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VERIFICATION OF SEISMIC PERFOMANCE OF IN-GROUND LNG TANK STRUCTURES
-STUDY ON STREAMLINING BY DYNAMIC NONLINEAR ANALYSIS METHOD -

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This report demonstrates that the design safety margin employed when designing the reinforced concrete structure of an in-ground LNG tank can be reduced through the use of a more sophisticated analytical methodology, thus streamlining construction. In conventional design, a method of equivalent linear analysis is applied to determine the amount of reinforcement required to achieve a particular ultimate sectional strength, even for Level 2 earthquake motion. The more sophisticated approach is to apply dynamic nonlinear analysis, after determining the amount of reinforcement required to withstand Level 1 motion, and then ensuring that ductility adequate to withstand Level 2 motion. The application of this more sophisticated analytical method makes it possible to more accurately analyze the behavior of members, thus not only reducing the amount of reinforcement needed but also improving safety.

Key Words: In-ground LNG tank, dynamic nonlinear analysis, streamlining reinforced concrete structure

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1. INTRODUCTION

This report describes a method that can be used to verify the seismic performance of a three-dimensional in-ground concrete structure during an earthquake based on "Guidelines for Verification of Structural Performance of In-ground LNG (Liquefied Natural Gas) Talk Structures" (JSCE Committee of Civil Engineering for Energy Equipment, 1999) [1]. The wall of an in-ground LNG tank is adopted as the model.

In-ground LNG tanks are used to store LNG. They are designed such that the LNG is always at or below the level of the surrounding ground. The JSCE guidelines relate to in-ground LNG talks that have a reinforced concrete in-ground structure consisting of the main structure, walls and a bottom slab. A membrane (consisting of a thin metal film designed for low-temperature) fitted within the reinforced concrete structure retains the LNG.

The guidelines describe the basic concepts and methods to be used for determining and verifying structural performance during the performance-based design of such reinforced concrete structures. In setting a framework for performance-based design, the guidelines have two principle aims:

(1) To ensure that suitable levels of safety and serviceability are achieved in the design of the main structure of an in-ground LNG tank, thus ensuring that it can withstand external forces such as strong earthquake motion.
(2) To achieve more streamlined and efficient design for a tank structure specific to a particular location on the basis of the latest technologies and knowledge, and at the same time to promote technological development toward more streamlined design.

In accordance with the intent of the newly introduced performance-based design approach, the guidelines give basic concepts and methods for determining and verifying the structural performance of main structure. At the same time, they offer more than one option as regards the verification method.

Four I analytical methods are given in the guidelines for obtaining the response of a structure to earthquake motion, depending on the level of the earthquake motion and seismic performance to be checked (Table 1.1). The methods range from traditional approaches to futuristic and ideal techniques. A designer can select whichever is suitable. In this case study, a method of verification for Seismic Performance 3 in the case of Level 2 earthquake motion is described for use when Method 2 or 3 is adopted.

The results of verification for Methods 2 and 3 are compared, and the influences of different analysis methods and seismic performance levels on the results (specifically the reinforcement arranged) are evaluated. Finally, it is suggested that the use of a more sophisticated method can lead to better streamlining (Figure 1.1).

Analysis methods given in the guideline

Method 1: Quasi-dynamic linear analysis method
(response displacement method) [2]
Method 2: Quasi-dynamic equivalent linear analysis method
(response displacement method) [3]

Method 3: Dynamic nonlinear analysis method
(structure: member nonlinearity, soil: total stress) [4]
Method 4: Dynamic nonlinear analysis method
(structure: material nonlinearity, soil: effective stress) [5] [6]

Table 1.1 Dynamic analysis methods for LNG tanks

		Method 1	Method 2	Method 3	Method 4
Method		Quasi-dynamic linear analysis method	Quasi-dynamic equivalent linear analysis method	Dynamic nonlinear analysis method (structural characteristics: nonlinearity of members, ground characteristics: total stress)	Dynamic nonlinear analysis method (structural characteristics: nonlinearity of materials, ground characteristics: effective stress)
Determined input ground motion		Single level	Multiple levels		
Input value for analysis		Response displacement of ground (simultaneous distribution calculated from ground response analysis)		Time history of acceleration waves (input to design ground)	
Static or dynamic analysis		Response displacement method (static analysis)		Time history response analysis (dynamic analysis)	
Analysis model	Separate or coupled analysis	Separate analyses of the ground and structure		coupled analysis of the ground and structure	
	Ground property	Ground spring: Elasto-plastic model		Nonlinear history-dependent model (total stress) **	Nonlinear history-dependent model (effective stress)
	Structural characteristics	Linear Initial stiffness (Thermal stress: 1/2 E_0I_0)*	Linear Equivalent stiffness (stiffness decrease)	Nonlinear Member-leve1, history-dependent macro model Cracking and yielding of reinforcement are considered indirectly.	Nonlinear Constitutive rule that materials Cracking and yielding of reinforcement are considered directly.
Major response values to be analyzed		Sectional force		Curvature and strain of element, and sectional force	Indicator that directly represents the deformation (residual displacement)or damage for entire structure
Check items for the main structure		Sectional force (checking of allowable stress	Sectional force (checking of limit state)	Strain of element, and sectional force (transverse shear)	Overall stability of the entire structure

* Initial stiffness (E_0I_0) is reduced by half when thermal stress is taken into consideration.

** In this case study, linear model with final stiffness calculated by one dimentional equivalent linear analysis

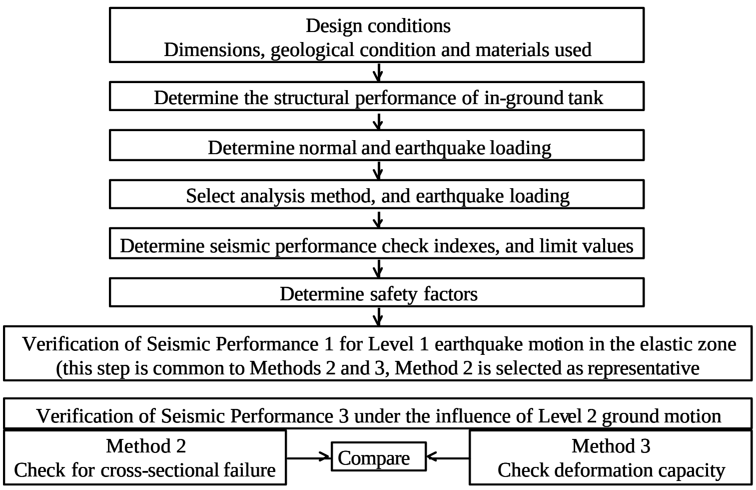


Figure 1.1 Review procedure in the case study

2. DESIGN CONDITIONS

2. 1 Dimensions

The major dimensions and structure of the in-ground LNG tank to be designed in the case study are shown in Table 2.1 and Figure 2.1, respectively. The main structure of the ta nk has an inner diameter of 69.9 m and a cylindrical shape. It is embedded in the ground. The tank consists of walls, a bottom slab and roof. Diaphragm walls are constructed outside the tank walls to allow for excavation work

Table 2.1 Major dimensions of in-ground tank.

Item	Dimension
Internal diameter of main structure	69.9m
Height of wall	37.8m
Thickness of component	Wall : 1.8m Bottom slab : 6.0 m Diaphragm wall : 1.1 m
LNG level	33.5m
Height of the fill	AP+14.0m
Embedded depth of diaphragm wall	AP-55.0m

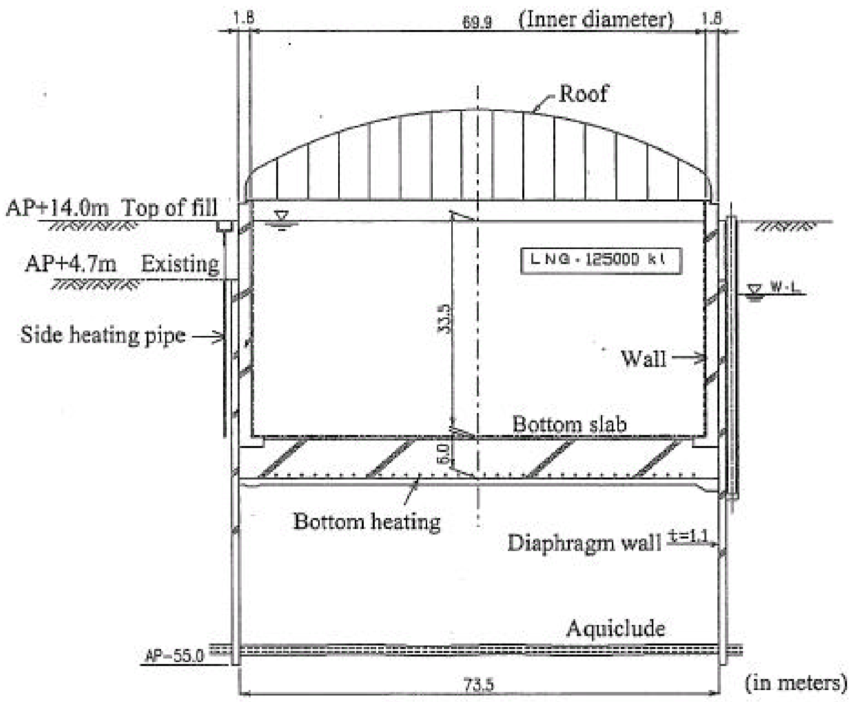


Figure 2.1 Structure of in-ground LNG tank

2.2 Geological conditions

The soil at the model point consists of fill at depths AP+14.0 m to AP+4.7 m, landf111 at AP+4.7 m t6 AP-6.1 m and diluvium at AP-6.1 and below. The fill around the tank has been consolidated enough to have an N-value of about 15. At the point, there exists a highly dense diluvial sandy layer uniformly, so the seismic design basement is set at AP-59.8 m.

2.3 Materials used

(1) Concrete
-Specifications: Ready mixed concrete satisfying JIS A 5308 with a nominal strength of 24 N/mm^2
-Characteristic compressive strength: fck-24 N/mm^2
-Young's modulus: E_c=25 kN/mm^2
(2) Reinforcement
-Specifications: SD345, steel bar satisfying JIS G 3112
-Characteristic yield strength under tension: f_{yk}=350 N/mm^2
(370 N/mm2: used for verification of seismic performance under the influence of Level 2H earthquake motion)
-Young's modulus: E_s=210 kN/mm^2

3. STRUCTURAL PEWORMANCE OF IN-GROUNI) LNG TANKS

In-ground LNG tanks are required to meet certain safety and serviceability requirements. These performance requirements are expressed in non-technical terms. However, the structural performance of an in-ground tank must be expressed in terms of engineering quantities as targets for design. Thus, normal and seismic performance levels are determined in terms of load-carrying capacity and watertightness. The service life an in-ground tank is set at 50 years.

3.1 Normal performance

The normal in-operation structural performance of the main structure of an in-ground tank must be defined such that load-carrying capacity and watertightness are adequate to meet normal safety and serviceability requirements. It must also be adequate to ensure that the tank remains usable for its service life without the need for major repairs.

3.2 Seismic performance

A major characteristic of earthquake activity is that strong earthquakes likely to have an effect on a tank are less likely to occur during its service life than less influential, weaker earthquakes. The rational approach, therefore, is to set seismic performance levels according to the probability of occurrence of different levels of earthquake motion such that the required level of safety attained. One aim of the guidelines is to clarify the required anti-seismic performance, including performance during the probable maximum earthquake, taking into account the importance of the tank, while at the same time streamlining the design process. The guidelines, therefore, set three different levels of earthquake motion and seismic performance. The required and target performance levels for the three levels of seismic performance are listed in Tables 3.1, 3.2, and 3.3.

