

IN-GROUND LNG STORAGE TANKS — TECHNOLOGICAL TRENDS AND LATEST TECHNOLOGICAL DEVELOPMENTS

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Liquefied natural gas (LNG) is a source of clean energy with stable long-term supplies that was first introduced into Japan in 1969. Since that time, sixty-six in-ground tanks for the storage of LNG, with a total capacity of 5,540,000 kiloliters, have been constructed in the country. Rapid developments in the technology used to construct these in-ground tanks, including the introduction of the super-deep slurry wall method and large-scale vertical NATM, have led to ever-increasing storage capacity — rising from 10,000 kiloliters in the early days to 200,000 kiloliters today. Completely buried tanks with concrete dome roofs have been constructed, and today's technology is such that tanks with rigid side wall to bottom connections are being constructed in large numbers for cost reduction while enhancing reliability and safety. This paper describes trends in LNG tank technology and the latest technological developments, as achieved by the author in his work at Tokyo Gas Co., Ltd.

Keywords: LNG in-ground tank; slurry wall; reinforced concrete dome roof; prestressed concrete; non-linear analysis; self-compacting concrete; rigid connection between side wall and bottom slab

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1. INTRODUCTION

Liquefied natural gas (LNG) is a low-grade mixture of hydrocarbons with a predominance of methane. It is a cryogenic liquid with a boiling point of -162°C at atmospheric pressure. A volume reduction to 1/600 results from liquefaction, which allows for efficient storage and transportation. 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 commissioned 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 66 in-ground tanks, providing about 5.54 million kiloliters of storage, or 43 percent of the country's total capacity, as shown in **Fig. 1**. A typical in-ground LNG tank is shown in **Photo 1**. Most of the tank structure is below ground, and only the roof is 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 by the early 1980s. Large-scale tanks were developed during the second generation, from the early

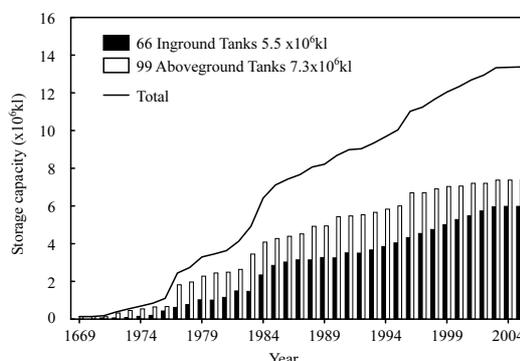


Fig. 1 Rise in LNG Storage Capacity

1980s up to the early 1990s; during this period tanks with capacities of 130,000 and 140,000 kl were constructed at the Sodegaura terminal operated by Tokyo Gas Co., At our Negishi terminal, the world's largest in-ground LNG tanks of 200,000 kl capacity were developed. Then, in 1998, the first completely buried LNG tank in the world was constructed at the Ohgishima terminal. The roof of this tank, as well as side wall and bottom slab, is of reinforced concrete. Now, the latest generation (third generation) of underground 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 jointless connection between the side wall and the bottom slab.

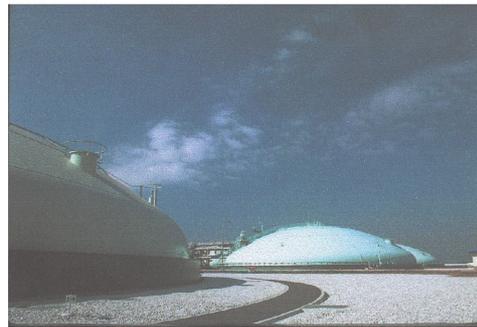


Photo 1 View of In-ground LNG Tanks (Sodegaura Terminal)

The author has been a civil engineer at Tokyo Gas Co., for more than 20 years, and has been engaged in various aspects of 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. In this paper, he gives an outline of LNG itself, describes in-ground LNG tanks, and discusses technological trends in the business. He then discusses the latest technological developments, including underground LNG tanks, rigid joints between side wall and bottom slab, etc.

2. LNG AND LNG STORAGE TANKS

(1) LNG

LNG is a colorless, odorless, and transparent gas that is cryogenic at a temperature of -162°C . In the process of liquefaction, impurities such as sulfur dioxide are removed — so it is acknowledged as a particularly clean source of energy. With abundant supplies of LNG distributed around the world, it offers the major advantage of a stable supply. Moreover, the release of sulfur oxides (SOx) and nitrogen oxides (NOx) during burning, and also of the global warming gas carbon dioxide (CO₂), is less than with other fuels such as petroleum and coal. (Refer to **Fig. 2**.) For these reasons, natural gas is known not only as a source of city gas but also a fuel for electricity generation. LNG presently accounts for about 12 percent of all energy consumed in Japan, taking an important position following oil and atomic power. Of the natural gas consumed in Japan, 97 percent is imported as LNG, and about 56 percent of the world's supply is imported into Japan. That is, Japan is the biggest importing country.

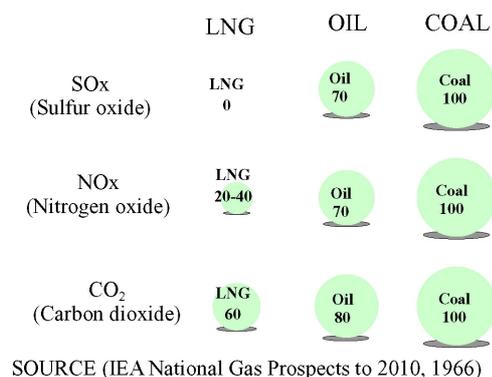


Fig. 2 Cleanness of LNG

(2) In-ground LNG tanks

The in-ground storage of cryogenic liquid LNG is desired for reasons of economic efficiency and safety. The main functionality required of an in-ground LNG storage tank is liquid/gas tightness, as well as the ability to withstand thermal shrinkage, liquid pressure, and other loads operating on the tank. Many types of LNG tank satisfied that meet these requirements have been developed and put into practical use. These can be divided into the following categories:

- ① Above-ground tank with bund wall
- ② Above-ground tank with PC outer wall
- ③ In-ground tank
- ④ Above-ground tank in pit

An in-ground tank is defined as one where the peak liquid level is below the level of the surrounding ground or below the rim of a stable embankment, and where the buried part of the tank is in contact with the surrounding ground. In-ground LNG tanks offer the following advantages over other designs: there is no possibility of

surface spills because the liquid is stored below ground; only the attractive roof with its relatively soft outline is visible at the ground's surface; and effective land use is possible because no protective dikes are required.

Figure 3 shows the configuration of an in-ground tank. The side wall and bottom slab are both of reinforced concrete, and these retain the tank against earth pressure and groundwater pressure. Within the side wall and bottom slab, a thin steel membrane provides liquid/gas tightness. This membrane has corrugations so as to absorb the deformation resulting from changes in temperature and gas pressure, and it is usually constructed from stainless steel (SUS 304) 2 mm thick. Surrounding the side and bottom of a tank is a heating system used to control ground freezing.

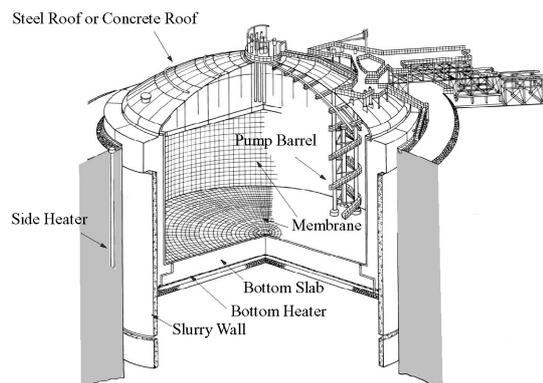


Fig. 3 Configuration of In-ground LNG Tank

A steel dome structure is usually adopted for the roof. The insulation used in the side wall and bottom slab must not only to preserve the cryogenic conditions in the tank interior but also transfer pressure from the membrane. For this reason, stiff polyurethane (PUF) foam with sufficient thermal isolation and compressive strength is normally used. The roof, on the other hand, is insulated by one of two methods. In a tank with suspended deck, glass wool insulation is spread over a deck suspended from the steel roof. Alternatively, PUF is affixed directly to the inner surface of the steel roof using bolts or other fixings.

3. TECHNOLOGICAL TRENDS FOR IN-GROUND LNG STORAGE TANKS

Tokyo Gas has been pushing forward the technology of in-ground LNG storage tanks 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 4** offers an outline of trends in technical development over this period.

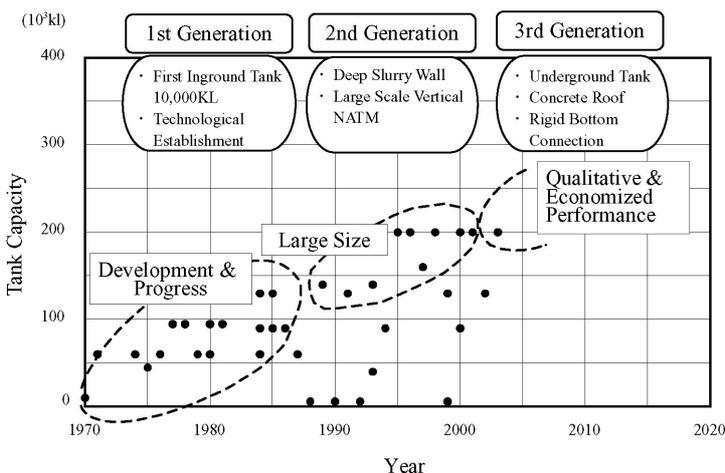


Fig. 4 Trend in Technological Development of In- or Under-ground LNG Tanks

(1) The first generation of in-ground LNG tanks (1970 to the early 1980s)

The idea of an in-ground LNG storage tank that would preclude spillage by keeping the peak liquid level below the ground surface was first realized in the form of excavations where the ground was frozen. Such tanks were constructed in Algeria and England in the 1960s. The sides and bottoms of these tanks consisted simply of frozen ground. This frozen ground sustains the groundwater pressure and earth pressure while at the same time maintaining gas/liquid tightness. (Refer to **Fig. 5**.)

