RECENT TECHNOLOGICAL DEVELOPMENT OF IN-GROUND LNG STORAGE TANK AT TOKYO GAS’ OHGISHIMA TERMINAL

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1 Introduction

The first imports of LNG into Japan began in 1969, while the first in-ground LNG tank (with a capacity of 10,000 kl) was completed at the Negishi terminal of Tokyo Gas Co., in 1970. Since then, LNG consumption has steadily increased, and total storage capacity has risen in line with the rising usage. Japan now has more than 69 in-ground tanks, providing about 5.99 million kiloliters of storage, or 45 percent of the country’s total capacity. Most of the tank structure is below ground, normally with, only the roof visible.

Roughly speaking, the development of in-ground LNG tanks can be divided into three generations. The early generation of tanks, with capacities up to 95,000 kl, was constructed through the early 1980s. Large-scale tanks were developed during the second generation, from the early 1980s through the early 1990s. During this period, tanks with capacities of 130,000 and 140,000 kl were constructed at the Tokyo Gas Sodegaura terminal. In 1995 at our Negishi terminal, in-ground LNG tanks, each with a 200,000 kl capacity (and each the world’s “largest”), were built. Then, in 1998, the world’s first completely buried LNG tank was constructed at the Ohgishima terminal. The roof of this tank, as well as side wall and bottom slab, was made of reinforced concrete (RC). Now, the latest generation (third generation) of in-ground LNG tanks is under construction with the priority on cost efficiency in addition to reliability and safety. The new tanks achieve these aims by means of a rigid joint-less connection between the side wall and the bottom slab.

The author has been a civil engineer at Tokyo Gas Co., for more than 15 years, and has been engaged in various aspects of the technological development and construction related to in-ground LNG tanks. The aim throughout has been to realize greater reliability, safety, and economy in the construction of in-ground LNG tanks.

2 Technological Trends for In-ground LNG Storage Tanks

Tokyo Gas has been pushing the technology of in-ground LNG storage tanks forward for more than quarter of a century. Over this period, three generations of requirements and technology have been developed: “Development and Progress” (the first generation), “Scaling Up” (the second generation) and “Qualitative and Economic Performance” (the third generation). Features of the construction technology involved in each of these generations are described below. Figure 1 offers an outline of trends in technical development over this period.

2.1 The First Generation of In-ground LNG Tanks (1970 to the Early 1980s)

The Tokyo Gas Co. developed its own in-ground storage tank design, in which the side wall and bottom slab were constructed of reinforced concrete (RC) to provide strength. This design has demonstrated remarkably good earthquake resistance. The tank is lined with a stainless-steel membrane to provide a gas/liquid seal and with insulation to maintain the LNG in its liquid state. The first prototype tank of this design, with a capacity of 10,000 kl, was completed at the Negishi terminal in 1970. Between then and the early 1980s, a number of larger tanks, with a similar design, were designed and constructed. In the sandy, permeable ground at the Sodegaura terminal, tanks with a capacity of 60,000 kl were constructed. However at the Negishi terminal tanks with capacities ranging from 60,000 kl to 95,000 kl were built in soft, impermeable rock using a construction method developed especially for these ground characteristics. During this first generation of in-ground tank construction, the focus of technological development was on establishing a basic design and construction methodology. As part of this effort, the Japan Gas Association established Committee on LNG In-ground Storage in 1976. The following three years saw lively discussion, experimentation, and the investigations, of frozen soil, tank structures, earthquake-proof engineering, and security.

“Recommended Practice for LNG In-ground Storage”, which established standard practices for technology and security from the planning and construction stages up to maintenance, was the result
of these efforts. It was publication in 1979 by the Japan Gas Association.

2.2 The Second Generation of In-ground LNG Tanks (from the Early 1980s to the end of the 1990s)

In this second generation, large-capacity in-ground LNG storage tanks were developed. To cope with remarkable growth in LNG demand as the Japanese economy grew and as the need for clean energy increased, high-capacity tanks had to be constructed on sites with limited land area. This led to tank capacities vastly greater than in the previous generation. In this period from the early 1980s up to the end of the 1990s, Tokyo Gas took up the technical development of construction methods based on super-deep slurry walls and large-scale vertical New Austrian Tunnelling method (NATM). This made possible the construction of in-ground LNG tanks with capacities from 130,000 to 140,000 kl at the Sodegaura terminal. Ultimately, in 1995, the world’s largest in-ground LNG tanks of 200,000 kl capacity that came into operation at Tokyo Gas’ Negishi terminal. In addition to this progress, 1998 saw the completion of a fully underground LNG tank with a concrete roof in addition to the concrete side wall and bottom slab at the Ohgishima terminal.