3.2.1 Earthquake motion for verification

Three levels of earthquake motion are defined for verification purposes: levels 1, 2L, and 2H. Level 2 earthquake motion is divided into two sub-levels, 2L and 2H, according to probability of occurrence. This division into multiple levels is thought to offer a better guarantee of performance for important structures while leaving room for design streamlining.

Level 1 earthquake motion: earthquake motion of intensity likely to be encountered once or twice during the service life of the in-ground tank

Level 2L earthquake motion: strong earthquake motion with a relatively low probability of occurrence at the tank location during the service life of the tank

Level 2H earthquake motion: very strong earthquake motion with an extremely low probability occurrence at the tank location during the service life of the tank

3.2.2 Combination of earthquake motion and seismic performance

The performance required of an in-ground tank is divided into the three levels outlined below.

Seismic performance level 1: structural performance during and immediately after an earthquake is such that the tank remains safe, loss in serviceability is not substantial, and the tank remains usable without major repair.

Seismic performance level 2: earthquake-induced loss in structural performance is not so great as to jeopardize tank safety. The tank remains usable without major repairs.

Seismic performance level 3: the main structure remains intact, and LNG storage is protected. With repairs, the tank can continue in use.

Seismic performance levels 1, 2, and 3 are combined with earthquake motion levels 1, 2L, and 2H, respectively (see Table 3.4).

Table 3.1 Performance of in-ground level tank during an earthquake

Variation of performance			Performance level		
			Seismic performance 1	Seismic performance 2	Seismic performance 3
Required performance	Focus of owners or managers	Safety	-No damage to life safety and safety, of property inside or outside the base -No danger restraining daily activities inside or outside the base	-No damage to life safe and safety of property inside or outside the base -No danger restraining daily activities inside or outside the bas	-No damage to life safety outside the base -No serious effect on life safety inside the base (protection of life) -No direct danger restraining daily activities outside the base for a long time
		Serviceability (effect on the function of LNG base)	-No damage to power generation or gas production -No damage to acceptance, storage and supply of liquid -Small deterioration in durability	-No serious danger to power generation and gas production (recoverable in short time) -Normal acceptance and supply can be resumed within short time of repair. No reinforcement is required. -Continuous storage is possible. -Increase of vaporized gas is below the allowable level. -No substantial deterioration in durability	-Inconvenience to power generation and gas production can be removed. -The facilities can be re-used after reinforcement. -Storage is possible in the meantime

Table 3.2 Load-carrying of in-ground tank during an earthquake
(target performance to satisfy the required performance shown in Table 3.1)

Variation of performance			Performance level		
			Seismic performance 1 [Sound]	Seismic performance 2 [Maintenance of functions]	Seismic performance 3 [No failure (no liquid diffusion)]
Target performance	Structural performance of in-ground tank	Load-carrying capacity (deformation capacity)	-Displacement or deformation of the main structure during or after an earthquake*1 is small enough to meet the following conditions. -Change in storage capacity (volume inside the main structure) is minute enough to be practically ignored. -Neither liquid-tightness nor airtightness decreases for the membrane and roof. -No decrease in strength of the main structure after an earthquake	-Displacement or deformation of the main structure after an earthquake*1 is at the level where the following conditions are met. -Change in storage capacity (volume inside the main structure) is below an allowable level. -Decrease in the liquid-tightness and airtightness of the membrane and in the airtightness of the roof are below an allowable level (where continuous use is possible). -No decrease in strength of the main structure after an earthquake	Displacement or deformation of the main structure after an earthquake*1 is at the level where the following conditions are met. -Change in storage capacity (volume inside the main structure) is below a level where LNG flows out. -Neither displacement nor deformation of the main structure progress under post-earthquake loading. -No substantial decrease in the liquid-tightness of the membrane (safe facilities on the premises ensure safety) -No substantial decrease in the airtightness of the roof (safe facilities on the premises ensure safety)

*1 Displacement or deformation of the main structure includes relative displacement between the side wall and the bottom slab, and the deformation of the circular crest of the side wall (radial deformation).

Table 3.3 Watertightness performance of in-ground tank during an earthquake
(target performance to satisfy the required performance shown in Table 3. 1)

Variation of performance			Performance level		
			Seismic performance 1 [Sound]	Seismic performance 2 [Maintenance of functions]	Seismic performance 3 [No failure (no liquid diffusion)]
Target performance	Structural performance of in-ground tank	Watertightness	-Post-earthquake inflow of surrounding groundwater into the main structure*1 is small enough to meet the following conditions. -No insulating capacity is deteriorated for the cold insulation layer contacting the inner surface of the main structure under conditions of appropriate control of water levels inside and outside the main structure, operation of the groundwater management system to maintain the water levels, and appropriate management of freezing temperature level.	Post-earthquake inflow of surrounding groundwater into the main structure*1 is at the level where the following conditions are met. -No insulating capacity is deteriorated for the cold insulation layer contacting the inner surface of the main structure under conditions of appropriate control of water levels inside and outside the main structure, operation of the groundwater management system to maintain the water levels, and appropriate management of freezing temperature level.	Post-earthquake inflow of surrounding groundwater into the main structure*1 is at the level where the following conditions are met. -No great water pressure acts that causes large deformation of the membrane under conditions of appropriate control of water levels inside and outside the main structure, operation of the groundwater management system to maintain the water levels, and appropriate management of freezing temperature level.

*1 The groundwater inflow into the main structure during an earthquake is allowed because it is small and has little effect on the insulation of the cold insulation layer since earthquakes act only for a limited time.

Table 3.4 Combinations of earthquake motion and seismic Performance level

Performance level / Earthquake motion	Seismic Performance level of tank		
	Seismic performance 1	Seismic Performance 2	Seismic performance 3
Level 1 earthquake motion	○		
Level 2L earthquake motion		○	
Level 2H earthquake motion			○

4. LOADINGS

In-ground tank structures are underground structures that are influenced by the very low temperature of LNG (162 ^oC). Loadings should be determined in view of these characteristics and the tank-specific conditions. The loadings to be used for verification purposes are determined separately for normal performance and for different levels of seismic performance.

4.1 Normal loading

The loads to be taken into account when verifying the normal performance of the walls of an in-ground tank are the weight of the walls themselves, earth pressure, internal gas pressure, liquid pressure, thermal loading, and roof load.

4.2 Determination of earthquake motion

Each earthquake motion to be used for verification is defined at the seismic basement and in terms of an acceleration response spectrum. A time-history waveform is set up so as to match the spectrum thus defined.

Level 1 earthquake motion is defined using a stochastic procedure, while Level 2 earthquake motion is defined using a deterministic procedure.

In this study, the service life is set at 50 years. For Level 1 earthquake motion, the acceleration at the seismic basement (2E) is defined as 230 gal based on the seismic hazard curve for the location, as shown ill Table4.1. The Level 2 earthquake motion is set using a simulated earthquake at 390 gal for Level 2L earthquake motion and at 620 gal for Level 2H (Table 4.2). The 620 gal motion is the value of the mean plus the standard deviation in view of the uncertainty involved in estimating the motion of the simulated earthquake. It is 1.6 times the Level 2L motion.

The simulated seismic waveform (artificial wave form) is designed to match the predetermined target response spectrum. For Level 1 earthquake motion, the simulated and observed waveforms are compared. Then, since the observed waveform is more severe on the structure than the simulated one here, the observed waveform is defined as the input earthquake motion.

The earthquake response in the nonlinear range should be taken into account. In this range, differences in waveform time history may have an impact on the nonlinear response even for an identical target spectrum. The guidelines recommend the use of multiple waveforms with different phase characteristics. In this case study, however, a seismic waveform with a certain phase characteristics is used for each level of earthquake motion.

Level 1 earthquake motion: Sodegaura waveform of the 1987 Chibaken-toho-oki earthquake, M6.7 (observed waveform)

Level 2L and 2H earthquake motions: La Union waveform of the 1985 Mexico earthquake, M8.I (simulated waveform)

An earthquake generated by an active fault at the model location is estimated to yield a maximum acceleration of about 280 gal at the seismic basement, so this is included, within the Level 2 earthquake motion described above. However, this is not specifically discussed as the design earthquake motion.

The time-history waveforms and response spectra of he input earthquake motions are shown in Figures 4.1 and 4.2, respectively.

4.3 Effects of earthquake

The earthquake-induced loads induced by earthquakes listed in Table 4.3 are considered in this study. In the analysis by Method 3, separate analysis is carried out for normal loading and the sectional force acting on the main structure is calculated. Analysis at the time of an earthquake then treats this normal sectional force as the initial state.

Table 4.1 Definition of Level 1 earthquake motion (by stochastic procedure)

Earthquake motion	Level 1 earthquake motion
Probability of occurrence	Expected once or twice during the service life of the structure
Maximum acceleration at seismic basement	230gal
Return period	About 70 years
Probability of occurring once during the service Life P (%) (reference value)*	About 50%

*The values in the table are for reference based on the assumption of a service life of 50 years. The following relation exists between recurrence interval T and probability of occurring once during the service life P (%): P/100-1-(1-1/T)t where, t is the service life (in years)

Table 4.2 Definition of Level 2 earthquake motion (by deterministic procedure)

Earthquake motion		Level 2L earthquake motion*	Level 2H earthquake motion**
Probability of occurrence		Strong earthquake motion with a relatively small probability of occurrence during the service life of the structure	Very strong earthquake motion with an extremely small probability of occurrence during the service life of the structure
Typical earthquake		Minami-Kanto earthquake	Minami-Kanto earthquake
Type of earthquake		Large earthquake along the Sagami trough (reoccurrence of earthquake with motion equivalent to that of the Great Kanto Earthquake)	Large earthquake along the Sagami trough
Magnitude		M8	M8
Shortest distance to the fault	Horizontal	8km	8km
	Vertica1	17km	17km
Maximum acceleration at seismic basement 2E		390gal	620gal
Return period T		About 300 years	About 1,000 years
Probability of occurring once During the service life P(%) (reference value)***		About 15%	About 5%

*Level 2L earthquake motion: Level of earthquake motion generally expected at the site with occurrence of an M8-class earthquake

** Level 2H earthquake motion: motion expected at the site with occurrence of an M8-class earthquake Upper limit is mean value plus 1 *s*