The Tokyo Gas Co. developed its own design of in-ground storage tank in which the side wall and bottom slab were

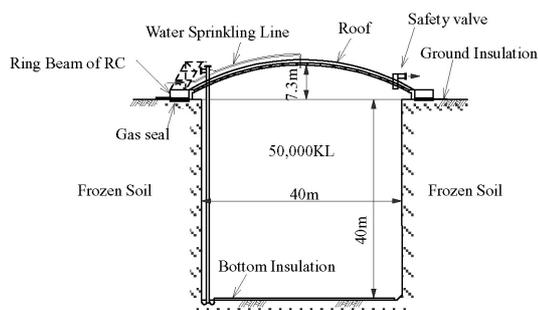


Fig. 5 In-ground Tank in Frozen Ground

of reinforced concrete to provide strength. This design demonstrated remarkably good earthquake resistance. The tank is lined with a membrane to provide a gas/liquid seal and insulation. The first prototype tank of this design, with a capacity of 10,000 kl, was commissioned at the Negishi terminal in 1970. Between then and the early 1980s, a number of similar tanks were developed and constructed. At Sodegaura terminal, on sandy permeable ground, tanks with a capacity of 60,000 kl were constructed, while at Negishi terminal tanks with capacities ranging from 60,000 kl to 95,000 kl were built on 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 Committee on LNG In-ground Storage was established by the Japan Gas Association in 1976. The following three years saw lively discussion, experimentation, and investigations of frozen soil, tank structures, earthquake-proof engineering, and security. The result of this was publication in 1979 by the Japan Gas Association of Recommended Practice for LNG In-ground Storage, which established recommended practice for technology and security from the planning and construction stages up to maintenance.

(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 the need for clean energy increased, high-capacity tanks had to be constructed on limited sites, and 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 NATM. This made possible the construction of in-ground LNG tanks with capacities from 130,000 to 140,000 kl at Sodegaura terminal and ultimately, in 1995, the world's largest in-ground LNG tanks of 200,000 kl capacity that came into operation at 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 Ohgishima terminal.

The following is a discussion of the major construction technologies that were developed to realize such large-capacity in-ground LNG storage tanks.

a) Super-deep slurry walls down to 100 m [1] (Sodegaura terminal)

The ground at Sodegaura terminal is soft reclaimed soil with an alluvium layer up to 15 m below the surface. Under this alluvium layer are diluvial layers. These diluvial layers mainly consist of permeable sandy material, but among these stiff sandy layers are impermeable diluvial silty layers at depths between approximately 40 m and 100 m.

Slurry walls are vertical continuous reinforced walls executed down from the ground surface. Their construction entails digging a length of deep trench using a special digging machine that uses a slurry method. Once the trench is complete, rebar cages are inserted into the trench and concrete is placed. This procedure is repeated in stages until a large cylindrical wall is obtained. This acts as a retaining and cut-off wall during later construction. However, if the gap between the wall sections is too large, the cut-off action is lost. In the early days, digging accuracy (measured as the ratio of gap in the horizontal direction to excavated depth) was only from 1/200 to 1/300, and this limited the depth of slurry walls to around 60 m. Further, certain ground conditions, such as those at Sodegaura terminal, limited the capacity of tanks constructed by conventional slurry wall technology to 60,000 kl because of ground heaving.

It was realized that if a slurry wall could be built to a depth of 100 m while maintaining safety against ground heave, in-ground storage tanks of much larger scale could be realized. Tokyo Gas took up the challenge of developing a digging machine able to dig a 100 m-deep trench to an accuracy of 10 cm in the horizontal direction (or an accuracy of 1/1000). On-site experiments involving digging under automatic control, construction of test joints between panels, and other aspects of the work were carried out. As a result of these experiments, it became possible to dig slurry walls using machines fitted with instruments such as inclinometers and depth indicators, thereby reducing error. Ultimately, super-deep trenching to 100 m in depth and with an error of less than 5 cm (namely, an accuracy of 1/2,000) was achieved for the first time.

This deep slurry wall technique was used to build cylindrical retaining and cut-off walls 100 m deep and 1.2 m thick. The side wall was made by reverse lining from top to bottom in a step-by-step manner as excavation

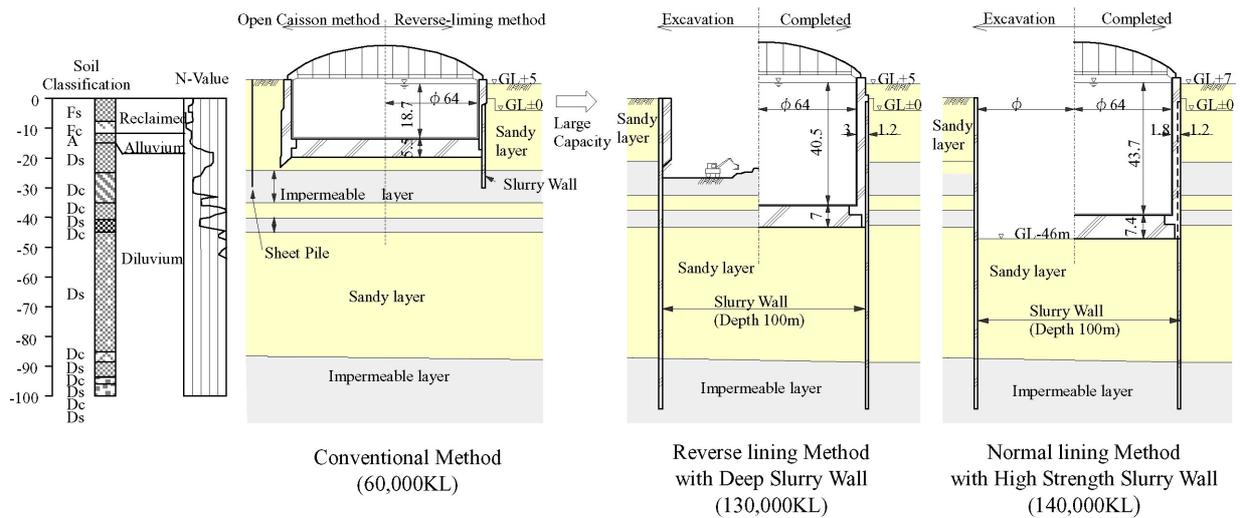


Fig. 6 In-ground Tank Features with Increasing Capacity (Sodegaura Terminal)

proceeded deeper. The final result was a large-scale in-ground LNG tank with a capacity of 130,000 kl. It measured 64 m in diameter and 41 m in depth.

In this tank, the slurry wall was merely a temporary structure used to assist construction. Further research was carried out to turn it into a permanent component of the side wall, where it could be used as a strengthening member. This would reduce the cost of construction while ensuring the quality of the finished tank. As a result of intensive effort, it became possible to build walls with a strength of 51 N/mm², much higher than the conventional 24 N/mm². A high-quality slurry wall of this design, which functions as part of the permanent structure as well as a retaining wall during construction, was used as the side wall of a 140,000 kl tank. The excavation was carried out in one stage, and the side wall was constructed from bottom to top by the normal lining method. (Refer to **Fig. 6**.)

b) Large-scale vertical NATM [1] (Negishi terminal)

At Negishi terminal, the ground is very soft reclaimed soil with an alluvium layer up to 10 m below the surface. Further below the surface, a soft rock known as mudstone is present. The unconfined compressive strength of this soft rock is 2-3 N/mm² and its permeability is very low. **Figure 7** shows the earlier retaining method used to construct an in-ground LNG tank with a capacity of 95,000 kl. The softer top layer down to 10 m was excavated using the sheet-pile sheathing method to retain the soil. Vertical steel piles were then driven into the soft rock along the line of excavation, and long anchors were inserted relatively deeply into the soil for attachment to the piles. These anchors support the piles against soil pressure.

A vertical NATM was developed as an economical and rational technology for super deep excavation in such conditions. This takes maximum advantage of mudstone's stable and consistent characteristics. A work cycle is applied in the excavation work, 3 m deep excavation is followed by excavated ground face protection with rock bolts. Then the consecutive step of another 3 m deep excavation is processed. Such cyclic work procedure is repeated subsequently. The protection is performed on the excavated surface in the manner of shotcrete with reinforcing mesh, horizontal boring hole, insertion of rock bolt and filling with mortar. Rock bolts measuring 5 m in length are set at a pitch of about 1.5 m.

