LNG TERMINAL IN CHINA – SEISMIC DESIGN OF TANKS ON 100m LONG PILES IN SOFT SOILS.

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Abstract

New 160,000m³ LNG tanks are under construction South of China. The Authors are in charge of geotechnical and earthquake engineering. European EN [1][2] and Chinese GB codes [3][4] have been followed. General offshore hydraulic backfill has been installed on a Chinese Sea island. To allow for site accessibility, soil improvement has been realized on the whole site resulting in a general surface settlement around 2.5m. According to soil investigations carried out from 2013 until 2015, the soil is made of 80 to 100m thick soft deposits, covering the Ignimbrite volcanic rocks. This very hard volcanic rock is outcropping at some places of the site.

Detailed seismotectonic studies have been conducted for the region to verify the absence of active faults on the site, and to provide the bedrock seismic levels through a probabilistic study PSHA. The horizontal Peak Ground Accelerations at bedrock level are :

- OBE →PGA = 0.09g
- SSE → PGA = 0.25g

Elevated rafts on 319 large bored piles in triangular layout have been designed as tanks foundation. The piles have been socketed (embedded) by adequate length inside rock to provide the requested bearing capacities.

An iterative process of Soil Structure Interactions (SSI) computations using SAP software has been conducted during some months. First SSI’s have been made using Spectrum as seismic input while the final SSI’s used Forced Response Analysis with accelerograms inputs. For preliminary SSI’s Spectrum, the piles have been considered as embedded in the raft without any isolator. Average pile length has been assumed from 2013 soil investigations. Bedrock seismic spectra have been used as earthquake input. Due to unacceptable bending moments at pile heads together with pile tractions impossible to be realistically taken by pile friction inside bedrock, use of isolators have been decided. Different types of isolators have been considered. Finally lead LRB isolators have been selected. During first SSI’s Spectrum, the resulting participating mass for 800 modes was horizontally admissible, but involved only 60% of the vertical mass. High traction loads have been computed at the pile heads of the two outer circles of piles. This computed traction, incompatible with isolator characteristics, was due to the too high stiffness of the isolators used for first trial and to the application of Square Root of Sum of Squares (SRSS) combination.

Additional soil investigations made end of 2015 have provided more accurate variable pile length together with dynamic parameters to carry on nonlinear SSI Forced Response Analysis, using bedrock accelerograms as input. Due to short planning project, computations have been made only for selected nodes and elements. Full nonlinear computations were not possible. Anyway, no more traction and lower compression loads have been obtained at pile heads.

By mid-2016, French and Chinese Third Party reviews have validated our design using ANSYS and stick models.

Keywords: LNG Tanks, Piled Raft, Soft Soils, LRB Isolators, Soil Structure Interaction

1. Introduction

The construction of a new LNG Terminal including many 160,000m³ LNG Tanks has been decided on an island close to shore line in the China Sea. Building of the 2 first tanks has been started beginning of 2016.

The site area is made of general backfill, about 667,000 m², covering the recent marine sedimentary layers. Ignimbrite volcanic bedrock is found around 100 m deep below surface. LNG tanks are built on elevated rafts supported by large bored piles stopped on the medium weathered bedrock.

This paper describes the foundation design methodology and the difficulties to realize adequate Soil Structure Interaction (SSI) for seismic loading cases.

2. Tectonic Data

According to regional data, the neotectonic activity (recent movements) of the region is mainly a global up-down movement with rare fault activity. No recent fault structure created by neotectonic activity has been observed during geological surveys.

No active faults have been discovered affecting the site. But a “dead” NE fault zone located at the northwest of the site will affect the foundation and especially the pile drilling works.

The historical seismicity of the 100 km surrounding region is therefore low. If no detailed PSHA study were available for the site, the China Peak Ground acceleration map provided in the Chinese GB18306 code “Seismic Zonation Map” would have proposed a bedrock PGA of 0.1g for 475 years return period.

Considering the offshore seismic activity, no risk of tsunami has been mentioned for this site.

3. Earthquake Data

Detailed seismotectonic studies, including Probabilistic Seismic Hazard Assessment (PSHA), have been conducted for the region to verify the absence of active faults on the site, and to provide the bedrock seismic levels. All regional faults and epicenters surrounding the site, which could have some influence on the site seismotectonic activities, have been reported and studied.

Geological studies and field investigations have been realized on all elements or lineation’s which could have indicated presence of active faults.

The return period of Operating Basis Earthquake (OBE) is 475 years, corresponding to 10% exceedance for 50 years operation life, while the return period of Safe Shutdown Earthquake (SSE) has been decided as 5,000 years, corresponding to 1% exceedance for 50 years operation life (or 2% for 100 years). The horizontal Peak Ground Accelerations at bedrock level are :

- OBE → PGA = 0.09 g (85 cm/s²)
- SSE → PGA = 0.25 g (242 cm/s²)

Geophysical borings with shear wave velocities measurements have been drilled at different locations of the site, from seabed and before any backfilling, to deduce the surface seismic spectrum from the bedrock levels by deconvolution. The resulting horizontal Peak Ground Accelerations at the surface are :

- OBE → PGA = 0.11 g (107 cm/s²)
- SSE → PGA = 0.23g (226 cm/s²)

Considering that backfills have been installed on existing soil after having realized the geophysical borings and that different soil improvement methods have been realized from the surface, the applicability of the provided “surface spectrum” is questionable. Bedrock spectrum shall therefore be used for the computations.

