A STUDY ON LIQUEFIED GROUND DISRUPTION EFFECTS ON LIQUID STORAGE TANK BEHAVIOR

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SUMMARY

Three liquid storage tanks located on a reclaimed land suffered severe damage during the 1995 Hyogo-ken Nanbu Earthquake. The three tanks (A, B, C) were different in terms of foundation and their location from the caisson type quay walls which moved laterally about 2.5m to 3.0m towards sea side. TANK A was supported by a group of bored cast-in-place piles whereas TANK B and C were founded on a soil improved by the vibro-flotation method. The distance from the quay wall to the center of TANK A, B and C were 58, 70 and 124 m, respectively. The settlement of TANK A, which was supported by piles, was negligible. TANK B showed an average settlement of 62cm and a maximum inclination of 1.25%. Whereas TANK C was same foundation soil as TANK B, the average settlement of TANK C was 44cm and almost no inclination of TANK C was observed. To illuminate the different settlement between TANK B and C, earthquake response analyses using an effective stress method were performed considering the influence of the liquid storage tanks with the actual amount of liquid, the liquefaction of foundation soils and surrounding soils, the large lateral movement of the quay wall, and the strong earthquake record of the 1995 Hyogo-ken Nanbu Earthquake. From the results of analyses, it is important to evaluate the effect of the boundary condition of analysis model, such as a quay wall lateral movement, for the calculation of tank's dynamic behavior under strong input motion. And the effective soil improvement is discussed based on the parametric study of simulation analyses.

INTRODUCTION

There are many fill and man-made islands around the coastal areas of the big cities (Tokyo, Osaka, Kobe, Yokohama, etc.) in Japan. In such areas, many chemical industries and power plants maintained a lot of liquid storage tanks. During the 1995 Hyogo-ken Nanbu Earthquake, some of them in the Hansin area (around Kobe and Osaka) suffered severe damage. Compared with the investigations on damage to buildings and bridges due to sand liquefaction, there have been very few detail studies on damage to liquid storage tanks. The seismic design of such liquid storage tanks considering the effect of liquefaction of foundation soils, however, is also a very important task for civil engineers.

The objectives of this paper are to explain a typical damage, during the 1995 Hyogo-ken Nanbu Earthquake, to the cylindrical type steel tanks for storing liquid petroleum gas (LPG) built on man-made fill near Kobe Port Island due to liquefaction of foundation soils, and also to investigate the effect of soil improvement using the vibroflotation work method, and to explain the influence of the large displacement of the tank site boundary(southern quay wall) on the settlement and inclination of the tanks by performing a series of earthquake response analyses using the 2-dimensional Finite Element Method (FEM) based on an effective stress method.

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DAMAGES OF LIQUID STORAGE TANKS

2.1 Specification of liquid storage tanks

TANK A, B and C, which are the liquid storage tank for liquid petroleum gas (LPG), were constructed in the manmade island Mikage-hama, Hyogo prefecture, at the middle of 1960's. TANK A, B and C have the same size (37 m both in diameter and height) and structure (cylindrical type double-cell structure made of metal with flat bottom). The actual amount of LPG at the moment of the 1995 Hyogo-ken Nanbu Earthquake was about 6700, 7400 and 7300 kilo liters for TANK A, B and C, respectively. These values were only about the 1/3 of the tank's capacity (the liquid depth was only 1/3 of the tank height).

TANK A was mounted on a concrete slab which was supported by a group of bored cast in-place reinforced concrete piles constructed using Benoto method (91 piles in total). The reinforced concrete piles, 1.1 m in diameter and 27 m in depth from the ground surface, were constructed in a layout of square with a side of 3.7 m.

TANK B and C were mounted on a concrete slab which was supported by improved foundation soil. The vibroflotation work method was used in this site for soil improvement. The depth of 7m is the maximum possible depth for performing this type of compaction method at that time. The foundation of TANK B and C is about 37m in diameter, and the area compacted is about 47 m in diameter.

