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SEISMIC BEHAVIOR OF MEMBRANE AND FULL CONTAINMENT TANKS INCLUDING SOIL STRUCTURE INTERACTION

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ABSTRACT

A comparative study of the seismic behavior of “classical” full containment LNG and membrane technology tanks is presented in this paper. The aim of the study is to evaluate the seismic response of the two technologies in areas subjected to severe seismic conditions.
A 9%Ni full containment tank and a membrane tank fitted with GST technology were analyzed. They are of similar net volume capacity of approx 180,000 m^3 and considered as resting on the same soil stratigraphy. Due to the poor site soil conditions, stone columns and vertical drains are required to increase foundation capacity and reduce settlements.
The seismic analysis is developed in a two-step approach with increasing accuracy, i.e. 1) following the simplified procedure reported in Eurocode 8, with references to other standards, in which the spring-mass analogy is considered, and 2) a detailed finite element model of the tanks in order to obtain the internal wall actions and the distribution of the soil pressure under the foundation. The soil structure interaction (SSI) and the fluid-structure interaction are considered in both the simplified and the refined methods for each tank.
The study demonstrates the feasibility of a GST membrane tank of large capacity in poor soil conditions and severe seismic environment, where a classical full containment LNG tank would instead require specific disposition/reinforcement to cope with such conditions.

Keywords: seismic analysis, LNG tanks, soil-structure interaction

1. INTRODUCTION

Nowadays the world is characterized by an increasing energy demand. As a result, more and larger LNG storage tanks are required. Often, large LNG tanks are built in zones prone to severe seismic conditions and/or near-shore where very poor soil conditions can be present. Typical LNG terminals have aboveground storage tanks with capacities ranging from 160,000 m^3 to 180,000 m^3. In the case of a containment failure, the economic damage would be substantial and the danger to life, property and environment correspondingly great. Consequently, storage tanks for liquefied gases at low temperature require most advanced design and construction techniques to ensure that the potential risk involved is reduced to a minimum.
This paper deals with the study of the seismic behavior of membrane GST technology and “classical” full containment LNG tanks. In general, a full containment tank is composed of an inner, self-standing, steel tank and an outer concrete tank (Figure 1, right). The inner tank is cylindrical and open at the top and made of cryogenic steel (9% Ni) in order to ensure adequate ductility at the operating temperatures and rests on thermal insulation placed on the base slab of the outer tank. The outer tank is made of concrete. The cylindrical wall is post-tensioned, both in the vertical and hoop directions. The base slab and the spherical dome consist of simply reinforced concrete. Adequate thermal insulation is provided between the two tanks.

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For a classical full containment tank majority of actions are absorbed by the outer tank, while the inner is protected by the outer tank and is only designed to contain the liquid gas. In case of strong seismic input the classical full containment tanks can be seismically isolated, with some implications on both experienced displacements and realization costs increase. Comparative assessments for seismic isolation are presented in few papers, e.g. Marì et al. 2010.
A tank fitted by membrane technology is similar to 9%Ni tank, they both have a robust pre-stressed concrete outer tank. Membrane technology combines tightness and thermal insulation functions, ensured by a multi layer sandwich fixed on the concrete wall. The structural functions in operations are ensured by the concrete tank. The GST system is made of insulation panels fixed to the wall, plus a corrugated membrane installed on the panels that ensures the tightness to gas and liquid. The system also includes a thermal Protection system at the wall to base connection to protect the corner in case of major leak as requested by EN14620 Standard (2006).
The aim of the study is to evaluate the behavior of these two different tank technologies under high seismic action. The LNG tanks considered have an internal radius of 40.47 m and a liquid level height of 37.90 m and are located in a zone with high seismic hazard.
The first part of the paper presents an overview of the soil conditions and of the main design parameters considered as well as the tank geometries and characteristics. The modelling approaches are described (simplified procedure and FEM analyses) in the second section. The paper shows the applicability and the main peculiarities of the different approaches reported in international codes and standards and it shows pros and cons of different finite element procedures. Only the seismic behavior of the two tank technologies is discussed in the paper. A comparison in terms of technological benefits is out of the scope of this study and interested readers can refer to Ezzarhouni et al. (2016).

2. DESCRIPTION OF THE CASE STUDY

2.1 Tank Geometries and Properties

In terms of seismic response, the main difference between membrane tank and full containment tank is related to how the seismic actions due to the fluid inertia are transferred to the foundations. In the case of the membrane tank, having only one external wall, the reinforced concrete mantel receives the inertial actions and transfers them to the foundations. On the contrary, the full containment tank has an outer reinforced concrete tank that receives only the inertial load due to its mass and a secondary inner steel tank that has to sustain the inertial load due to the fluid mass. Figure 1 shows a graphical representation of the two different tank technologies.