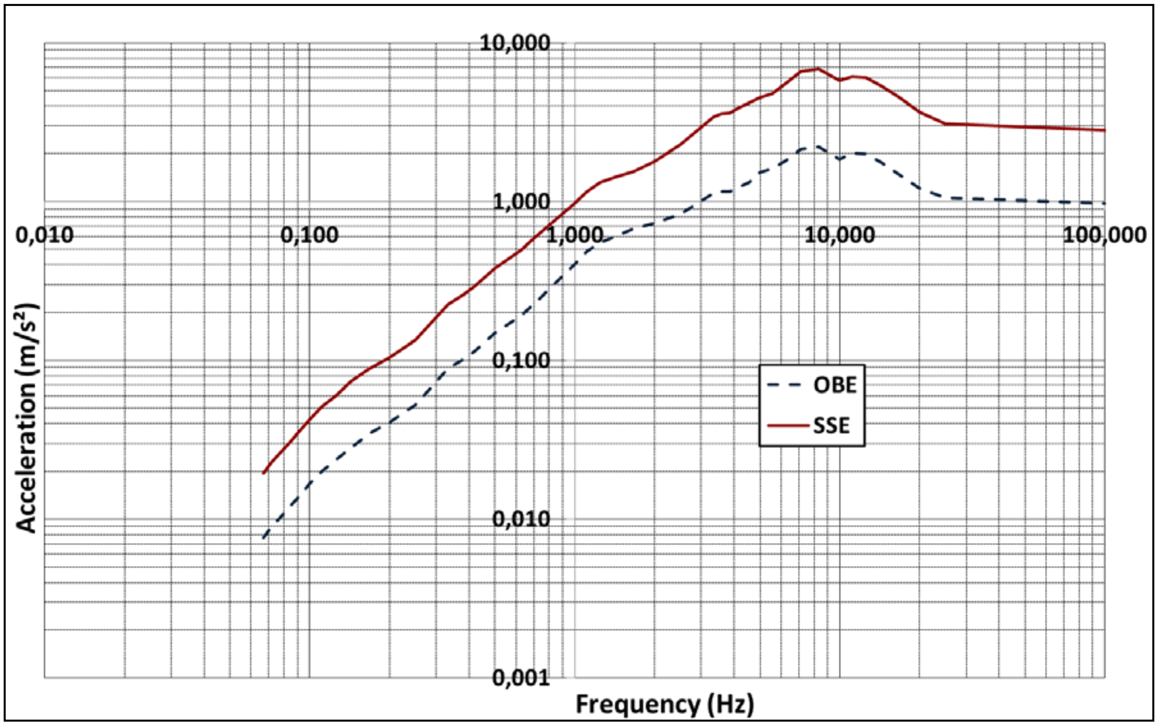


Fig. 1 – OBE and SSE Bedrock Spectra

The two spectra have been contractually provided by Client based on the detailed PSHA study. Chinese GB 50011 code could also have provided similar spectra

4. Soil Data

According to geotechnical investigation, the sub-soil is made of a thick Quaternary marine sediment soil facies covering the volcanic bedrock. Depth of bedrock is varying on the site; it is even outcropping at some locations. At the two tanks area location, there are 80m to 100m thick soils covering bedrock.

Following boring descriptions, the Quaternary soil could be classified into 8 main layers and 15 sub-layers. The Quaternary soil layers are made of muddy clays and silty clays, which are mostly soft and compressible. The bedrock is made of Ignimbrite in different weathering stages. Ignimbrite is a volcanic deposit of pyroclastic flows, resulting from gigantic explosions of pyroclastic ash, lapilli and blocks flowing down the sides of volcanoes.

The Ignimbrite bedrock has been classified into completely weathered, highly weathered and medium weathered rocks. The completely weathered Ignimbrite has a sand-like structure with low resistance, while the highly weathered unit is a massive structure made of tuffs and blocks which can be broken by hand.

The medium weathered Ignimbrite is a massive unit made of fused tuffs and rocks, having good mechanical characteristics. A total of 10 Unconfined Compression Tests have been carried out on selected medium weathered samples. The natural UCS compression varies from 49.6 to 96.6 MPa.

Since backfill is more permeable than the lower soft soils, the ground water has been located at the base of the backfill, at elevation 0.00m corresponding to mean sea level.

Correlation between soil descriptions given by borings and soil behavior provided by geophysical borings has allowed setup the subsoil layering of the tanks site. Dynamic soil parameters like Edyn and Gdyn have been deduced from wave velocities measurements.

A general backfill of some meters has been installed on the soft soils in order to reach the project platform level. Different improvement methods, including dynamic compaction and deep band drains with vacuum preloading, have been realized to consolidate the soil and to obtain accessible working platform.

The Finite Element Model (FEM) requested for the SSI computations has been elaborated with 6 main soil layers based on boring descriptions, shear wave velocities and soil improvements depth. These main layers are :

- 1 : sands representing the backfills installed on the existing muddy seabed
- 2 : part of the muddy soil improved by vertical band drains and vacuum loading
- 3 : muddy soil in place without improvement
- 4 : silty clays becoming stiffer with depth
- 5 : silty clays with gravels becoming also stiffer with depth
- 6 : volcanic bedrock made of Ignimbrite