The vibroflotation method, which was used for soil improvement of TANK B and C, is a technique of compaction of loose soil by vibration intended for reinforcing the ground and preventing liquefaction by making use of a horizontal vibration and water compaction effect. The SPT N-value after compaction is converted from Swedish cone penetration resistance performed in 1964, and is almost larger than 20, which was the design target value.

TANK A is located 58m from the southern side quay wall to the tank center, and TANK B is 70m, and TANK C is 124m. These are summarized in Table 1.

Table 1: Specification of tanks, foundation type and soil improvement

Tank No.	A	B	C
Tank size	Diam :37m Height :37m Distance*1:58m	Diam :37m Height :37m Distance*1:70m	Diam :37m Height :37m Distance*1:124m
Foundation type	Pile foundation	Raft foundation	Raft foundation
Specification of Piles and ground Improvement	Bored cast-in-place pile Pile dia. = 1.1m Pile length :27m Pile distance:3.7m	Vibroflotation method Depth=7.0m Dia.=47m Pile distance:1.4 m	Vibroflotation method Depth=7.0m Dia.=47m Pile distance:1.4 m

*1: Distance from southern side quay wall

2.2 Damages of liquid storage tanks

The manmade island consists of weathered granite (locally called Masado) from the ground surface to a depth of about 17m, and underlain by Holocene silty clay and fine sand. This fill was considered to be liquefied during the 1995 Hyogo-ken Nanbu Earthquake [HPGSIJ, 1995 A].

The quay walls on the southern side of the manmade island moved and tilted to the sea side due to liquefaction of foundation soils. They were constructed at the middle of 1960's without considering the effect of soil liquefaction. The maximum lateral displacement of the quay wall was measured to be about 2.5 m to 3.0m from an aerial photograph as reported by Hamada et al. (1996). The minimum settlement of the ground at this site was about 70 cm at the northern side. The maximum settlement, about 150cm, was observed near the southern side quay wall.

Figure 1 is the tanks' relative vertical displacement between before and after main shock. As shown in Figure 1, the settlement of TANK A was negligible. However, due to the settlement difference between TANK A and surrounding ground, some of the connecting pipes to TANK A were broken. TANK B showed an average settlement of 62cm and a maximum inclination of 1.25% (about 80cm of the maximum differential settlement) from north to south. The average settlement of TANK C was 44 cm, about 18cm less than that of TANK B. And almost no inclination of TANK C was observed.

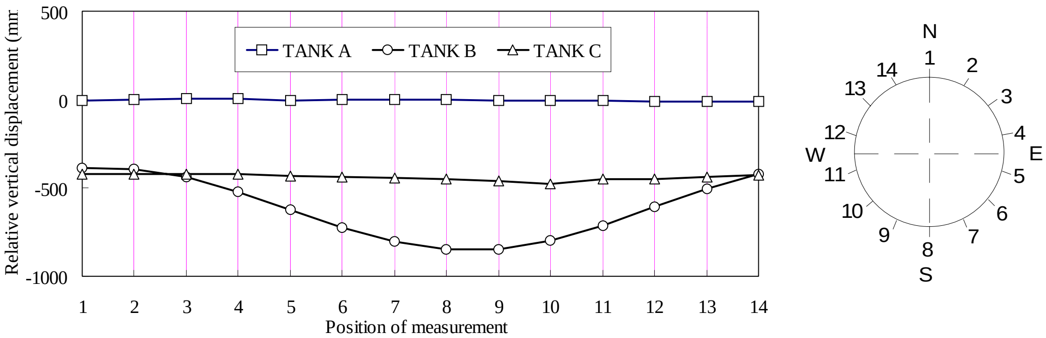


Figure 1: Vertical displacement of foundation for TANK A, B, and C

NUMERICAL ANALYSES TO INVESTIGATE THE DAMAGE OF LIQUID STORAGE TANKS

3.1 Numerical analyses model

To investigate the effect of soil liquefaction on damage of tanks, especially paying attention to settlement of the tanks, numerical analyses were performed by using Finite Element Method (FEM) under plane strain condition. TANK B and C were selected for analysis target. The modeling of soils, tanks, quay wall are as follows.

