Geomechanics and Engineering, Vol. 5, No. 2 (2013) 165-185

Experimental analysis on FEM definition of backfill-rectangular tank-fluid system

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(Received September 19, 2012, Revised January 15, 2013, Accepted February 22, 2013)

Abstract. In the present study, the numerical and experimental investigations were performed on the backfill- exterior wall-fluid interaction systems in case of empty and full tanks. For this, firstly, the non-linear three dimensional (3D) finite element models were developed considering both backfill-wall and fluid-wall interactions, and modal analyses for these systems were carried out in order to acquire modal frequencies and mode shapes by means of ANSYS finite element structural analysis program. Secondly, a series of field tests were fulfilled to define their modal characteristics and to compare the results from proposed approximation in the selected structures. Finally, comparing the theoretical predictions from the finite element models to results from experimental measurements, a close agreement was found between theory and experiment. Thus, it can be easily stated that experimental verifications provide strong support for the finite element models and the proposed procedures themselves are the meritorious approximations to the real problem, and this makes the models appealing for use in further investigations.

Keywords: rectangular tank; forced vibration tests; finite element model; fluid-structure-soil interaction

1. Introduction

Rectangular tanks form crucial links in the water supply network due to the storage of water for drinking and fire fighting, and are vulnerable to intensive earthquake loading. Since these lifeline structures are becoming more and more prevalent by reason of fast growing population and industry, and since they are an integral part of the infrastructure of modern societies in the world, their preservation and structural safety can be considered as a fundamental issue. In the past, however, it was stated many times that the post-earthquake damages to the structures were often experienced, and significant financial expenses were made in repair or replacement. In addition to these seismic failings, during the in-situ investigations of authors, it is seen that, through the improper foundation design, wall design and mistakes made in selecting tank built areas, there are many examples failed statically; and hence many of them are not be able to contain water. In order to avoid this situation, the tanks have to be properly designed against earthquake and should remain functional in the post-earthquake period to ensure water supply in earthquake prone areas.

Depending on design conditions and load bearing mechanisms, the tanks are classified into

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*Corresponding author, Assistant Professor, E-mail: cakirtufan@hotmail.com different categories, e.g., rectangular tanks, elevated tanks, underground tanks, ground-level cylindrical tanks, etc. The seismic responses of various types of tanks were examined by a number of researchers in the past, either experimentally, analytically, or numerically. While most of these studies have concentrated on ground-level cylindrical tanks, the behavior of rectangular tanks during seismic loading has been studied by a few researchers. A review of chronological developments pertaining to the analyses of water storage tanks can be found in various publications. The publications performed on the behavior of rectangular tanks due to soil-structure interaction (SSI) effects can be attained from Livaoglu (2008). In addition to the studies presented in it, Chen and Kianoush (2009) presented a simplified method using the generalized single degree of freedom system for seismic analysis and design of rectangular tanks. Then, effects of wall flexibility and earthquake frequency content on dynamic behavior of rectangular tanks were investigated by Ghaemmaghami and Kianoush (2010) and Kianoush and Ghaemmaghami (2011). Jeong (2011) developed an analytical method based on the Rayleigh-Ritz approach for calculating natural frequencies of rectangular tanks, and verified the method by observing an agreement with finite element analysis results. Furthermore, studies on dynamic behavior of rectangular tanks considering backfill-wall-fluid interaction were carried out by Cakir (2010), Livaoglu *et al.* (2011) and Cakir and Livaoglu (2012).

The effect of backfill soil pressure is of great importance for a number of problems stemming from retaining walls, sheet pile walls and basement walls etc. Earthquakes have unfavorable effects on lateral soil pressures acting on retaining walls or exterior walls of the tanks, because backfill exerts large dynamic forces on walls and causes severe failures. The damages of the exterior walls are mainly associated with the movement and failure induced by strong earthquake motion and high seismic soil pressure. The seismic soil pressures during earthquakes may be affected by various factors such as frequency components of earthquake ground motions, the motions of foundations or underground structures, compositions and properties of soil layers around the structures, types of earthquake waves, incident angle of seismic waves and so on (Minowa and Sadohiro 2001). Hence, the assessment of seismic lateral soil pressures is of practical significance in most seismic designs of walls. Discussion of all the research work on the seismic soil pressure is extensive and beyond the scope of this study. Rather, only some milestones that have influenced the design practice are described below. So, earlier investigations on the dynamic behavior of retaining walls during earthquakes can be broadly divided into three major areas: (1) analytical investigations, (2) numerical investigations, (3) experimental investigations. The investigations performed by various researchers in these three areas are widely discussed by Nazarian and Hadjian (1979) at the end of the 70's. Thus, the other studies carried out after those are briefly discussed below.

In the analytical investigations, many researchers have used elasticity principles and wave propagation theory to obtain the dynamic response of soil-structure systems and model the effects of wall-soil interaction with different approaches, assumptions and simplifications. Scott (1973) analytically studied earth pressure on rigid retaining walls rotating about the base, and concluded that forces and moments were significantly higher than those calculated by Mononobe-Okabe (M-O) analysis. Arias (1981) developed a model for the case of fixed, rigid walls under an arbitrary horizontal dynamic excitation, and compared the results with those of Wood (1973) and with other finite element solutions. Then, Dimaragona (1983) analytically showed that, in the behavior of the wall, modes of the motion have clear importance on earth pressures on retaining walls subjected to dynamic loading. Finally, the most important and considerable studies have been carried out by Veletsos and his co-worker. Veletsos and Younan (1994a,b) developed a simple approximate expression to simulate the dynamic pressures, the associate forces, and the responses induced by ground shaking on a straight, vertical rigid wall retaining soil with a semi-infinite, uniform viscoelastic layer of constant thickness. The solutions for frequency-dependent and frequency-independent parameters were studied and compared with the results proposed by Scott (1973). The elastic constrained bars with distributed mass were used to represent the soil stratum in backfill. They concluded that Scott's model, which ignores radiational soil damping and considers the wall pressure to be proportional to the relative motions of the wall and the soil at the far field, does not adequately describe the action of the system and may lead to large errors. Veletsos and Younan (1997), continuing and expanding their work, developed a solution technique to compute the dynamic response of cantilever retaining walls that are elastically constrained against base rotation subjected to horizontal ground motion, and Younan and Veletsos (2000) investigated dynamic response of flexible retaining walls and educed that the wall displacements and pressures are quite sensitive to the flexibility of the wall. Mylonakis *et al.* (2007) proposed a closed-form stress plasticity solution which is essentially an approximate yield-line approach, based on the theory of discontinuous stress fields, for the gravitational and earthquake-induced earth pressures on retaining walls. Furthermore, displacements of the retaining walls may be induced during earthquakes. Then, a displacement-based design needs to be introduced. Thus, some researchers carried out displacement-based designs taking the permissible displacements of the wall into account (Richards and Elms 1979, Siddharthan *et al.* 1991, Rafnsson 1991, Wu 1999, Choudhury 2004).