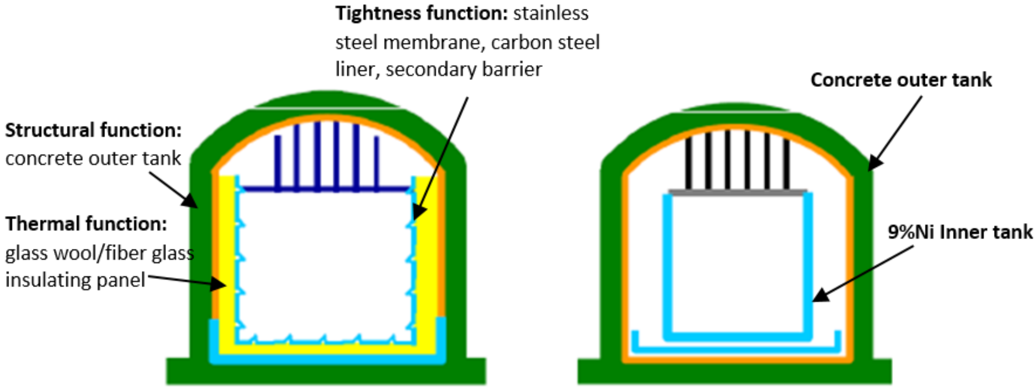

Figure 1. Conceptual simplified representation of membrane tank (left) and full containment tank (right).

The main geometrical properties derived from the net volume capacity for the two tank technologies (180,000 m^3) are described in Table 1.

Table 1. Geometrical properties of the membrane and full containment tanks.

		Membrane Tank	Full Containment
Property	**Symbol**	**Value**	**Value**
Height of the tank wall (m)	H_{tot}	43.32	n.a
Height of the outer tank wall (m)	$H_{tot,outer}$	n.a	38.44
Height of the inner tank wall (m)	$H_{tot,inner}$	n.a	34.97
Height to the free surface of the liquid for the OBE (m) - Design	H	37.90	31.98
Height to the free surface of the liquid for the SSE (m) - Normal	H	37.11	31.90
Radius inside concrete polygon (m)	R	40.40	45.00
Radius inside membrane polygon (m)	R	39.99	44.00
Tank slenderness (-)	H/R	0.94	n.a.
Inner tank slenderness (-)		n.a.	0.79
Equivalent uniform thickness of the tank wall (m)	s_w	0.80	n.a.
Equivalent uniform thickness of the outer tank wall (m)	$s_{w,outer}$	n.a.	0.80
Equivalent uniform thickness of inner tank wall (m)	$s_{w,inner}$	n.a.	0.02
Equivalent uniform thickness of the tank foundation slab (m)	s_b	0.50	0.50
Foundation diameter (m)	D	84.50	n.a.
Outer tank foundation diameter (m)	D	n.a.	93.60
Net capacity volume (m^3)	V	175,000.00	180,772.00
Liquid volume for the OBE (m^3)	V	190,306.00	194,329.00
Liquid volume for the SSE (m^3)	V	186,309.00	193,903.00

In the previous table the liquid levels are related to the two different seismic scenarios: the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE) based on the definition of the NFPA (2013).
The liquid level assigned to the SSE, which is the seismic event with the largest return period, is the normal operating level while the fluid height assigned to the OBE is the design liquid level. This is because the less frequent earthquake is associated with the most probable level of liquid and vice versa.
Some observations can arise from Table 1. First, the membrane tank has a smaller gross volume than the full containment tank for very similar net volume capacity. The small difference in terms of net capacity volume, 4%, is due to differences in the design assumptions developed at the concept functional design phase but this difference will not affect the results of this study. Second, the membrane tank has a greater slenderness than the full containment tank. Last observation is that the membrane tank has a smaller foundation diameter than the full containment tank.
Concerning the material properties considered in the present study, the concrete of the tank wall and foundation is a C45/55 class concrete. The LNG fluid density is equal to 480 kg/m^3. The inner tank is made of 9%Ni ASTM A533 steel with allowable stresses equal to 320 MPa and 306 MPa for the SSE and OBE respectively.

2.2 Soil Conditions

The soil stratigraphy at site comprises about 18-20 m of sediment deposit overlaying bedrock. Given the poor mechanical properties of the soil, installation of stone columns and preloading has been considered. The improvement of the soil properties has been evaluated following Priebe's methodology (1995). Resulting dynamic soil properties are synthetized in the Table 2.

Table 2. Soil stratigraphy and dynamic soil properties of the ground after improvement

Layer	Depth[1] (m)	Mass density, ρ (t/m^3)	Shear Wave Velocity, V_s (m/s)	Small Strain Shear Modulus, G_0 (MPa)
Structural Fill	0.0 to -4.0	1.8	170	52.0
Fill	-4.0 to -6.0	1.4	210	61.7
Very Soft to Soft Clay	-6.0 to -8.5	1.2	84	8.5
Silt/Sand Clayed Matrix	-8.5 to-17.3	1.9	186 to 228	65.7 to 98.8
Coarse Sand	-17.3 to -22.0	1.9	210	83.8
Clay	-22.0 to - 26.0	1.9	500	475.0
Claystone	-26.0	1.9	760	1097.0

Note: (1) Depth from foundation level.

2.3 Seismic Input

The area is characterized by severe seismic conditions, having peak ground accelerations (PGAs) on rock of 0.84 g and 0.52 g for the SSE and OBE, respectively. A 1D site response analysis was performed in order to evaluate the seismic response spectra at the ground surface. The results in terms of response spectra of accelerations are shown in Figure 2.