*** Value calculated by the same method as in Table 4.1

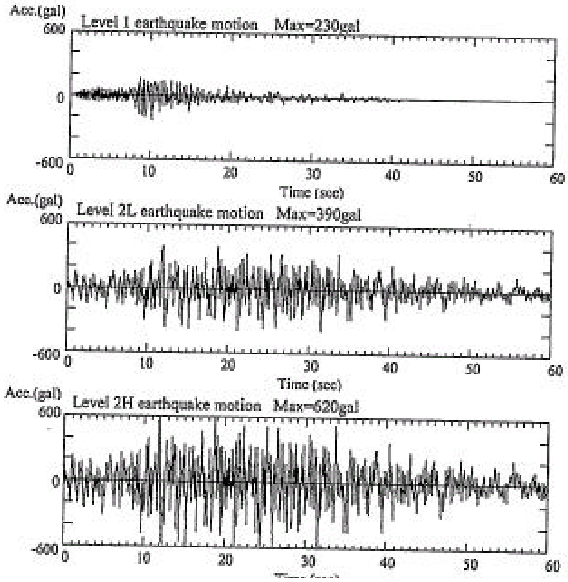


Figure 4.1 Time history waveforms of input ground motions

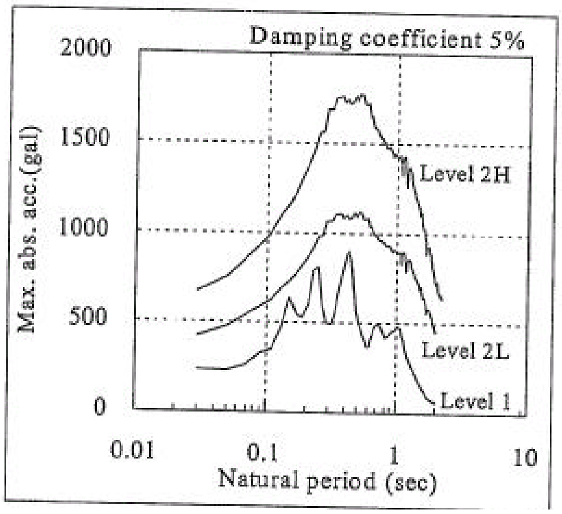


Figure 4.2 Acceleration response spectrum

Table. 4.3 Effects of earthquake

Method / Load	Method 2 (Quasi-dynamic equivalent linear analysis method)	Method 3 (Dynamic nonlinear analysis method)
Loads applied by dynamic interaction between in-ground tank and soil	The maximum relative displacement between superstructure and substructure is considered. physical properties of soil defined according to the final stiffness.	With coupled analysis of soil and structure, the load acts through the interaction of the tank with the soil when the earthquake motion is input.
Inertial force induced by volume of the in-ground tank and roof load	The inertial force of the main structure is calculated using the seismic intensity at a depth equivalent to half the embedded depth of the main structure. The roof load is calculated using the seismic intensity obtained by multiplying the horizontal seismic intensity of the main structure by a correction factor corresponding to the response characteristics of the roof.	Since the weight of the main structure of the in-ground tank is defined as its own weight, the inertial force of the main structure is applied when the earthquake motion is applied.
Load applied by LNG stored in the tank	Dynamic liquid pressure is calculated using the seismic intensity at a depth equivalent to half the embedded depth of the main structure	Since the model includes LNC as a component, dynamic liquid pressure is applied when earthquake motion is input.

5. ANALYSIS METHODS FOR VERIFICATION OF SEISMIC PERFORMANCE

In order to streamline the earthquake-resistant design of an in-ground tank, it is necessary to strictly evaluate the deformation capacity of the tank's main structure. If the actual behavior of the structure can be accurately simulated, the deformation capacity and stress state of different-parts of the structure can also be determined. As a result, a stricter verification can be carried out with the real limit state of the structure taken into consideration. Generally, for a three-dimensional structure such an in-ground tank, stricter analysis allows for more streamlined design than simple analysis. However, in cases where a sufficient amount of relevant data is available and well-proven performance is involved, a simple method may be adequate to meet the needs of analysis.

Two of the methods given in the guidelines are investigated in this study of seismic performance verification. The quasi-dynamic equivalent linear analysis method (Method 2) has a history of use in design and has proved effective in streamlining. The dynamic nonlinear analysis method (Method 3) is expected to contribute to greater streamlining in the future through its greater sophistication.

5.1 Quasi-dynamic equivalent linear analysis (Method 2)

5.1.1 Analysis of earthquake response of soil

To calculate the relative displacement of the soil in the application of Method 2, an earthquake response analysis is carried out for soil. Table 5.1 shows the results of total stress analysis (equivalent linear analysis) under the influence of both Level 1 and Level 2 earthquake motions. For values of relative displacement used in analysis by Method 2 are those at the ground surface and at the base of the diaphragm wall, because in the model the diaphragm wall is integrated with the walls of the tank

.

Table 5.1 Results of soil response analysis

Earthquake motion	Level 1 Earthquake motion	Level 2L Earthquake motion	Level 2H Earthquake motion
Maximum acceleration at ground surface	319gal	523 gal	696gal
Maximum displacement at ground surface	5.9cm	16.2cm	31.7cm
Maximum relative displacement between superstructure and substructure*	5.8cm	15.8cm	31.3cm

* Relative displacement between ground surface and base of diaphragm wall

5.1.2 Structural analysis model

(1) Response displacement analysis model

Figure 5.1 shows the analysis model used. The model represents the tank walls and diaphragm walls as shells, while the surrounding ground, bearings, and interface between wall and diaphragm wall are treated as springs.

(2) Determination of soil spring value

The soil spring used in analysis by the response displacement method is modeled as an elastoplastic soil spring (Figure 5.2). The initial gradient of this soil spring is obtained using a finite element method. Specifically, a unit load is applied to a model of the soil containing no structure in the direction of the desired soil spring value. The resulting load-displacement relationship is then used to calculate the soil reaction force. The soil stiffness used in the analysis is the final value obtained by soil equivalent linear earthquake response analysis. The upper and lower limits of the soil spring reaction are chosen such that a uniform value is reached when the spring reaction reaches the active or passive earth pressure.

(3) Equivalent stiffness of structural members

The values of equivalent stiffness for the tank walls as used in Method 2 (quasi-dynamic equivalent linear analysis method) are shown in Table 5.2. The equivalent stiffness of the main structure is taken to be the residual stiffness at a point where some of the members suffer yielding of the reinforcement; this is obtained by iterative computations of sectional force and residual stiffness using a formula that reflect the effects of cracking.

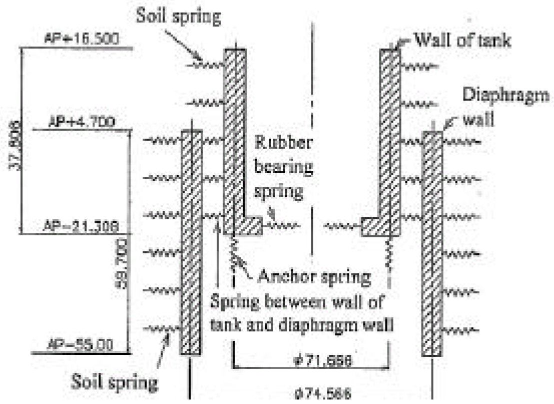


Figure 5. 1 Response displacement analysis model (Method 2)

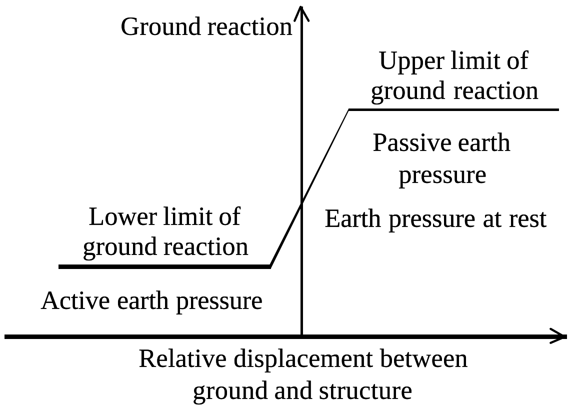


Figure 5.2 Mechanical properties of ground

Table 5.2 Equivalent stiffness of in-ground tank walls

	Circumferential	Vertical
Level 1 earthquake motion	2/3	Semicircle on the loading side 2/3 Semicircle on the unloading side 1
Level 2L earthquake motion	1/2	Semicircle on the loading side 1/2 Semicircle on the unloading side 1
Level 2H earthquake motion	1/3 Bottom end 1/5	Semicircle on the loading side 1/3 Semicircle on the unloading side 1

Note: Figures in the table indicate the ratio with respect to stiffness effective for the full face
Bending, axial, and shear stiffnesses are treated as uniform.

5.2 Dynamic nonlinear analysis method (Method 3)

5.2.1 Dynamic coupled analysis model

The analysis model is shown in Figure 5.3. This is a coupled model of the soil and structure. The wall consists of three-dimensional shell elements, while the soil and bottom slab consist of solid elements. The bearing, and interface between wall and diaphragm wall are modeled as springs. The side and bottom boundaries of the analysis model are assumed to be viscous boundaries.

5.2.2 Model of soil mechanical properties

The physical properties of the soil used for dynamic analysis are assumed to be linear and equivalent to the ultimate physical properties of the soil based on earthquake response analysis of the soil. The values used in the analysis are listed in Table 5.3.

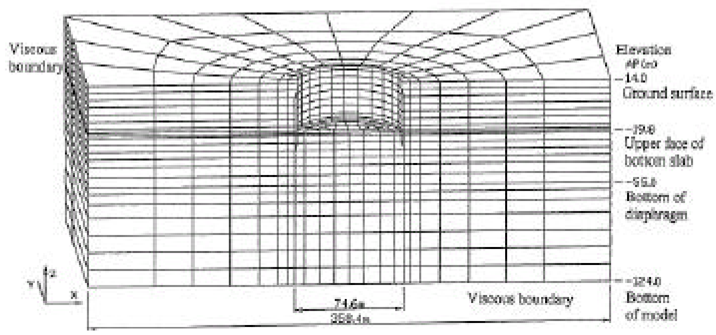


Figure 5.3 Analysis model and boundary conditions

5.2.3 Model of structural member mechanical properties

(1) Outline of mechanical properties model

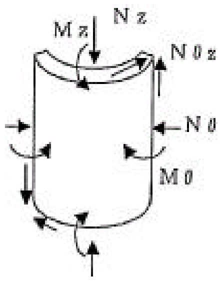
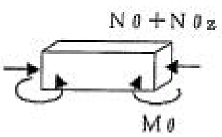
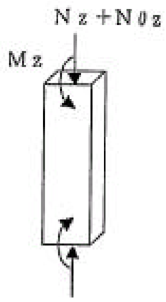
The in-ground LNG tank is subjected to three-dimensional analysis using general-purpose dynamic nonlinear analysis codes. In analyzing the wall and diaphragm wall of the in-ground tank, which are modeled with shell elements, the stiffness is determined with a subroutine.

The nonlinear property model of each member used to calculate stiffness consists of a history-dependent macro model reflecting reductions in stiffness due to flexura1 and axial forces in two directions, as well as the in-plane shear force (Table 5.4). The nonlinearity of the flexural and axial forces is assumed to be different in the vertical and circumferential directions. Thus an orthotropic model is assumed.

Table 5.3 Physical properties of soil used in dynamic nonlinear analysis

Depth AP m	Layer Thickness m	Layer type	Unit weight kN/m^3	Final stiffness $\times 10^3$kN/m^2	Damping Coefficient %	Poisson's ratio
14.00 - 9.35	4.65	Fill	18.0	48.4	6.7	0.45
9.35 - 4.70	4.65		18.0	24.1	17.6	0.45
4.70 - -0.20	4.90	Fill and landfill	19.0	15.2	22.6	0.48
-0.20 - -5.10	4,90	Landfill	18.0	9.5	20.5	0.49
-5.10 - -10.00	4.90	Diluvium	18.2	42.6	12.4	0.48
-10.00 - -14.90	4.90		17.0	37.6	10.9	0.48
-14.90 - -17.90	3.00		18.0	135.6	4.8	0.47
-17.90 - -19.80	1.90		18.0	135.6	4.9	0.48
-19.80 - -21.30	1.50		18.0	134.4	5.1	0.48
-21.30 - -26.10	4.80		18.0	131.6	5.4	0.48
-26.10 - -31.10	5.00		18.0	127.6	6.1	0.48
-31.10 - -36.10	5.00		18.0	124.8	6.5	0.48
-36.10 - -41.10	5.00		17.5	120.1	7.1	0.47
-41.10 - -48.05	6.95		17.5	145.2	8.8	0.47
-48.05 - -55.00	6.95		17.2	208.5	7.1	0.46
-55.00 - -59.80	4.80		17.0	234.8	6.5	0.46
-59.80 - -124.00	64.20		18.5	333.0	2.0	0.47

Table 5.4 Concept of macro models of members

Model for structural analysis	Macro model for calculating stiffness (subroutine)		
	Circumferential Flexural and axial forces	Vertical flexural and axial forces	In-plane shear
Shell 			In-plane shear stiffness is determined based on the reinforcement strain calculated using a model of flexural and axial forces (Aoyagi's formula)
	In-plane shear force is replaced by axial tensile force		

(2) Modeling of nonlinearity

The nonlinearity of members is modeled as a secant stiffness according to the presence of cracking, reinforcement yield, and member stress, as shown below. The stiffness used for structural analysis is tangential stiffness, so tangential stiffness is calculated by the subroutine based on the sectional force-strain relationship.