Numerical modeling efforts have been applied to verify the seismic design methods in practice and to provide new insights to the problem. Bryne and Salgado (1981) and Steedman (1984), adopting the elastoplastic models, investigated dynamically induced pressures on retaining walls. Nadim and Whitman (1983) developed a finite element solution and concluded that amplification of motion in the backfill plays an important role with regard to the permanent displacement of the wall when the ratio of dominant frequency of ground motion to the fundamental frequency of the backfill is greater than 0.3. Gazetas *et al.* (2004), using finite element modeling and considering both linear and non-linear soil behavior, explored the magnitude and distribution of dynamic earth pressures on several types of flexible retaining systems. Psarropoulos *et al.* (2005) built up a more general finite element method of solution and showed that the obtained results are in good agreement with the available analytical results for the distribution of dynamic earth pressures on rigid and flexible walls. Madabhushi and Zeng (2007) presented the results of a finite element simulation of a flexible cantilever retaining wall with dry and saturated backfill under earthquake loading, and compared the results with those of a centrifuge test.

The experimental investigations on wall-soil systems can be broadly classified into three different categories as follows: (1) shaking table tests, (2) centrifuge tests, (3) full scale tests.

A number of shaking table tests have been fulfilled to investigate dynamic wall-soil response. Most of these tests were usually conducted on small models and the recorded experimental data have been usually compared to those predicted by the widely known M-O solution (Richards and Elms 1979, Sherif *et al.* 1982, Ishibashi and Fang 1987, Elms and Richards 1990, Ishibashi *et al.* 1994). Additionally, a large body of centrifuge tests have been focused primarily on wall-soil deformations under strong dynamic excitation conditions, and wall-base translation and rotation modes have been investigated (Ortiz 1982, Ortiz *et al.* 1983, Bolton and Steedman 1982, 1984, Anderson *et al.* 1987, Pahwa *et al.* 1987, Whitman and Ting 1993, Ting and Whitman 1995, Stadler 1996, Zeng 1998, Madabhushi and Zeng 1998, Dewoolkar *et al.* 2001). In the third category of experimental investigations, few dynamic full scale tests have been performed on retaining wall-soil systems (Fukuoka and Imamura 1984, Chang *et al*. 1990, Alampalli 1990, Elgamal *et al*. 1996).

Literature investigations show that although many studies can be found in technical literature for water storage tanks and/or retaining walls, there are relatively few studies on the behavior of rectangular tanks among them. Furthermore, from the above discussion, many researchers have primarily dealt with the developments of analytical models and different numerical techniques as a tool for analyzing the influences of different parameters on the vibration characteristics of walls to compare with experimental studies which are performed on laboratory model investigations only for particular cases. On the other hand, there is almost no investigation regarding the in-situ tests conducted on rectangular tank wall and/or retaining wall. When examined the codes (Eurocode 8 2003, 2006, ACI 2001) about the tanks and/or retaining walls, it can also be clearly seen that a specific method is unavailable regarding how the backfill interaction effects should be taken into consideration. Notwithstanding the significant advances in developing theoretical solutions to problems of vibration, experimental verification of such theories via field tests remains a necessary prerequisite for their adoption and reliable application in practice. So, it can be clearly emphasized that there is no adequate number of in-situ experimental studies and/ or investigation on the exterior wall of reinforced concrete (R/C) rectangular tank to define the dynamic characteristics subjected to both backfill and fluid interaction effects. For these reasons, the main subject of this study is selected as to submit a procedure to literature, comparing its results with those derived from in-situ experimental investigation, and prove that the numerical results may be useful for determining the seismic behaviour of such a fluid-structure-backfill system.

2. Description of the rectangular tanks under consideration

Despite their structural simplicity, rectangular tanks are rather complicated fluid-structure-soil interaction systems, the dynamic responses of which have not yet been fully understood. In this study, both a reinforced concrete rectangular (prismatic) tank with a container capacity of 5000 m^3 in case of empty container (TANK_A) and a rectangular tank with a container capacity of 8000 m^3 in case of partially filled container (TANK_B) were investigated, respectively. There are several parts in a rectangular tank since these structures were constructed as segmental, not monolithic in practice. In this connection, each part of the structure is subjected to different loads, and exhibits different behaviours. In this study, only the exterior walls of the rectangular tanks which interact with both the backfill in one side and the fluid in the other side were tackled, as each part of the structure shows considerable differences in terms of both the load bearing mechanisms and the geometrical and positional differences. During the in-situ tests, since the TANK_A is empty, the system consists of backfill medium and exterior wall. So, the model was named as “backfill-exterior wall system”. On the other hand, as the depth of water within the container of TANK_B was measured as 2.5 m, the system consists of backfill medium, exterior wall and fluid medium, and the considered model was named as “backfill-exterior wall-fluid system”. The aforesaid tanks were partly suffered from non-structural damage during 1992 Erzincan earthquake, but any structural damage did not occur and only some repairments were made in the components such as mortar, plaster and ceramic etc. The rectangular tanks under consideration have two main divisions. The roofs of the tanks were constructed as beam slab supported by 5 m-height-slender columns which have 0.3 m x 0.3 m plan geometry. The thicknesses of exterior walls of the tanks are 0.3 m. The roof coverings consist of the granular material, and their thicknesses are 0.7 m. In the below subtitles, in-situ structural properties and other information from the authorized person who is responsible for servicing of the tank are given.

2.1 TANK_A: empty container situation

The rectangular tank examined was constructed in Erzincan (NE Turkey) in 1978. Both the top and side views and in-situ defined geometrical properties of the TANK_A were given in Figs. 1 and 2, respectively. As mentioned before, because of the unavailability of fluid in the container, the problem analyzed consists of backfill-exterior wall system. The mechanical properties of the TANK_A were determined with in-situ non-destructive testing, and its structural properties were determined by making measurements on it. In the modelling of the TANK_A, to obtain the dynamic characteristics of tank wall, Young's Modulus, Poisson's ratio and the weight of concrete per unit volume were taken to be 26160 MPa, 0.2 and 25 kN/m^3, respectively. Moreover, taking representative samples of soils from the field, the samples were tested in the laboratory, and it is determined that the backfill soil can be classified as silty sand. Thus, examining the mechanical and physical properties recommended in the literature for abovementioned soil classes, Young's Modulus, Poisson's ratio and the unit weight of soil were taken to be 20 MPa, 0.3 and 19 kN/m^3, respectively.

2.2 TANK_B: partially filled container situation

The rectangular tank examined was constructed in Erzincan (NE Turkey) in 1976. Both the front and top views and in-situ defined geometrical properties of the TANK_B were given in Figs. 3 and 4, respectively. As stated previously, due to the fluid with a height of 2.5 m in the container, the problem analyzed consists of backfill-exterior wall-fluid system. In the modelling of the TANK_B, to obtain the dynamic characteristics of tank wall, Young's Modulus, Poisson's ratio and the weight of concrete per unit volume were taken to be 28000 MPa, 0.2 and 25 kN/m^3, respectively. In addition to these, Young's Modulus, Poisson's ratio and the unit weight of soil were taken to be 20 MPa, 0.3 and 19 kN/m^3, respectively.