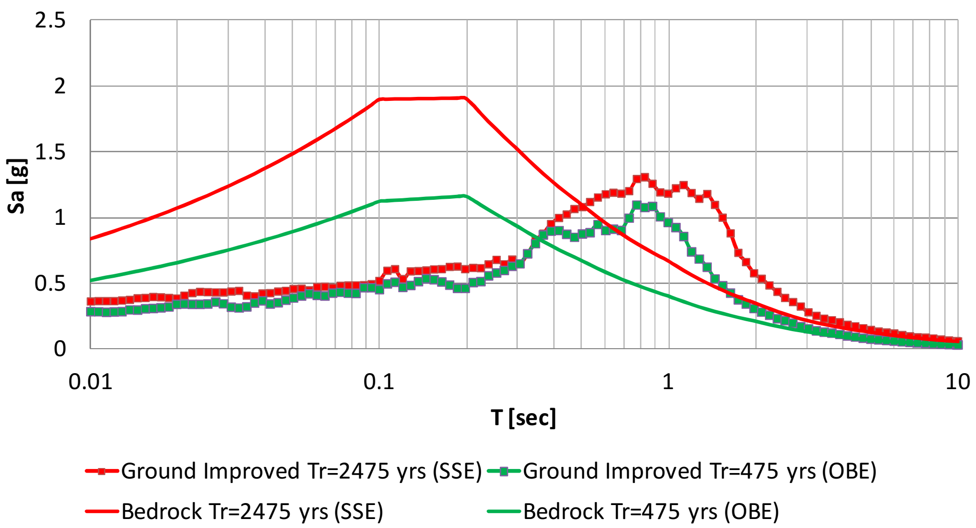

Figure 2. Horizontal response spectra at 5% damping

Vertical spectra were assumed as 2/3 of horizontal spectra at bedrock.
As shown in Figure 2, both the SSE and the OBE response spectra have their maximum values at about 0.8 sec. This characteristic of the horizontal response spectra has some implications in the results of this study as highlighted in the following.

3. DYNAMIC SOIL STRUCTURE INTERACTION PARAMETERS

Dynamic soil impedances (springs and dampers) for soil-structure interaction earthquake analyses shall take into account the soil layering, its geomechanical properties at the level of strain induced by seismic actions and the mass, mass moment of inertia and geometry of the foundation-tank system.
The effects of the soil stratigraphy on the dynamic response of the foundation are taken into account following the approach by Christiano et al. (1979) implemented in the RINA Consulting S.p.A. in house computer program WGTMOD.
It is well known that soils exhibit nonlinear behavior in shear (Darendeli, 2001), with the result that secant shear modulus decreases with increasing strain amplitude. Shear modulus at small strains, at which soil behavior is linear, is referred to as small-strain shear modulus, G_0. The relationship between shear modulus and strain amplitude is typically characterized by a normalized modulus reduction curve.The soil shear moduli considered in spring calculations are here derived from resulting average shear strains along the soil column defined through the 1D site response analysis. Shear moduli vary from 10% to 49% and from 5% to 35% of G_0 for OBE and SSE, respectively. The G considered for the calculation of stiffness is thus taken as a portion of the G_0 described in Paragraph 2.2. The resulting dynamic soil springs obtained for SSE and OBE are reported in Table 3.

Table 3. Dynamic soil stiffness

	Membrane		**Full Containment**	
Dynamic Stiffness	**OBE**	**SSE**	**OBE**	**SSE**
Vertical $K_{v,dyn}$ (kN/m)	9.95e06	6.85e06	1.10e07	7.60e06
Horizontal $K_{h,dyn}$ (kN/m)	2.80e06	1.95e06	3.11e06	2.16e06
Rocking $K_{\theta,dyn}$ (kNm/rad)	4.90e09	3.35e09	6.62e09	4.52e09

Table 3 shows that SSE dynamic stiffness is smaller than OBE stiffness, because of the larger strain expected for the SSE, and that the full containment soil-foundation is stiffer than the membrane tank because of its larger diameter.
In order to consider the energy dissipation, two sources of damping are considered in the soil-structure interaction analyses: radiation and material damping combined together as reported in the Eurocode 8 (2006). The radiation damping is estimated from WGTMOD and compared with the formulas available in the Eurocode 8 (2006). Given that the radiation damping estimated with both methods is found higher than the limit of the 25% imposed in the Eurocode 8 (2006), the 25% is applied. The material damping values are estimated as 2% and 5% for the full containment, inner tank only, and for the membrane tank respectively. The hysteretic damping in the soil is estimated based on the average shear strain obtained in the 1D site response analysis as 4% and 8% for the OBE and SSE, respectively. The resultant system damping values are reported in the following table:

Table 4. Equivalent system damping.