(i) Equations for calculating stiffness where no cracking occurs in concrete

Bending stiffness : $EI_{eff} = E \cdot I_g$ (5.1)

Axial stiffness : $EA_{eff} = E \cdot A_g$ (5.2)

In-plane shear stiffness: $E_v = E \cdot A_g / [2(1+n)]$ (5.3)

(ii) Equations for calculating stiffness where flexural cracking occurs in concrete

The stress and strain in the reinforcement, as required to calculate stiffness, are obtained on the assumption that the reinforced concrete is behaving correctly based on the sectional force obtained by analysis.

Bending stiffness: $EI_{eff} = E \cdot [(s_{scr} / s_s)^4 \cdot I_g + \{1-(s_{scr} / s_s)^4\} \cdot I_{cr}]$ (5.4)

(obtained by extending Branson's formula to the case where flexural and axial forces are applied)

Axial stiffness: $EA_{eff} = X_{eff} \cdot E \cdot A_g / h$ (5.5)

In-plane shear stiffness: $E_v = K \cdot A_g / e_{fm}$ (Aoyagi's formula) (5.6)

$$e_{fm} = e_{smq} + e_{smz}$$

(iii) Equations for calculating stiffness where cracking occurs throughout the cross section of concrete (full-face tension)

The strain in the reinforcement, as used to calculate stiffness, is obtained on the assumption that the reinforced concrete is behaving correctly based on the sectional force obtained by analysis.

Bending stiffness : $EI_{eff} = M / f = M \cdot L / (e_{sm1} - e_{sm2})$ (5.7)

Axial stiffness : $EA_{eff} = N / e_{sm}$ (5.8)

$$e_{sm} = (e_{sm1} + e_{sm2}) / 2$$

In-plane shear stiffness: $E_v = K \cdot A_g / e_{fm}$ (Aoyagi's formula) (5.9)

(iv) Out-of-plane shear stiffness

For the out-of-plane shear stiffness, an equivalent stiffness based on an assumption of proportionality to axial stiffness is used,

where

E : Young's modulus of concrete

I_{eff} : Effective moment of inertia

A_{eff} : Effective sectional area

I_g : Gross section equivalent moment of inertia

A_g : Gross section equivalent sectional area

n : Poisson's ratio

I_{cr} : Moment of inertia with concrete in tension being ignored

s_s : Reinforcement stress

s_{scr} : Reinforcement stress at the time of (immediately after) cracking

X_{eff} : Equivalent height of neutral axis corresponding to I_{eff}

h : Height of member

K : Constant (360 t/m^2)

f : Curvature
e_{sm1}: Mean strain in outer reinforcement
e_{sm2} : Mean strain in inner reinforcement
e_{fm} : Mean strain perpendicular to crack direction
e_{smq} : Mean strain in circumferential reinforcement
e_{smz} : Mean strain in vertical reinforcement
L: Distance between outer and inner 1-einforcing bars
M: Bending moment
N: Axial force (including in-plane shear force)

(3) Hysteresis behavior of mechanical property model
The mechanical property model of a member basically exhibits the hysteresis behavior described below.

(i) Before cracking
-The sectional force depends on the initial stiffness.

(ii) After cracking occurs
-The hysteresis point moves to the points determined by the equations for stiffness during loading. For the flexural and axial forces, the hysteresis point moves to, the points determined by the equations for stiffness according to whether the hysteresis point is in a state of gross sectional compression, the flexural cracking occurs in concrete, or the hysteresis point is in the gross section tension.
-Bending stiffness is assumed to be asymmetrical depending on whether the curvature is positive or negative, in view of the difference in the amount of reinforcement on the inside and outside of the cross section.
-An origin-oriented model is assumed where the hysteresis point moves toward the origin during unloading.

(iii) After reinforcement yield
-The hysteresis point follows a path of a tangential stiffness equivalent to 1% of the gross section stiffness (initial stiffness). The reinforcement yield point is defined as a point where any of the reinforcing bars first yields.

Figure 5.4 shows a hysteresis loop representing the relationship between the in-plane shear force and shear strain.

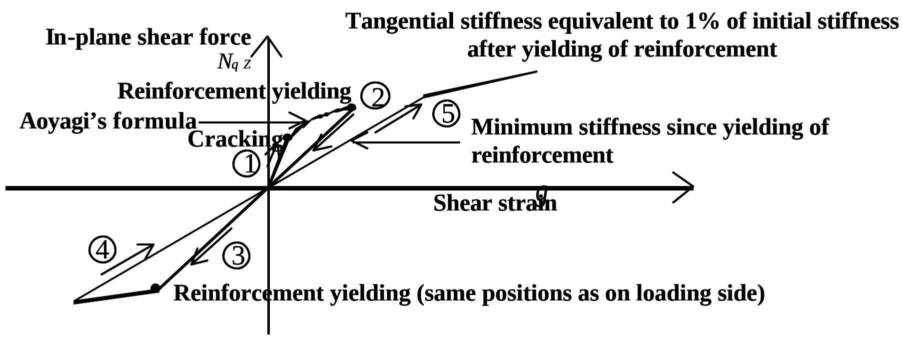


Figure 5.4 Hysteresis loop of in-plane shear and shear strain

6. CHTECK INDEXES AND LIMIT VALUES FOR SEISMIC PERFORMANCE

To verify the performance of an in-ground tank, it is necessary to define suitable check indexes and also limit values of the check indexes for judgment of whether target performance can be achieved. The task is then to ascertain that the response of the structure to the design loads does not reach the chosen limit values.

The check indexes used to judge the seismic performance of the main tank structure are defined according to individual analysis methods used for the verification, since different response values are calculated for different methods. Tables 6.1, 6.2, and 6.3 list the limit values [6] of indexes used for verification of the structural performance of the main tank structure during an earthquake.

In verifying structural performance during an earthquake, limit values are established for verification of load-carrying capacity so as to ensure that the in-ground tank is able to retain the desired level of performance after the earthquake. To identify load-carrying performance, the strength and deformation capacity of the wall, the relative displacement between the wall and the bottom slab, and the deformation of the uppermost part of the wall are checked. There is no verification of watertightness because the groundwater level around the in-ground tank is to be lowered.

Method 3 is unable to directly confirm seismic performance Level 3 by calculation, which requires that the main structure never suffers failure in an earthquake. In this case study, therefore, the load-carrying capacity of the main structure is checked at the element level on the premise that structures such as in-ground tanks never fail as long as they pass element-level verification.

For limit values, the deformation that corresponds to the maximum strength is defined for seismic performance Level 2. For seismic performance Level 3, deformation similar to that for seismic performance Level 2 is defined as the limit value to ensure design on the safe side. Different safety factors are, however, defined for seismic performance Levels 2 and 3.

6.1 Check indexes and limit values for seismic performance where Method 2 is used

6.1.1 Check indexes for seismic performance level 1 against Level 1 earthquake motion

(1) Verification of load-carrying capacity

1) Verification of strength of main structure

a. Verification of cross-sectional strength against frexura1, axial, and in-plane shear forces

To simplify the verification, the in-plane shear force is converted into an axial force and then the check is carried out for flexural and axia l forces. The design bending moment M_d is assumed to be equal to or less than the design flexural strength due to yielding M_{yd}.

$$g_i \cdot M_d / M_{yd} \leq 1.0 \qquad (6.1)$$

b. Verification of cross-sectional strength against out-of-plane shear force

The design shear force is assumed to be equal to or less than the design shear force of the member V_{yd}.

$$\gamma_i \cdot V_d / V_{yd} \leq 1.0 \qquad (6.2)$$

2) Relative displacement of the tank wall and bottom slab and deformation of the uppermost part of the wall

The membrane and roof fully meet the target performance for liquid-tightness and airtightness. Verification is omitted because seismic performance Level 2 is checked.

(2) Verification of watertightness

The main structure of the in-ground tank fully meets the target performance for watertightness because the main structure of the in-ground LNG tank remains within the elastic range. Verification is therefore omitted.

Table 6.1.Limit values for verification of load-carrying performance (Seismic performance Level 1)

Target performance	Seismic performance Level 1 (tank is sound)		
	Main structure maintains its strength after earthquake	Liquid storage capacity altered but remains within allowable range	Deterioration in liquid-tightness and airtightness of the membrane and in the airtightness of the roof is less than the allowable level
Method 2	-Sectional forces during an earthquake less than load-carrying capacity -Yield sectional strength against bending and axial forces -Out-of-plane shear strength -In-plane shear strength	-Verification omitted because the tank remains in the elastic range	-Since the limit values for the liquid-tightness and airtightness of the membrane and roof for seismic performance Level 1 are set so that the main structure of the tank will remain in the elastic range, the target performance is easily achieved. This check may be omitted if a check for seismic performance Level 2 is carried out

6.1.2 Check indexes and limit values for seismic performance levels 2 and 3 against level 2 earthquake motions

(1) Verification of load-carrying capacity

1) Verification of strength of main structure

The verification methods for flexural and axial forces, in-plane shear force, and out-of-plane shear force are similar to those for seismic performance Level 1, so the explanation of verification is omitted.

2) Verification of relative displacement between tank wall and bottom slab

The relative displacement at the point where the tank wall meets the bottom slab is assumed to be less than the allowable displacement of the membrane as determined from the effects of repetitive loading and unloading during overhaul inspections or while the tank is filled and emptied.

$$\gamma_i \cdot \delta_{rbd} / \delta_{bd} \leq 1.0 \qquad (6.3)$$

where

δ_{rbd} : Relative displacement at point where tank wall meets bottom slab

δ_{bd} : δ_b / γ_b

δ_b : Allowable displacement; 45 mm (in the radial direction)

Allowable displacement where the slab (equipment for absorbing relative displacement) is installed between the wall of the tank and the bottom slab, which is set at 45 mm

g_b : Member factor

3) Verification of deformation at the uppermost part of the wall

The rim deformation (oval deformation) of the uppermost part of the wall is assumed to be less than the limit value determined by the deformation capacity of the roof.

$$g_i \cdot d_{rtd} / d_{td} \leq 1.0 \qquad (6.4)$$

where

d_{rtd} : Radial relative displacement at uppermost part of the wall in the range from 0^o to 180^o

d_{td} : d_t / g_b

d_t : Limit value determined by deformation capacity of the roof; 8.7 cm

In this case study, the limit value is set at 1/800 of the roof diameter (so the stress of the steel roof remains below its yield strength)

g_b : Member factor

(2) Verification of watertightness

Verification is omitted because the main structure of the in-ground tank is not required to be watertight. The groundwater level around the tank is assumed to have been lowered so as to avoid problems with watertightness.

Table 6.2 Limit values for verification of load-carrying Performance (Seismic performance Level 2)

	Seismic performance Level 2 (tank remains functional)		
Target performance	Post earthquake loading causes no progressive displacement or deformation of the main structure	Change in liquid storage capacity is less than the level causing LNG outflow	No substantial deterioration occurs in the liquid-tightness of the membrane and in the airtightness of the roof
Method 2	-Sectional forces during earthquake are less than load-carrying capacity -Ultimate sectional strength against bending and axial forces -Out-of-plane shear strength -In-Plane shear strength	-The allowable deformation of the roof is small enough to allow verification to be omitted	-Relative displacement at the interface between wall and bottom slab less than allowable deformation (45mm: radial direction) determined by the fatigue limit)* -Oval deformation of the wall rim less than allowable deformation (1/800 of roof diameter)
Method 3	-Compressive strain of element less than ultimate compressive strain of concrete (e'_{cu} :3500m) -Tensile strain of element less than allowable strain (14000 m) determined by the fatigue limit of membrane -Out of-plane shear force during earthquake less than load-carrying capacity		

* Allowable deformation is the limit value for a case in which the slab (equipment for absorbing relative displacement) is used. In this case study, the relative displacement is assured to be no more than 45mm.