3.1.1 Modeling of tanks

For the simplicity in this study, the tank with LPG was modeled as a lumped mass, located at a height of the center of gravity of the tank (3.25m from the ground surface), with a period equal to the period of the first order of the liquid. The period of the first order of the liquid stored in TANK B and TANK C was estimated as 0.103 sec and 0.102 sec, respectively, by using the simplified theory proposed by Sakai and Ogawa (1979).

The mat foundation of reinforced concrete of the tank was modeled as a shear beam with flexural rigidity having a shear modulus of concrete. The thickness of the beam (= the thickness of the two-dimensional model) was determined by equalizing the area of the tank foundation (a circle with a diameter of 37 m) to a rectangular having a longer side of 37 m. The detailed values of tank models used in analyses are indicated in Table 2.

Table 2: Specification of tanks

	Young's modulus (kPa)	Shear modulus (kPa)	Area of Cross-section (m^2)	Geometrical momrnt of inertia (m^4)	Weight
TANK B and C	$2.058*10^8$	$7.915*10^7$	1.3506	0.145	648 *1
Foundation	$1.7052*10^7$	$7.35*10^6$	1.4	0.018736	19.6 *2

*1 : Total weight (kN) *2 : Unit weight

3.1.2 Modeling of soil stratum

The soil stratum for numerical analyses was basically modeled on the field observation data of the tank site. Some properties of soil, which were not conducted soil test at this site, were modeled from representative data near the tank site.

The unit weight and shear wave velocities were determined from the test results performed after the earthquake at the tank site. The ground water table before the earthquake was estimated to be about 2.5m from the ground surface, based on the filed investigation performed after the 1995 Hyogo-ken Nanbu Earthquake. The SPT N-value is converted from Swedish cone penetration resistance performed in 1964.

The soil properties of non-improved Masado fill for analyses were determined based on the studies performed by Hatanaka et al. (1997,1997 B) for Masado fill from Kobe Port Island. The internal friction angle of the fine sand below the Masado fill was estimated from the empirical correlation proposed by Hatanaka and Uchida

(1996) by using the SPT N-value. The internal friction angle for gravel was estimated based on the test results of high-quality undisturbed gravel presented by Suzuki et al. (1993). The permeability coefficient of fine sand and gravel were estimated based on the test results shown by Hatanaka et al. (1997 A and 1996), respectively.

The soil properties of the compacted Masado fill were determined based on the test results for compacted Masado fill obtained by "GRCCHAE" (1997). The soil properties are summarized in Table 3.

Table 3: Soil properties used in analysis

(1) Original ground

Depth (m)	Soil type	Unit weight (kN/m^3)	Shear wave velocity (m/s)	Cohesion (kPa)	Internal friction angle (deg)	SPT N-value (N)	Normalized N-value (N_1)	Liquefaction strength at 15 cycles	Permeability coefficient (cm/sec)
2.5	Crushed stone	19.6	110	0.1	41.8	8	-	-	-
7.0	Masado fill	19.6	110	0.1	41.8	8	12	0.19	$2.2*10^{-3}$
13.6	Masado fill	19.6	170	0.1	39.5	8	12	0.19	$1.1*10^{-2}$
17.0	Masado fill	19.6	170	0.1	39.8	15	13	0.20	$1.1*10^{-2}$
19.0	Sandy silt	17.15	150	76.44	11	-			$1.0*10^{-5}$
28.0	Fine sand	18.62	210	0.1	40	25	-		$1.0*10^{-3}$
32.0	Gravel	19.6	320	0.1	42	25			$1.0*10^{-3}$