	Membrane		**Full Containment**	
System damping	**OBE**	**SSE**	**OBE**	**SSE**
ξ horizontal	29.0 %	33.0 %	29.1 %	33.1 %
ξ vertical	27.0 %	29.0 %	27.0 %	29.0 %

These damping values are used to adjust the 5% response spectra to calculate the maximum impulsive spectral accelerations. Spectral adjustment for damping considered the following reduction factor η:

$$\eta = \sqrt{\frac{10}{5+\xi}} \tag{1}$$

The effect of the smaller material damping of the 9%Ni tank is negligible when looking at the whole system damping estimated with the soil structure interaction. However, if a fixed based analysis is performed, the unique source of damping is related to the tank material damping and hence the membrane tank has a damping reduction factor of 20% bigger than the full containment tank. The system damping is almost the same in both tanks when considering soil-structure interaction.

4. "SIMPLIFIED" CODE-BASED ANALYSES

The simplified dynamic soil structure interaction (DSSI) is carried out according to Annex A of Eurocode 8 – part 4 (Eurocode 8, 2006) with the hypothesis of a rigid tank for the membrane technology and of a flexible tank for the full containment structure. Both tanks are assumed to rest on deformable soil. In general, the effect of soil structure interaction is to lengthen the fundamental period of vibration of the system and to increase the damping. This simplified approach is used to define parameters for the 3D FE model and as reference for the seismic actions resulting from the FE model. The tank-liquid system is modeled by means of two single degree-of-freedom systems, one corresponding to the impulsive component, moving together with the concrete wall, and the other corresponding to the convective component as reported in Malhotra (1997). The two responses, impulsive and convective, are then combined by taking their numerical sum. The simplified systems used for the rigid and deformable tanks are different and are summarized in the following figure:

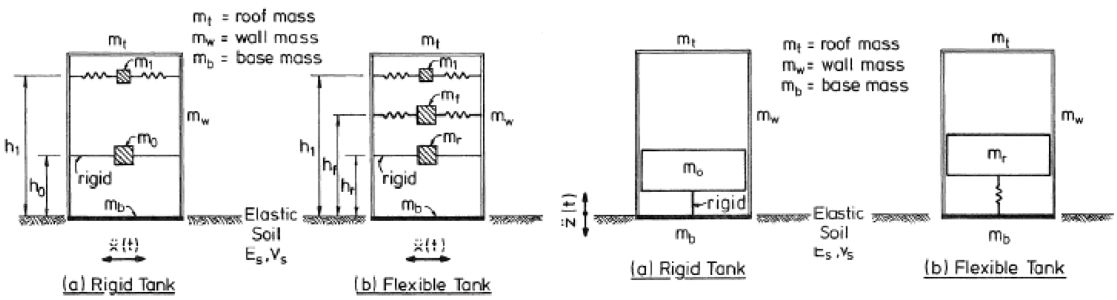

Figure 3. Spring-mass analogy for horizontal motion (left) and vertical motion (right) – after Priestley et al. 1986

In the simulation of Figure 3, m_o is the portion of the liquid moving together with the tank mass and m_1 is the first-mode sloshing mass. These masses are acting at heights h_o and h_i from the tank base.
For flexible tanks, a category in which the majority of steel tanks are included, the inertia mass m_0 is divided in two contributions: m_r and m_f acting at heights h_r and h_f representing the proportion of liquid that continue to act as rigidly connected to the tank wall and the proportion that interacts with the lateral flexibility of the tank walls. Vertical response for both rigid and flexible tanks must consider foundation flexibility.
In the simplified procedure proposed by the Eurocode 8 (2006), the recommended design values of dynamic parameters are originally defined by Malhotra (1997). These values are related to the first impulsive and convective modes of vibration and they depend on the ratio H/R, where H denotes the height of liquid.
Table 5 summarizes the main parameters obtained from the equations reported in the Eurocode 8. As expected, the membrane tank shows significant greater t/R ratio and slenderness than the full containment tank. The minor slenderness of the full containment tank implies that the convective component is higher in this technology than in the membrane tank: in other words, the sloshing effect is more pronounced in the full containment technology.

Table 5. Parameters for the spring-mass analogy for the case study tanks

Property	**Symbol**	**Membrane**	**Full Containment**
Flexibility Index (-)	t/R	0.0198	0.0005
Tank slenderness (-)	H/R	0.9400	0.7948
Total mass of liquid (kg)	m	97,474	93,121
Impulsive mass affected by the walls deformability (kg)	m_f	n.a.	37,532
Impulsive mass (kg)	m_i	46,151	2,057
Impulsive mass/total liquid mass ratio (-)		51.58%	40.30%
Convective mass (kg)	m_c	43,322	53,532
Convective mass/total liquid mass ratio (-)		48.42%	57.49%
Half wave length of bending (m)	L	13.51	2.35
Height of the impulsive mass (m)	h_o	15.39	11.59
Height of the convective mass (m)	h_1	22.46	18.33

Moreover, the half wave length of bending, that describes the distance from the tank base in which the bending effects are concentrated (Calvi and Nascimbene, 2011) is much less for the flexible steel tank than for the concrete tank.