Table 6.3 Limit values for verification of the load-carrying performance (Seismic Performance Level 3)

	Seismic Performance Level 3 tank never collapse (no liquid leaks)]		
Target performance	The main structure maintains its strength after earthquake	Change in liquid storage capacity is within the allowable range and	Deterioration in liquid-tightness and airtightness of the membrane and in the airtightness of the roof is less than the allowable level
Method 2	-Sectional forces during earthquake less than load-carrying capacity -Ultimate sectional strength against bending and axial forces -Out-of-plane shear strength -In-Plane shear strength	-The allowable deformation of the roof is small enough to allow verification to be omitted	-Relative displacement at the interface between wall and bottom slab less than allowable deformation (45mm: radial direction) determined by the fatigue limit)* -Oval deformation of the wall rim less than allowable deformation (1/800 of roof diameter)
Method 3	-Compressive strain of element less than ultimate compressive strain of concrete (e'_{cu}: 3500m) -Tensile strain of element less than allowable strain (14000 m) determined by the fatigue limit of membrane -Out of-plane shear force during earthquake less than load-carrying capacity		

*Allowable deformation is the limit value for a case in which the slab (equipment for absorbing relative displacement) is used. In this case study, the relative displacement is assured to be no more than 45mm.

6.2 Check indexes and limit values for seismic performance where Method 3 is used

6.2.1 Check indexes and limit values for seismic performance level 1 against level 1 earthquake motion

Verification is omitted because the verification for seismic performance level 1 against Level 1 earthquake motion using Method 2 can be used as an alternative.

6.2.2 Check indexes and limit values for seismic performance level 2 and 3 1 against Level 2 earthquake motions

(1) Verification of load-carrying capacity

1) Verification of deformation capacity of main structure of in-ground tank

a. Verification of compressive strain of concrete

It is confirmed whether or not the compressive principal strain of an element is below the limit value of compressive strain of concrete.

$$gi \cdot e'_{rd} / e'_{cd} \leq 1.0 \qquad (6.5)$$

where

e'_{rd} : Compressive principal strain of element

e'_{cd} : e'_{cu} / g_b

e'_{cu} : Limit value of compressive strain of concrete (Ultimate compressive strain: 3500 m)

g_b : Member factor

b. Verification of tensile strain of element

It is confirmed whether or not the tensile principal strain of an element is below the allowable strain determined by the fatigue limit.

$$gi \cdot e'_{rd} / e'_{td} \leq 1.0 \qquad (6.6)$$

where

e'_{rd} : Tensile principal strain of element

e'_{td} : e'_{t} / g_{b}

e'_{t} : Limit value of tensile axial strain of element: 14,000 m

Allowable strain confirmed by membrane deformation test by repetitive loading and unloading: set at 14,000 m in this case study

g_{b} : Member factor

2) Verification cross-sectional strength against out-of-plane shear force

The design shear force V_d is set equal to or lower than the design shear force of the member V_{yd} .

$$gi \cdot V_d / V_{yd} \leq 1.0 \qquad (6.7)$$

3) Verification of relative displacement between tank wall and bottom slab, and deformation of uppermost part of the wall

Verification is omitted because it is similar to the verification by Method 2 for seismic performance.

(2) Verification of watertightness

Verification is omitted because the main structure of the in-ground tank is not required to be watertight, as already mentioned.

7. SAFETY FACTORS

The safety factors that are used in normal and seismic performance checks are the following: material factor, member factor, load factor, structural analysis factor, and structure factor. These safety factors must be determined in view of such variability as undesirable changes in the characteristic values of materials used and in expected loads, uncertainties associated with structural analysis, calculation, or the determination of limit values, and the importance of the tank concerned.

7.1 Safety factors for Method 2

Table 7.1 lists the safety factors adopted for verification of the strength of the tank wall when using Method 2. The factors were established based on "A study on rationalization of design of reinforced concrete in-ground LNG tanks"[3] as described below. The safety factors used for verification of the relative displacement between the tank wall and the bottom slab, and the deformation of the uppermost part of the wall are described in the section related to safety factors for Method 3.

(i) Characteristic values of materials and material factors

The characteristic value of the reinforcement is set at 370 N/nm^2, 5% higher than the standard value of 350 N/mm^2, for an earthquake of Level 2H motion. This is a level of motion rarely encountered. The value is based on statistical analysis of materials data obtained in tests on similar construction work where quality control was excellent. For earthquakes other than Level 2H motion, existing standard values as given in the Standard Specifications for Reinforced Concrete are used without modification to ensure safety because there is no guarantee of the characteristic values obtained in tests being reproduced for concrete strength. For the material factors, the standard values given in the Standard Specifications are used.

(ii) Member factors

The structural members of an in-ground tank are 2 to 6 m thick, so the effects of dimensional errors on cross-sectional strength can be safely ignored (though for some members, the effect ranges from 5% to 10%). In view of this, some of the member factors are set below the standard values given in the Standard Specifications for Reinforced Concrete. The degree of reduction varies according to the performance level that is to be guaranteed by the limit state.

For shear strength against Level 2 earthquake motion, a value larger than the flexural and axial forces is set according to the Standard Specifications for Reinforced Concrete to increase the ductility of the member.

(iii) Load factors

All seismic load factors are set at 1.0 because uncertainty is already taken into account when determining the characteristic values of earthquake load and because the probability of different loads occurring at the same time is low.

Table 7.1 Safety factors for Method 2 (cross-sectional strength of wall

Earthquake motion			Level 1	Level 2L	Level 2H
Performance level			Seismic performance Level 1	Seismic performance Level 2	Seismic performance Level 3
Check index			Sectional force		
Material factor g_m	Concrete g_c		1.3	1.3	1.3
	Reinforcement g_s		1.0	1.0	1.0
Member factor g_b	Bending and Axial forces	Bending force is predominant	1.1	1.1	1.05
	Out-of-plane Shear	Concrete	1.3	1.55	1.5
		Reinforcement	1.15	1.4	1.35
Load factor g_f	Self weight		1.0	1.0	1.0
	Incremental earth pressure due to filling		1.0	1.0	1.0
	Normal unsymmetrical pressure		1.0	1.0	1.0
	Unsymmetrical pressure due to filling for disaster control		1.0	1.0	1.0
	Gas Pressure		1.0	1.0	1.0
	Liquid Pressure		1.0	1.0	1.0
	Thermal load		1.0	1.0	1.0
	Roof load during an earthquake		1.0	1.0	1.0
	Inertia force of the main structure		1.0	1.0	1.0
	Dynamic liquid pressure		1.0	1.0	1.0
	Response displacement (Level 1)		1.0	-	-
	Response displacement (Level 2L)		-	1.0	-
	Response displacement (Level 2H)		-	-	1.0
Structural analysis factor g_a	General load		1.0	1.0	1.0
	Thermal load		1.0	1.0	1.0
	Load during an earthquake		1.1	1.1	1.0
Structure factor g_i			1.1	1.05	1.0

Note: The table lists safety factors for a tank filled with LNG and subjected to thermal loading

(iv) Structural analysis factors
For verification by Method 2, the structural analysis factor for the tank wall during an earthquake is set at 1.1, because uncertainty is involved in the equivalent linear analysis used to approximate the nonlinear characteristics of members and in the evaluation of ground reaction using a spring model. For verification of seismic performance Level 3, the structural analysis factor is set at 1.0 considering the low probability of Level 2H earthquake motion occurring.

(v) Structure factors
The importance of an in-ground LNG tank in the limit state is evaluated in terms of the impact of damage on society, its effect on the functioning of the facility, the difficulty of repair or restoration, the need to predict irregularities, and vulnerability to loading. Consequently, the selected structure factors range from 1.1 during Level 1 earthquake motion to 1.0 during Level 2H earthquake motion, according to the limit state.

7.2 Safe Factors for Method 3

The check indexes used for Method 3 consist of not only the conventional sectional forces but also indexes related to deformation of the main structure, such as the compressive strain and tensile strain of elements. Consequently, safety factors are also established for these additional check indexes.
Tables 7.2 and 7.3 show the safety factors used for verification by Method 3. The safety factors used to verify seismic performance Levels 2 and 3 by Method 3 are determined based on the principles outlined below.

(i) Characteristic values of materials and material factors
The characteristic value of the reinforcement is set at -370 N/mm^2 against Level 2H earthquake motion, as for Method 2: For the material factors, the standard values given in the Standard Specifications are used.

(ii) Member factors
-Compressive strain and tensile strain of elements
The member factors related to element strain are set at the same level as the flexural and axial forces for Method 2. They are set at 1.1 for seismic performance Level 2, and 1.05 for seismic performance Level 3.

-Relative displacement between tank wall and bottom slab, and deformation of the uppermost part of the wall
The limit values of the relative displacement between the tank wall and the bottom slab, and of deformation of the uppermost part of the wall are determined by deformation capacities of the membrane and roof. The limit values are determined to provide some redundancy, so the member factors are set at 1.0.

-Member factors for out-of-plane shear force
The member factor for the out-of-plane shear strength against Level 2 earthquake motion is set at a value larger than the flexural and axial forces according to the Standard Specification for Reinforced Concrete to increase the ductility of the member.

(iii) Load factors

All of the seismic load factors are set at 1.0 as for Method 2.

(iv) Structural analysis factors

The structural analysis factors are set at 1.2 in view of the uncertainty involved in the nonlinear structural analysis model. For verification of seismic performance Level 3, the structural analysis factors are set at 1.0 as for Method 2 in view of the low probability of Level 2H earthquake motion occurring.

(v) Structure factors

The structure factors are set as for Method 2 because they are determined based on the importance of the in-ground LNG tank and do not depend on the analysis method.