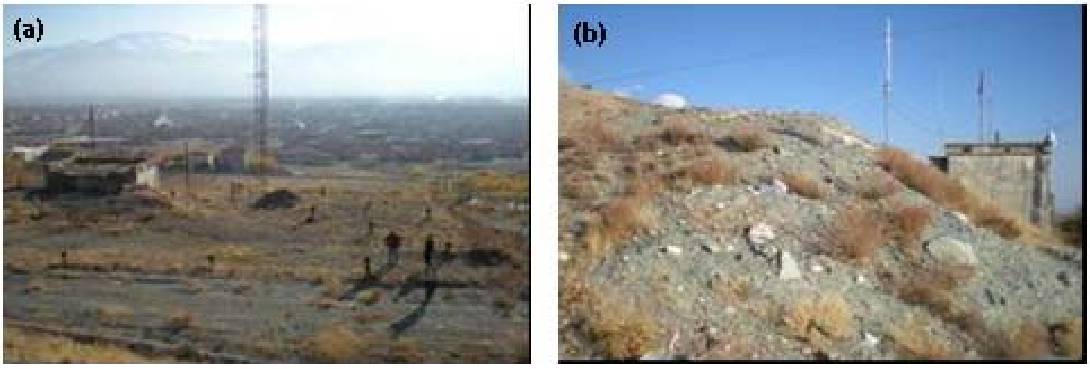


Fig. 1 (a) The top view; and (b) side view of the TANK_A

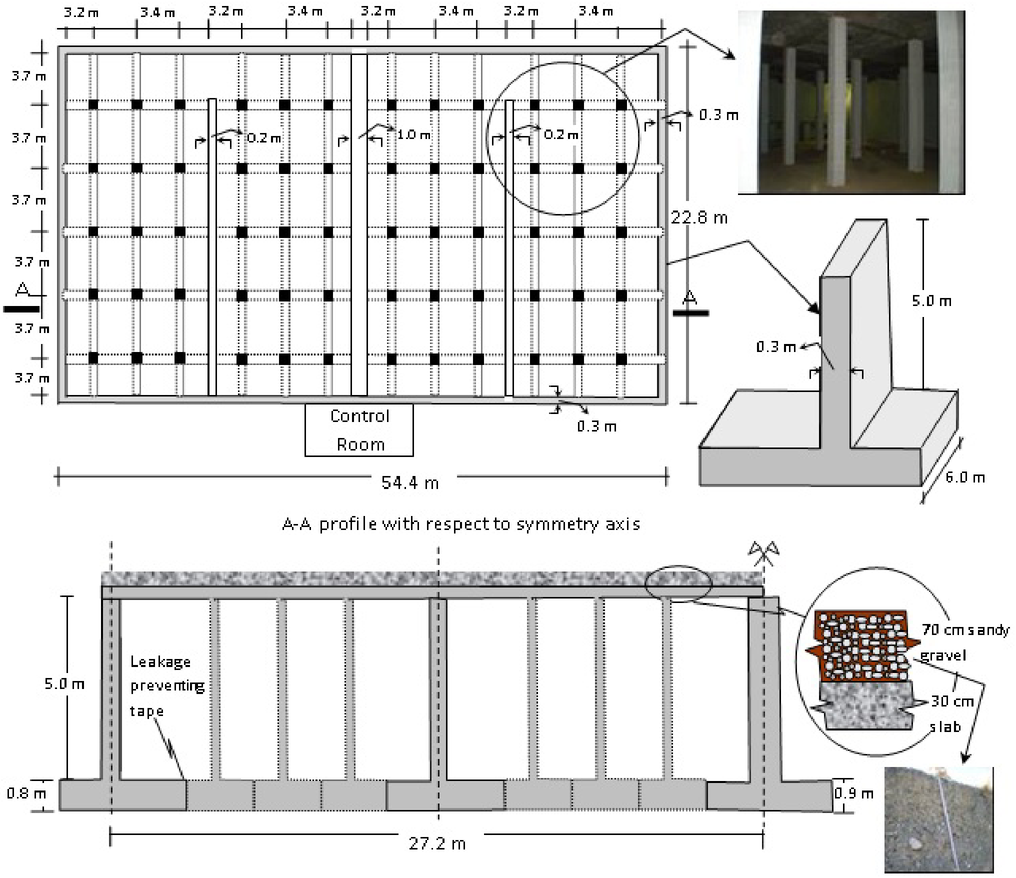


Fig. 2 The geometrical properties of TANK_A

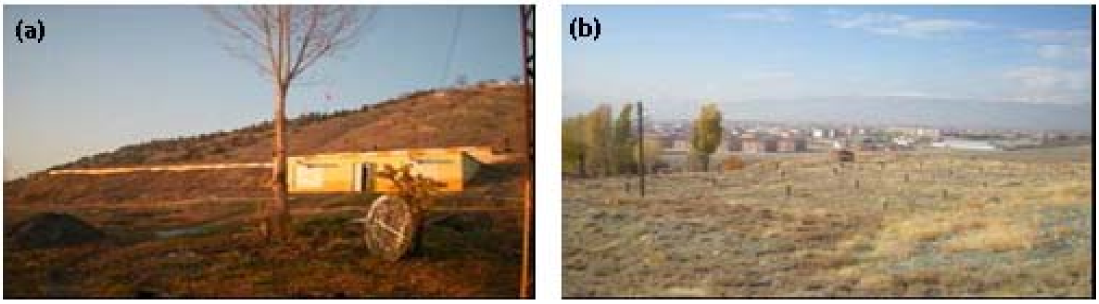


Fig. 3 (a) The front view; and (b) top view of the TANK_B

3. The proposed numerical models

The ANSYS (2006) finite element program is used to obtain the frequencies of modes and mode shapes of the combined both backfill-wall and backfill-wall-fluid systems. The finite element models for the aforementioned systems are presented below. The models were extended to their most general and comprehensive cases since they were designed to be used in future