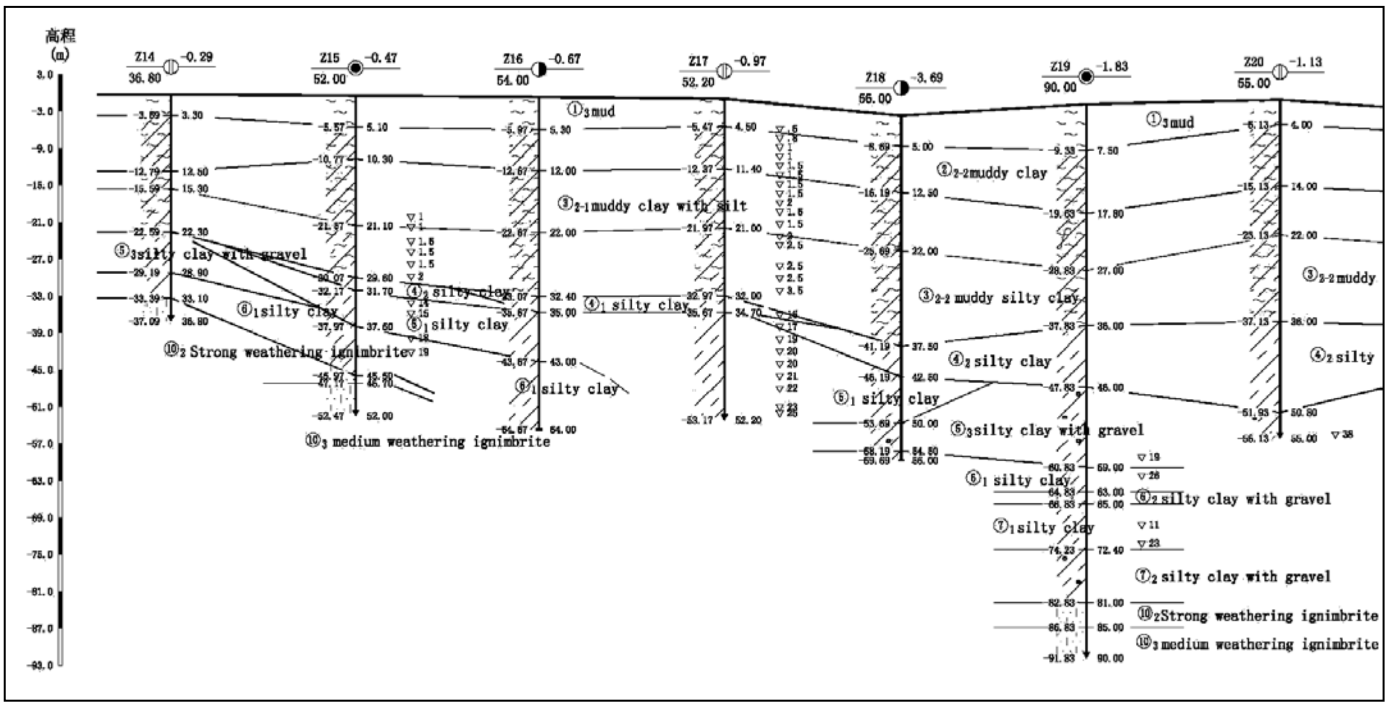


Fig. 2 – Typical Soil Cross-Section

5. Foundations

The soil layers covering bedrock are loose and compressible. They are not able to provide any foundation support to the tanks having each of them a static load of 1,300,000 kN. Tanks must therefore be founded on large bored piles drilled to the medium weathered Ignimbrite bedrock, located at 80m to 100m deep below surface

Piling works were difficult because :

- In some areas, the top layer includes some erratic blocks usually 30 to 50 cm diameter.
- Drilling in soft soils is inducing local hole collapses and significant shafts oversizing.
- The Ignimbrite is a very difficult rock medium to create adequate pile socket
- There is a NE direction “dead” fault zone crossing the northwest part of the site.

Use of percussion equipment has been requested during blocks crossing and when penetrating in Ignimbrite. Special care has been taken to avoid hole collapse during pile drilling by using adequate bentonite mud with controlled density and by use of temporary steel casings.

Tanks are built on elevated raft supported by 319 large bored piles of 1.20m diameter, organized in a triangular layout. There are 2 peripheral rows of piles, the 60 outer and the 60 inner row piles, and 199 inner piles.