Table 6. Periods of vibration for the tanks obtained with the simplified method

Period of vibrations (Eurocode 8-part 4)	**Symbol**	**Membrane**	**Full Containment**
Period of the impulsive response fixed base (sec)	T_{imp}	0.20	0.43
Period of the impulsive response with DSSI (sec)	T^*_{imp}	1.59	1.00
Period of the convective response	T_c	9.73	10.57
Period of the impulsive flexible response fixed base (sec)	T_f	n.a.	0.37
Period of the impulsive flexible response with DSSI (sec)	T^*_f	n.a.	0.98

From the previous table is possible to notice that in the case of fixed base, the membrane concrete tank is stiffer than the full containment inner tank. The effect of DSSI is more pronounced in the membrane tank than in the full containment: for the concrete tank the period lengthening effect due to the DSSI inclusion goes from 0.20 sec to 1.59 sec with an increment factor of 7.95. On the other hand, the effect of DSSI in the period of vibration for the impulsive flexible response of the full containment tank is limited to a period lengthening of a factor of 2.65 (from 0.37 sec to 0.98 sec). From the response spectrum reported in Figure 2 it is possible to see that the effect of the period lengthening is to move the period of vibrations where the spectrum shows higher accelerations. However, the effect of the damping associated with the dynamic soil structure interaction mitigates this increment. Finally, it is possible to notice that with the inclusion of DSSI, the full containment tank is stiffer than the membrane tank. This can be explained by the simplified equation used to compute the impulsive period of vibration considering DSSI (Eurocode 8 2006):

$$T^*_{imp} = 2\pi\sqrt{\frac{m_o + m_b}{K_{h,dyn}} + \frac{m_o h_0^2}{K_{\theta,dyn}}} \tag{2}$$

where m_b is the mass of the tank base. From Equation 2 it is possible to see that the higher is the horizontal and rocking stiffness of the foundation, the smaller is the impulsive period of vibration. On the other hand, smaller is the height of the impulsive mass and smaller is the period of vibration of the liquid that moves connected to the tank wall.

Based on the results in terms of period of vibrations and masses, the actions on the tank foundations are computed for both technologies by entering in the acceleration response spectrum lowered with the damping evaluated from the DSSI as indicated in Equation 1. The resultant actions below the base plate are reported in the following table for the SSE.

Table 7. Actions below base plate for the SSE for the membrane tank and full containment tank

	Membrane Tank			**Full Containment Tank**		
Load combinations	**N (kN)**	**Q (kN)**	**M (kNm)**	**N (kN)**	**Q (kN)**	**M (kNm)**
G+ E_h +0.3 E_v	1,371,095	397,480	11,741,198	1,532,044	553,754	11,921,650
G+ E_h -0.3 E_v	1,166,100	397,480	11,741,198	1,262,283	553,754	11,921,650
G+0.3 E_h +0.3Ev	1,71,095	119,244	3,522,359	1,532,044	166,126	3,576,495
G+0.3 E_h -0.3 E_v	1,166,100	119,244	3,522,359	1,262,283	166,126	3,576,495
G+0.3 E_h + E_v	1,610,257	119,244	3,522,359	1,846,765	166,126	3,576,495
G+0.3 E_h - E_v	926,939	119,244	3,522,359	947,562	166,126	3,576,495

5. 3D FINITE ELEMENT MODELS

The simplified approach presented in the previous chapter is developed in order to have reference values of internal pressures and foundation actions to be compared with the results of refined finite element models.
The models are used to define the distribution of the soil pressure under the foundation and the distribution of the internal pressure. The finite element analysis of a large tank has to consider some important aspects: the liquid-structure interaction, the soil-foundation-structure interaction, the correct distribution of the masses of the liquid content and of the tank structures. All these aspects might be included in very refined finite element models but the aim of this study is to define a “practice oriented” approach to be compatible with design project schedule without loss of reliability. To this extend, the finite element models for both tanks are developed with the software SAP2000 (CSI, 2017) considering shell elements for both the walls and the foundation plate. A sensitivity analysis was first developed in order to define the mesh sizing of the elements by comparing the hoop force and the out of plane moment of the finite element models with those of the analytical solutions available in bibliography (Calvi and Nascimbene, 2011).
For the consideration of the fluid-structure dynamic interaction, a simplified added masses approach is applied. The added mass approach is first derived by Westergaard (1931) for dams and then applied to the seismic analysis of circular tanks by Virella et al. (2006) and Amiri (2014) among the others. In this method the impulsive mass of the liquid is rigidly connected to the tank walls and its vertical distribution $m_i(z)$ is defined from the impulsive pressure distribution $p_i(z)$:

$$m_i(z) = \frac{p_i(z)}{S(T^*_{imp})} \tag{3}$$

being $S(T^*_{imp})$ the spectral acceleration at the impulsive period of the tank modified with the soil-structure interaction as per Equation 2 and defined in the acceleration response spectrum lowered at the system damping as per equation 1. The lumped masses obtained with this method have the same vertical distribution of the impulsive pressure from which they are derived and a uniform distribution around the tank circumference.
The convective part of the liquid content is instead connected to the tank wall by means of flexible links having stiffness compatible with the first sloshing mode in any radial directions. The stiffness of the flexible links, k_c, is defined as follows:

$$k_c = 4\pi^2 \frac{m_c}{T_c^2} \quad (4)$$

where m_c and T_c, reported in Table 5 and Table 6, are the convective mass and the sloshing period respectively. In addition, also the static pressure on the tank walls and base plate are considered in the model. As far as the soil-foundation stiffness is concerned, the lumped springs defined in Table 3 are distributed in the foundation mesh to obtain a "bed" of linear springs (NERHP, 2012). The stiffness of an individual vertical spring in the interior portion of the foundation can be taken as the ratio between the lumped vertical stiffness and the foundation area. If this approach would have been applied across the entire area of the base plate, the vertical stiffness of the soil-foundation system would be correctly reproduced but the whole rocking stiffness would be underestimated. This occurs because the vertical soil reaction is not uniform but it is higher near the foundation edges. To correct this underestimation of the rocking stiffness, stiffer springs were assigned at the zone near the foundations edges.