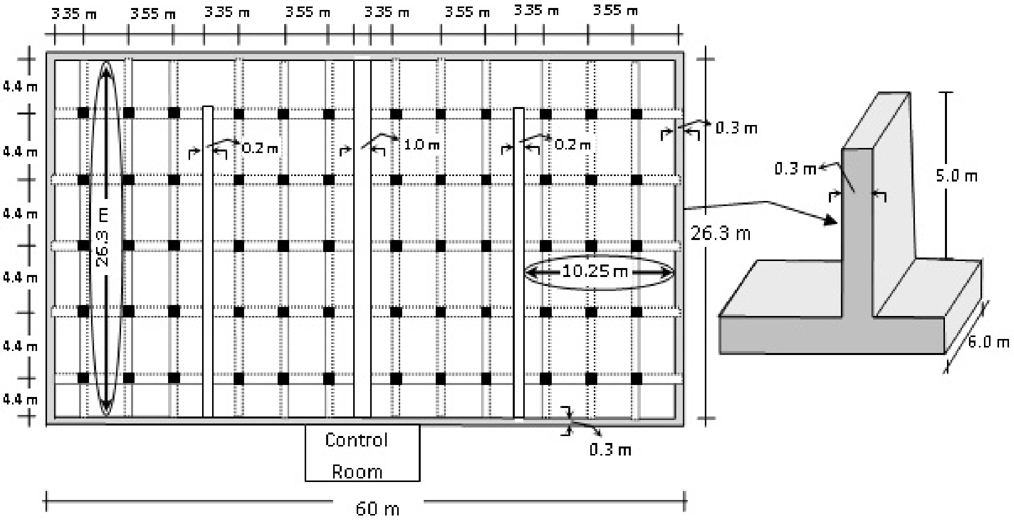


Fig. 4 The geometrical properties of the TANK_B

investigations in order to examine the seismic behaviour of rectangular tanks considering fluid and soil interactions through nonlinear time history analysis. However, it is a well known fact that taking into account the material nonlinearity is not a practical way for the modal analysis. In this context, the modal analyses were conducted assuming elastic material responses in this study. Furthermore, it is worth mentioning that both the viscous boundary and the nonlinear properties of the material considered in the proposed model are for clear understanding of the system dynamic behaviour during time history analysis.

3.1 Numerical model for backfill-exterior wall system

The finite element model for backfill-wall system is shown in Fig. 5. The considered system is supported on rigid foundation. Structural wall is modeled with solid elements (SOLID65) having three degrees-of-freedom at per node; the roof system is modeled with quadrilateral shell element (four nodes, six degrees-of-freedom at per node) and also with additional mass of cover. The backfill soil is also modeled with solid elements (SOLID185). Actually, despite its structural simplicity, the dynamic response of exterior walls of the rectangular tanks is part of a rather complex dynamic system. What makes that response so complicated is the dynamic interaction between the wall and backfill soil. Understanding the behavior of this system requires the consideration of the mass and stiffness of the wall, the backfill and/or underlying ground and the interaction among them. In this situation, the deformation and strength of the backfill and the underlying soil are important issues, which need to be modeled using reasonable soil model. Moreover, reasonable modeling of the wall-soil interaction requires using special interface elements between the wall and adjacent soil to allow for separation and sliding. Thus, to model backfill-wall interaction, unidirectional element with nonlinear generalized force-deflection capabilities is used in the analysis. The element has longitudinal or torsional capability in 1-D, 2-D, or 3-D applications. The longitudinal option is a uniaxial tension-compression element with up to

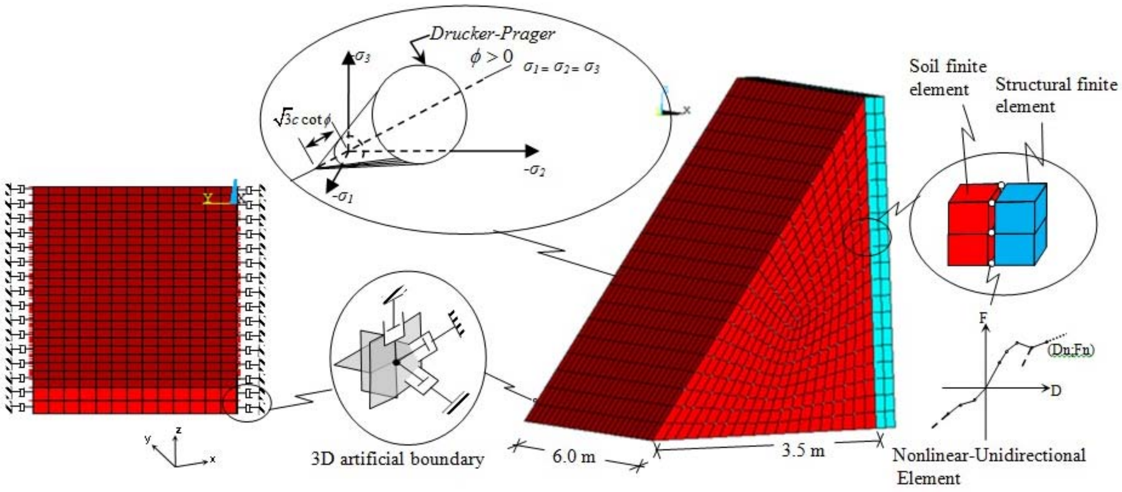


Fig. 5 The finite element model of backfill-exterior wall system

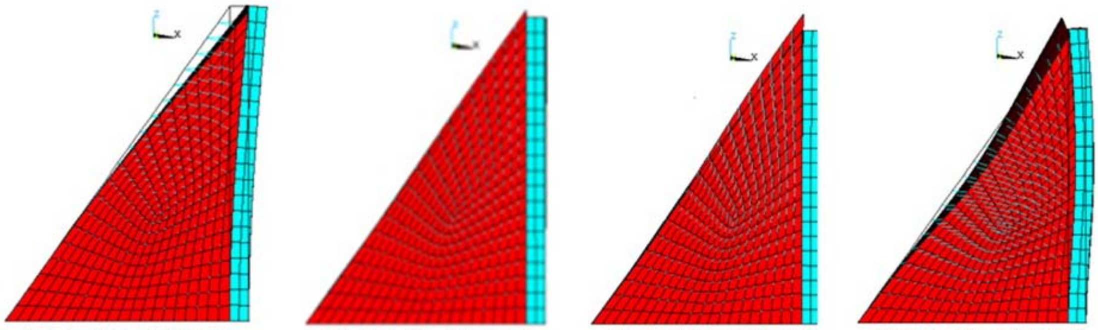
three degrees of freedom at each node: translations in the nodal *x*, *y*, and *z* directions. Backfill soil behind the exterior wall of the tank interacts with wall in compression, but it is assumed that there is no interaction in tension. Then, the unidirectional nonlinear element is used having very rigid compression characteristics with tensionless in interaction face of the backfill-wall system. Furthermore, no bending or torsion is considered, and the vertical friction between the wall and backfill is ignored. Mathematical details of modeling of the bounded media can be found on another study of the author (Livaoglu and Dogangun 2007).

As known, the simulation of the infinite medium in the numerical method is an extremely important topic for the dynamic soil-structure interaction problems. Although the backfill soil under consideration may be finite in some cases, the dimension perpendicular to the wall may be larger than the dimension of the wall for the other circumstances. More appropriate approximations include utilization of the artificial and/or transmitting boundaries. Furthermore, reflecting and radiation effects of propagating waves from the structure-foundation layer may be avoided by means of these types of boundaries. There are different types of boundaries in frequency or time domain with different sensitivities (Wolf and Song 1996). Firstly, Lysmer and Kuhlemeyer (1969) developed a viscous boundary using one-dimensional beam theory. This theory has been commonly used with the FEM (Livaoglu and Dogangun 2007). In this study, viscous boundary is used in three dimensions to consider radiational effect of the seismic waves through the soil medium in the direction perpendicular to the normal of wall (Fig. 5).

