Anti-floatation design test and simulation study of large LNG underground storage tanks

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Abstract. With the changes in China’s energy demand, the proportion of liquefied natural gas (LNG) is expected to account for 10% of the country’s total energy resources by 2020; thus, the demand for large storage tanks has become urgent. However, all LNG storage tanks in China are located aboveground, with no precedent set for fully- or semi-underground storage tank implementation. Therefore, the present study analyzes the displacement and pore water pressure characteristics of underground LNG storage tanks under anti-floatation conditions based on the construction of a 250,000 kL underground storage tank that is 91 m in diameter and 42 m in depth. Centrifuge tests were conducted to explore the development laws of displacement and pore water pressure of the bottom plate under different load conditions for fully- and semi-underground tanks. The displacement response of the tank under the flotage condition clearly exhibited three-stage characteristics, that is, the safe, dangerous, and failure stages. The development laws of each stage were then summarized, analyzed, and explained by numerical and analytical calculations to provide a theoretical basis for storage tank design and anti-floatation monitoring.

1. Introduction
Global efforts to protect the environment and reduce emissions of greenhouse gases, such as carbon dioxide, have led to significant increases in the supply and demand for clean fossil fuels, such as natural gas. In particular, liquefied natural gas (LNG) has become an important product in the natural gas industry owing to its advantages of safe storage and easy transportation. In recent years, China has actively sought to transform its energy structure and has vigorously promoted the use of clean energy. In particular, China’s natural gas industry has entered a golden period of development by strongly supporting pipeline construction, gas storage, LNG receiving stations, and other infrastructure. [1] Relevant data show that the abundance of cushion gas needed in underground gas storage results in a low investment recovery rate of only 85%, whereas the storage recovery rate of LNG is as high as 98%. [2]

The first successful development of LNG occurred in 1959 in the United States. After the 1970s, the country began to build receiving stations for exporting and importing LNG. In recent years, LNG receiving stations have been constructed on a large scale. [3]
Currently, Japan is the world’s largest builder of large LNG storage tanks, owning 62% of the world’s LNG supply. Although some of the storage tanks in this country are aboveground, most are underground tanks. In addition, Indonesia, Brunei, Algeria, and other LNG exporting countries, as well as LNG importing countries, such as Britain and France, maintain a large number of low-temperature LNG storage tanks under normal pressure. Korea Natural Gas has built the world’s largest LNG storage tank, with a capacity of 270,000 m³. However, most LNG tanks built in China are situated aboveground even though underground tanks are more functional owing to their characteristics of less land occupation, high safety, good sealing of stored liquid, strong seismic resistance, and good durability. Although the application prospects of underground tanks are broad, little research has been conducted on this topic and its implementation in China. Thus, further research should be conducted. However, the process of transferring LNG storage tanks from aboveground to underground remains challenging owing to significant differences in the boundary conditions of the tank structure as well as the anti-floatation and anti-seismic properties required for underground tanks. Moreover, LNG receiving stations need to be situated along coastlines, highlighting the requirement for the anti-floatation design of large underground LNG storage tanks. To realize the engineering application of underground storage tanks, the conflict between the safety and economy of the anti-floatation design should be resolved by research combining experiments, numerical calculation, and theoretical analysis. Considering the construction scheme of a 250,000 kL underground storage tank, the present study adopts a method combining a centrifuge model test and numerical simulation in order to simulate the anti-floatation condition during the operation of a large underground storage tank. This study discusses the development law of displacement of such tanks under different water levels to provide technical support for the anti-floatation design and monitoring of underground tanks.

2. Centrifuge model test

2.1. Introduction to centrifuge model test

The purpose of a geotechnical centrifuge model test is to improve the volume force of the model by using the centrifugal force field generated by the rotation of the centrifuge to simulate the behavior of the prototype under the natural gravity field. By controlling the centrifugal acceleration, the deformation characteristics and failure modes of the geotechnical structure prototype can be reconstructed in a small-scale model under conditions of similar geometry and mechanical properties and the same stress and strain values and failure mechanism. The centrifuge test has several advantages, such as its high centrifugal acceleration field. With the objective of obtaining 1:1 stress–strain between the model and the prototype, each particle of the model can establish a stress level and distribution scheme as close as possible to that of the prototype along the depth. On this basis, the complex nonlinear deformation characteristics and failure mechanisms in actual engineering cases, such as the site, rock and soil structure, and soil-bonding action, can be accurately, rapidly, and effectively reconstructed.