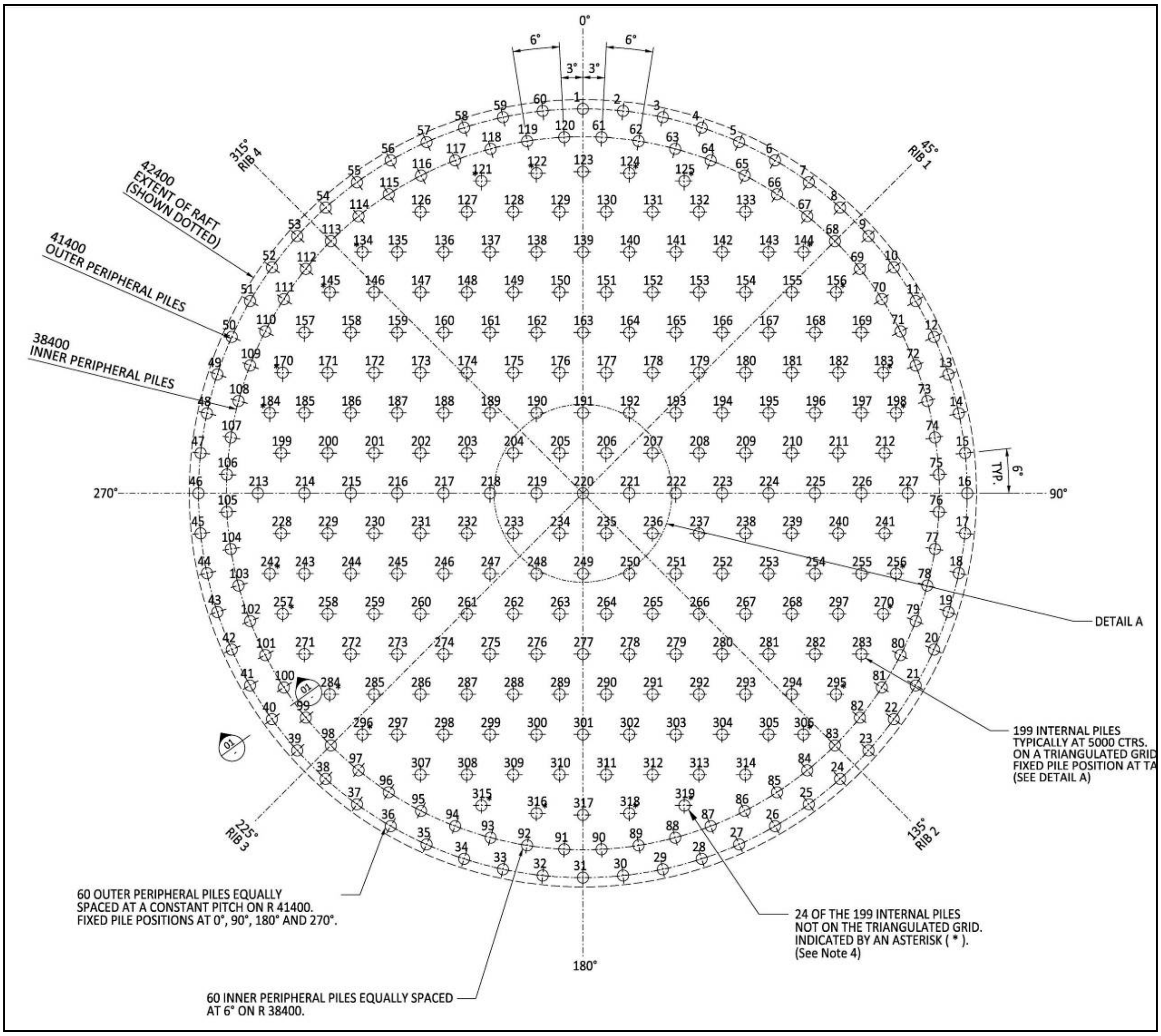
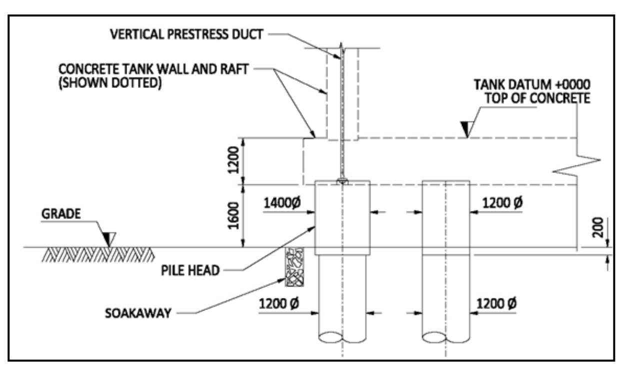


Fig. 3 – 319 Piles in a Triangular Layout

Fig. 4 – Elevated Raft on Large Bored Piles



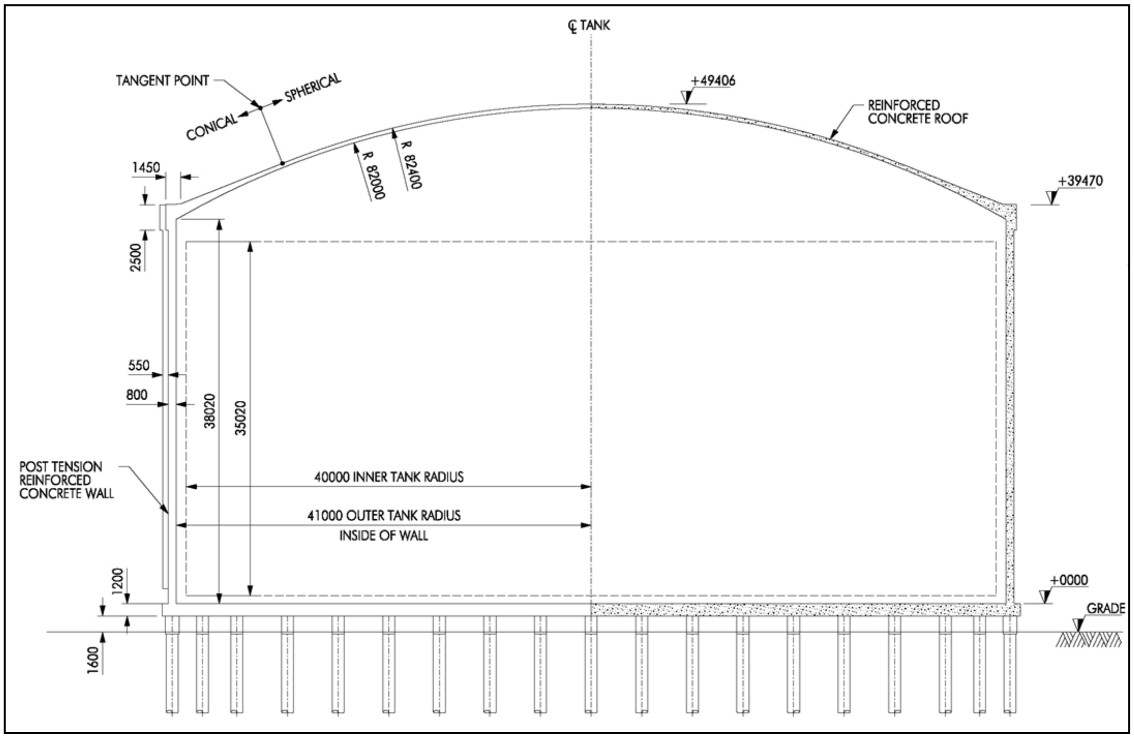


Fig. 5 – Tank Cross Section

According to contractual requests and bidding constraints, the raft thickness is constant and the 60 outer row piles are located at tank wall axis.

6. Design Methodology

LNG tanks are to be designed for static and seismic loading cases according to EN1473 [1] and EN14620 [2]. The OBE is taken as a normal loading case, while SSE is considered as an accidental case. During SSE there must be no negative impact on the surrounding people and environment, and the safe shutdown of the tank must be organized. Interaction between Soil and tank Structure (SSI) computations must be done to adequately design the tanks for seismic loading cases.