(2) Improved area

Depth (m)	Soil type	Unit weight (kN/m^3)	Shear wave velocity (m/s)	Cohesion (kPa)	Internal Friction angle (deg.)	Liquefaction strength at 15 cycles	Permeability Coefficient (cm/sec)
2.5	Improved fill	19.6	120	0.1	42	-	-
7.0	Improved fill	19.6	175	0.1	42	0.35	$1.0*10^{-3}$
13.6	Improved fill	19.6	175	0.1	42	0.35	$1.0*10^{-3}$
17.0	Improved fill	19.6	175	0.1	42	0.35	$1.0*10^{-3}$

3.1.3 Modeling of southern side quay wall and surrounding ground

A caisson type quay wall and the surrounding ground modeled for the present analyses are indicated in Figure 2. The properties of the surrounding ground are listed in Table 4. The permeability coefficient were determined based on the previous studies of Hatanaka et al. (1996,1997 A). The internal friction angle of mound rooks and backfill were determined from the test results by Suzuki et al. (1993). The internal friction angle of replaced sand was estimated by using the empirical equation proposed by Hatanaka and Uchida (1995).

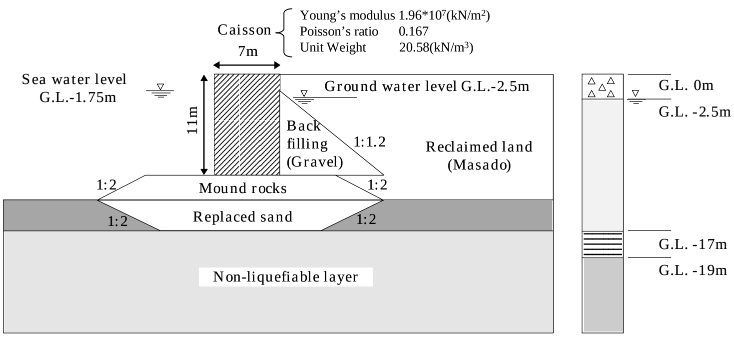


Figure 2: Model of southern quay wall and surrounding ground for analyses

To simulate the large horizontal displacement of the caisson, a joint element was adopted to connect the caisson and mound rocks by friction angle 31(deg.). And also a joint element was adopted to connect the caisson and back filling by friction angle 15(deg.) . These values of the joint elements were referred to the similar analysis by Iai et al. (1995) .

he sea water was modeled as an added mass upon the seaside surface of the caisson based on Westergaard's formula (Westergaard, 1931).

Table 4: Properties of soil around the caisson

Soil type	Unit weight (kN/m^3)	Shear modulus (kPa)	Reference effective confining stress (kPa)	Cohesion (kPa)	Internal friction angle (deg.)	Permeability coefficient (cm/sec)
Mound rocks	19.6	180000	98	0.1	40	$1.0*10^{-1}$
Replaced sand	17.64	58320	106	0.1	36.7	$5.0*10^{-3}$
Back fill	19.6	180000	98	0.1	40	$1.0*10^{-1}$

3.2 Analysis method and models

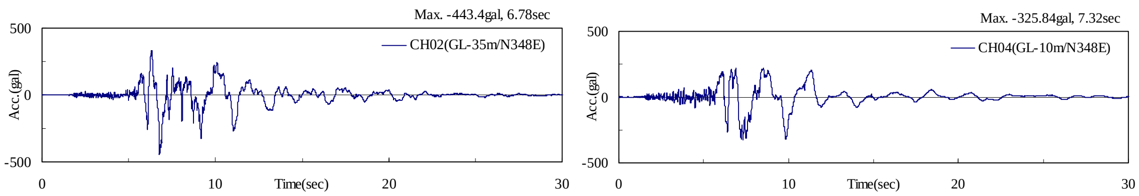
The effective stress analysis is useful to capture the liquefaction phenomena. In this study, the effective analysis code "MuDIAN"[Shiomi et al,1998] was used for numerical analysis. The u-U formulation was adopted, where u is the displacement of the soil skeleton and U indicates the displacement of water.