To analyze all aspects of the seismic response of backfill-wall system is currently impossible due to complex interacting phenomena and the inherent variability and uncertainties of soil properties. In addition, soil behavior is highly sensitive in general when it is exposed to an earthquake-induced motion. Thus, elasto-plastic and/or perfectly plastic behavior of the backfill soil are frequently observed in the SSI, especially for the system subjected to the lateral force or system excited by seismic actions. Lateral responses are generally the most important parts of the SSI. In view of all reasons mentioned above, Drucker-Prager material model is used in modeling of the backfill soil medium in this study.

3.1.1 Numerical results for backfill-exterior wall system

The dynamic characteristics of backfill-exterior wall system were obtained from the modal analyses by using the ANSYS finite element structural analysis program. The first four frequencies and corresponding mode shapes were shown in Fig. 6. As can be seen in this figure, the frequencies of modes were calculated as 2.96, 5.14, 8.34 and 13.34 Hz, respectively. Since combining the response contributions of all the modes gives the total response of the system, the response contributions of all the modes should obviously be included in order to obtain the exact value of the response, but few modes can usually provide sufficiently accurate results and it may not be necessary to include the contributions of all the modes in computing the response. Actually, the modes which have relatively larger modal contribution factors or effective modal masses are more efficient on the response of the system under consideration. So, it can be easily stated that the first four modes obtained from finite element model proposed here can be evaluated as sufficient in representing the response since the 65%, 1%, 2% and 21% of the total mass are represented by first, second, third and fourth mode, respectively.



Mode 1 (f = 2.96 Hz) Mode 2 (f = 5.14 Hz) Mode 3 (f = 8.34 Hz) Mode 4 (f = 13.34Hz)

Fig. 6 The finite element mode shapes and corresponding frequencies of backfill-wall system (TANK_A)

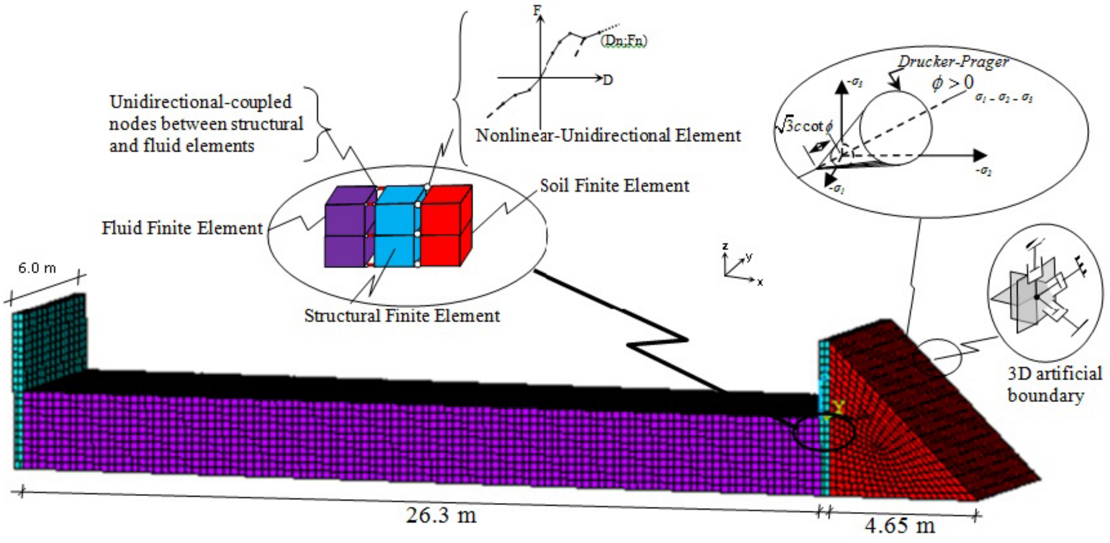


Fig. 7 The finite element model of backfill-exterior wall-fluid system for front face (TEST_1)

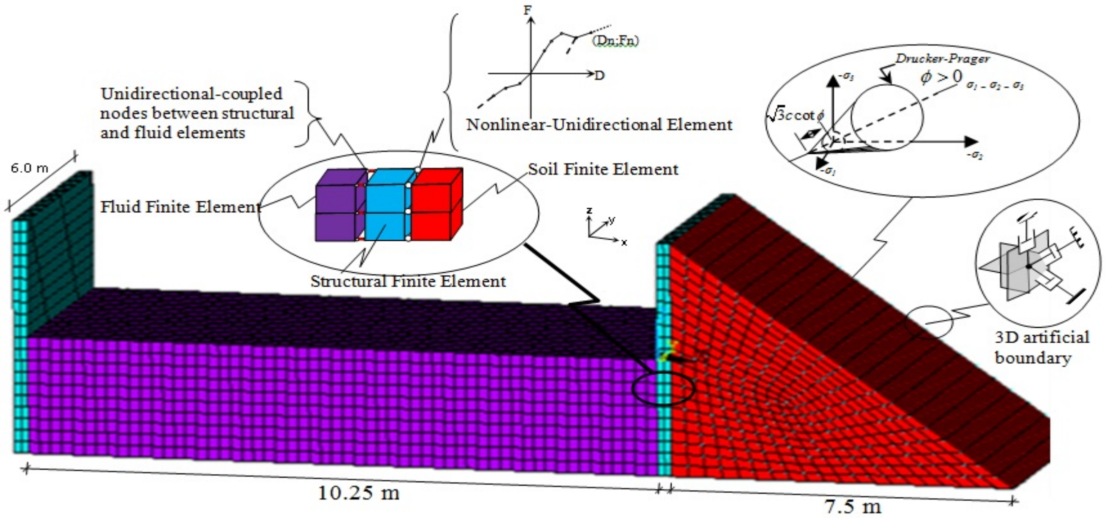


Fig. 8 The finite element model of backfill-exterior wall-fluid system for side face (TEST_2)

3.2 Numerical models for backfill-exterior wall-fluid system

The finite element models of backfill-exterior wall-fluid system for both front face and side face of the tank are shown in Figs. 7 and 8, respectively. The models proposed for front face and side face of the tank are named as TEST_1 and TEST_2, respectively. The details regarding the finite element models were given in previous section. In addition to the details, the fluid within the container is also modeled. The fluid elements (FLUID80) were specially formulated to model fluid within container having no net flow rate. Mathematical details of modeling of fluid can be found on another study of the author (Livaoglu and Dogangun 2007).

3.2.1 Numerical results for backfill-exterior wall-fluid system

The dynamic characteristics of backfill-exterior wall-fluid system for front face (TEST_1) were obtained from the modal analyses by using the ANSYS finite element structural analysis program.