Apparently, the elevated raft on 319 long piles could have been guessed to be rather flexible in the horizontal direction. In reality, these 319 piles are vertically equivalent to a single 21.4m diameter concrete column drilled in a thick soft soil. The surrounding soil is so soft that it has very little influence on the piles behavior.

It is also obvious that the vertical component of the bedrock vibration will be directly transmitted through the piles rather than through the ground. It’s even more obvious because the raft does not rest on the ground foundation. Indeed, there is a 1.60 m gap between the backfill surface and the lower face of the raft.

Beside the fact that the proposed surface spectrum has been elaborated from questionable soil data, this is another reason why the spectra at bedrock level were used, modeling the upper ground layers with their own stiffness and unit mass.

7. First Tentative : SSI Spectrum and Piles Embedded in the Raft

Tank design has been first started considering the piles embedded in the concrete raft and supported at their base by adequate socket length inside the bedrock. No seismic isolators were considered at that stage taking into account that seismic loading cases were not that high. No detailed soil investigation was available. All soil layers were therefore considered as horizontal and homogeneous. Pile lengths were supposed to be 80m long.

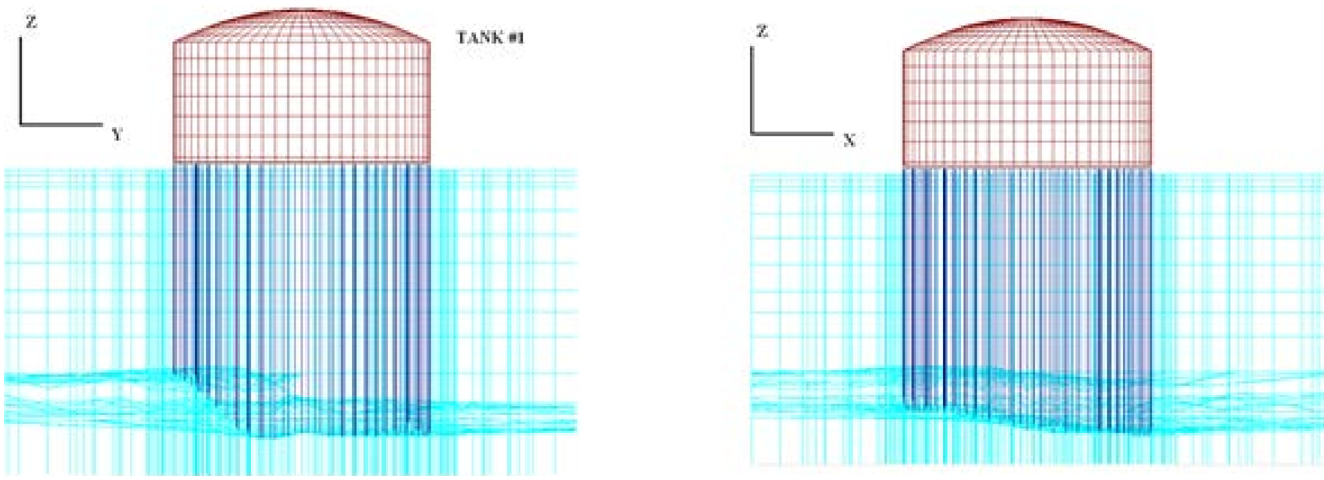


Fig. 6 – Finite Element Model of Tank #1

The SSI computations have been affected by the FEM complexity and by the project planning.

The ground has been featured by 8-nodes brick elements, with, for each layer, the suitable ground Young and shear modulus, mass density, Poisson's ratio and damping ratio. Weighted damping has been used.

On one hand, the finite element model was elaborated by almost 10,000 nodes to represent the tank, the raft, piles and foundation ground down to level -150.00 m and to a radius equal to 350.00 m around the tank axis. The great quantity of nodes associated to corresponding element lump masses is requiring a high number of modes to take in account the total mass of the model.

On the other hand, it was required to quickly provide estimation of quality and quantity of materials to be provided by the builders. The SSI computations were therefore started with the 800 first modes of the model for a Response Spectrum Analysis, with a missing mass correction.

For the selected earthquake loading cases, it immediately appeared that the internal forces at the upper part of the piles were too high : the bending moment would require too much steel reinforcement as well as the shear force.

The pile compression was also high, but the most disturbing problem was the high traction loads in the peripheral pile impossible to be taken by the piles. Indeed, piles should have been drilled by more than 20m inside ignimbrite which was unrealistic with classic drilling equipment at 100m depth inside so hard rock.

8. Second Tentative : SSI Spectrum and elastomeric isolators between slab and piles

The above mentioned concerns have resulted from the horizontal displacement of the tank and its slab under OBE and SSE earthquake loading cases.

As a solution, it was proposed to use elastomeric isolators placed at the top of the piles just under the slab. Several types of isolators were tried in fast computation evaluations. It was rapidly discovered that the horizontal problems of shear and bending were completely solved by the use of horizontally soft elastomeric supports.

However, a new vertical problem appeared which was previously partly hidden by the horizontal concerns. Indeed, the compression in the pile decreased, the traction decreased too, but the some traction remained too high in the peripheral piles.

At that stage of project development, an additional soil investigation has provided detailed pile lengths and more accurate dynamic parameters. The high traction values discussed above were then increased when the real length of each pile was taken in account : the feet of the 319 piles of tank#1 lie at bedrock level from -60.00

m to -83.00 m, while those of tank#2 vary from -84.00 m to -90.00 m. Taking into account the platform level, the elevated raft and pile socket lengths, pile lengths vary from 70m to 100m.