Table 7.2 Safety factors for Method 3 (Seismic Performance Level 2)

Performance level			Level 2L earthquake motion				
			Seismic performance 2				
Check index			Compressive strain of element	Tensile strain of element	Out-of-plane Shear strength	Relative displacement between wall and slab	Radial deformation of wall rim
Material factor γ_m	Concrete γ_c		1.3	1.3	1.3	1.3	1.3
	Reinforcement γ_s		1.0	1.0	1.0	1.0	1.0
Member factor γ_b	Out-of-plane Shear	Concrete	-	-	1.55	-	-
		Reinforcement	-	-	1.4	-	-
	Deformation of structure		1.1	1.1	-	1.0	1.0
Load factor γ_f			1.0	1.0	1.0	1.0	1.0
Structural analysis factor γ_a	Sectional force		-	-	1.2	-	-
	Deformation of structure		1.2	1.2	-	1.2	1.2
Structure factor γ_i			1.05	1.05	1.05	1.05	1.05

Table 7.3 Safety factors for Method 3 (Seismic Performance Level 3)

Performance level			Level 2H earthquake motion				
			Seismic performance 3				
Check index			Compressive strain of element	Tensile strain of element	Out-of-plane Shear strength	Relative displacement between wall and slab	Radial deformation of wall rim
Material factor γ_m	Concrete γ_c		1.3	1.3	1.3	1.3	1.3
	Reinforcement γ_s		1.0	1.0	1.0	1.0	1.0
Member factor γ_b	Out-of-plane Shear	Concrete	-	-	1.5	-	-
		Reinforcement	-	-	1.35	-	-
	Deformation of structure		1.05	1.05	-	1.0	1.0
Load factor γ_f			1.0	1.0	1.0	1.0	1.0
Structural analysis factor γ_a	Sectional force		-	-	1.0	-	-
	Deformation of structure		1.0	1.0	-	1.0	1.0
Structure factor γ_i			1.0	1.0	1.0	1.0	1.0

8.VERIFICATION OF SEISMIC PREFORMANCE LEVEL 1 AGAINST LEVEL 1 EARTHQUAKE MOTION

8.1 Verification of seismic performance level 1 by Method 2

8.1.1 Sectional forces

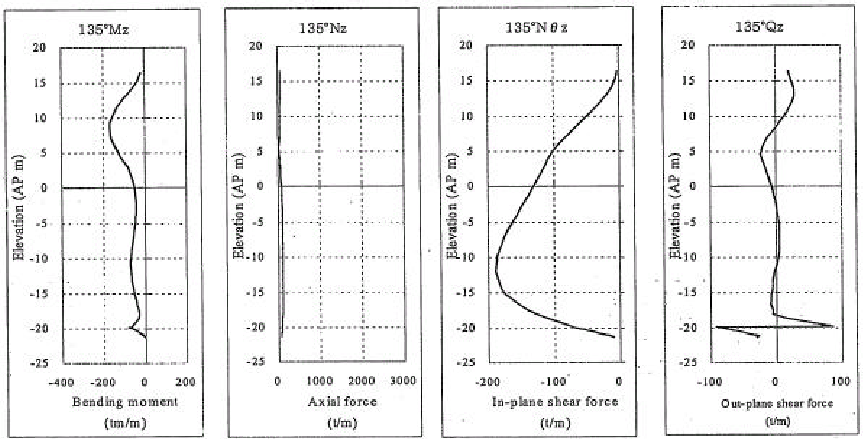
The sectional forces generated by level 1 earthquake motion are shown in Figure 8.1. The figure shows the sectional forces in the 135^{o} range in the cross section that are severe on vertical reinforcement, and the sectional forces in the 180^{o} range in the cross section (on the loading side) that are severe on the circumferential reinforcement in the lowermost part.

8.1 2 Verification of load-cycling capacity

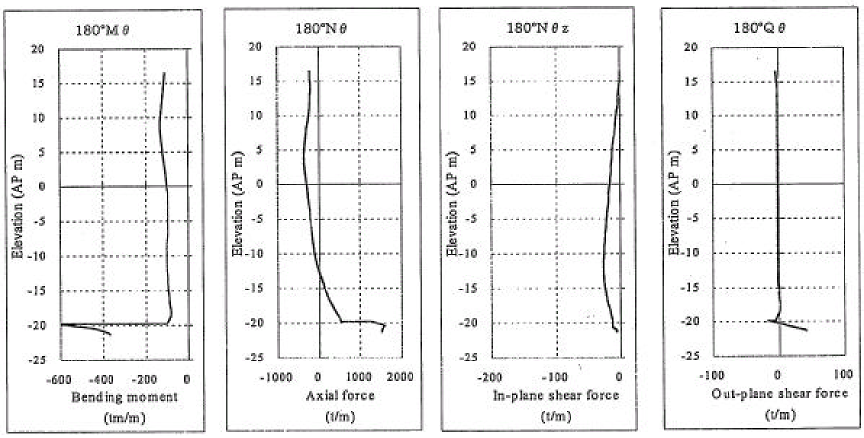
(1) Verification of strength of the main structure

1) Verification of cross-sectional strength against flexural and axial forces and in-plane shear forces
The reinforcement arrangement based on the sectional forces generated by Level 1 earthquake motion shown in Figure 8.1 is shown in Figure 8.2(solid line). Based on the reinforcement arrangement, the parts with the largest amount of reinforcement are checked. The results are shown in Table 8.1. In-plane shear forces are converted to axial forces for verification. The standards are met in all the cross sections.

2) Verification of cross-sectional strength against out-of-plane shear forces
Verification for out-of-plane forces is omitted because the amount of shear reinforcement is determined for Level 2 earthquake motions that apply heavy loading.



Vertical cross section in 135^{o} range



Circumferential cross section in 180^{o} range

Legend: Positive bending moment: Tension on the outside Positive axial force: Tensile force

Figure 8.1 Sectional forces in wall (Level 1 earthquake motion)

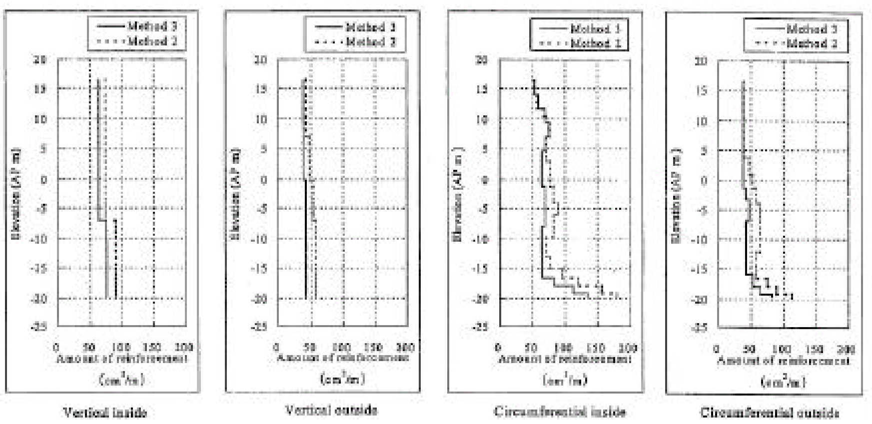


Figure 8.2 Amounts of reinforcement determined for Methods 2 and 3(main reinforcement)

Table 8.l Verification of cross-sectional strength against flexural and axial forces and in-Plane shear force

Cross section		Vertical direction AP-12.0 m 135° range		Circumferential direction AP-19.8 m 180° range		Remarks
Width B (cm)		100		100		
Height H (cm)		180		180		
Sectional forces acting in the cross section	M_d (kN·m/m)	-712		-984		$g_a = 1.1$
	N_d^* (kN/m)	2,956		5,329		$g_a = 1.1$
Outside reinforcement	d_1 (cm)	18.0	33.0	14.5	29.5	
	A_{s1} (cm^2/m)	D29@300 21.4	D29@300 21.4	D41@300 44.7	D38@300 38.0	
Inside reinforcement	d_2 (cm)	166.0	151.0	161.6	146.6	
	A_{s2} (cm^2/m)	D38@300 38.0	D38@300 38.0	D51@300 67.6	D51@300 67.6	
M_{yd} (kN·m/m)		-870		-1,201		$g_c = 1.3$ $g_s = 1.0$ $g_b = 1.1$
N_{yd} (kN/m)		-3,610		-6,506		$g_c = 1.3$ $g_s = 1.0$ $g_b = 1.1$
$g_i \cdot M_d / M_{yd}$		0.90		0.90		$g_i = 1.1$
Check result		OK		OK		

* Including in-plane shear force   Positive axial force: Tension   Negative bending moment: Inward bending

(2) Verification of relative displacement of the wall of the talk and the bottom slab and deformation of the uppermost part of the wall.

Verification is omitted because the liquid-tightness and airtightness of the membrane and the roof fully achieve the target Performance as the main structure of the in-ground LNG talk behaves in the elastic range and because seismic performance level 2 is checked.

8.2 Verification of seismic performance level 1 by Method 3

Verification is omitted because the verification for seismic performance level 1 by Method 2 can be used as an alternative.

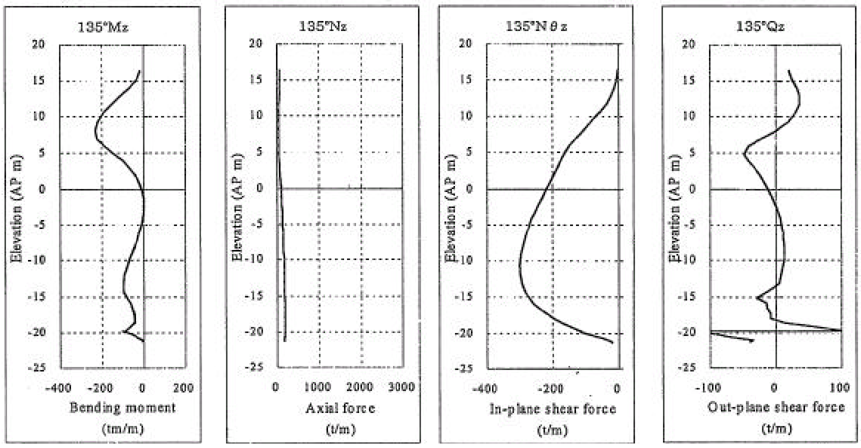
9.VERIFICATION OF SEISMIC PERFORMANCE LEVEL 3 AGAINST LEVEL 2 EARTHQUAKE MOTION

The verification of seismic performance Level 2 is omitted because the Level 3 verification uses the same check indexes and limit values, and is carried out for the stronger for Level 2H earthquake motion.

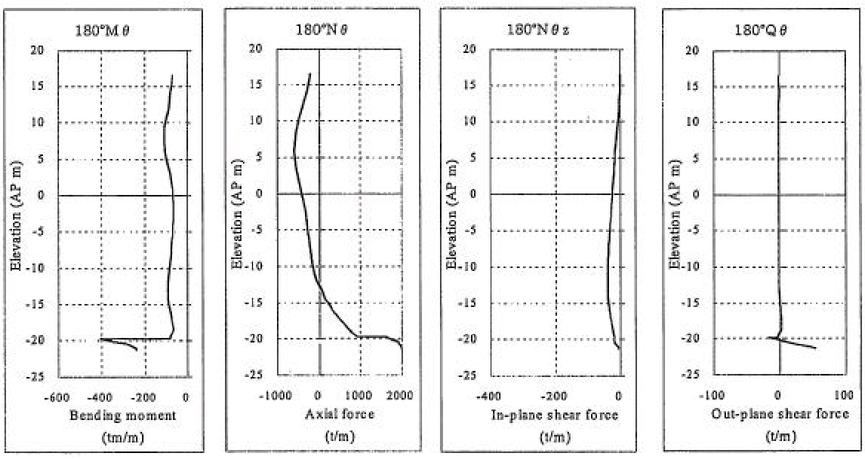
9.1 verification of seismic performance Level 3 by Method 2

9.1.1 Sectional forces

The sectional forces generated by Level 2H earthquake motion are shown in Figure 9.1. This demonstrates that severe sectional forces in the 135° range act on the vertical reinforcement, while severe sectional forces in the 180° range act in the cross section (on the loading side) on the circumferential reinforcement in the lowermost part. These sectional forces generated by Level 2H earthquake motion are greater than those caused by Level 1 earthquake motion For example, the in-plane shear force is about 1.5 times larger.



Vertical cross section in 135° range



Circumferential cross section in 180° range

Figure 9.1 Sectional forces in the wall (Level 2H earthquake motion)

9.1.2 Verification of load-carrying capacity

(1) Verification of cross-sectional strength

1) Verification of cross-sectional strength against flexural and axial forces and in-plane shear force
A reinforcement arrangement designed on the basis of the sectional forces generated by the Level 2H earthquake motion shown in Figure 9.1 is illustrated in Figure 8.2(dotted line). The sectional forces are greater than those generated by Level 1 earthquake motion, so the amount of reinforcement is approximately 400 t greater than in the case of a design for Level 1 earthquake motion. Based on this reinforcement arrangement, the parts with the greatest quantity of reinforcement are checked. The results are shown in Table 9.1. The in-plane shear forces are converted to axial forces for verification purposes. The standards are met in all cross sections.