A modal response spectrum analysis is then performed using input spectra at different levels of damping: the spectrum applied to the convective mode is defined with a 0.5 % damping while the response spectrum for the impulsive component of the fluid is calculated considering the damping arising from the soil-structure interaction assessment. Figure 5 shows the finite element models developed for the two tank technologies.

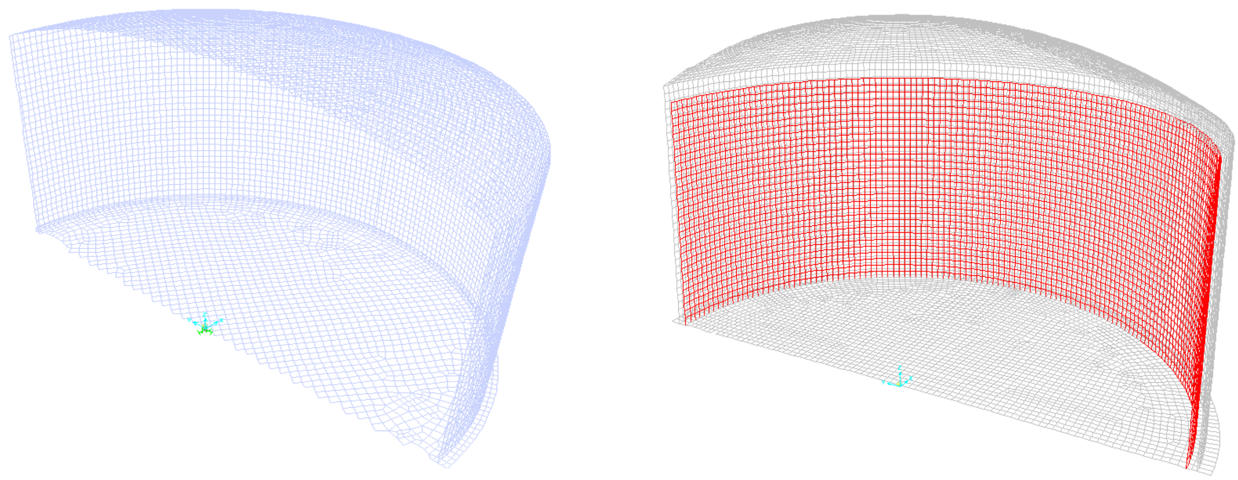

Figure 4. Finite element model: membrane tank (left) and full containment tank (right). In grey are shown the concrete elements and in red the steel elements

The eigenvalue analysis of the two models is developed first in order to investigate the differences between the results obtained with simplified analytical methods.

Table 8. Periods of vibration for the tanks obtained with the finite element model

Period of vibrations (FEM)	**Symbol**	**Membrane**	**Full Containment**
Period of the impulsive response with DSSI (sec)	T^*_{imp}	1.67	1.49
Period of the convective response (sec)	T_c	9.81	10.68
Period of vertical response with DSSI (sec)	T^*_v	0.92	0.90

From Table 8 it is possible to see that the convective period of vibration defined with the finite element model is in good agreement with the period defined with the simplified method shown in Table 6. The same conclusion applies for the vertical response considering DSSI.

As far as the first impulsive mode is concerned, for the membrane tank the periods estimated with the two methods are in good agreement. On the other hand, for the full containment tank, the simplified approach underestimates the period of the inner tank. This is because the simplified method considers inner tank and outer tank uncoupled but, with inclusion of DSSI, the two parts are coupled. If we consider two springs in series, one for the inner tank and one for the outer tank, a simplified alternative calculation of the impulsive period is possible as proposed in the following equation (Rotzer et al, 2005).

$$\frac{1}{\omega^2_{\text{system}}} = \frac{1}{\omega^2_{\text{outer tank}}} + \frac{1}{\omega^2_{\text{inner tank}}} = \frac{T^2_{\text{outer tank}}}{4\pi^2} + \frac{T^2_{\text{inner tank}}}{4\pi^2} = \frac{1.00^2}{4\pi^2} + \frac{0.98^2}{4\pi^2} = 0.05\ \frac{1}{\text{rad/sec}^2} \tag{5}$$

$$T_{\text{system}} = 1.40\ \text{sec} \tag{6}$$

Under this assumption, the period of the impulsive response obtained with the analytical calculation corresponds to the one obtained with the finite element model.