Initially, a 60,000 kl in-ground LPG tank was constructed using this vertical NATM only for the section deeper than 16 m. A slurry wall was constructed for the first stage down to 17 m in depth. The diameter, length, and pitch of rock bolts was determined in accordance with experience using NATM for tunnel construction. The axial tension acting on the rock bolts and deformation of the excavated surface (using an inclinometer) were monitored during construction in order to accumulate technical data for use in a future large-scale excavation. Following this success, the method was applied to the excavation of a 85,000 kl in-ground LNG tank to a depth of 46 m. In this case, the arrangement of rock bolts was planned after carrying out ground behavior analysis for stress and deformation. During NATM work, a series of instruments was monitored and ground behavior investigated. Based on the instrumentation results and accumulated expertise, quantitative predictions of mudstone

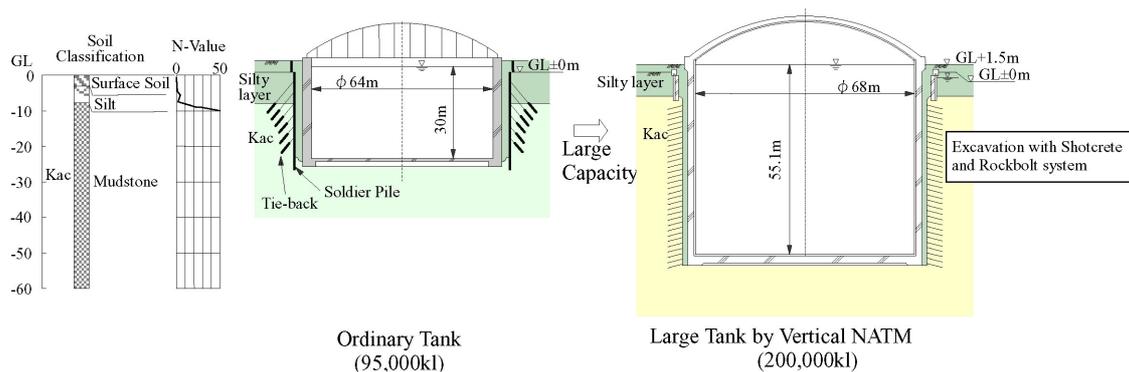


Fig. 7 Large Tank Constructed by Large-Scale Vertical NATM (Negishi Terminal)

behavior can now be provided analytically. This practical experience improved the reliability of vertical NATM in soil conditions like those at Negishi terminal. Ultimately, it led to large-scale excavations down to 60 m and the realization of in-ground LNG tanks with a capacity of 200,000 kl. (Refer to **Fig. 7.**)

(3) The third generation of in-ground LNG tanks (from the end of 1990's)

In a conventional in-ground LNG tank, the bottom slab has to withstand the up-lift due to groundwater pressure. The side wall and bottom slab are regarded as isolated structural members with a split-hinged connection from an economic point of view. In contrast to this approach, Tokyo Gas developed a new type of in-ground storage tank of 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. Details of this tank are given in Section 6.

4. DEVELOPMENT OF CONSTRUCTION TECHNOLOGY FOR IN-GROUND STORAGE TANKS

(1) Expanding the application of in-ground storage tank technology

In constructing large-scale in-ground LNG storage tanks, Tokyo Gas has developed and used new methods of excavation that form large underground spaces. These methods can be applied to the construction of other large-scale underground structures aside from LNG tanks. The super-deep slurry wall, to take a prominent example, was used in large numbers during construction of an artificial island off Kawasaki for the Tokyo Bay Aqua Line highway. It has also been used for the foundations of bridges, such as the anchorage of the Akashi Straits Bridge. Recently, a slurry wall was constructed as the 140 m in diameter retaining wall for an underground power plant. The vertical NATM method, too, is considered an excellent excavation method from an economic perspective, and it is expected to find application where advantage can be taken of mudstone's stable and consistent characteristics.

(2) Overseas application of construction technology for in-ground storage tanks

The concept of in-ground storage tanks was developed uniquely in Japan. Now, however, the safety and environmental advantages of this type of construction are being noted with interest in other countries, and such tanks have been built in the earthquake-prone countries of Taiwan and South Korea.

At the Yon-an terminal in Taiwan, three in-ground LNG tanks with a capacity of 100,000 kl and three with a capacity of 130,000 kl have been constructed. In this case, the upper layer of ground had just been reclaimed from the seabed and was consequently very soft. The old seabed layer was also relatively soft, so a careful study on use of a retaining wall method was carried out. As a result, the tanks were constructed using a reverse lining method with a rigid slurry wall.

On the other hand in Incheon LNG terminal of Korea two 140,000kl tanks, six 200,000kl tanks are now simultaneously under construction. The ground deeper than 70m from ground surface consists of granite, and concrete bodies are being constructed by a normal lining method with slurry wall.

5. LATEST CONSTRUCTION TECHNOLOGY FOR IN-GROUND STORAGE TANKS

— underground LNG tanks —

At Tokyo Gas' Ohgishima terminal, fully underground LNG tanks are in service. These tanks enhance safety while preserving the landscape (**Photo 2**).



Photo 2 Aerial view of Underground Tanks (Ohgishima Terminal)

The roof is a dome structure of reinforced concrete (RC) that can support its own weight plus the weight of the covering soil. A concrete roof of this type is considered the most suitable shell structure because compressive forces act on it. However, careful attention to buckling is necessary. A RC roof without covering soil is made stable against buckling by choosing a rise-diameter ratio from 1/6 to 1/10, as specified in ACI 1344 R-70 [2]. In the case of an underground tank that is to be covered with soil, the lower this rise-diameter ratio, then the less soil fill is required and the more economical construction becomes.

A small ration, however, makes it more susceptible to buckling. In order to clarify the buckling characteristics, experiments, non-linear analysis, and other investigations were carried out.

The development and design of an underground tank with a capacity of 200,000 kl is described below.

(1) Characteristics of the underground tank

Figure 8 shows the structural configuration of the 200,000 kl underground LNG storage tank. The vertical section is schematically illustrated in **Fig. 9** together with a conventional design for comparison. The conventional steel roof not only supports its own weight and loads such as imposed by earthquake, gas pressure, and so on, but also maintains gas-tightness. The purpose of the RC dome roof, in contrast, is only to withstand loading; gas-tightness is ensured by a stainless steel membrane with a thickness of 2mm.

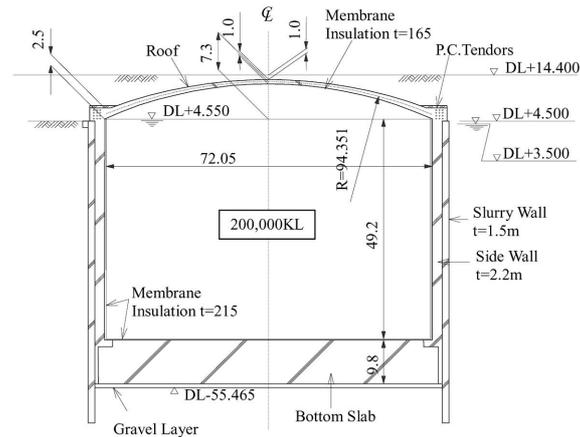


Fig. 8 Sectional View of Underground LNG Tank (200,000kl)

The dome roof has a diameter of 72.8 m, a rise of 7.3 m and a rise-diameter ratio of 1/10. The concrete thickness varies from 1 m at the center to 2.5 m at the circumference so as to provide support for the total weight of 40,000 tons of cover soil and 15,000 tons of concrete. The cover soil at the tank center is 1 m thick; this is enough to grow grass on top of the tank. The side wall top is designed to withstand an effective compressive force of 12,000 tons so as to prevent cracking due to the thrust of the roof.

(2) Development of low-rise concrete dome roof

In developing a concrete dome roof, it is crucial to deal with the dome instability problem when the rise is small. It is necessary to clarify in advance the buckling characteristics of the roof. To do this,

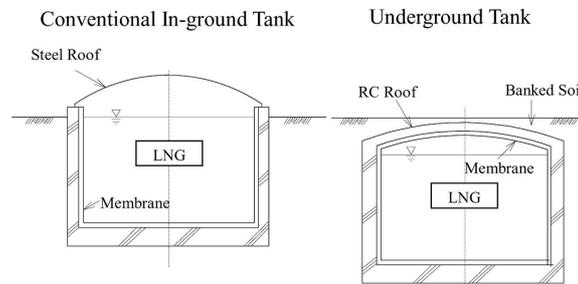


Fig. 9 Comparison of Conventional In-ground Tank with Underground Tank

experiments, non-linear analysis, and an examination of stability against buckling using the conventional design method were carried out to verify that there were no potential buckling instabilities.

a) Experiments [3]

A series of model tests was performed on a 1/20-scale concrete dome roof, as shown in Fig. 10, using the test arrangement shown in Fig. 11.

Table 1 shows the test cases and failure patterns. The three patterns illustrated in Fig. 12 were considered the possible failure patterns of RC dome structures. The concrete dome models in CASE2 and CASE5 can be characterized as having lower compressive strength and normal shell thickness. On the other hand, the CASE1, CASE3, and CASE4 models have higher compressive strength and thinner shells. All of these models suffered flexural failure in the tests, but the thin shell models exhibited a clear trend toward large deformation secondary effects. Moreover, in CASE4 (where the compressive strength of the dome mortar was about 80 N/mm^2 and its thickness is 1.74 cm — representing 34.8 cm in a full-size tank) the failure pattern indicated large deformation secondary effects. Yet this test model is considered to have reached the ultimate state of flexural failure without buckling.

This demonstrates that even if the rise-diameter ratio is reduced below $1/16$ in practice, a dome roof approximately 1 m thick is adequate to reduce secondary effects enough to prevent buckling.

b) Non-linear analysis

In analyzing structural stability phenomena such as buckling, it is considered that large-deformation (geometrically non-linear) theories need to be used in place of micro-deformation (geometrically linear) theories. The non-linear characteristics of materials can be also reflected in forms of structural analysis that trace displacement relations and stress paths to failure.

The author has confirmed that the experimental results can be simulated, the ultimate strength of the dome roof computed, and the failure pattern judged by means of a general FEM analysis program if the non-linear characteristics of materials are taken into account as stated below.