2.2. Storage tank engineering plan

In this study, integrated underground storage tanks were taken as the object. As shown in Figure 1, the storage tanks are composed of an internal tank and an external one. The internal tank is constructed of stainless steel and cold insulation material and contains LNG, whereas the external tank is made of reinforced concrete.
Figure 1. Schematic diagram of integrated underground storage tank.

Two external tank engineering plans for a fully-underground tank and a semi-underground tank, respectively, were tested, and the results were analyzed. Schematic diagrams of these tanks are shown in Figure 2 and Figure 3, respectively, and the dimensions of each member of the tank body are given in Table 1.

Figure 2. Schematic diagram of the fully-underground tank.

Figure 3. Schematic diagram of the semi-underground tank.

| Items                              | Fully-underground tank | Semi-underground tank |
|------------------------------------|------------------------|-----------------------|
| Prototype size (m)                 | Model size (m)         | Prototype size (m)    | Model size (m)         |
| Inner diameter of the external tank| 91                     | 0.364                 | 91                     | 0.364                 |
| Height of the external tank        | 42                     | 0.084                 | 42                     | 0.084                 |
| Thickness of external tank         | 2                      | 0.008                 | 2                      | 0.008                 |
| Thickness of the bottom plate      | 4                      | 0.016                 | 4                      | 0.016                 |
| Depth of the ground wall           | 84                     | 0.336                 | 42                     | 0.168                 |
| Thickness of the ground wall       | 1.2                    | 0.005                 | 1.2                    | 0.005                 |
| Diameter of the foundation pit     | 97.4                   | 0.390                 | 97.4                   | 0.390                 |
| Depth of the foundation pit        | 42                     | 0.084                 | 21                     | 0.042                 |

2.3. Centrifuge test plan

The tank body and retaining wall were constructed of an aluminum alloy, and the soil layer was simulated using standard sand. The good permeability of the sand enabled the pore water pressure to dissipate or increase rapidly, thus preventing significant influence between two adjacent groups of
tests. Because floatation resistance is the main consideration of this study, the LNG load was simulated by using water of the same mass.

The centrifuge acceleration was set at 100 g, where g represents gravitational acceleration. The generalized scaling factors, determined on the basis of the generalized similarity ratio theory and application, \(^{[9-10]}\) are given in Table 2.

| Quantity       | Scaling factors for 1 g test (prototype/virtual model) | Scaling factors for centrifuge test (virtual model/physical model) | Generalized scaling factors (prototype/physical model) | Value of generalized scaling factors \((\mu = 2.5, \eta = 100)\) |
|----------------|-------------------------------------------------------|------------------------------------------------------------------|-------------------------------------------------------|--------------------------------------------------|
| Length         | \(\mu\)                                               | \(\eta\)                                                        | \(\mu\eta\)                                           | 250                                              |
| Density        | 1                                                     | 1                                                                | 1                                                     | 1                                                |
| Time           | \(\mu^{0.75}\)                                        | \(\eta\)                                                        | \(\mu^{0.75}\eta\)                                    | 198.8                                            |
| Permeability   | \(\mu^{0.75}\)                                        | \(\eta\)                                                        | \(\mu^{0.75}\eta\)                                    | 198.8                                            |
| Pore pressure  | \(\mu\)                                               | 1                                                                | \(\mu\)                                               | 2.5                                              |
| Stress         | \(\mu\)                                               | 1                                                                | \(\mu\)                                               | 2.5                                              |
| Strain         | \(\mu^{0.5}\)                                         | 1                                                                | \(\mu^{0.5}\)                                         | 1.58                                             |
| Displacement   | \(\mu^{1.5}\)                                         | \(\eta\)                                                        | \(\mu^{1.5}\eta\)                                     | 395.3                                            |

As shown in Table 2, under the test conditions, the generalized scaling factor for length is 250; therefore, the model was created at 1/250 of the prototype size. The specific size data are given in Table 1.

The centrifuge model test system of the fully-underground tank is shown in Figure 4. Although not pictured, the semi-underground tank was designed with the same scheme.

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2.4. Model test monitoring system

To study the anti-floatation characteristics of the tank, this test measured the pore water pressure and the displacement of the bottom plate under the action of buoyancy. The pore water pressure was monitored by micropore pressure sensors with an outside diameter of 10 mm, thickness of 10 mm, and range of 500 kPa. Structural displacement was measured by noncontact laser displacement sensors. Because there was no need to make contact with the surface of the measured object in the measurement process, it has no influence on the tank model. Four equidistant measurement points were distributed in a radius at half the distance from the center of the circle of the bottom plate, as shown in Figure 4.