Densification model was used to describe the dilatancy behavior of Masado fill below the ground water. The densification model is a simple constitutive model based on Mohr-Column's yield criteria, and its characteristic of dilatancy is based on the empirical relationship between the accumulated strain and the excess pore water pressure, which was generalized by Zienkiwicz et al (1978). Hardin-Drnevich model was used to describe the nonlinear characteristic of the soil layer below the fill and the Masado fill above the ground water table.

2% of Rayreigh damping was used for the stability of the numerical solution on the Masado fill below the ground water table. While, 5 % of Rayreigh damping was used for the Masado fill above the ground water table. No Rayreigh damping was used for the soil layer below the Masado fill.

3.3 Input motion

Figure 3 is the strong record in the N-S direction observed at the Higashi Kobe Bridge Station, which is the nearest vertical array record from the tank site. The record of "G.L.-35m,N348E" was used for input motion for the analysis model at the depth of 32m.



(1) G.L.-35m, E348N　　　　(2) G.L.-10m, E348N

Figure 3: Vertical array acceleration record observed at the Higashi Kobe Bridge Station

3.4 Analysis results and discussions

Figure 4 is the 2D-FEM numerical analysis models. The difference between TANK B model and TANK C model is the distance from the southern quay wall to the tank center. To discuss the effect of the lateral displacement of the southern quay wall, " No quay wall" model was also adopted in this analyses.

The effect of the lateral movement of the southern side quay wall on damage to TANK B and C could be seen in Figure 5 and Figure 6 by comparing the results of three models (TANK B, TANK C, and TANK B without quay wall). The maximum response acceleration at the ground surface (Line-1) is about 310gal. These values are nearly equal to that observed at the ground surface of Kobe Port Island (341 gal)[CEORKA, 1995].

Figure 5 shows the vertical distribution of the response value of three models at Line-1 and Line-A. The excess pore water pressure showed no significant difference among three models. Shear strain and pore water pressure were diminished by improved soil at the depth from 0m to 7m, however large shear strain and pore water pressure occurred at the Masado fill at the depth from 17m to 7m. The distributions of shear strain were slightly different each other, and also the significant differences occurred at the horizontal and vertical displacement of tanks at 30sec. The quay wall top moved about 110cm towards the sea side in this analysis. This value was quite smaller than observed values, but this caused the significant differences of tank's remained displacement.