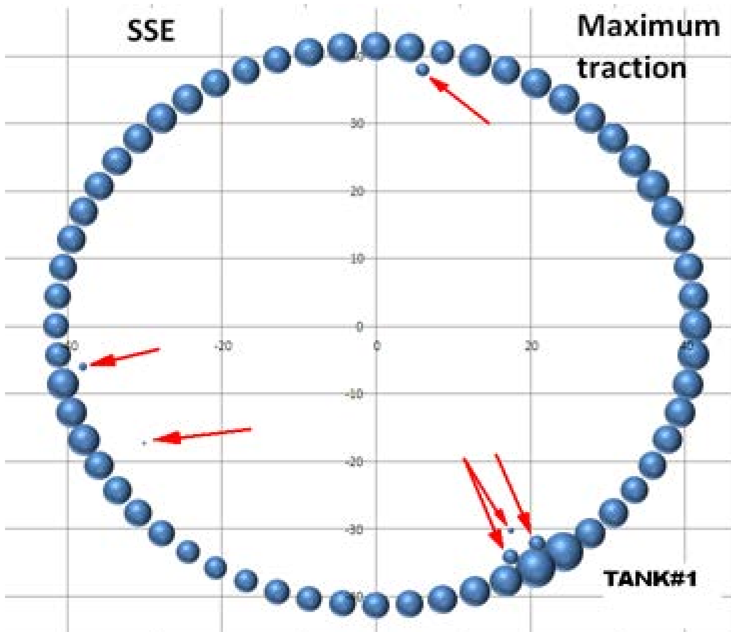


Fig. 7 – Tank #1 : location of piles with maximum tractions

Figure 7 shows that the whole external circle of piles of Tank #1 is affected by traction under SSE loading case, plus four isolated piles of the second outer row and two inner piles. The circle sizes are proportional to the traction values. Different tractions values are resulting from the different pile lengths. The situation is similar for Tank #2 but the traction values are lower and only outer row inner piles are in traction.

The horizontal stiffness of the elastomeric supports is about thousand times less than the vertical one. It was therefore not surprising that some unacceptable values are still computed in the vertical direction, while all horizontal problems disappeared

Another elastomeric support having softer vertical characteristics has been therefore searched. The best one was finally selected, and the new computation showed that, as reducing the vertical stiffness involves that the horizontal one is reduced too, the horizontal forces were at most reduced while the vertical ones came back to a reasonable intensity. Meanwhile some traction still appeared in some outer piles. This last computation was realized using Response Spectrum input, taking into account the total mass of the ground lumped on the maximum feasible number of nodes, and 800 modes.

9. Final Design : SSI Forced Response Analysis

The SSI Response Spectrum approach uses the Square Root of the Sum of Squares (SRSS) of the effect of each mode to combine the results. This SRSS combination was statistically established at the very beginning of the seismic studies of structures by computers. It is applicable for structures whose mass is totally taken in account by some dozen of modes, under earthquakes duration between 30 and 60 seconds.

For the 800 modes taken in account, the formula supposes that they are always acting in the same direction and that they may occur in phase during an earthquake. This hypothesis results in significant overestimation of the results.

Moreover, many modes of the ground volume show deformation shapes which are nor compatible with an earthquake movement (dissymmetric shapes, axisymmetric shapes,…) that would never be excited by an earthquake, but are still taken in account in the SRSS formula.

This is the reason why the more complex SSI based on Forced Response Analysis by Modes Superposition was carried out using synthetic accelerograms matching the OBE and SSE bedrock spectra as seismic input. Three synthetic accelerograms have been elaborated for the OBE, and 3 others for the SSE. One of these synthetic accelerograms is shown as an example on figure 8.

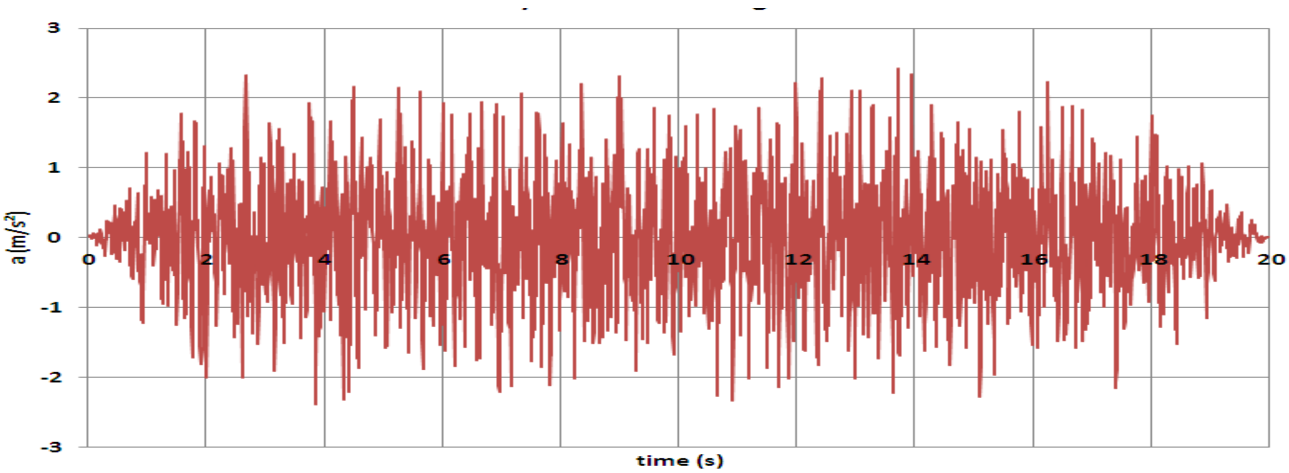


Fig. 8 – SSE synthetic accelerogram number 3

The time step of the accelerograms is 0.007 second. These accelerograms have been created providing that the envelopes of the spectra of these accelerograms were above the OBE and SSE bedrock spectra, given as contractual seismic input values, as shown on figure 9 for SSE.