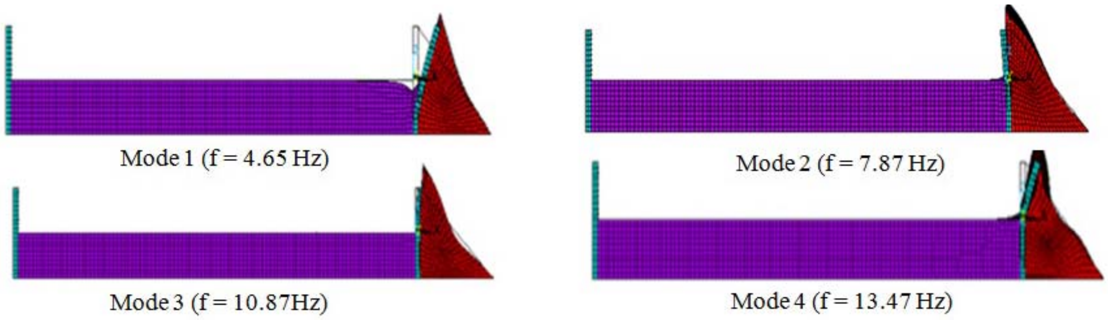

Mode 1 (f = 4.65 Hz)
Mode 2 (f = 7.87 Hz)
Mode 3 (f = 10.87Hz)
Mode 4 (f = 13.47 Hz)

Fig. 9 The finite element mode shapes and corresponding frequencies of backfill-wall-fluid system for front face

The first four frequencies and corresponding mode shapes were shown in Fig. 9. As can be seen in this figure, the frequencies of modes were calculated as 4.65, 7.87, 10.87 and 13.47 Hz, respectively. As mentioned before, the modes which have relatively larger modal contribution factors or effective modal masses are more efficient on the response of the system. So, it can be easily stated that the first four modes obtained from finite element model proposed here can be evaluated as sufficient in representing the response since the 60%, 10%, 9% and 14% of the total mass are represented by first, second, third and fourth mode, respectively.

Similarly, the dynamic characteristics of backfill-exterior wall-fluid system for side face (TEST_2) were obtained from the modal analyses by using the ANSYS finite element structural analysis program. The first four frequencies and corresponding mode shapes were shown in Fig. 10. As can be seen in it, the frequencies of modes were calculated as 3.40, 7.39, 9.23 and 11.56 Hz, respectively. The modes which have relatively larger modal contribution factors or effective modal masses are more efficient on the response of the system. So, similar to above-mentioned examples, it can be easily stated that the first four modes can be evaluated as sufficient in representing the response since the 53%, 6%, 21% and 5% of the total mass are represented by first, second, third and fourth mode, respectively.

4. Modal testing

4.1 Testing equipment

During the experimental studies, the electrodynamic shaker with a force capacity of 250 N, which induces different types of motion as sinusoidal, random etc., was used as a vibration source to produce shaking force in a frequency range of practical importance of 20-150 Hz. Besides, three seismic accelerometers (Dytran 3100D24 5 g range) were employed to obtain generated values that are stored in computer by using signal calculator program, and a four channel Dynamic Signal Analyzer (Data Physics Quattro) was used to digitize and record time histories. The cooling system was utilized to cool the electrodynamic shaker by means of blower. A GW-300 W power

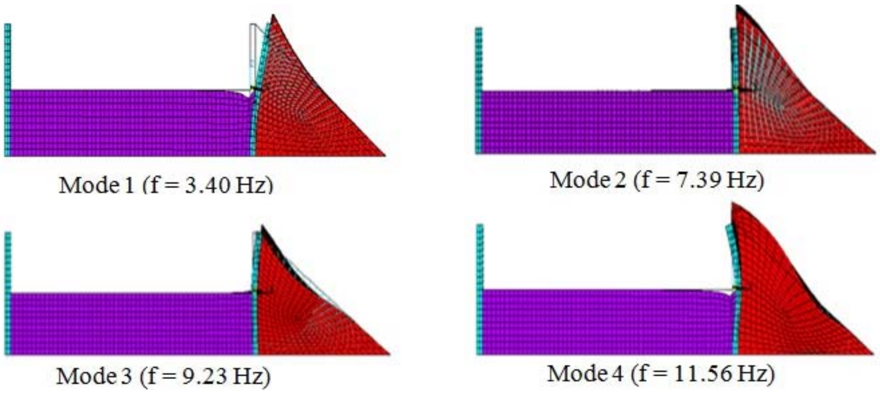

Mode 1 (f = 3.40 Hz)
Mode 2 (f = 7.39 Hz)
Mode 3 (f = 9.23 Hz)
Mode 4 (f = 11.56 Hz)

Fig. 10 The finite element mode shapes and corresponding frequencies of backfill-wall-fluid system for side face

(a) Shaker system (shaker, cooling and power amplifier)

(b) Data analyzer

(c) Accelerometer

(d) 300 W power amplifier

Fig. 11 The configuration of testing equipments used in the field tests

amplifier was operated to provide excitation for electrodynamic shaker during the signal processing. The configuration of testing equipments mentioned above was given in Fig. 11.

4.2 General test setup and testing procedure

The views of the electrodynamic shaker and seismic accelerometers mounted on identified locations of the exterior wall of the TANK_A, both front and side exterior walls of the TANK_B were shown in Figs. 12-14, respectively. Since two different tests were carried out on exterior walls of the TANK_B, the tests performed on front and side exterior walls are named as TEST_1 and TEST_2, respectively. As seen from the figures, both the seismic accelerometers and the electrodynamic shaker system were located on the maximum elevation of the rectangular tank walls, and not only the shaker force but also all measurements were oriented in the direction perpendicular to the wall face. As emphasized before, the electrodynamic shaker system which is particularly efficient in producing dynamic forces even at low frequencies and can be used for dynamic testing of a number of structures including retaining and rectangular tank walls was used to provide shaking force within the desired range of frequencies. Thus, the electrodynamic shaker was designed to induce sinusoidal motions in a frequency range of 20-150 Hz during the signal processing. The accelerations of both backfill-wall system (TANK_A) and backfill-wall-fluid systems (TANK_B) were measured by three seismic accelerometers mounted on identified locations, and the signals obtained from accelerometers were collected in the four channels Dynamic Signal Analyzer. Then, the recorded signals were analyzed by the Dynamic Signal Analyzer in order to digitize and record time histories and the desired modal parameters were obtained.