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2.5. Water level control system
The water level control system used a solenoid valve to control the water level. A water tank was set up on the upper part of the model box, and a system using water pipes with an inner diameter of 2 mm was arranged on the lower part of the soil body of the model box. A solenoid valve connected to the water pipe was used to control the water flow in order to simulate water level changes in the actual process. During this period, the water level was monitored through pore water pressure sensors as shown in Figure 4.

2.6. Centrifuge test process
The centrifuge test applied the following steps.
(1) The model and device were arranged in accordance with Figure 4. A particular amount of water was injected into the tank to simulate the load condition of the tank and to conduct debugging to ensure that the displacement meter, hole pressure sensor, solenoid valve, oil pump, and operation monitoring system were working normally.
(2) The centrifuge was started and was gradually accelerated to 100 g within 1000 s. During this procedure, monitoring was conducted to ensure that the centrifuge monitoring system and model box were not damaged accidentally, which guaranteed the safety of the test and normal operation of the centrifuge.
(3) After accelerating to 100 g and stabilizing, the solenoid valve of the water tank was opened to increase the water level from the bottom plate to the ground and to observe, detect, record, and analyze real-time data.
(4) After the monitoring data were stable, the power supply system of the centrifuge was removed to stop the machine, conduct drainage, and lower the water level to the bottom of the tank (initial state).
(5) The above steps were repeated for different working conditions.

Photographs of the test process are shown in Figure 5.

Figure 5. Photographs of the centrifuge model test procedures.
2.7. Test results and analyses

According to the test monitoring data and the generalized scaling factors given in Table 2, the displacement and pore pressure curves of the prototype structure under the condition of increased water levels are shown in Figure 6–Figure 9.

![Figure 6. Pore pressure diagram of the fully-underground tank.](image)

As shown in Figure 6 and Figure 7, in the fully-underground tank test process, the pore pressure and displacement of the bottom plate increased continuously owing to the increased water level and presented an almost linear feature. Moreover, the pore pressure and displacement of the bottom plate were close under different load conditions.

![Figure 7. Displacement diagram of the fully-underground tank.](image)

![Figure 8. Pore pressure diagram of semi-underground tank.](image)

As shown in Figure 8 and Figure 9, for semi-underground tank under the full LNG load, the pore pressure and displacement of the bottom plate increased with the increase in water level, which also presented a nearly linear trend. However, under the empty tank condition, the displacement of the semi-underground tank presented typical three-stage characteristics. At Stage I, or the safe stage, the results were similar to those of the full tank condition: With the increases in water level and pore pressure, the tank experienced slow and slightly upward displacement. At Stage II, or the dangerous stage, the speed of the displacement began to increase significantly although the duration was short and the displacement amplitude was still not
very large. At Stage III, or the failure stage, the displacement of the tank increased sharply and large floatation displacement occurred. This indicates that the tank body was damaged by the flotage. The results were plotted in Figure 10 and Figure 11 to directly analyze the response of the displacement to pore pressure.

![Figure 10](image1.png)  ![Figure 11](image2.png)

**Figure 10.** Response of displacement of the bottom plate to pore pressure for the fully-underground tank.

**Figure 11.** Response of displacement of the bottom plate to pore pressure for the semi-underground tank.

As shown in Figure 10 and Figure 11, in the safe stage, an obvious linear relationship was present between the pore pressure and displacement. However, once the pore pressure exceeded a certain critical value, it quickly entered the dangerous stage. If the pore pressure had continued to increase, floatation failure of the underground tank would have occurred.

3. Three-dimensional numerical simulation

To evaluate the test results and to further discuss the displacement characteristics of LNG underground storage tanks under the effect of groundwater, PLAXIS 3D software was used to simulate the anti-floatation condition for the fully-underground tank.