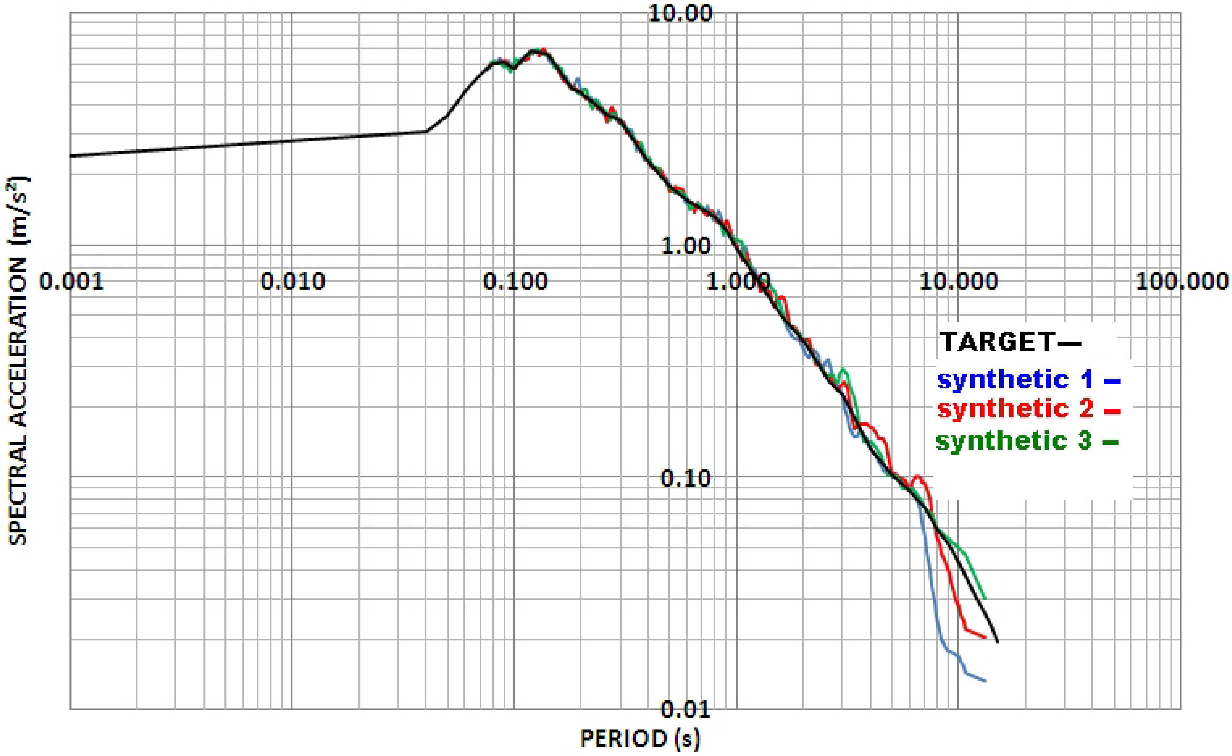


Fig. 9 – Superposition of 3 synthetic accelerogram spectra with the SSE bedrock spectrum

The SSI computation results have demonstrated that the last selected elastomeric support type was the best one. Indeed, no more traction appeared in any piles, under OBE or SSE, and only a minimum 350kN compression remained in the most vertically upwards stressed pile.

To take in account the total mass of the modeled ground, the masses of some adjacent nodes of a soil volume were brought together as a lumped mass to the central node of this volume, according to Pr. Zienkiewicz works [5].

The 800 first modes were recomputed, involving 98 % of the total mass in the horizontal directions, and 95 % in the vertical one.

The forced response analysis was conducted using a computation time step of 0.005 sec, which allows to satisfactory take in account all the modes up to a 20 Hz frequency.

Table 1 – Characteristics in kN/mm of the selected elastomeric support

Elastomeric Support number	**LRB700**
yield force (kN)	106.1
stiffness after yield (kN/mm)	1.40
equivalent horizontal stiffness (kN/mm) at 100% ratio	2.22
equivalent horizontal stiffness (kN/mm) at 120mm displacement	2.28
vertical stiffness(kN/mm)	1975

The computation was carried out in the linear elastic domain, as the maximum shear strain in the ground meets a maximum θ = 0.0006 at level -35.00 m under SSE. According to Seed and Idriss [6], the shear modulus reduction factor is very close to 1.00, and the damping ratio of the ground is very low within this range of deformation. This behavior demonstrates that the foundation ground does not play a big role in the resistance against earthquake loading, and that the foundation piles withstand quite alone the main part of the earthquake effect.

The results showed that the last chosen elastomeric support type was the best one, as no more traction appeared in any piles, under OBE or SSE, and in the worst case a 250 kN compression remained in the most vertically upwards stressed pile.