- ① The relationship between equivalent unconfined stress and strain for mortar
- ② The failure curve of mortar under biaxial stress
- ③ Residual tensile rigidity for mortar after cracking
- ④ Reduction modulus of shear rigidity for mortar after cracking
- ⑤ The relationship between stress and strain for rebars

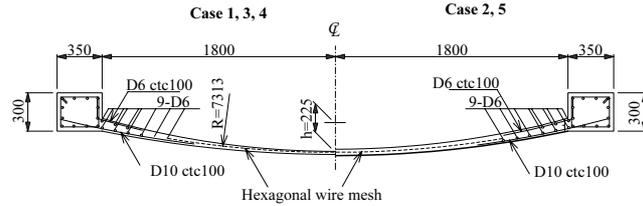


Fig. 10 Details of Model Concrete Roof

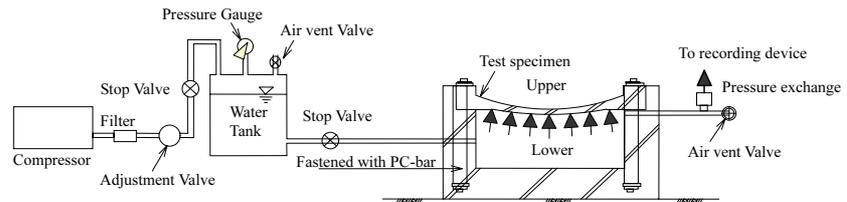


Fig. 11 Testing System

Table 1 Test Cases and Failure Patterns

CASE NO.	Rise h (m)	Span D (m)	h/D	Dome curvature R(m)	Crown thickness t_c (cm)	Mortar strength (N/mm ²)	Mortar Young's modulus (N/mm ²)	Failure load (kN/m ²)	Radius of failure zone r (mm)	Failure pattern
CASE1	0.225	3.6	1/16	7.313	3.33	83	3.36×10^4	477	300	②
CASE2	0.225	3.6	1/16	7.313	5.50	33	1.74×10^4	376	600~900	①
CASE3	0.225	3.6	1/16	7.313	1.74	76	3.20×10^4	133	300	②
CASE4	0.225	3.6	1/16	7.313	1.89	81	3.36×10^4	191	300	②
CASE5	0.225	3.6	1/16	7.313	4.67	41	1.81×10^4	379	600~900	①

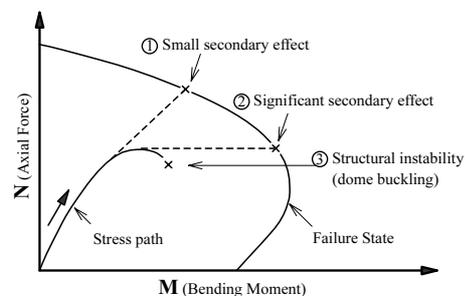


Fig. 12 Failure Pattern due to Secondary Effect of Large Deformation

c) Examination of buckling stability by conventional design method

The IASS (International Association for Shell and Spatial Structures) has published “Recommendations for the Buckling of Reinforced Concrete Shells” [4].

Table 2 Analytical Predictions of Applied Load at Buckling

	CASE1	CASE2	CASE3	CASE4	CASE5	
Failure pattern	②	①	②	②	①	
Test result (kN/m ²)	477	376	133	191	379	
Non-linear analytical value (kN/m ²)	471	382	146	206	402	
Recommendation by IASS	Linear analysis (kN/m ²)	1371	2037	384	470	1611
	Buckling load (kN/m ²)	651	815	192	229	740

In these recommendations, stability against buckling is defined to examine the comparison the buckling load obtained by following method and load worked actually on the dome roof. First, the linear buckling load obtained by linear eigenvalue analysis is calculated. Then the final buckling load is calculated using a reduction factor to take into account concrete plasticity, initial deformation and cracking, and the safety factor.

Table 2 shows the analytical predictions of the load at buckling in the case of a dome roof under the same conditions as in the experiments already described. The buckling load is calculated using both IASS and non-linear analysis.

In comparing the IASS buckling load with the experiments, the eigenvalue is high and the resulting buckling load obtained by IASS is differs considerably from the test results for failure patterns such as CASE2 or CASE5, where the secondary effects due to deformation are comparatively small. On the other hand, in the case of a failure pattern like CASE1, CASE 3, and CASE4, where the secondary effects due to deformation are greater, the buckling load obtained by IASS is almost the same as that in the tests. The excessive IASS value in the former cases is thought to arise because the IASS recommendations define the failure pattern as buckling whereas flexural failure occurred before buckling in the tests.

This means that careful selection of a reduction factor and safety factor would allow a rough value of buckling load to be acquired using the IASS recommendations for buckling.

(3) Design method for RC roof

The design procedure used for the RC roof is shown in **Fig. 13**. After determining the specifications of the roof (such as its size and the materials to be used, etc.) the standard design procedure (which ignores buckling) is first implemented. This entails allowable stress

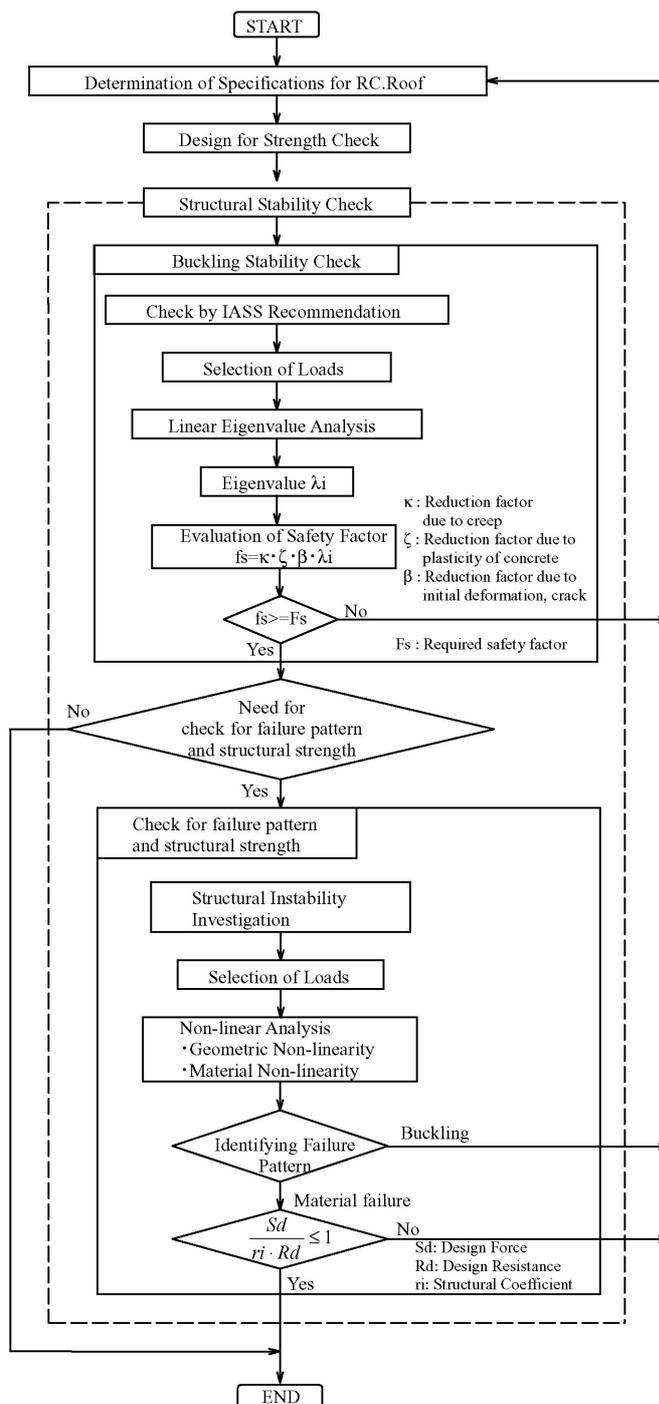


Fig. 13 Design Procedure for Reinforced Concrete Roof

design based on elastic theory as well as an examination of the failure pattern using the limited design method. Then, in order to check the stability of the dome roof against buckling, this is followed by linear analysis (according to the IASS recommendations) and an examination of failure pattern and structural strength by non-linear analysis.

(4) Construction of RC dome roof

Currently, three tanks in commercial use at Ohgishima terminal have RC dome roofs of the type described: two underground LNG storage tanks with a capacity of 200,000 kl and one underground LPG tank with a capacity of 60,000 kl. A third LNG tank is now under construction. The dome roof of the first LNG tank was constructed with a truss girder support so as to ensure shape accuracy of the inner roof for attachment of the membrane and to ensure problem-free construction. The air-raise & air-support method was adopted for the second tank, aiming at further cost savings and to reduce the construction period. On the other hand, a lift-up method was adopted for the RC dome roof of the LPG tank for the first time. The dedicated efforts of all design and construction engineers involved led to successful testing of these three methods and enabled the various technical issues to be overcome. The three methods are described in detail below.

a) Truss girder support method [5]

The insulation and membrane components fitted inside the concrete roof are in a configuration consisting of 421 polyhedrons. This requires extreme precision in the surface finishing on the roof interior. For example, the grooves for fillet welding of membrane elements require an accuracy of 6 mm or better. The allowable gap around the polyhedron edges and the plane irregularity of the polyhedrons is 10 mm or less. Considering these severe requirements, a truss girder method was adopted for constructing the RC dome roof 50 or 60 m above the bottom slab because of its proven reliability and long track record. The advantages of this method are easily controlled displacement through use of a high-rigidity truss (refer to **Fig. 14**) and easily workable plywood formwork for weather resistance. The truss girder support consists of an umbrella-shaped three-dimensional steel truss supported on a central steel pedestal and by brackets around the side wall. The truss structure weighed approximately 2,000 tons. (Refer to **Photo 3**.)

The roof concrete was placed in one session so as to ensure a precise finished surface for the membrane and to avoid the gaps that would appear between adjacent blocks due to displacement of the truss girder if staged placement were adopted. The following three requirements were taken into consideration at this time:

- ① The specified concrete strength must be reached at 28 days and a form removal strength of 18 N/mm² must be assured at 14 days.
- ② The roof concrete must not sag even after vibrating the concrete at the steepest point on the roof, 22 degrees, around the periphery, while it must remain moderately soft for at least 60 minutes after shipping to enable raking upward and surface leveling.
- ③ No harmful thermal cracks must be generated.

To meet these requirements, the slump flow was set at 10 cm plus or minus 2.5 cm, with reference to past construction records for RC dome roofs. The mix proportion given in **Table 3** was selected as a result of mixing tests with several cement types and trial placing with a mock-up section of the roof periphery. Concrete shear strength based on Vane shear tests was used as a selection criterion. A total concrete volume of 5,430 m³ was placed over a period of 28 hours in a concentric pattern from the periphery to the center. The surface of the placed concrete was covered with curing mats and plastic sheeting, from the periphery inward, as placing proceeded.