2.3 The Third Generation of In-ground LNG Tanks

In a conventional in-ground LNG tank, the bottom slab has to withstand the up-lift due to groundwater pressure. From an economic point of view, the side wall and bottom slab are regarded as isolated structural members with a split-hinged connection. In contrast to this approach, Tokyo Gas has developed a new type of in-ground storage tank with improved economy, reliability, and safety in which the side wall and bottom slab are joined in a rigid unit. This latest type of tank is currently under construction at Ohgishima terminal.

![Fig.1 Trend in Technology Development for In-or-Underground LNG Tanks](image)

3 NEW TECHNOLOGIES APPLIED TO RECENT LNG TANK

3.1 Rigid Side Wall and Bottom Slab Connection
Conventionally, the side wall and bottom slab of an in-ground storage tank are separate structures with a joint between them, as shown on the left side of Figure 2. This joint is known as a split hinged connection. A cushioning material and other components are fitted at the joint so as to transfer sectional force between the side-wall and the bottom slab.

On the other hand, it was also understood that a tank with a rigid connection between the side wall and bottom slab, would have enhanced seismic resistance and deformation properties, as well as additional redundancy due to the combined structural strength of the rigid connection. However, practical implementation of such designs faced previously unsolved problems, however, such as progressive corner cracking due to stress concentration caused by rebar congestion at the bottom of the side wall. Now, after solving these problems by carrying out a design analysis using three-dimensional RC non-linear analysis, an advanced tank design of this type has been developed using large-diameter prestressed cables, adopting self-compacting concrete, and introducing a haunch structure. This new design offers inherent reliability and security, as well as economic efficiency and is shown on the right side of Figure 2. Details of the studies carried out are given below.
(1) Design for in-ground LNG tank with rigid connection

The right side of Figure 2 shows the tank configuration. Figure 3 shows a close-up of the rigid connection detail. The tank design's ability to withstand applied loads and groundwater cut-off performance were checked by allowable stress design and limit state design.

a) Examination of the design’s Ability to Withstand Loading

Section forces were calculated by Finite Element Method (FEM) analysis using three-dimensional solid modeling (a 180-degree model divided into 7.5-degree elements in the circumferential direction), as shown in Figure 4. This analysis considered the many loads that act on the body of an in-ground LNG tank. The surrounding ground and slurry wall were modeled as springs. Then the tank was analyzed for two cases: assuming no influence from the slurry wall (TYPE-A) and considering its influence (TYPE-B) in the case of a level-2 earthquake.

Seismic design was carried out for the earthquake motion given in “Recommended Practice for In-ground LNG Storage”, which stipulates 0.15 as the horizontal seismic coefficient (or 0.15 G as the horizontal seismic acceleration) at the seismic foundation (and half of these values in the vertical direction). Seismic design was also carried out for level-2 motion. The static seismic intensity method was adopted for the former case while, on the other hand, the seismic response method and dynamic analysis for coupled tank-ground interaction were implemented for the latter.

The results of this analysis led to a rebar arrangement at the rigid corner where the side wall meets the bottom slab as shown in Figure 3. Here, stirrups and additional rebar were arranged according to the results of the RC non-linear analysis to be described later. Prestressing forces were designed such that a compression zone of at least 10 cm would be secured in the member section, or such that a force no more than 100 N/mm² develops in the rebar. This secures more reliable cut-off capacity during normal operation.

![Fig. 4 Numerical Idealization of Tank with Rigid Connection (TYPE-A)](image)

b) Examination of Cut-off Performance

A concrete in-ground tank, kept cold by the cryogenic LNG it contains, must provide reliable cut-off under all operational conditions. Cut-off ability was examined using linear FEM analysis to confirm that no harmful residual cracks would develop in the concrete, even after an earthquake. Figure 5 illustrates one of the results of this analysis. The tank can be appraised as having adequate cut-off capacity since the rebar stress does not exceed the allowable stress and no residual cracks are present even after an earthquake.
(2) Safety Verification of Rigid Corner by Non-linear Analysis

A study of the rigid corner safety was carried out to ensuring that the rigid corner does not fail ahead of the side wall or bottom slab. It resulted in a quantitative assessment of whether additional corner reinforcement were necessary or not, and of the structural strength in the vicinity of the rigid corner.

a) Study Method

The groundwater uplift pressure beneath the bottom slab is considered to be the most significant load acting on the rigid corner. In the design analysis, this uplift pressure was increased beyond the design load level until failure. At the top of the side wall, the boundary condition is that deformation in the vertical direction is inhibited, while horizontal deformation is free to take place.