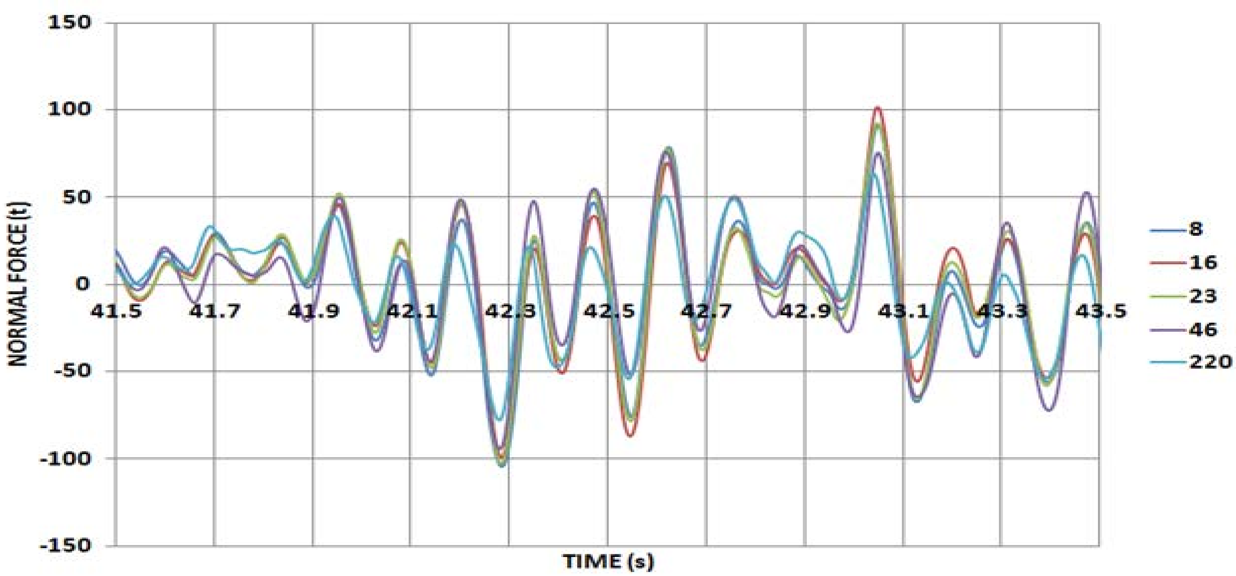


Fig. 10 – Tank #1 : Normal maximum dynamic forces in the supports for vertical SSE

Figure 10 shows the variation of the dynamic vertical force on some piles and supports of Tank #1 under vertical SSE loading case.

Loads at the outer row piles have been increased due to constant raft thickness and particular location at tank wall axis.

Table 2 – Tank #1 : Normal forces (in kN) on piles heads

		Operating (kN)	OBE (kN)	SSE (kN)	Operating + OBE (kN)	Operating + SSE (kN)
outer Circle	Min	1540	- 410	- 1287	1130	253
	Max	5460	410	1287	5870	6747
Inner Circle	Min	1240	- 299	- 929	941	311
	Max	4440	299	929	4739	5369
Inner Piles	Min	1640	- 301	- 875	1339	765
	Max	5530	301	875	5831	6405

Table 2 shows, for Tank #1, the combinations of the maximum and minimum normal forces on pile heads at operating stage combined with OBE and SSE loading cases. Tank #1 has the most stressed piles since its foundation is more dissymmetric than the foundation of Tank #2.

All values requested by design (bending moments, shear forces, displacement and accelerations) were extracted from the results files containing around 2 billion of data.

10. Conclusion

The SSI design has been developed for project under construction and not for academic purpose. The short planning imposed by the project and the delays between contractor's questions and clarifications resulting from computations did not allow for developing complex and more scientific theories. Therefore, response spectrum analysis was mainly used in the search of the optimal solution during SSI.

More than twenty computations on a finite element model containing 10,000 nodes were performed. Each computation has involved the modal extraction of 800 frequencies and mode shapes, the response spectrum analysis for OBE and SSE with combination of static and seismic loads in X, Y and Z directions, the determination of the internal forces in the piles and the accelerations and displacements of the tank. Considering project planning, this computation work was only feasible in the linear domain. Anyway, the first results have quickly shown that the strain in the foundation ground was very low in the vicinity of the group of piles supporting the tank.

The initial computations on the raft with embedded piles using the preliminary ground characteristics, then with the backfill, and after with ground improvement have shown excessive bending and shear in the upper part of the piles. Impressive pile tractions have also been computed.

The use of elastomeric supports was then considered. After some trials with common supports provided in the catalogues of different manufacturers, elastomeric supports with lead core were chosen.

Several runs were necessary to select the most flexible device to reduce the internal forces in the pile heads. A significant decrease of the bending and shear values in the piles was observed. However, the vertical dynamic forces were still inducing tractions in the outer circle of piles which was located directly under the concrete tank wall, because the vertical component of the earthquake is transmitted directly from the bedrock to the tank through the piles. The vertical stiffness of the elastomeric support being approximately 1,000 times higher than the horizontal one, the reduction of the dynamic normal forces in the piles was smaller than the improvement observed in the horizontal direction.

In the meantime, the additional soil investigation has shown a deeper bedrock than first assumed. Consequently, the pile lengths increased, and varied from pile to pile as the bedrock surface is not flat and horizontal. This complicated the interpretation of the computation results, as some inner piles showed traction too. To make the study more accurate, the use of lumped ground masses far from the tank allowed taking in account 95% of the total mass in the vertical direction and 99% in the horizontal directions.

Finally, a really vertically softer support was found and available. The resulting SSI computations have shown that the horizontal shear and bending was very low, and that some traction still occurred during SSE under the outer circle of piles.

It was assumed that the use of the Response Spectrum Analysis with its Square Root of the Sum of Squares combination of 800 modes has overestimated all the results. To demonstrate it, a Forced Response Analysis by mode superposition has proved that in the worst case no traction occurs, and that a 250 kN compression remains in the most dynamically stressed pile under SSE.

In conclusion, SSI using Spectrum Analysis is the fast methodology to issue preliminary design and to identify the problems. Special care must be taken to review the participating masses during computations. Indeed, lower participating masses together with SRSS combinations will result in significant overestimation of the results. A final SSI using Forced Response Analysis is the best methodology to obtain realistic design values.

By mid-2016, Third Party reviews made by Chinese Institute and followed by French Control Organizations have validated the SAP results, using their own independent ANSYS and Stick Models.

11. References

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