6. COMPARISON BETWEEN MEMBRANE AND FULL CONTAINMENT TANKS

After the finite element models are compared with the results of the analytical models, the refined models are used in the verifications for both the tank technologies. As results of the response spectrum analysis, the base shear and overturning moments below the base plate are obtained and are presented in the following table:

Table 9. Actions below base plate for the SSE for the membrane tank

	Membrane Tank			**Full Containment Tank**		
Load combinations	**N (kN)**	**Q (kN)**	**M (kNm)**	**N (kN)**	**Q (kN)**	**M (kNm)**
G+ E_h +0.3 E_v	1,565,826	363,012	11,389,037	1,549,056	458,935	15,295,171
G+ E_h -0.3 E_v	1,244,090	363,012	11,398,661	1,220,516	458,935	15,292,453
G+0.3 E_h +0.3Ev	1,532,236	91,740	3,073,092	1,541,626	138,982	4,655,312
G+0.3 E_h -0.3 E_v	1,277,680	91,740	3,082,716	1,227,946	138,982	4,652,595
G+0.3 E_h + E_v	1,828,283	92, 392	3,119,376	1,900,155	143,319	4,874,678
G+0.3 E_h - E_v	981,633	92,392	3,129,000	869,417	143,319	4,871,960

The actions at foundation base listed in Table 9 are in good agreement with the actions obtained with the simplified method and presented in Table 7. This means that the simplified method is able to accurately define the global actions on the foundation. Furthermore, the base shear for the full containment tank is higher than for the membrane tank because the spectral acceleration at the impulsive period of vibration is higher (0.5 g at T^*_{imp}=1.49 sec and 0.4 g at T^*_{imp}=1.67 sec for the full containment tank and membrane tank respectively).

In order to address the capability of both the tank technologies to withstand the seismic forces, the following verifications are performed: foundation geotechnical verifications for the membrane tank, walls structural verifications and uplift check for the full membrane tank.

6.1 Membrane tank verifications

The geotechnical verifications are carried out according to Eurocode 7 (2005). In order to include the soil inertial effects in seismic condition, reduction factors are taken into account in the geotechnical verification (Paolucci and Pecker, 1997).

The bearing capacity calculation for the membrane tank has been verified using the in house program DAPTEC (D'Appolonia, 2002). The program has been developed for the evaluation of bearing capacity of foundations supported by a layered soil profile. The program is based on the Lauritzen and Schejtne (1976) slip surface method, modified to take into account drained failure through cohesionless layers and undrained failure in cohesive strata. DAPTEC determines the safety factor against bearing capacity failure of an embedded rectangular foundation subjected to inclined and eccentric loading. The minimum safety factor is determined considering different depths of analyzed failure surfaces below the foundation level. The

computation of the safety factor for a particular failure surface is performed using an iterative procedure. The minimum factors of safety for OBE and SSE are 2.32 and 1.44, respectively. Therefore, the bearing capacity verification is satisfied (minimum Fs > partial resistance factor = 1.4).
Sliding verification consists in the evaluation of the capability of the foundation to sustain horizontal forces, including seismic loads. The soil-concrete friction resistance is developed along the base, while the contribution of lateral surfaces is conservatively not considered. The minimum factors of safety for OBE and SSE are 2.56 and 1.67, respectively. Therefore, the sliding verification is satisfied (minimum Fs > partial resistance factor = 1.1).

6.2 Full integrity tank verifications

Regarding the wall thickness verification for the full containment tank, the following checks are computed. The stress levels in the wall base sections are defined for hoop forces and bending moment considering also the effects of hydrostatic pressure, impulsive and convective pressures and vertical excitation. The wall verifications are performed considering an allowable stress of 320 MPa and 306 MPa for the SSE and OBE respectively. The maximum vertical stress due to the hoop forces at the wall base is equal to 232 MPa. The wall base thickness is also verified for the elastic diamond buckling and for the elastic-plastic collapse, also called "elephant foot buckling". In order to withstand the elastic-plastic collapse, a minimum thickness of 42 mm is required. The remaining verifications for the 9%Ni inner tank dealt with the uplift mechanism, verified in accordance to the procedure proposed in the New Zealand guidelines (NZSEE, 2009). The uplift mechanism occurs if the overturning moment is bigger than the restoring moment computed considering the walls and roof weight. An iterative procedure is followed to find the radius of the circular base surface, r, that corresponds to the equilibrium (overturning moment equal to the restoring moment): this happens for r=42.75 m meaning that a loss of contact of 1.03 m is expected. The uplift might be avoided by applying steel anchors of the steel walls to the reinforced concrete plate. However, this might causes technological issues: the welding for fixing the anchor straps on the inner tank shell causes additional stresses in an already heavily loaded area. Furthermore, the anchors penetrate the bottom insulation, the secondary tank bottom and the vapor barrier. In case that uplift mechanism is accepted, the effects of the vertical displacement of the steel base plate are: increment of the wall axial stress at the base section and increment in the radial stress of the base plate. In order to satisfy the strength and stability verifications under the uplifting mechanism, a minimum thickness for the base wall section of 50 mm is thus required.