The computer program code used was WCOMD-SJ, which accounts for the non-linear behavior of reinforced concrete over a wide range of stress levels. Both side wall and bottom slab are numerically idealized as axial symmetric solid elements, but the roof and slurry wall are not idealized. Tables 1 and 2 show the material properties and failure criteria, respectively. Two analysis cases, (with and without additional reinforcement) were performed as shown in Table 3. The rebar arrangement in the vicinity of the corner is illustrated in Figure 6; the additional corner reinforcement (as a result of the analysis) is also shown.
Table 1  Properties of Materials

<table>
<thead>
<tr>
<th>Member</th>
<th>Concrete</th>
<th>Reinforcing bar</th>
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<tr>
<td></td>
<td>Compressive strength (N/mm²)</td>
<td>Tensile strength (N/mm²)</td>
</tr>
<tr>
<td>Bottom slab</td>
<td>24</td>
<td>1.9</td>
</tr>
<tr>
<td>Side wall</td>
<td>60</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Table 2  Failure Criteria

<table>
<thead>
<tr>
<th>Classification</th>
<th>Principal strain at failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tension failure</td>
<td>0.030 (3.0%)</td>
</tr>
<tr>
<td>Compression failure</td>
<td>-0.010 (-1.0%)</td>
</tr>
<tr>
<td>Shear failure</td>
<td>±0.020 (±2.0%)</td>
</tr>
</tbody>
</table>

Table 3  Analytical Case

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Additional corner reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>None</td>
</tr>
<tr>
<td>Case 2</td>
<td>Added</td>
</tr>
</tbody>
</table>
b) Results

The load vs. displacement relationship and the development of cracks are illustrated in Figures 7 and 8, respectively. For the case without additional corner reinforcement, it is predicted that failure is initiated deep in the corner zone. The structure exhibits a less ductile response that terminates at loading state A, as shown on Figure 7. On the contrary, when additional corner reinforcement is provided, greater deformability is ensured beyond loading state A. No critical cracks appeared in the corner even at loading state A, while the final loading state was reached when flexural failure occurred at the base of the side wall slightly distant from the corner. The crack induction process was different and the ultimate failure mode is clearly different, in the case of additional corner reinforcement.

![Graph showing load vs. displacement relationship for corner reinforcement](image)

Fig. 7 Load-Displacement Relations with / without Corner Reinforcement

![Diagrams showing crack development](image)

Fig. 8 Crack Development around Rigid Corner with/ without Additional Corner Reinforcement

(3) Rationalization of Rebar, in the Side Wall Foot

A study to reduce the amount of stirrups was carried out using non-linear analysis and equivalent linear analysis.

a) Study Method

Figure 9 is a flow chart of the process used to rationalize the rebar arrangement. Assuming a reduced stirrup arrangement, three-dimensional static analysis that takes into account the non-linear behavior of reinforced concrete was performed under the most severe stirrup loading condition; (i.e., level-2 earthquake motion without thermal loading and with LNG in the tank). The computer code used was COM3 developed in the concrete laboratory of the civil engineering department at the University of Tokyo. In order to allow detailed examination of the behavior of the rigid corner, the foot of the side wall and the bottom slab are idealized using solid elements, while the roof and slurry wall are not idealized. (Refer to Figure 10.) Material properties are the same as given in Table 1. The seismic load...
is assumed to be forced displacement under level-2 earthquake motion, as shown in Figure 11, and this was calculated by linear dynamic analysis for coupled tank-ground-LNG interaction. This forced displacement was applied beyond the design displacement until failure. Failure of the reinforced concrete was assumed to occur when structural strength began to fall, or when the tensile primary strain of the concrete in the tank body exceeded the allowable stainless-steel membrane strain.

**3D RC-nonlinear Analysis (COMS)**

**Purpose**: Re-examination of reinforcement quantity at bottom of side wall

**Seismic Load**: Forced displacement resulting from soil-structure-LNG interaction analysis to be applied to sidewall. The displacement increases in direct proportion until failure occurs.

**Normal Load**: Water pressure on bottom slab

**Analysis**: Displacement control

**Performance Verification**:
- Acceptable reduction of flexural stiffness and in-plane shear stiffness during level-2 earthquake
- No extreme change in structural system because of rapid development of crack

**Stiffness**:
- As a result of 3D RC-nonlinear analysis, the stiffness of side wall and bottom slab at the moment of failure is obtained. This stiffness is defined as the "equivalent stiffness"

**3D Equivalent Linear Analysis (ABAQUS)**

**Purpose**: To verify the safety of side wall and bottom slab during level-2 earthquake under various operational statuses. The equivalent stiffness is applied for the analysis model. The rebar arrangement is the one reexamined above.