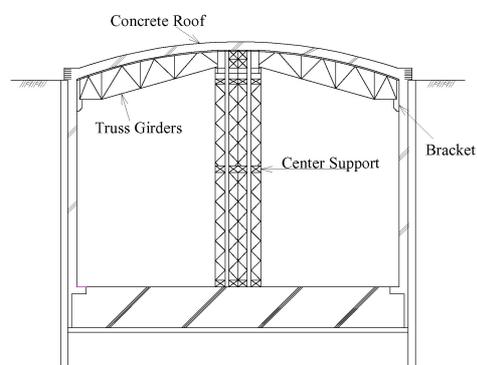


Fig. 14 Truss Girder Support for Roof Concrete Placement

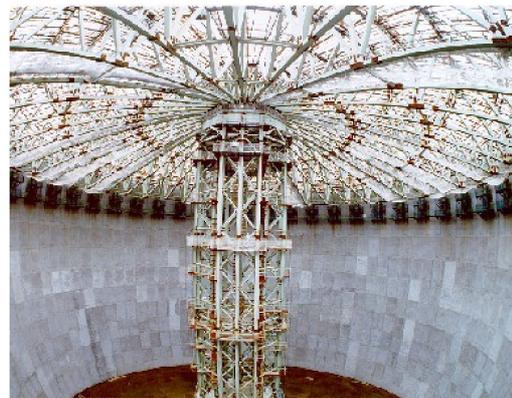


Photo 3 View of Truss Girder Support

Table 3 Mix Proportion of Roof Concrete

Type of Cement	Maximum size of coarse aggregate (mm)	Slump (cm)	Air content (%)	Water-cement ratio (%)	Sand-aggregate ratio (%)	Unit quantity (kg/m ³)				Admixture C*W _f (%)
						W	C	S	G	
MKC*	20	10±2.5	5±1	45.0	44.2	144	320	803	1057	MT 2000 R 1.20

*Portland blast-furnace slag cement for mass concrete: Type B

Curing was under moist conditions with water sprinkling for 28 days. (Refer to **Photo 4.**) As a result of this curing process, no harmful cracks were observed. Vertical displacement of the roof and rebar stress at the top of the side wall were measured in order to monitor the situation while the roof concrete was being placed, as the truss girder support was removed, and while prestressing the strands. A good match was obtained between the measured values and the calculations, confirming that the concrete roof behaved as an elastic body.



Photo 4 Concrete Dome Roof during Curing

b) Air-raise & air-support method [6]

In the case of the second LNG storage tank, a temporary steel roof was installed under the concrete dome. This was fabricated on the bottom slab, fitted with the insulation and membrane material, and then lifted in position at the tank top by blowing air pressure underneath. The air pressure was then raised during construction of the formwork and during concrete placing.

Using this method, the roof and side wall were constructed simultaneously for a time saving of 4 months and a cost saving of 400 million yen.

Although 60 cases have already proven the reliability of this method in Japan, certain characteristics of this situation required us to carefully examine steel stresses and buckling properties for all states (from fabrication on the bottom slab to air-raising):

- ① The rise-diameter ratio is smaller than in previous cases.
- ② Roof rigidity is low because of the H-100 steel main frames.
- ③ The flat part in the center of the roof disrupts the perfect geometry of the spherical shell

The temporary roof was raised using pneumatic force under posture control with balancing wires, as shown in Fig. 15. Prior to raising the structure, balancing weights on the steel roof were adjusted by measuring the total weight balance, and the deformation and tension of the balance wires, during a test raising.

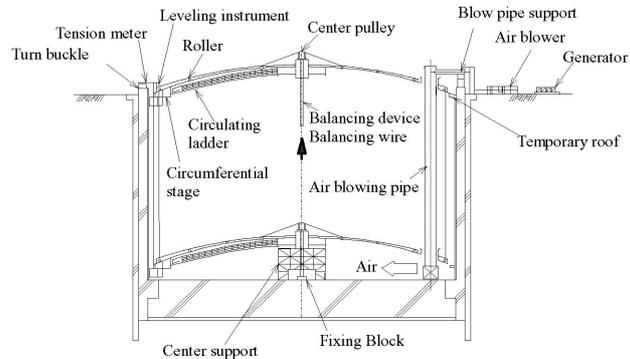


Fig. 15 Lifting of Steel Roof by Air Raising Method

In the actual raising procedure, the steel roof was raised by 46.4 m in 3.5 hours. Lift rate was controlled by adjusting the air blower flow rate such that it did not exceed the designated value shown in Table 4. The center of the roof deviated from level by only 9 mm, indicating the extremely good precision achieved. The RC dome roof, measuring 100 to 200 cm in total thickness, was constructed in two

Table 4 Control Items during Air-Raise

Item	Control value	Allowable limit value
Inclination	Max 160 mm	Max 200 mm (D/350)
Wire tensile force	Max 19.6 kN	Max 22.5 kN
Air pressure	1.7 from 3.0 kPa	—
Raising rate	Max 300 mm/hr	—
Misalignment	Target 15 mm	Max 30 mm

layers. The first layer was 50 cm thick and the second from 50 to 150 cm depending on the required roof thickness at each point. The first layer was constructed with the air support, while the second relied on support from the hardened first layer.

Strength and structural stability during concrete placing were examined by carrying out a stress analysis for the step-by-step construction procedure was well as by non-linear analysis. Residual stresses remaining after construction of the first layer were considered in the stress analysis, even for the completed state. This structural stability proving took into account non-linear concrete material characteristics and non-linear geometrical characteristics, and demonstrated that the first layer would not buckle even under 3.4 times the second layer weight.

Table 5 Control of Pressure during Air-Support

Item	Description		
Pressure	3.0kPa	At fixing of roof	
	17.0kPa	During rebar work	
	19.5kPa	Confirmation test for soundness during concrete placing	
Control range of pressure	-0.20kPa from +0.25kPa	-0.20kPa	Supply valve open
		+0kPa	Supply valve closed
		+0.05kPa	Release valve closed
		+0.25kPa	Release valve open
Alarm for pressure control	less than -0.50kPa	Red lamp + buzzer	
	more than +0.50kPa	Yellow lamp + buzzer	
Alarm for operation control	10 seconds after operation directive out of operation	Yellow lamp + buzzer	
Alarm for power supply	Normal	Green lamp	
	Power failure	Buzzer (by battery)	

Table 5 shows the pressure control values used for air-support. The internal pressure used during concrete placing was set at 19.5 kPa based on the results of stability analysis and stress analysis of the steel support. These same results indicated that the steel roof would lose its ability to resist compression buckling when placing of the first 50 cm-thick layer of concrete reached halfway from the periphery. To overcome this problem, concrete was placed to a thickness of 35 cm over the whole roof and then the remaining 15 cm layer was constructed before setting of the lower layer. The target initial setting time was extended to more than 12 hours by means of adding a setting retarder to the roof concrete. Observations of roof behavior (deformation of the temporary steel roof was measured every 30 minutes) indicated that actual deformation was less than the analytical value.

c) Lift-up method

The concrete dome roof of the 60,000 kl underground LPG tank was constructed by the lift-up method in order to save costs and reduce the construction period. **Figure 16** gives an outline of the construction process. The total weight of the concrete roof is 4,130 tf.

High-precision lifting of the large concrete dome roof reduced the work period by three months as compared with conventional support system formwork method a result of a highly detailed execution scheme worked out in advance.

Sixteen lifting points were selected in consideration of balancing the jack arrangement at the top of side wall and the jack capacity, which was 500 tf. The hydraulic chain was divided into four paths for controllability during lifting, with four jacks in each path all receiving the same hydraulic pressure (**Fig. 17**).

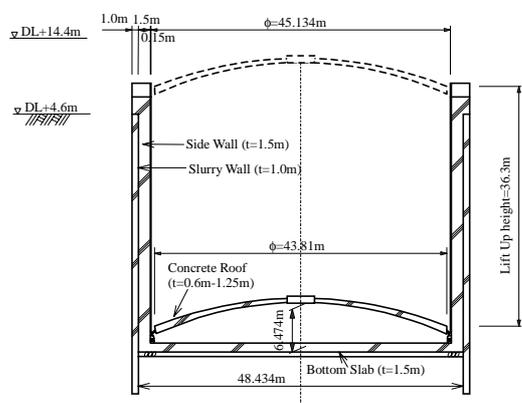


Fig. 16 Lifting of Reinforced Concrete Dome Roof

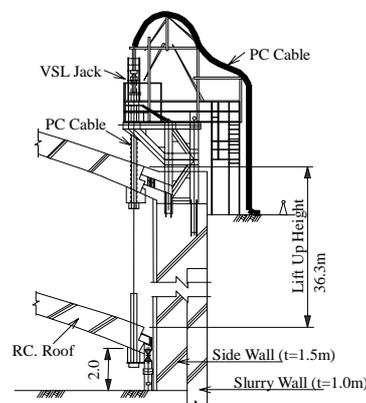


Fig. 17 Lifting Jack System

The maximum stress in roof members did not exceed 120 N/mm² and structural safety was proven through three-dimensional FEM analysis on a model in which the lifting cables were represented by spring elements. Lifting took three days. First, the dome roof was carefully lifted up to 5 m while measuring the displacement. Thereafter, the roof was lifted into its final position at a rate of 2.2 m/hour while controlling differences in vertical displacement among each jack path. After three days, the roof had reached the top of the side wall and was temporarily fixed in place.