2) Verification of cross-sectional strength against out-of-plane shear forces
Reinforcement is arranged as shown in Figure 9.2(dotted line) based on the sectional forces generated by the Level 2H earthquake motion shown in Figure 9.1. The results of checking the cross-sectional strength at the base of the wall are shown in Table 9.2. The standards are met.

(2) Verification of relative displacement between tank wall and bottom slab
The relative displacement (1.4 cm) of the tank wall with respect to the bottom slab is below the limit value (4.5 cm). Thus, the joint between the wall and the bottom slab is considered to have the desired deformational capacity.

Table 9.1 Verification of cross-sectional strength against flexural and axial forces and in-plane shear force

Cross section		Vertical direction AP-12.0 m 135° range			Circumferential direction AP-19.8 m 180° range			Remarks
Width B (cm)		100			100			
Height H (cm)		180			180			
Sectional forces acting in the cross section	M_d (kN·m/m)	-925			-837			$g_a = 1.0$
	N_d^* (kN/m)	4,607			9,218			$g_a = 1.0$
Outside reinforcement	d_1 (cm)	18.0	33.0		14.2	29.2		
	A_{s1} (cm²/m)	D35@300 31.9	D32@300 26.5		D41@300 44.7	D51@300 67.6		
Inside reinforcement	d_2 (cm)	166.0	155.0	136.0	162.2	147.7	132.2	
	A_{s2} (cm²/m)	D35@300 31.9	D35@300 31.9	D32@300 26.5	D41@300 44.7	D51@300 67.6	D51@300 67.6	
M_{ud} (kN·m/m)		-974			-927			$g_c = 1.3$ $g_s = 1.0$ $g_b = 1.05$
N_{ud} (kN/m)		-4,850			10,208			$g_c = 1.3$ $g_s = 1.0$ $g_b = 1.05$
$g_i \cdot M_d / M_{ud}$		0.95			0.90			$g_i = 1.1$
Check result		OK			OK			

*Including in-plane shear force   Positive axial farce: Tension   Negative bending moment: Inward bending

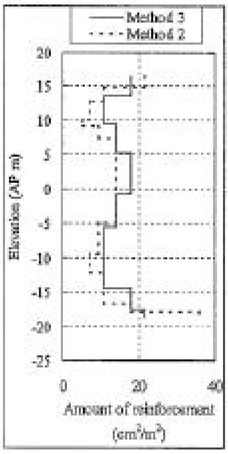


Figure 9.2 Amounts of reinforcement determined for Methods 2 and 3

Table 9.2 Verification of cross-sectional strength against out-plane shear force

Cross section		Vertical direction			Circumferential direction			Remarks
		AP-19.2 m in the 180° range						
Width B (cm)		100			100			
Height H (cm)		180			180			
Sectional forces acting in the cross section	M_d (kN·m/m)	-1,461			-743			$g_a = 1.0$
	N_d^*(kN/m)	2,599			8,297			$g_a = 1.0$
	V_d(kN/m)	1,115			12			$g_a = 1.0$
Outside reinforcement	d_1 (cm)	18.0	33.0		14.2	29.2		
	A_{s1} (cm²/m)	D35@300 31.9	D32@300 26.5		D41@300 44.7	D51@300 67.6		
Inside reinforcement	d_2 (cm)	166.0	151.0	136.0	162.2	147.2	132.2	
	A_{s2} (cm²/m)	D35@300 31.9	D35@300 31.9	D32@300 26.5	D41@300 44.7	D51@300 67.6	D51@300 67.6	
Shear reinforcement	A_w (cm²/m)	10.6			0.7			
	S_s (cm)	30.0			60.0			
V_{cd} (kN/m)		0			0			
V_{sd} (kN/m)		1,266			18			$g_b = 1.5$
V_{yd} (kN/m)		1,266			18			$g_s = 1.35$
$g_i \cdot V_d / V_{yd}$		0.88			0.67			$g_i = 1.0$
Check result		OK			OK			

* Including in-plane shear force   Positive axial force: Tension   Negative bending moment: Inward bending

(3) Verification of deformation of the uppermost part of the wall

The deformation (6.2 cm) of the uppermost part of the wall is below the limit value (8.7 cm) for verification as determined from the deformation capacity of the roof. The main structure is, therefore, considered to have the desired deformation capacity.

9.2 Verification of seismic performance level 3 by Method 3

For analysis by Method 3, a model is constructed with the reinforcement arrangement determined for Level 1 earthquake motion. The input earthquake motion used for this analysis is the data obtained during a five-second interval from 9.0 to 14.0 seconds (Figure 4.1) during which the stress on the structure reaches a maximum.

9.2.1 Response values of structural members

(1) Deformation mode of in-ground LNG tank

The stress on the in-ground tank reaches a maximum at the point when the relative displacement between the uppermost and lowermost extremities of the wall becomes largest. The time history waveforms in the free field and of relative displacement between the uppermost and lowermost extremities of the wall are shown in Figure 9.3 for the case of Level 2 motion. The maximum relative displacement of the wall is about 18 cm. Method 3 is able to track the increasing deformation of elements once the reinforcement has yielded, so the relative displacement exceeds the maximum value found by Method 2 by about 6 cm. The relative displacement reaches this maximum value after 3.8 seconds, which is the same time frame as the maximum in analysis by the response displacement method.

The mode of deformation of both ground and wall at the point of maximum relative deformation (3.8 seconds) is shown in Figure 9.4. The wall deforms in the shear mode, corresponding to the deformation of the ground.

The free field relative displacement (relative displacement between the base of the wall and the ground surface) is about 26 cm. In a pushover analysis [6] of an in-ground tank wall, the wall displacement had to exceed 60 cm for the stress on the in-ground tank to reach a maximum for a reinforcement ratio of 1% or less. The reinforcement ratio of the in-ground tank in this case study is 1% or less in most areas, so the deformation capacity of the tank exceeds the relative displacement of the ground. The tank is, therefore, unlikely to collapse during an earthquake.

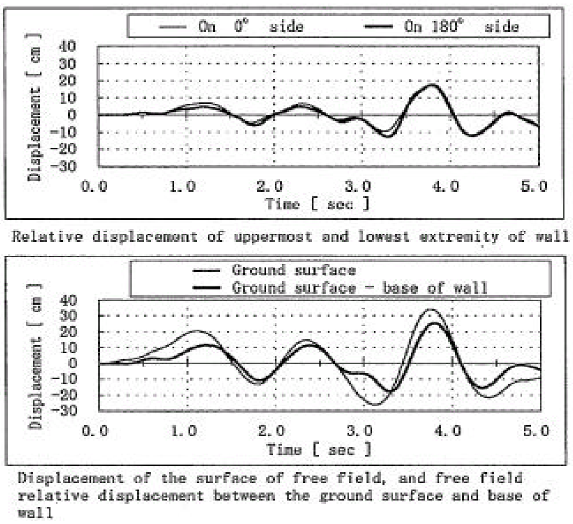


Figure 9.3 Time history response of free field and wall

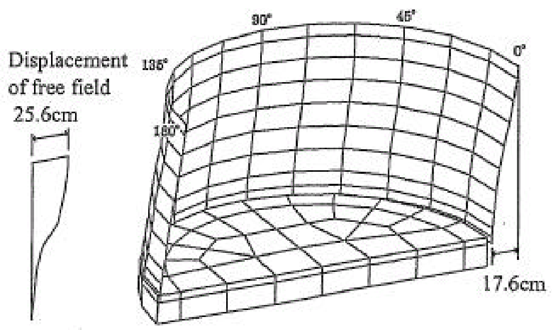


Figure 9.4 Distribution of displacements of free field and wall (at 3.8 seconds)

Elevation (AP m)	172.5°	157.5°	142.5°	127.5°	112.5°	97.5°	82.5°	67.5°	52.5°	37.5°	22.5°	7.5°
16.3	Tθ	Tθ	Tθ	Tθ					Tθ	Tθ	Tθ	Tθ
11.7	Tθ	Tθ	Tθ	Tθ				Tθ	Tθ	Tθ	Tθ	Tθ
7.0	Tθ	Tθ	V	V	V	V	V	V	V	Tθ	Tθ	Tθ
2.3		Tθ	V	V	V	V	V	V	V	V	V	
-2.7	V	V	V	V	V	V	V	V	V	V	V	
-7.6		V	V	V	V	V	V	V	V	V		
-12.5				V	V	V	V	V				
-16.4			V	V	V	V	V	V	V			
-18.9	Tz	Mz	Mz	V	V			V	V	Mz	Mz	Tz

Legend
Tθ : Circumferential axial force is predominant
Tz : Vertical axial force is predominant
V : In-plane shear force is predominant
Mz : Vertical moment is predominant
Blank : Element in which no reinforcement has yield

Figure 9.5 Sectional forces governing element stress

(2) Sectional forces governing the response of elements

The sectional forces that govern element stress are determined before verification of the deformation capacity of each element. The sectional forces predominant in each element are extracted based on the sectional forces at the time when the reinforcement yields. The results are listed in Figure 9.5.

In the main structure of the in-ground tank, the predominant forces are in-plane shear force and circumferential axial force. The predominance of in-plane shear force in the 45° to 135° range in the wall corresponds to the fact that the wall is in shear deformation mode. The analysis model does not take into account delamination or sliding of the ground and the structure, so the wall deforms in line with displacement of the ground as a result of circumferential tension near the ground surface (near the uppermost extremity of the wall). This is where the displacement response of the ground reaches a maximum. The circumferential tension is, therefore, predominant. In-plane shear force is a predominant sectional force throughout the wall.

9.2.2 Verification of load-carrying capacity

(1) Verification of load-carrying capacity

As the sectional forces governing the response of wall elements, the in-plane shear force at the center of the wall is extracted in the 45° to 135° range, and the axial force and bending moment are found for the loading and unloading side of the wall (Figure 9.5). For verification of the deformation capacity of each element against the sectional forces, compressive and tensile principal strains (response values) are calculated for comparison with the limit values of element strain.

1) Verification of deformation capacity of main structure

a. Verification of compressive strain of concrete

The distribution of maximum compressive principal strain inside and outside the wall is shown in Figure 9.6. The compressive principal strain exceeds 1500 m in the range 45° to 135° in the wall, where in-plane shear force is predominant. The time history waveform of compressive principal strain for elements that experience the maximum compressive principal strain (67.5° AP-7.6 m) is shown in Figure 9.7. The compressive principal strain is below the limit value for verification of concrete compressive strain (3300 m). Thus, it is assumed that no members (elements) suffer compressive failure.

b. Verification of tensile strain of elements

For verification of the tensile strain in the area where the membrane is installed, the maximum tensile principal strain inside the wall is determined. The distribution of maximum tensile principal strain of the wall is shown in Figure 9.8. Tensile principal strain is predominant at the uppermost extremity of the wall on the loading and unloading sides and at the center of the wall in the 45° to 135° range, which corresponds to the area where circumferential axial tensile force and in-plane shear force are predominant, as shown in Figure 9.5. The time history waveform of tensile principal strain for elements that experience the maximum compressive principal strain (22.5° AP+15.3 m) is shown in Figure 9.9. The tensile principal strain is below the limit value for verification of strain (13,300 m) determined from the fatigue limit of the membrane. Thus, the members (elements) are assumed to have the desired deformation capacity.

2) Verification of cross-sectional strength against out-of-plane shear forces

The layout of reinforcement based on the sectional forces generated by Level 2H earthquake motion is shown in Figure 9.2(Solid line). The standards for verification of the cross-sectional strength against out-of-plane shear forces are achieved.