**Load**: Various loads under a variety of operational statuses

**Operational Status**: Classified with water level, LNG level, thermal load

**Analysis**: Equivalent linear analysis with equivalent stiffness model

**Verification**: Limit state design method

Fig. 9 Flow Chart for Rationalization of Rebar Arrangement
non-linear analysis was adopted as the equivalent stiffness. The ABAQUS computer program code was used.

b) Results and Consideration

As a result of this non-linear analysis on the reinforced concrete, it was verified that no failure would occur in the rigid connection. Rather, failure occurred as in-plane shear failure between the foot and mid-height of the side wall at an orientation of 90 degrees to 130 degrees. Figure 12 shows the relationship between in-plane shear force and forced displacement in the element where in-plane failure occurred. Further, Figure 13 shows the reduction factors of flexural stiffness and in-plane shear stiffness at failure.
This result led to a reduction in the amount of stirrups by about 20% as compared with conventional design based on linear analysis only. It also confirmed that the failure mode of this type of tank under level-2 earthquake motion is not out-of-plane shear failure near the rigid connection but rather in-plane shear failure. Forced displacement at failure is 18 times the design load, hinting that this tank is particularly tough. This application of non-linear analysis was shown to be useful in tank design. It is a subject for future research to validate the (chose one and remove the other) tank design method using the knowledge acquired here. Doing so will be in accordance with the desire to actively introduce performance-based design in the future.

### 3.2 Application of Self-compacting Concrete

In the bottom region of the sidewall, the reinforcing bars arrangement was so congested that the minimum clear gap between the bars was left only 78 mm. In addition to that, ducts and anchor plates for the pre-stressing tendons, would have made it extremely difficult to place concrete in the conventional way. Therefore, Tokyo Gas decided to employ, self-compacting concrete, in the sidewall’s congested zone. The total volume used was 9,800 m³, including that used in the top portion of the sidewall where pre-stressing was introduced to support the roof. The technology of self-compacting concrete was developed in Japan and has found a variety of applications in concrete constructions. The properties of the concrete specified were:

- design compressive strength at the age of 91 days: 60 Mpa
- slump-flow: 65 ± 5 cm
- time limit for place ability: 90 minutes
- entrained air content: 4 ± 1 %

The mix proportion used is listed in Table 1. To do a 100% check of the flowability and the non-blockage properties of concrete to be placed, we constructed a special testing apparatus as shown in Figure 14. After the fresh concrete transported to the site by a mixer truck had been made to pass through the apparatus, it was pumped to be distributed in the placement area in the sidewall. The properties of concrete during placing were continuously monitored by video cameras installed in the site control room, from where instructions were conveyed for modifications if necessary.
3.3 Construction of Reinforced Concrete Roof

In the conventional LNG in-ground tanks, the roof structure was installed above ground level. The main design loads for the roof are its own weight, earthquake force and upward boil-off gas pressure, which makes it possible to use a steel grid-work and plate structure. However, the roof of the newly developed completely underground tank design, is required to support the weight of cover soil. To meet this severe loading condition a RC domed roof was employed, with thermal insulation and stainless steel gas tight membrane lining on the underside. The demand to make the tank’s embedment depth as shallow as possible along with lightening the cover soil burden on the roof and thereby to reduce the construction cost of the tank motivated Tokyo Gas to design the roof with as low a rise as possible. Scale model tests, non-linear finite element analyses, assessment on potential buckling etc., led to the adoption of a low rise/diameter ratio (h/D = 7.3m/73m = 1/10). The depth of soil cover at the edge of the roof could be reduced to 7.5 m with that at the center being 1 m. This was still equivalent to a total soil weight of 40 thousand ton, but much less than it would have been with a roof of normal h/D. To counteract the large horizontal thrust load coming from the roof a circumferential pre-stressing force of 12 thousand ton was applied in the periphery of the dome. Concrete was poured on a steel truss support for No.1 tank, but an airlift support system was used for pouring the roof of No.2 tank. As is illustrated in Figure 15, the steel roof lined with thermal insulation and gas tight membrane was assembled on the bottom slab. The framework-cum- roof support was airlifted up to the position where concrete could be placed in a layer of 50 cm thick, supported by and maintaining the air pressure. After the concrete was hardened the air pressure was removed and the concrete layer served as a support for the remaining upper layer pour of concrete with a depth of 50 to 150 cm. The total volume of concrete placed on the roof was 5,400 m³. Slump of concrete was controlled at $10 \pm 2.5$cm to prevent the fresh concrete from flowing down.

Adoption of the air-supported system for roof construction was instrumental in reducing construction period and cost by 4 months and 400 million yen, respectively.
4 CONCLUDING REMARKS

Demand for LNG a “clean energy”, is expected to continue to increase, as global environmental problems become more important. As a result, there will be a growing demand for underground LNG tanks, for which the technology has been developing very rapidly. Storage capacity, reinforced concrete roofs, as well as and rigid side-wall and bottom slab connections are all symbolic of the high level that the technology has already reached. This has come about because the engineers engaged in the development and construction of in-ground LNG tanks have continued to pursue new goals with unflagging enthusiasm. I hope to continue my involvement in new ideas for in-ground tanks without being satisfied with the current state of the art.