7. CONCLUSIONS

The seismic behavior of a full containment and membrane LNG tanks are compared. The comparison is developed considering two tanks of the same net volume capacity, under identical soil properties and seismic input. Moreover, the same level of detail is considered in the analyses of both technologies. The main objectives of the paper are to verify the capability of the two tank technologies to withstand the seismic forces and to investigate some modeling issues for practical applications. The soil structure interaction (SSI), and the fluid-structure interaction are considered both in the simplified and in the refined methods for both tanks.
Based on the case study investigated, the main outcomes are:

a) the membrane tank can sustain seismic loads without isolators and the shallow foundation is verified considering specific ground improvement. The amount of reinforcement in the concrete elements is within technical feasibility;
b) full containment tank shows uplift during the seismic event: verifications of wall buckling during uplift would lead to more thick walls and base plates compared to typical values;
c) uplift of inner shell is not allowed by the codes so that for 9% Ni it is mandatory to install anchors. If the anchors become overdesigned due to the seismic level, the designer shall put the whole tank into seismic isolators. Anchorages are an additional potential source of gas leakage as they cross the corner protection and the carbon steel bottom liner, in an area were welding and control is difficult. Furthermore, the welding for fixing the anchor straps on the inner tank shell causes additional stresses in an already heavily loaded area. The counterparty is that for the membrane technology without seismic isolators, the concrete wall and its reinforcement shall be designed for the resulting seismic forces;

d) a practice-oriented finite element model is described with the inclusion of dynamic soil structure interaction together with the added mass approach for the consideration of the fluid-structure interaction. The results of the FE model are compared to simplified code-based approaches. There is a general agreement between FE results and simplified approaches. A modification on the simplified estimate of the inner tank natural period is proposed to take into account also the soil-structure interaction.

8. REFERENCES

Amiri M (2014). Added Mass Approach for Time History Analysis of Dome Roof Tanks under Horizontal Earthquake Excitation, Sylvan Journal English Edition, Vol. 6, pp. 82-89

Calvi G M, Nascimbene R (2011). Progettare i Gusci, IUSS Press, Pavia, Italy, *in Italian*.

Christiano P P, Rizzo P C, Jarecki S J (1974). Compliances of Layered Elastic Systems, *Proceedings - Institution of Civil Engineers*, Part 2, Vol. 57, pp. 673-683.

Computers and Structures Inc. (CSI) (2014). Analysis Reference Manual for SAP 2000, Berkeley, CA.

D'Appolonia (2002). Program DAPTEC, User's Manual, Bearing Capacity of Foundations on Layered Soil Profiles, Doc. A-COM-H3-Rev.5.

Darendeli M B, (2001). Development of a new family of normalized modulus reduction and material damping curves. *Doctor of Philosophy*, University of Texas at Austin.

EN 14620-1, (2006). Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0°C and -165°C, December 2006.

Eurocode 7 (2005). Eurocode 7 Geotechnical Design - Part 1: General Rules, EN 1997-1, July 2005 European Committee for Standardization.

Eurocode 8 (2006). Eurocode 8 - Design of Structures for Earthquake Resistance -Part 4: Silos, Tanks and Pipelines, EN 1998-4, July 2006, European Committee for Standardization.

Ezzarhouni A, Powell J, Elliott S (2016). Why a Membrane Full Integrity Tank, LNG 18, Perth, PO-8, 11-15 April 2016

Lauritzen R, Schjetne K (1976). Stability Calculations for Offshore Gravity Structures, *Offshore Technology Conference*, Paper No. OTC-2431, Houston.

Malhotra P K (1997). Seismic Response of Soil-Supported Unanchored Liquid-Storage Tanks, Journal of Structural Engineering, No. 123, pp. 440-449.381-387.

Martí J, Crespo M and Martínez F (2010). Seismic Isolation of Liquefied Natural Gas Tanks: A Comparative Assessment, Mathematical Sciences Publishers Anti-Seismic Systems International Society

National Fire Protection Agency (NFPA) (2013). Standard for the Production, Storage and Handling of Liquefied Natural Gas (LNG), NFPA 59A.

NEHRP (2012). Soil-Structure Interaction for Building Structures, NIST GCR 12-917-21.

NZSEE (2009). Seismic Design of Storage Tanks: 2009, Recommendations of New Zealand Society for Earthquake Engineering on Seismic Design of Storage Tanks, New Zealand Society for Earthquake Engineering.

Paolucci R, Pecker A (1997). Seismic Bearing Capacity of Shallow Strip Foundation on Dry Soil, *Soil and Foundation*, Vol. 37, No.3, pp.95-105.

Priebe H J (1995). The Design of Vibro Replacement, Reprint from Ground Engineering, Technical Paper GT 07-13E.

Priestley M J N, Wood J H, Davidson B J (1986). Seismic Design of Storage Tanks, *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 19, No. 4.

Rotzer J, Douglas J, Maurer H (2005), Hazard and safety investigation for LNG tanks, LNG Journal, July/August 2005

Virella J C, Godoy L A, Suarez L E (2006). Dynamic buckling of anchored steel tanks subjected to horizontal earthquake excitation. *Journal of Constructional Steel Research*. Vol. 62, pp. 521-531.

Westergaard H M (1931). Water Pressures on Dams during Earthquakes, ASCE Transactions, pp. 418-433.