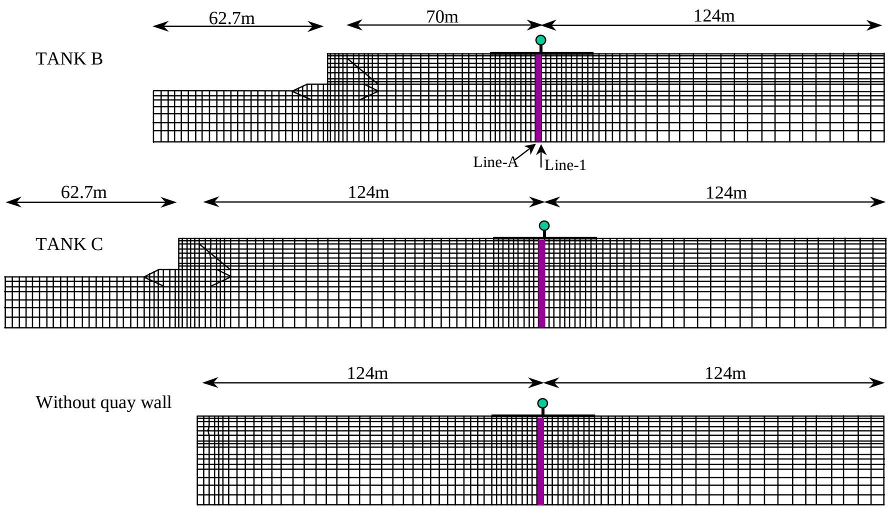
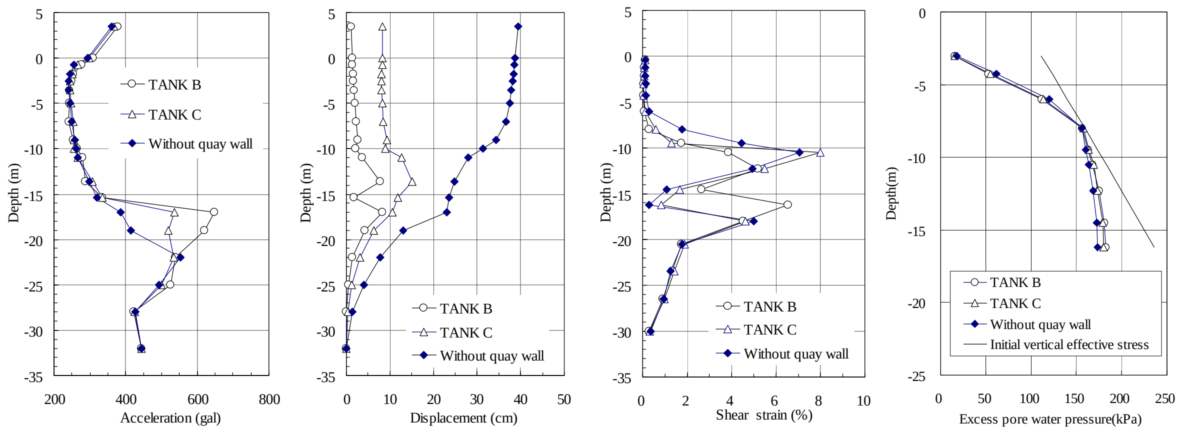
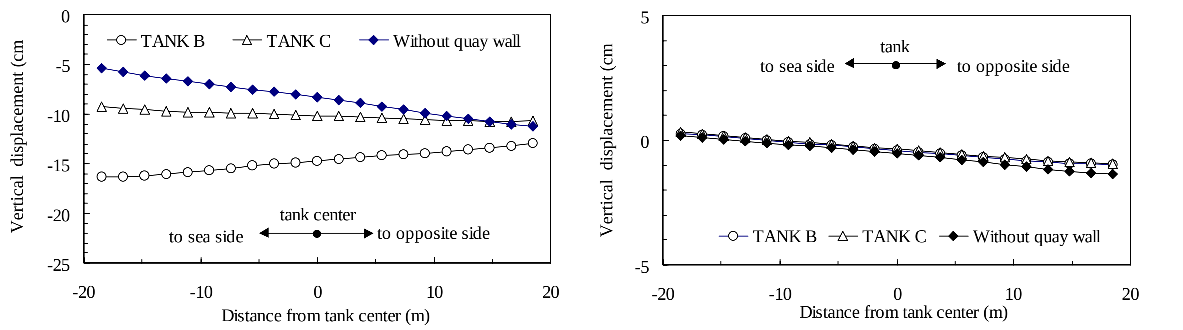


Figure 4: Numerical models used for the analysis



(1) Maximum acceleration at Line-1 (2) Horizontal displacement of Line-1 at 30sec (3) Maximum shear strain at Line-A (4) Pore water pressure of Line-A at 30sec

Figure 5: Vertical distribution of response values (at Line-1 and Line-A)



(1) Vertical displacement at tank mat (2) Vertical displacement at tank mat (half size of input motion)

Figure 6: Comparison of the vertical displacement at the tank's base mat

Figure 6(2) is a comparison of the vertical displacement at the tanks, while the input ground motion was adjusted to half size of the observed record. In this case, the response acceleration on the ground was about 230gal, and the quay wall top moved about 20cm towards the sea side. There are no significant differences among the three models.