(2) Verification of relative displacement between tank Wall and bottom slab
The results of verification of the relative displacement between the wall and the bottom slab are shown in Figure 9.10. The limit values are the allowable displacement as determined by the fatigue limit of the membrane and the radial relative displacement established considering the structure of the slab fitted between the wall and the bottom slab (to absorb the relative displacement). The relative displacement between the tank wall and the bottom slab is less than the limit value for verification (4.5 cm). Thus, the joint between the wall and the bottom slab is considered to have the desired deformation capacity.

(3) Verification of deformation of upper extremity of the wall
The time history waveform of oval deformation of the wall rim is shown in Figure 9.11. This figure indicates the radial relative displacement at the rim of the wall at 0^{o} and at 180^{o}. The relative displacement of wall rim is below the limit value for verification as determined by the deformation capacity of the roof (8.7 cm). The wall is, therefore, considered to have the desired deformation capacity.

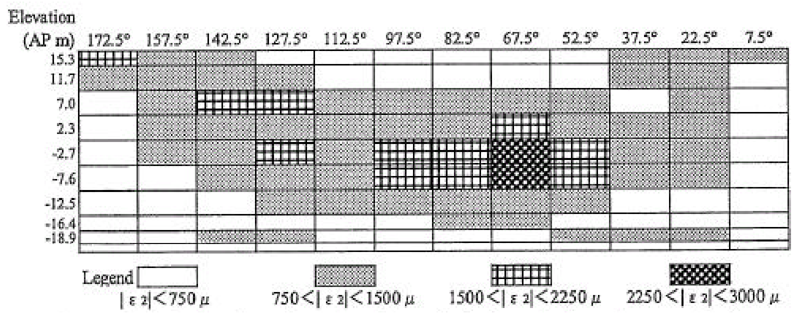


Figure 9.6 Distribution of compressive principal strain (maximum value)

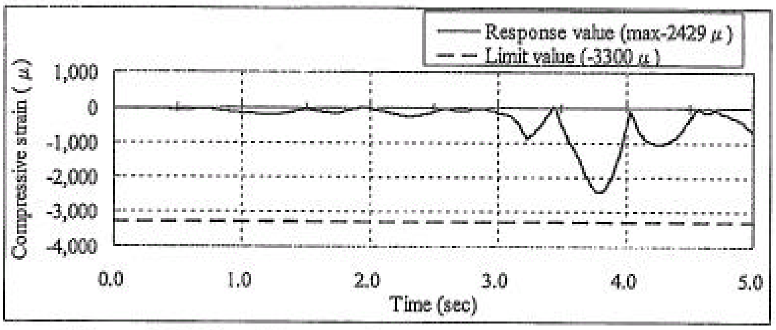


Figure 9.7 Time history of compressive principal strain

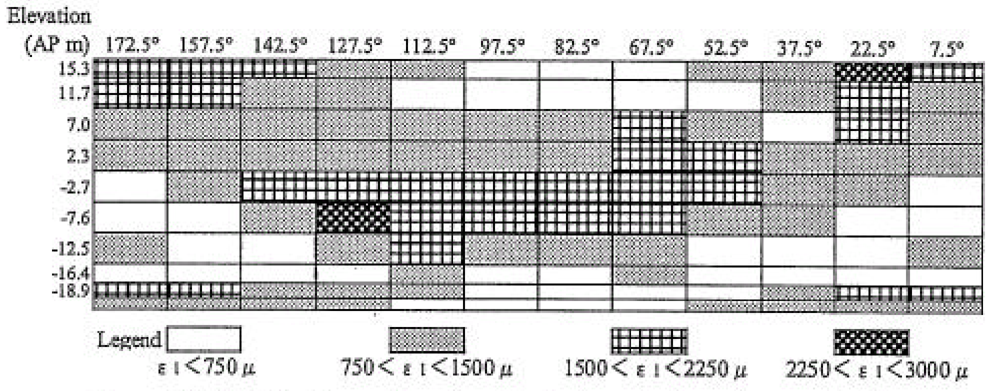


Figure 9.8 Distribution of tensile principal strain (maximum value)

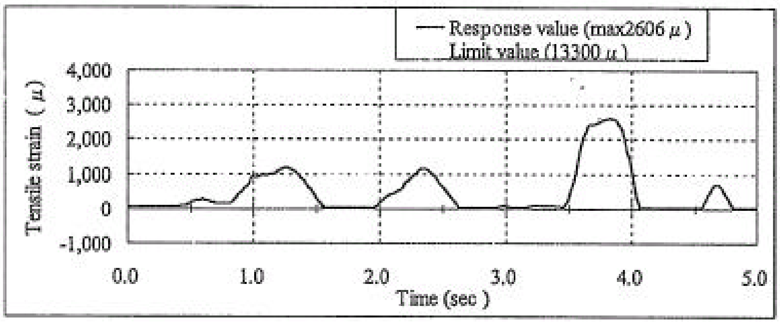


Figure 9.9 Time history of tensile principal strain

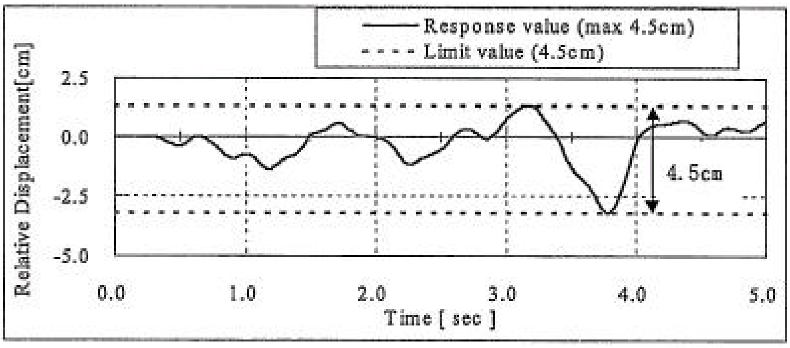


Figure 9.10 Time history of relative displacement between wall and bottom slab

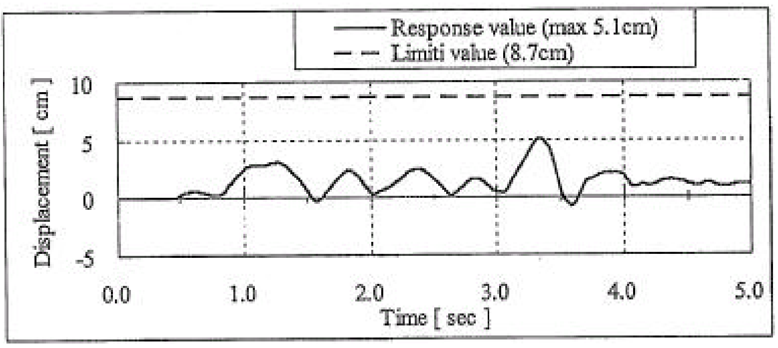


Figure 9.11 Time history of deformation of wall rim

10.COMPARISON OF VERIFICATION BY METHODS 2 AND 3

10.1 RELATIONSHIP BETWEEN SECTIONAL FORCE AND STRAIN

Method 3 represents the nonlinearity of the main wall structure of the wall using a member-level history-dependent macro model. Figure 10.1 shows the relationship between in-plane shear force and in-plane shear strain for elements in this model for which the in-plane shear force is predominant (97.5^{o} AP-2.7m). On the other hand, Method 2 treats the nonlinearity of a member as an equivalent stiffness and sets it at a level one-third of the initial stiffness (Figure 10.1). Method 3 yields lower stiffness in comparison with the increase in in-plane shear force than Method 2 does. As a result, Method 3 is capable of analyzing the deformation of members more accurately after the reinforcement has yielded.

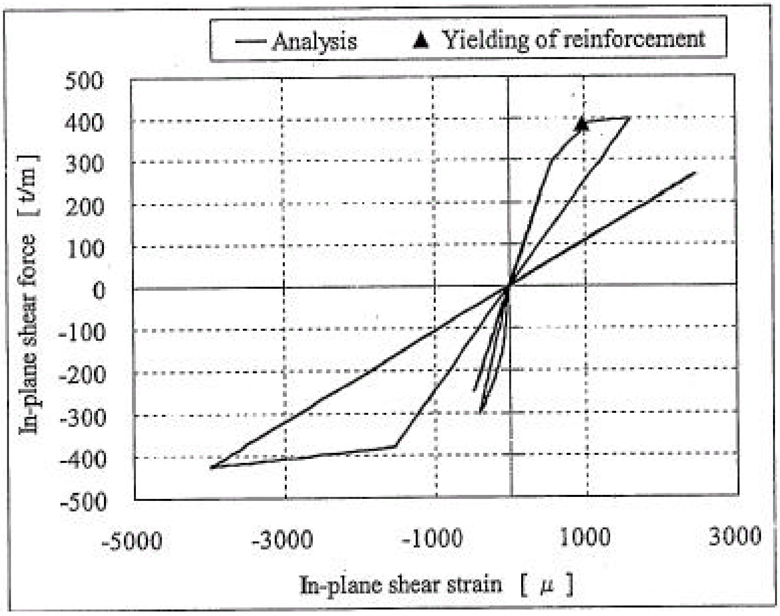


Figure 10.1 Relationship between in-plane shear force and shear strain (97.5^{o} AP-2.7m)

10.2 COMPARISON OF AMOUNT OF REINFORCEMENT REQUIRED

The amount of main reinforcement is determined on the basis of the sectional forces determined by analysis using Level 1 earthquake motion. The deformation capacity of the in-ground LNG tank is then checked against Level 2H earthquake motion by analysis using Method 3 (the dynamic nonlinear analysis method). This procedure demonstrates that the in-ground LNG tank has the desired deformation capacity. Using Method 2 (the quasi-dynamic equivalent linear analysis method), on the other hand, the reinforcement is arranged so as to provide adequate strength to withstand the sectional forces obtained in analysis for Level 2H earthquake motion.

The amounts of main reinforcement determined using Methods 2 and 3 are compared in Figure 8.2. The reinforcement arrangement determined using Method 3 contains about 400 t less steel than when using Method 2. The amount of shear reinforcement is the same with both methods (Figure 9.2).

11. EFFECT OF STREAMLINING BY MOW SOPHISTICATED ANALYSIS METHOD

By checking the deformation capacity of the main structure of an in-ground LNG tank using a dynamic

nonlinear analysis method (Method 3), it is found that less reinforcement is required than when a quasi-dynamic equivalent linear analysis method (Method 2) is used.
When implementing Method 2, the equivalent stiffness is set such that the cross-sectional strength is sufficiently high for a safe-side design. As a result, this method determines the amount of reinforcement required to withstand Level 2 earthquake motion and thereby secure the required cross-sectional strength. With Method 3, on the other hand, deformation beyond yielding of the reinforcement can be analyzed, so it is possible to design the structure to withstand larger Level 2 earthquake motion by making effective use of the deformation capacity of members and thus absorbing the energy. As a result, since Method 3 determines the amount of reinforcement on the basis of Level 1 earthquake motion as shown in Table 11.1, less reinforcement is required than when using Method 2.

Table 11.1 Relation between analysis method and Performance verification

Performance / Method	Level 1 earthquake motion	Level 2L earthquake motion	Level 2H earthquake motion
	Seismic Performance 1	Seismic Performance 2	Seismic performance 3
Method 2	Verification of yield strength	Verification of ultimate strength	Verification of ultimate strength
Method 3	Verification of yield Strength (Method 2 substituted)	Verification of deformation capacity Verification of out-of-plane shear strength	Verification of deformation capacity Verification of out-of-plane shear strength

Legend

Combination of analysis methods used for determination of amount of reinforcement and seismic performance

From the above discussion, it is clear that the use of the more sophisticated analysis method results in a more accurate analysis of member and element behavior. This in turn means that the safety allowance can be reduced and allows for more streamlined design of the main structure of an in-ground LNG tank.

Acknowledgements

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