wave Height [m]	Peak Wave Direction [degree]	Wind Direction [degree]	Wind Direction [degree]
28 Aug 2011 15:00	0,83	199,55	1,22	80,46	177,39	186,30
19 Dec 2011 08:00	2,09	154,01	2,36	108,95	57,93	30,33
01 Feb 2012 12:00	3,33	198,48	3,89	5,90	38,60	31,67
01 Feb 2012 14:00	3,16	166,45	3,84	9,56	37,81	34,23

The choice of storms was made for different directions and intensity of the sea. both Bora and the Scirocco winds were involved. The stochastic subspace identification analysis has led to the results reported in Table 3, taken into account the relative earthquakes and sea storms.

Table 3. Mode shapes identified for earthquake and storm events.

	FEM	20 May 2012	03 June 2012	28 Aug 2011	19 Dec 2011	01 Feb 2012	01 Feb 2012
f_1 (Hz)	1,20	1,11	0,96	0,95	1,15	1,26	1,30
f_2 (Hz)	1,25	1,33	1,18	1,14	1,47	1,60	1,70

Tables 2 and 3 show that the structural dynamic behavior is influenced by the stiffness of the soil, in particular to an action of stronger intensity corresponding to a more rigid structural behavior. Furthermore in Figures 7 and 8 show the mode shapes obtained from the FEM analysis.

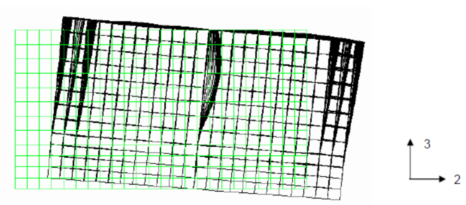

Figure 7. FEM Lumped parameter foundation model, mode number 3 at 1.20 Hz.

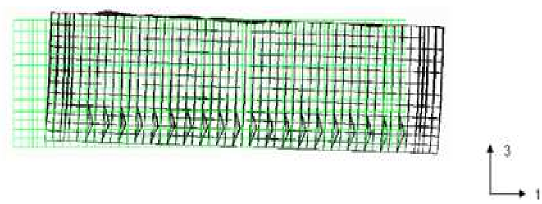

Figure 8. FEM Lumped parameter foundation model, mode number 4 at 1.25 Hz.

In addition, the stabilization diagram can be used to define the probability density of structural resonance. The Probability Density Function (PDF) could be built by means of a Gaussian base according to Eq. (3) where O_{min} and O_{max} represent the minimum and maximum order of the SSI model and N_f is the number of identified main frequencies.

$$p(f) = K \sum_{h=O\,\min}^{O\,\max} \sum_{k=1}^{Nf} \exp\left(-\frac{(f - f_{hk})^2}{2\sigma_h^2} \right)$$

where,

$$K = \left[\int_{-\infty}^{+\infty} \sum_{h=O\,\min}^{O\,\max} \sum_{k=1}^{Nf} \exp\left(-\frac{(f - f_{hk})^2}{2\sigma_h^h} \right) df \right]^{-1} \quad (3)$$

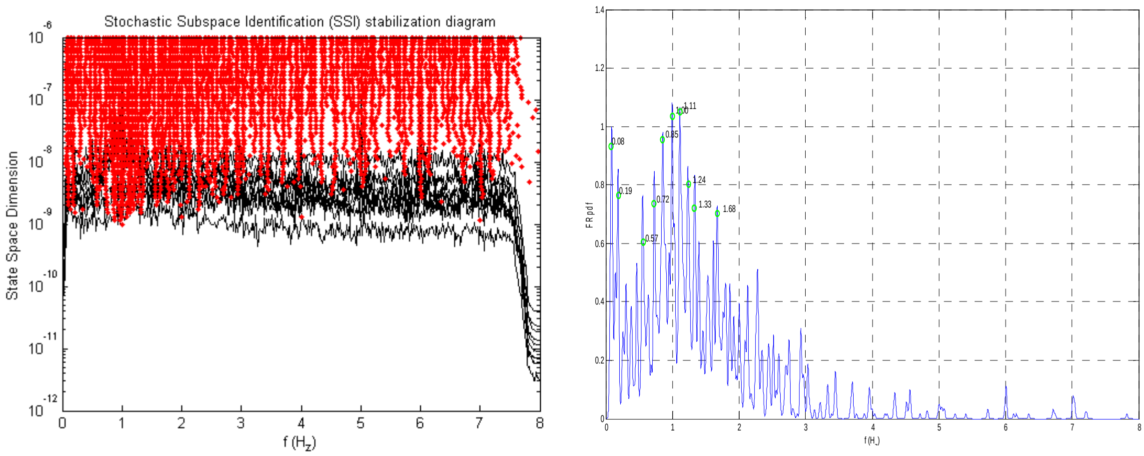

Figure 9. SSI analysis, stabilization diagram and PDF for earthquake 2012/05/20.