4.3 Test results

In view of the above explanations, two series of measurements were carried out to determine the modal frequencies of the system under consideration. In the first series, accelerations were measured within the frequency range of 0-20 Hz, and all the acceleration data were recorded up to the frequency of 20 Hz over the 40 s. In the second one, accelerations were measured within the frequency range of 0-10 Hz, and all the acceleration data were recorded up to the frequency of 10 Hz over the 80 s. Since ten measurements of accelerations which are consecutively taken from each of three different channels are sufficient for the purposes of seismic design, the exponential averages of the obtained measurements were given. Converting the acceleration records taken from each channel from time domain to frequency domain via Fast Fourier Transform (FFT), the

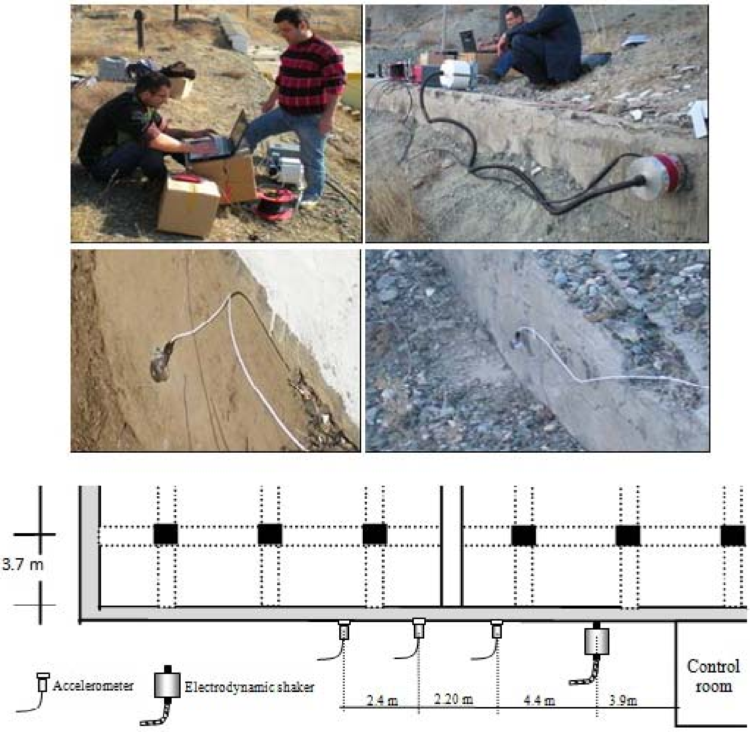


Fig. 12 The general test setup and instrumentation on the exterior wall of TANK_A

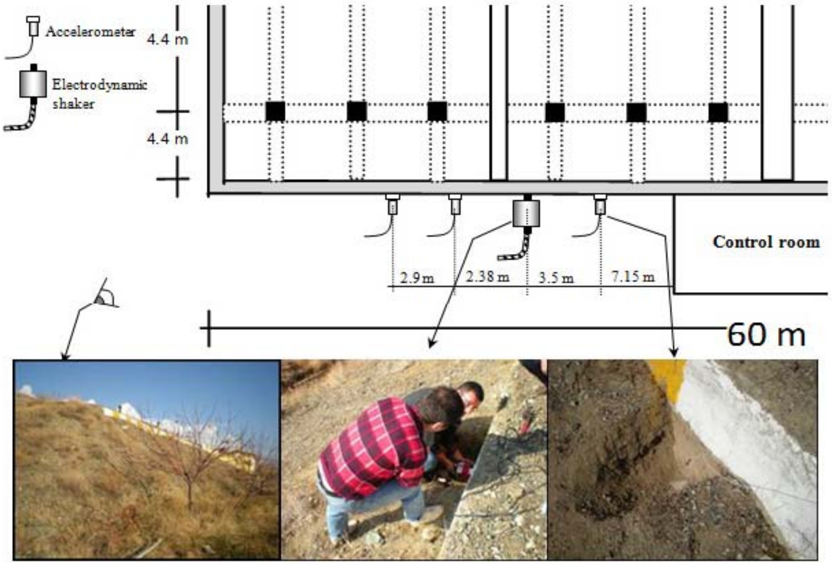


Fig. 13 The general test setup and instrumentation on front exterior wall of TANK_B (TEST_1)

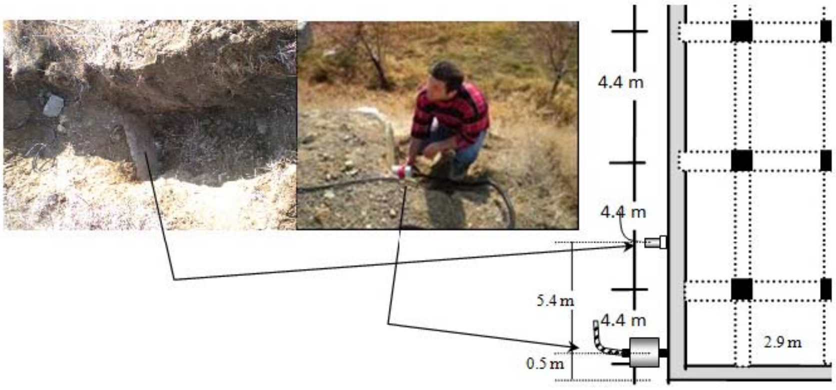


Fig. 14 The general test setup and instrumentation on side exterior wall of TANK_B (TEST_2)

power spectrums were obtained. It is worth emphasizing that the accelerometer signals were digitally filtered to remove the presence of any extraneous random excitation noise or interference effects such as the traffic and/or machine-induced vibrations before the calculation of corresponding power spectrums.

4.3.1 Test results conducted on TANK_A

The variation of power spectrums obtained from each of three channels and their enhanced arithmetic averages were given in Figs. 15 and 16 for the frequency range of 0-20 Hz and 0-10 Hz, respectively. As can be seen in these figures, the mode frequencies of the system under investigation were obtained as 3.11, 5.63, 8.48 Hz for the frequency range of 0-10 Hz, and 3.07, 5.65, 8.55, 13.55 Hz for the frequency range of 0-20 Hz, respectively.

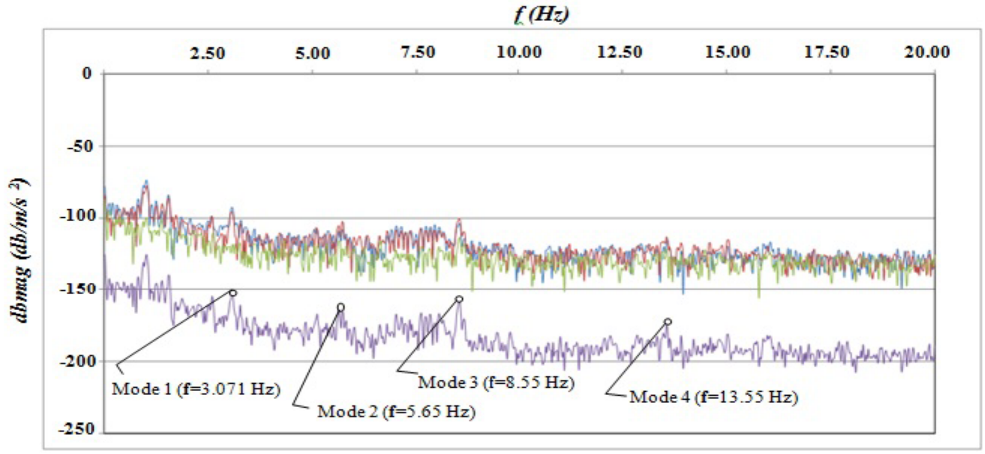


Fig. 15 The variations of power spectrums and their enhanced arithmetic averages for the frequency range of 0-20 Hz (f_{Sine} = 25 Hz)