3.1. Parameter settings

The material parameters used in the numerical calculation are shown in Table 3 and Table 4. Among them, the strength of the concrete structure was relatively high and is thus assumed to be linear, indicating elastic material, and standard sand is assumed to be elastic–plastic material, which meets the Mohr–Coulomb failure criterion. \(^{[11]}\)

| Parameters | Symbol | Value | Dimension |
|------------|--------|-------|-----------|
| Density    | \(\gamma\) | 25 kN/m\(^3\) |
| Poisson ratio | \(\nu\) | 0.2 | —— |
| Young modulus | \(E\) | 30 GPa |

| Parameters | Symbol | Value | Dimension |
|------------|--------|-------|-----------|
| Unsaturated soil density | \(\gamma_{\text{unsat}}\) | 17 kN/m\(^3\) |
3.2. Model settings

A numerical model was then created according to the prototype size given in Table 1, with each component described as a “plate.” The tank model is shown in Figure 12. To eliminate the influence of the boundary effect, the soil boundary was set at 1000 m long, 1000 m wide, and 500 m deep, as shown in Figure 13.

| Property                  | Value |
|---------------------------|-------|
| Saturated soil density    | 20 kN/m³ |
| Elasticity modulus        | E = 80 MPa |
| Poisson ratio             | ν = 0.2 |
| Cohesion                  | c = 1 KPa |
| Angle of internal friction| ϕ = 45° |

Figure 12. Numerical model of the tank.

Figure 13. Numerical model of the soil.

3.3. Condition settings

According to the pressure–time curve shown in Figure 6, the same time interval and water level conditions were set in the numerical calculation to restore the working conditions of the increased water level used in the test. The simulated pore pressure curve is shown in Figure 14. In addition, the LNG load was set as a uniform load acting on the bottom plate.

![Figure 14. Simulated pore pressure curve](image-url)
Figure 14. Test and simulation results of pore pressure in the fully-underground tank.

Figure 15. Test and simulation results under full LNG load of the fully-underground tank.

According to the numerical simulation results shown in Figure 15, the response of the structural displacement to the pore pressure was similar to that shown by the test data; that is, the displacement showed a linear relationship with the pore pressure but little relationship with the LNG load. Therefore, the numerical calculation simulation and test results were mutually verified.

4. Analyses and discussions

4.1. Analyses of anti-floating forces of underground tanks

Many scholars have conducted abundant research in the field of anti-floating of underground structures. Li and Wu calculated the uplift force of underground water [12] and further analyzed the accuracy of the effective stress principle, the measurement of pore water pressure in clay, and the uplift pressure on the basement. They concluded that the principle of effective stress is applicable and effective in both saturated sand and clay [13]. Mei et al. studied the problem of reduced groundwater buoyancy derived from sand and clay by model testing, and they analyzed the variation in buoyancy with the water level and the dynamic process of buoyancy with time. [14] Liu et al. researched an underground granary in which backfill with a lime to soil ratio of 2:8 was considered in an indoor scale model test used for calculating the buoyancy resistance of underground structures. [15] Jeong et al. studied the reinforcement effect of an anchored pile on the horizontal and uplift resistances of a submerged breakwater by combining an analytical method and numerical analysis. [16] These analysis results of underground tanks indicate that the horizontal side pressures cancel each other, and the vertical forces meet the static equilibrium Equation (1) [13]:

\[ F_s + R_s = F_f + F_t + G_s + G_h \]  \( (1) \)

\( F_f \): Flotation force acting on the bottom plate (kN).
\( R_s \): Soil reaction force (kN).
\( R_t \): Foundation reaction force (kN).
\( F_s \): Side friction force (kN).
\( G_s \): Gravity on the tank structure (kN).
\( G_h \): Gravity on the LNG (kN).

On the basis of Equation (1), we calculated the forces occurring at the critical turning point of the semi-underground tank under no LNG load as shown in Figure 11. When the tank was transferred from the safe stage to the dangerous stage, the pore water pressure was 170 kPa. Thus, the flotation on the tank was calculated to be \( 134.2 \times 10^4 \) kN at this point, which is close to the gravity of the tank, at \( 133.4 \times 10^4 \) kN. When the tanks entered the failure stage, the pore water pressure was 220 kPa. The flotation on the tank at that time was calculated to be \( 171.4 \times 10^4 \) kN. Moreover, the side friction force acting on the tank reached the maximum level of \( 38 \times 10^4 \) kN.

Considering Equation (1) and the foregoing calculation results, the side friction force is obviously an important part of anti-floating force, accounting for 20%. According to the soil mechanics data, the maximum side friction stress is calculated in Equation (2). [17]

\[ f_{s,max} = \sigma_h \tan \delta = K \sigma_v \tan \delta \]  \( (2) \)

\( f_{s,max} \): Maximum side friction stress (kPa).
\( \sigma_h \): Horizontal effective stress (kPa).
\( \sigma_v \): Vertical effective stress (kPa).
\( K \): Coefficient of static side pressure.
\( \delta \): Angle of friction between the structure and the soil.