Table 6 Monitoring Items

Status	Item	Monitoring method
Before initial raise from slab	Rebar stress	Rebar stress meter
During initial raise	Hydraulic jack	Pressure sensor
	Vertical displacement	Water-level pipe
During main raise	Rebar stress	Pressure sensor
	Hydraulic jack	Water-level pipe Electro-optical distance meter
	Horizontal displacement	Laser pointer
During temporary fixing to top of side wall	Hydraulic Jack	Pressure sensor
	Strain of temporary support member	Strain gauge
	Vertical displacement	Scale
	Horizontal displacement	Scale
After temporary fixing to top of side wall	Rebar stress	Rebar stress meter

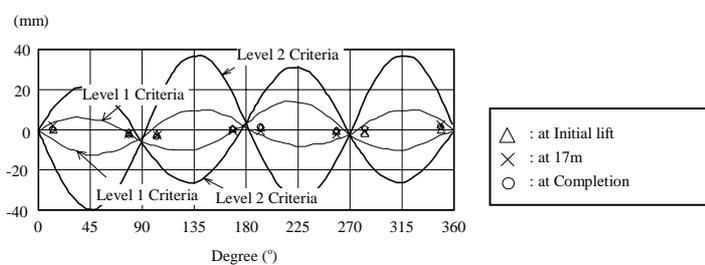
One crucial point was to control the RC dome roof such that it remained as close to horizontal as possible during initial lifting from the slab and during lifting into final position. Failure to control this could lead to harmful cracking. To achieve such control, the measurements listed in **Table 6** were taken and control values set up according to the criteria described below.

First criterion:

Values for a situation in which a 10 mm vertical differential is measured at points that 4.2 m distant from the boundary of each jack path. (Set up in consideration of adding a safety factor to the second criterion.)

Second criterion:

Values for a situation where the rebar stress in the roof reaches 120 N/mm². (A level of rebar stress at which cracking is assumed not to occur.)



Degree	12	78	102	168	192	258	282	348
Level 1 Criteria	4.1	-1.0	2.9	6.0	9.3	4.0	3.3	5.6
	-4.4	-6.6	-7.9	-0.1	-2.0	-4.1	-6.0	-2.3
Level 2 Criteria	11.2	4.9	12.9	18.8	17.1	10.5	14.6	17.1
	-15.6	-19.3	-15.6	-6.3	-11.7	-15.3	-13.5	-8.8
Initial lift	0.5	-1.0	-1.8	1.0	1.7	-0.1	-0.9	0.5
At 17m	2.6	-1.2	-2.1	0.0	-0.1	-1.9	-0.3	3.1
Completion	0.6	-2.3	-3.0	0.6	1.9	-0.1	0.6	1.8

Fig. 18 Variations in Vertical Displacement at Lifting Jacks

Figure 18 illustrates these control values and the measured vertical displacement differentials for each path. Maximum values of only 2 mm were observed. These differentials all fell under the level 1 criterion and lifting was completed with no trouble. Rebar stress in the roof fluctuated by a maximum of 30 N/mm² before and after lifting, and no substantial variations in roof stress conditions were noted.

6. TECHNOLOGY FOR THE LARGEST IN-GROUND STORAGE TANKS

— Rigid side wall and bottom slab —

Conventionally, the side wall and bottom slab of an in-ground storage tank are separate structures with a joint section, as shown in **Fig. 19**. This 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 understood that a tank with a rigid side wall and bottom slab structure would have enhanced seismic resistance and deformation properties, as well as additional redundancy due to the combined structure of rigid connection. Practical implementation of such designs faced unsolved problems,

however, such as progressive cracking at the corner and rebar congestion due to stress concentration at the bottom of the side wall. Now, having solved these problems by carrying out a safety examination using three-dimensional RC non-linear analysis, applying large-diameter prestressing cables, adopting self-compacting concrete, and introducing a haunch structure, an advanced tank design of this type has been developed. This new design offers inherent reliability and security, as well as economic efficiency. Details of the studies carried out are given below.

(1) Design for in-ground LNG tank with rigid connection

Figure 19 shows the tank configuration, together with the ground characteristics at Ohgishima terminal. Ability to withstand applied loads and groundwater cut-off performance were checked by allowable stress design and limit state design.

a) Examination of ability to withstand loading

Section forces were calculated by FEM analysis using the three-dimensional solid model shown in **Fig. 20** (180-degree model divided into 7.5-degree elements in the circumferential direction). 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 this earthquake motion as given in “Recommended Practice for In-ground LNG Storage”, which stipulates 0.15 as the horizontal seismic coefficient or 150 gal as the horizontal seismic acceleration at the seismic foundation (and half of those 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 **Fig. 21**. Here, stirrups and additional rebars were arranged according to the results of RC non-linear analysis as 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

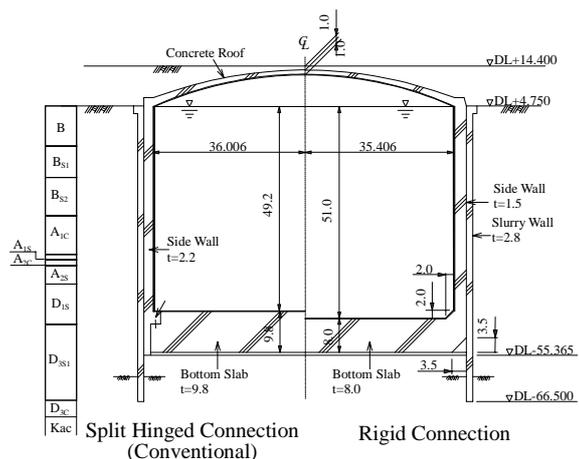


Fig. 19 Comparison of Conventional Tank and New Tank with Rigid Connection between Bottom Slab and Side Wall

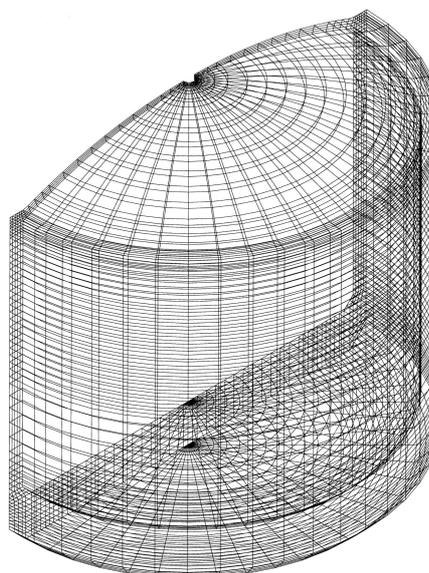


Fig. 20 Numerical Idealization of Tank with Rigid Connection (TYPE-A)

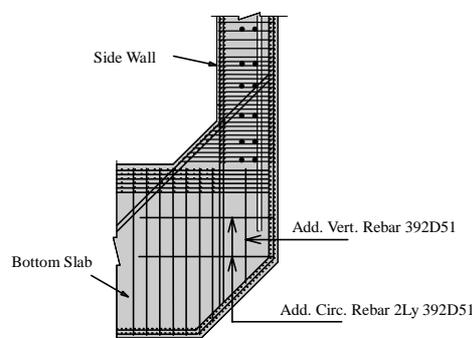


Fig. 21 Rigid Corner Reinforcement Details

develops in the rebars. This secures more reliable cut-off capacity during normal operation.

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 so as to confirm that no harmful residual cracks develop in the concrete even after an earthquake. **Figure 22** illustrates one of the results of this analysis. The tank can be appraised as having adequate cut-off capacity inasmuch as the rebar stress does not exceed the allowable stress and no residual cracks are present even after an earthquake.

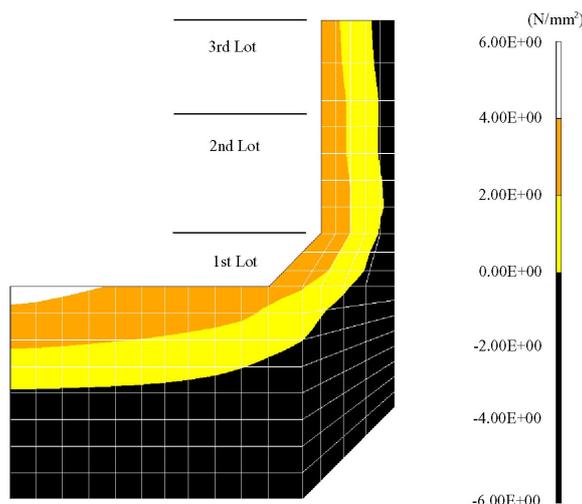


Fig. 22 Stress Distribution for Evaluating Watertightness Requirement under Ground Water, LNG and Thermal loading

(2) Safety verification of rigid corner by non-linear analysis

This study of the safety of the rigid corner was carried out with the aim of 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 is necessary or not and of the structural strength in the vicinity of the rigid corner.

a) Study method

The uplifting groundwater pressure beneath the bottom slab is considered the most significant load acting against the rigid corner. This uplift pressure was increased beyond the design load level until failure (**Fig. 23**). 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.

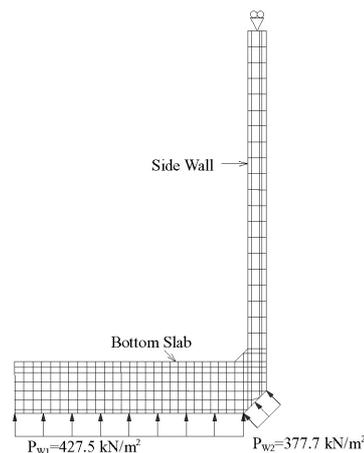


Fig. 23 Loads and Support Conditions

The computer code used was WCOMD-SJ [7], which accounts for the non-linear behavior of reinforced concrete over a wide range of stress levels. Both side wall and bottom slab are

Table 7 Properties of Materials

Member	Concrete		Reinforcing bar
	Compressive strength (N/mm ²)	Tensile strength (N/mm ²)	Class
Bottom slab	24	1.9	SD390
Side wall	60	3.5	SD345

Table 8 Failure Criteria

Classification	Principal strain at failure
Tension failure	0.030 (3.0%)
Compression failure	-0.010 (-1.0%)
Shear failure	± 0.020 (± 2.0%)

Table 9 Analytical Case

Case No.	Additional corner reinforcement
Case 1	None
Case 2	Added

numerically idealized as axial symmetric solid elements, but the roof and slurry wall are not idealized. **Tables 7** and **8** show the material properties and failure criteria, respectively. Two cases of analysis (with and without additional reinforcement) were performed as shown in **Table 9**. The rebar arrangement in the vicinity of the corner is illustrated in **Fig. 26**; the additional corner reinforcement is also shown.

b) Results

The load displacement relation and the development of cracks are illustrated in **Figs. 24** and **25**, 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. 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 exhibits clearly distinguishing features in the case of additional corner reinforcement.