So the difference of damages between TANK B and C is supposed to be caused by following reasons, (1) the distance from the tank center to the quay wall which moved largely to the sea side, (2) the much larger input motion than design earthquake, (3) the inadequate vertical range of soil improvement.

Due to the limitation for performing the vibroflotation method in 1960', the compaction depth was 7 m from the ground surface at the site of TANK B and C. So simulation analysis was performed to estimate the effective soil improvement area against the settlement. Numerical models for TANK B was used for this simulation, because it suffered more damages than TANK C from the quay wall's dynamic behavior. CASE-1 is the real improvement area as designed, CASE-2 is the extension of horizontal improved area, and CASE-3 is the extension of vertical improved area. CASE-2 is assumed to be realized in 1960'.

Figure 7 shows the comparison of vertical displacement at the tank among the three cases. It is clear that CASE-2 is almost same as CASE-1, and CASE-3 is clearly improved the settlement of tank. This result indicates that the soil improvement by compaction to a depth of 17m from the ground surface reduces the effect of the southern side quay wall.

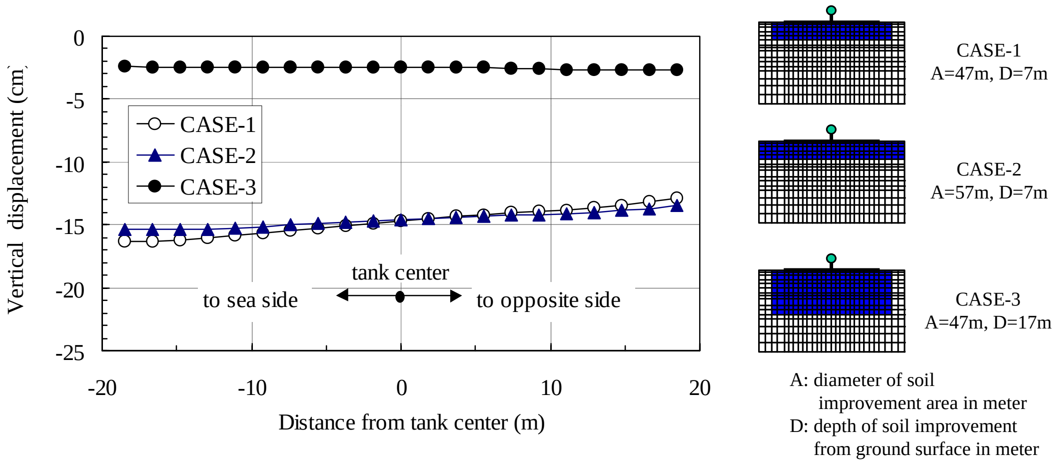


Figure 7 : Comparison of vertical displacement at tank mat (TANK B)

CONCLUSIONS

Based on this study, following conclusions can be stated:

1. Based on the damage to the foundation of TANK B and C and surrounding ground due to liquefaction, the settlement and inclination of the tanks are considered to be affected by the large movement of the quay wall located on the southern side of the site. These observations are also qualitatively supported by the results obtained in the simulation analyses.
2. The foundation soil of TANK B and C was improved by using the vibroflotation method based on the design earthquake at the time of construction. These tanks, however, suffered damage due to liquefaction of foundation soil. The main reason for this result is considered due to the unexpected large ground motion that occurred during the 1995 Hyogo-ken Nanbu Earthquake based on the results of simulation analyses. Another important reason is that the depth of soil improvement (7 m) was not enough for the large ground motion observed in the 1995 Hyogo-ken Nanbu Earthquake. This could also be pointed out from the analyses results.
3. Based on the analytic results from CASE-1, 2 and 3 for TANK B, it is important to point out that it is more effective to compact the foundation soil to a much greater depth than enlarging the width of the compaction area outside the tank. Judging from the results obtained in the present analyses, the width of 5 m for compaction out side the tank in a horizontal direction (compaction depth is 7 m) is considered appropriate under the design earthquake.

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