As can be deduced from Equation (2), with the decrease in effective stress caused by the increased water level, the maximum side friction stress will also decrease. For underground tanks, even though a
shallower burial depth results in less flotage force at the same water level, the maximum side friction force decreases more, which weakens its anti-floatation capacity. 

On the basis of the above analyses, Figure 16 was drawn to directly analyze the anti-floatation force of the underground tank.

Figure 16. Analysis of soil reaction force of underground tanks. For all forces, positive values reflect the upward direction and vice versa.

As shown in Figure 16, at Stage I, when the pore water pressure on the bottom plate was 0, the soil supported the weight of the tank and the LNG. As the flotage increased, the soil reaction force began to decrease until the soil reaction force reached 0, which means that the flotage force supported the weight of the entire structure and the LNG. At Stage II, as the flotage force continued to rise, the downward soil reaction force, which was mostly side friction force at this stage, resisted the flotage force. At that point, the tank began to separate from the foundation soil although large displacement still did not occur under the constraint of the soil body. However, the side friction force has a certain limit value that decreases with an increase in pore water pressure. When the soil reaction force reached the maximum side friction force, it entered the next stage. At Stage III, the pore water pressure increased. Because the soil could no longer provide additional side friction force, the tank body floated up under the total upward force.

4.2. The law of displacement response to pore pressure at Stage I

Considering the analysis results given in Section 4.1, we further analyzed the displacement development law of Stage I. At that stage, as shown in Figure 16, the effective stress of the soil decreases with an increase in flotage. This results in elastic rebound of the soil, which is related only to the unloading amount, or the flotage increment. Therefore, at Stage I, the displacement and pore pressure of the bottom plate show a good linear relationship, and the displacement under different LNG loads is not significantly different. This characteristic is demonstrated in Figure 10, Figure 11, and Figure 15.

4.3. Displacement development law at Stage II

At Stage II, which is the dangerous stage, the soil reaction is mostly due to side friction force. As the pore water pressure increases, the friction force on the downward side also increases. The displacement at this stage is attributed mainly to shear deformation of the surrounding soil. The shear modulus of the soil is usually 1/2–1/3 of the elastic modulus; therefore, the response of the displacement to the pore water pressure is more intense than that occurring at Stage I, as shown in Figure 11. In addition, the displacement amplitude during Stage II is still low although the duration of
the stage is relatively short. Therefore, this stage should be regarded as a warning of structural failure and should not be designed for normal operating conditions.

5. Conclusions
A 250,000 kL underground storage tank constructed with a diameter of 91 m and a depth of 42 m was analyzed to evaluate the displacement characteristics of underground LNG storage tanks under anti-floating conditions. According to the analyses results of a centrifuge test, numerical simulation, and analytical calculation, the following conclusions were drawn:

1. The LNG load and side friction force play important roles in the anti-floating condition. A semi-underground tank without an LNG load is obviously the most dangerous condition owing to the low load and burial depth. Therefore, the semi-underground tank scheme is not recommended.

2. The development of floatation displacement of an underground tank during operation is characterized by three stages. At Stage I, which is the safe stage, increases in the water level and pore pressure cause slow and slightly upward displacement related mainly to floatage; the LNG load is of little consequence. At Stage II, which is the dangerous stage, the stage duration is short, and the response of the displacement to the pore pressure is more intense. At this point, the structural displacement has not reached a large value, which can be regarded as an early warning of failure. At Stage III, which is the failure stage, the maximum side friction force of the soil is unable to resist the floatage. Consequently, the tank will quickly float under total upward force, resulting in destruction.

3. Because the responses of the displacement characteristics are obvious and an early warning is indicated before complete damage, it is recommended that more attention be paid to the response of displacement to pore pressure during monitoring.

4. The side friction can change owing to pore water pressure, earthquakes, and other factors. Thus, for safety concerns, this factor should be considered a safety reserve rather than an accurate resistance value at the design stage unless the ground wall is specifically designed to resist pulling.

5. In this study, sand was used for testing and analysis. Therefore, complex factors such as buoyancy reduction and soil cohesiveness were not considered. For practical application to engineering cases, more complex soil conditions should be used in future research.

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