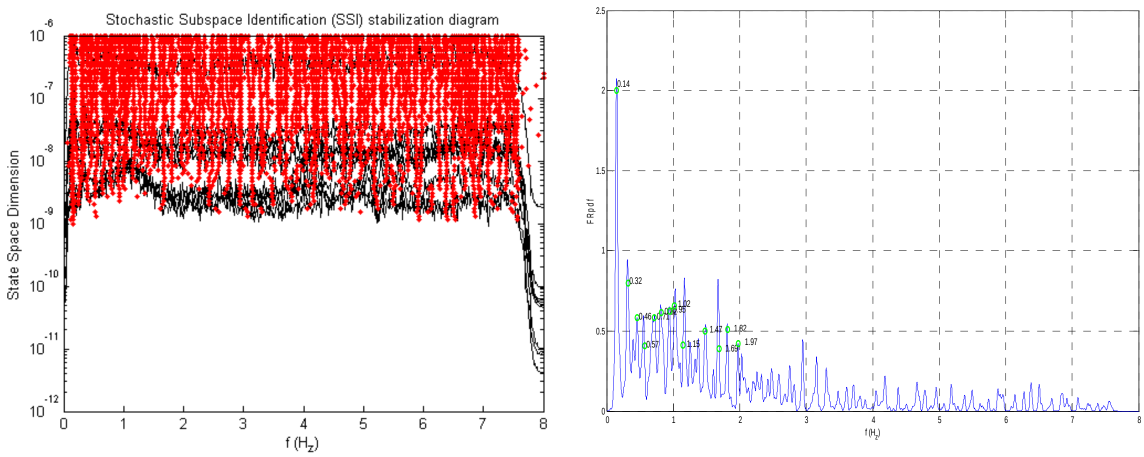

Figure 10. SSI analysis, stabilization diagram and PDF for sea storm 2011/12/19.

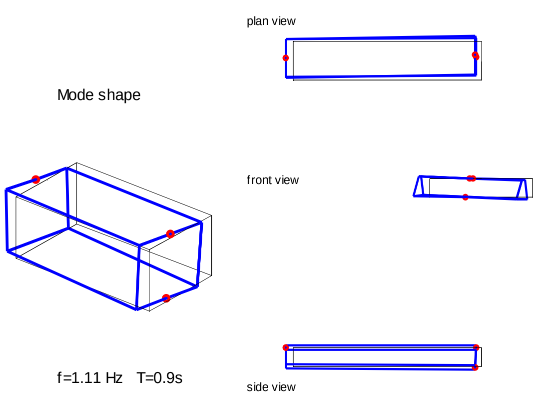

Figure 11. SSI analysis, earthquake 2012/05/20, mode at 1.11 Hz.

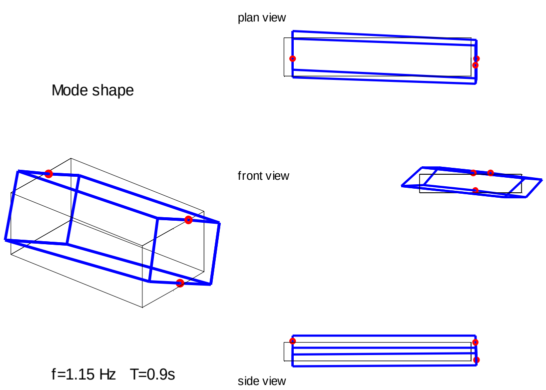

Figure 12. SSI analysis, sea storm 2011/12/19, mode at 1.15 Hz.

Figure 9 and 10 is an example of the stabilization diagram and the PDF of structural resonance.

Once the eigenfrequencies were estimated in Tab. 3, the corresponding mode shapes are retrieved by means of the Singular Value Decomposition. The estimated mode shapes are reported in Figure 11 and 12, for the earthquake of 2012/05/20 and for the sea storm of 2011/12/19.

6 CONCLUSIONS

The comparison of monitored data and computed acceleration data, maximum acceleration of 0.02g (same order of magnitude of OBE design scenario), and computed acceleration data has confirmed the coherence of experimental data with design data. The values of amplification at top of the GBS are lower and therefore the seismic loads applied to LNG tanks and main topside structures during these events are less stressed thus ensuring safer conditions.

The frequencies, computed also during the sea storms, show a natural vibration frequency in the N-S direction, lower than computed for the FE model during the design. The monitored behavior should reduce the seismic static loads and stresses of the GBS concrete structure and also in the more stiff topside and sidewall structures. Otherwise the presence of a lower natural period of the GBS could activate some stress amplifications of supported structures with a natural period near to that of the GBS.

Future developments will be the validation and elaboration of environmental data and the structural monitoring system in an organic manner which will check the structural response under wave, wind and seismic actions. The response under significant actions, wave and seismic events, are useful to compare the structural response with the effect level under actions assumed in the design phase.

The results of the monitoring system are a valuable tool for evaluation of the structural response during the life of the GBS terminal and offer support in the risk based inspections.

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