(3) Rationalization of rebars in 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 27 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 severest stirrup loading condition; that is, level-2 earthquake motion without thermal loading and with LNG in the tank. The computer code used was COM3 [8] 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 **Fig. 28**.) Material properties are the same as given in **Table 7**. The seismic load is assumed to be forced displacement under level-2 earthquake motion, as shown in **Fig. 29**, and this was calculated by linear dynamic analysis for coupled tank-ground-LNG interaction. This forced displacement was applied beyond

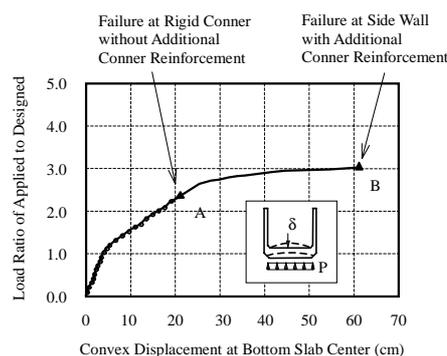


Fig. 24 Load-Displacement Relations with / without Corner Reinforcement

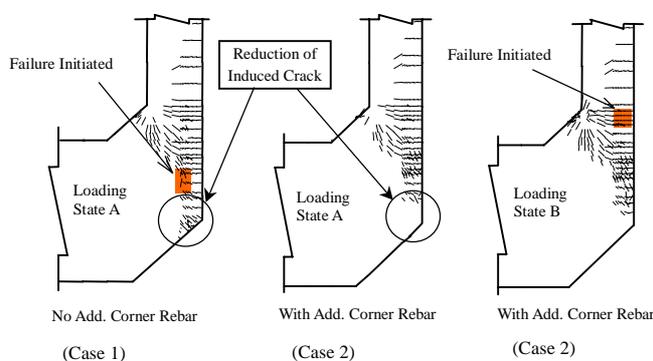


Fig. 25 Crack Development around Rigid Corner with/ without Additional Corner Reinforcement

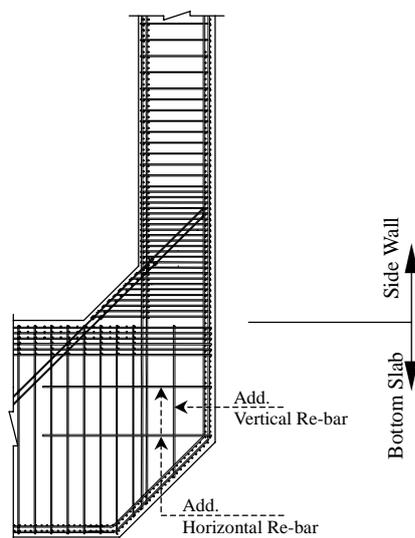


Fig. 26 Reinforcement Details at Rigid Corner

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 membrane strain.

Next, equivalent linear analysis using an equivalent stiffness model was carried out to verify the safety of the side wall and the bottom slab under level-2 earthquake motion when the tank is in various operational states. The residual stiffness of the tank body obtained from the result of RC non-linear analysis was adopted as the equivalent stiffness. The ABAQUS computer code was used. All parts of the tank body were idealized using shell elements.

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 30** shows the relationship between in-plane shear force and forced displacement in the element where in-plane failure occurred. Further, **Fig. 31** 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 takes place at 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 rationalize a tank design method

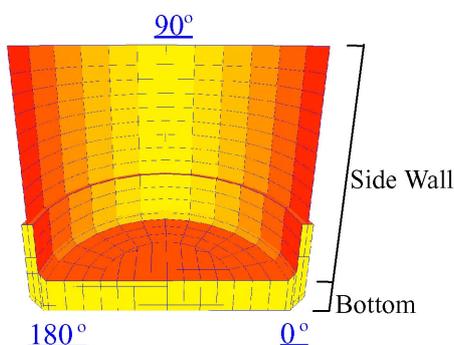


Fig. 28 Modeling for Non-Linear Reinforced Concrete Structural Analysis

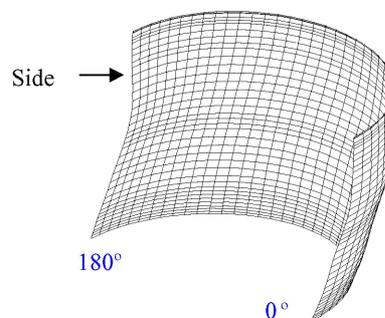


Fig. 29 Forced Displacement of Tank to Simulate Level-2 Earthquake

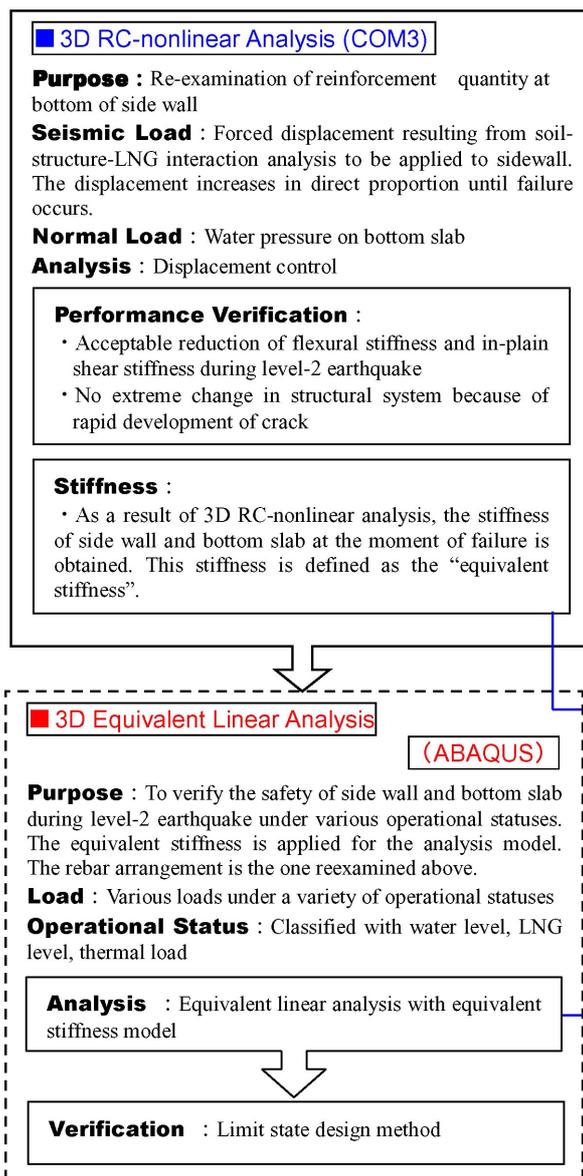


Fig. 27 Flow Chart for Rationalization of Rebar Arrangement

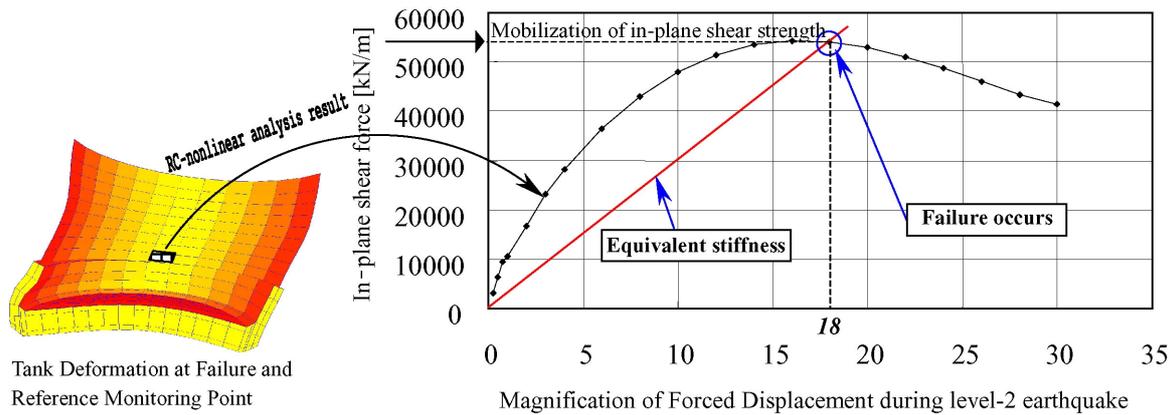


Fig. 30 Relationship between In-plane Shear Force and Forced Displacement

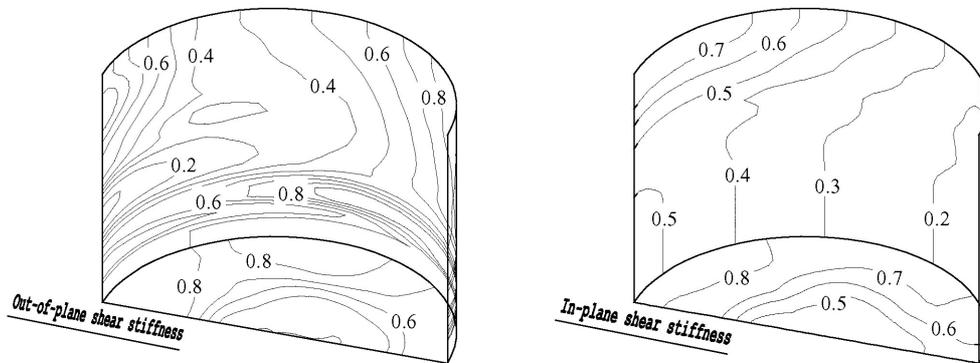


Fig. 31 Reduction Factors for Flexural/In-plane Shear Stiffness at Failure

using the knowledge acquired here. Doing so will be in accordance with the desire to actively introduce performance-based design [9] in the future.

(4) Application of self-compacting concrete

a) Self-compacting concrete requirements

The side wall of the tank is 2.8 m thick and 53 m high. Circumferential as well as vertical prestressing tendons are set into the lower portion of the side wall. The arrangement of rebars (D51) in this area is so congested that the minimum clearance is just 78 mm. Further, ducts and anchor plates for the pre-stressing tendons would have made it extremely difficult for workers to enter this space for conventional concrete placement. Consequently, it was decided to introduce self-compacting concrete for this congested part of the side wall.

Table 10 shows the quantitative

Table 10 Qualitative Requirements for Self-Compacting Concrete

Item	Unit	Specification	Remarks
Design compressive strength	N/mm ²	60.0	Design age 91 days
Proportioning strength	N/mm ²	68.3	Coefficient of variation: 7.0% Over design factor: 1.138
Slump flow	cm	65 ± 5	Standard value at site test
O-funnel time	sec	(9~15)	Reference value: likely to vary with ambient temperature
Air content	%	4 ± 1	Considering high-strength concrete and freeze-thaw durability
Filling height of U-Type	cm	30.0	Barrier type R1
Retention time of quality	min	90	Considering time from mixing to placing

requirements of the self-compacting concrete. These requirements, including slump flow, O-funnel time, and filling height in a U-tube, were selected based on a target of level-1 self-compactability as defined in “Recommendations for Self-Compacting Concrete”[10]. A value of 60 N/mm² as the specified design strength was selected to achieve cost reduction by reducing the thickness of the side wall. The air content was set so as to maintain freeze-thaw resistance.

b) Application of self-compacting concrete [11]

Tables 11 and 12 give the properties of the various materials and the mix proportion, respectively. Because groundwater pressure as high as 0.6 N/mm² acts on the side wall, the concrete used is required to have high compressive strength and good water tightness. A rich powder mix was chosen for the self-compacting concrete. Low-heat Portland cement was used, and limestone powder, which has high Blaine value, was added. A chemical air-entraining agent and a polycarboxylic acid superplasticizer were used. Resistance to segregation was secured by adjusting the amount of powder while self-compactability was maintained by reducing the amount of coarse aggregate.

The quality of a self-compacting concrete used without vibration depends on its characteristics at the time of manufacture. For this reason, concrete that did not satisfy the required quality standards had to be discarded before pouring. For this reason, a prototype test unit designed to check all concrete batches for the required properties immediately before placing was studied.

Table 11 Material Specifications

Item	Material	
Cement	Low-heat Portland cement	ρ: 3.24g/m ³ Blaine: 3230cm ² /g
Admixture	Lime stone powder	ρ: 2.70g/m ³ Blaine: 7280cm ² /g
Chemical admixture	High-range water reducer Air entraining agent and superplasticizer	Polycarboxylic acid type
Fine aggregate	Mountain sand	ρ: 2.60 g/m ³ Absorption: 1.32%, F.M: 2.60
Coarse aggregate	Crushed stone	ρ: 2.70 g/m ³ Absorption: 0.632%

Table 12 Mix Proportion of Concrete

F ² ck(91) (N/mm ²)	W/C (%)	s / a (%)	Unit quantity (kg/m ³)					
			W	C	Admixture (LSP.)	S	G	Chemical Admixture (AE, SP)
60	38.0	52.4	160	421	108	858	810	8.99

c) Improving quality assurance with new test unit

The prototype test unit for self-compactability [2] consists of a box with obstacles representing the rebars. Concrete is evaluated as self-compacting if it passes through the obstacles without the use of force. In general, self-compactability is judged from the performance of concrete passing through small gaps in a test box, while flowability and viscosity are examined by sampling. However, when large amounts of concrete are being placed, the number of samples increases and quality control becomes very complex. Attempts were made to improve the prototype so as to reduce the need for so many conventional sampling inspections for flowability and viscosity. Three different designs were tested, including the prototype. The performance of concrete in passing through each was observed, and the time taken to pass through was measured.

- 1, Type 1: prototype
- 2, Type 2: with extended horizontal distance within box
- 3, Type 3: Type 2 with added barriers

With Type 2, the horizontal distance was increased in order to give greater differentiation in the time taken by concrete to pass through. With Type 3, the dynamic force was reduced when concrete came into contact with the barriers. **Table 13** shows test results on the fluidity of self-compacting concrete passing through the three types of test unit.

Even with varying slump flow and O-funnel time, little difference was seen in the time taken to pass through the test devices, and all concrete passed successfully through the boxes. When using Types 1 and 2, the dynamic force involved in pouring the concrete into the test device acted on the concrete, and it was difficult to clearly differentiate variations in flowability and viscosity. As a result, concrete performance could not be judged. In test unit Type 3, concrete with a slump flow of about 45 cm was blocked by the barriers. However, concrete with slump flows 55 cm and 60 cm passed through the box at the same rate, and no blocking occurred. Consequently, it was judged that this design was suitable for managing quality control for all

concrete in this project.

In practice, air content and slump flow were measured by sampling in addition to the above quality assurance testing on all the concrete. **Photo 5** shows the test unit.

Table 13 Results of Pouring Self-compacting Concrete through Three Test Units for Self-Compactability Evaluation

Type of test unit for self-compactability evaluation	Slump flow passage time through test unit			Remarks
	60cm	55cm	45cm	
Prototype	2min 33sec	--	2min 38sec	4.8m ³ /1 car
With extended horizontal distance within unit	3min 34sec	--	3min 12sec	4.8m ³ /1 car
With barriers	2min 40sec	2min 14sec	Blocked	2.3m ³ /1 car



Photo 5 Testing Apparatus with Barrier for Self-Compactability Evaluation



Photo 6 Using the Testing Apparatus at Construction Site

d) Concrete placement in side wall using new test equipment

The first lot of side wall concrete was placed using eight pumps arranged around the tank. It took about 10 hours to place a volume of 1,815 m³. The test unit was located between the concrete pump and the agitator truck, as shown

Photo 6. Moreover, video cameras were installed at the testing unit to allow continuous monitoring by a technical expert in the control room. Quality control of the poured concrete using this method was very good, and it was possible to save considerable labor by reducing the frequency of sampling.

7. THE LATEST CONSTRUCTION TECHNOLOGY FOR IN-GROUND STORAGE TANKS

— Revision of Recommended Practice for In-ground LNG Storage —

It is now about 20 years since the current “Recommended Practice for In-ground LNG Storage” was issued in March 1979. In the intervening years, this code of practice has been quoted in the Electric Utilities Industry Law and the Gas Utility Industry Law, indicating the importance of its recommendations. The code of practice is expected to increase in importance in the years to come. For this reason, the Japan Gas Association has been revising the code to reflect the current state of the art in updated and rationalized contents.

The basic premise on which the revision process is based is as follows. First, since both the revised code and the current one are to be quoted in the Gas Utility Industry Law and the Electric Utilities Industry Law, the revised version needs to take into account the new versions of these laws. Second, the recommended seismic design method should correspond to the high-level earthquake motion defined in the disaster prevention master plan that the governmental Central Disaster Prevention Council published in July 1995. Thirdly, the concept of performance-based design is being adopted in the fields of civil engineering and building work.

Finally, the contents should include explanation of the latest technology as described in this paper, while the technical standards require review and editing to comply with international standards of units, etc.

8. THE LATEST CONSTRUCTION TECHNOLOGY FOR IN-GROUND STORAGE TANKS

— Future technological developments —

Japan's first 10,000 kl in-ground tank was constructed over 30 years ago. Since then, construction technology has developed such that we can now build tanks with a capacity of 200,000 kl. It is through such advances that the 100 m slurry wall used to build the tanks at Sodegaura terminal became possible. Recently, the technology used for these large-scale tanks has been adopted in the construction of other large-scale civil structures, such as shafts and retaining walls for an underground substation.

Completely buried underground tanks with RC dome roofs have made a great contribution to realization of Ohgishima LNG terminal, which is in a densely populated section of Yokohama city. The newly developed rigid connection between side wall and bottom slab, as used at this terminal, makes use of the latest in civil engineering technology, including highly prestressed concrete. It is certain that such technology will see more widespread use in the future for its economic efficiency and ease of operation/maintenance.

Further developments in the technology of in-ground tanks are likely to bring about further economic and quality improvements. On the other hand, competition between energy sources is increasing as deregulation of the gas industry proceeds, so it is necessary to take a proactive stance towards such developments as a way to further reduce construction costs and construction periods.

From a design perspective, a rational approach rooted in performance-based design is becoming more necessary. In particular, seismic design for level-2 earthquake motion needs to take into consideration the material and geometric nonlinearities of the concrete tank body so as to make hoped-for reductions in the amount of rebars and concrete. Other areas needing attention are the possibility of reducing the amount of reinforcement by introducing high-strength bars of SD390 or other similar classes.

Concerning the actual construction of in-ground tanks, one area of potential technical development is rationalization of the design and construction of slurry walls; these temporary retaining and cut-off walls account for a large part of the total cost of in-ground tanks. Further, the possibility of reducing the slurry wall's thickness by using concrete of higher compressive strength, adopting a cut-off membrane instead of a slurry wall, and construction by means of a pneumatic caisson needs to be considered. Although many issues remain to be solved, such as more efficient excavation methods and automation, I believe that the caisson method is a very attractive option for in-ground tanks as it eliminates the need for a slurry wall. The establishment of these new construction methods is also important for the reductions in construction cost and construction period that they can bring.

9. CONCLUDING REMARKS

Demand for the so-called "clean energy", LNG, is expected to continue increasing as global environmental problems become more important. As a result, there will be growing demand for underground LNG tanks, for which the technology has been developing very rapidly. Storage capacity, reinforced concrete roofs, 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.

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