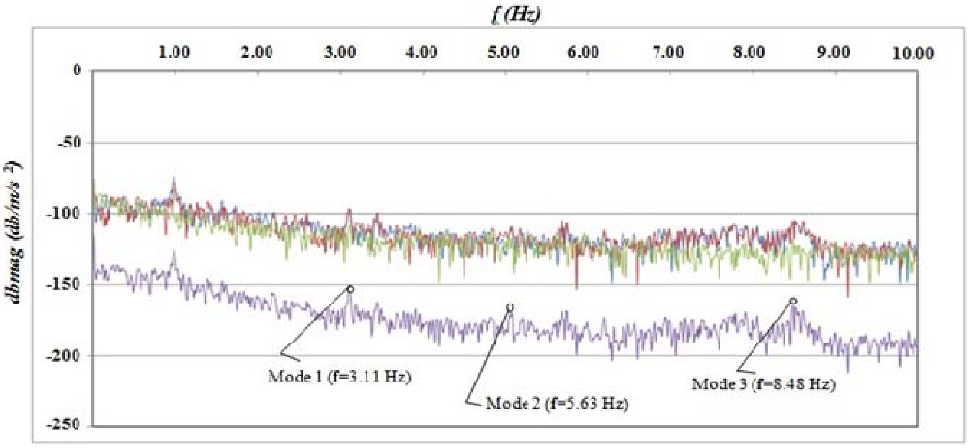


Fig. 16 The variations of power spectrums and their enhanced arithmetic averages for the frequency range of 0-10 Hz (f_{Sine} = 25 Hz)

4.3.2 Test results conducted on TANK_B

The variation of power spectrums obtained from each of three channels mounted on identified locations for TEST_1 at an excitation frequency of 25 Hz and their enhanced arithmetic averages for the frequency range of 0-20 Hz were given in Fig. 17. Furthermore, the variation of power spectrums obtained from a channel mounted on identified location for TEST_2 at three different excitation frequencies of 20, 25, 50 Hz and their enhanced arithmetic averages for the frequency range of 0-20 Hz were given in Fig. 18. As can be seen from Figs. 17 and 18, the mode frequencies of the systems examined were obtained as 4.30, 7.35, 10.30, 12.45 Hz for TEST_1, and 3.65, 7.75, 9.35, 11.30, 16.5 Hz for TEST_2. Moreover, it is clearly seen that not only the measurements acquired at different excitation frequencies but also the measurements taken from different channels coincide.

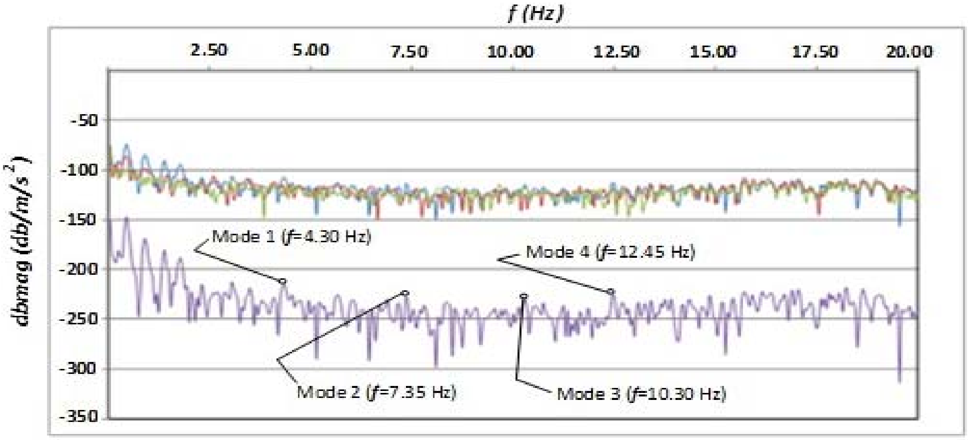


Fig. 17 The variations of power spectrums and their enhanced arithmetic averages for TEST_1 (f_{Sine} = 25 Hz)

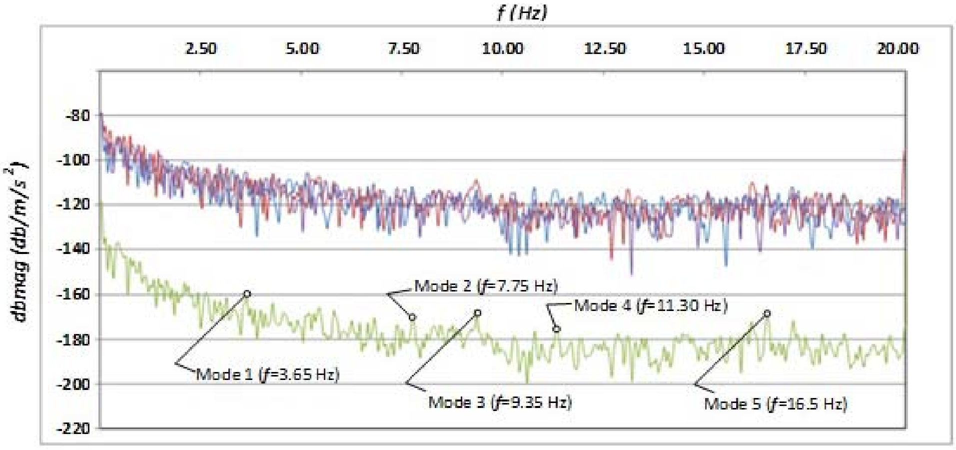


Fig. 18 The variations of power spectrums and their enhanced arithmetic averages for TEST_2 (f_{Sine} = 20 Hz, f_{Sine} = 25 Hz, f_{Sine} = 50 Hz)

5. Evaluation of numerical and test results

The modal frequencies predicted by a three dimensional finite element model developed by using ANSYS commercial package program and corresponding frequencies measured by in-situ tests conducted on wall-backfill system (TANK_A) were given in Table 1. As shown in this table, the experimental frequencies are a little bit higher than those of theoretical for all modes and the comparison between the predicted and measured dynamic quantities resulted in an overall average error of about 4%. Similarly, considering the backfill-wall-fluid system (TANK_B), the modal frequencies obtained by means of finite element techniques and in-situ tests for TEST_1 and TEST_2 were given in Tables 2-3, respectively. It is clearly seen from these tables that the overall average errors for TEST_1 and TEST_2 are approximately 7% and 4%, respectively. Indeed, these reflect successful predictions knowing that there can be some uncertainties and difficulties

Table 1 Comparison of numerical and experimental modal frequencies for TANK_A

Mode Number	Modal Frequencies (Hz)		Error (%)
	Numerical Model (FEM)	Test	Mode Number
1	2.96	3.07	3
2	5.14	5.65	9
3	8.34	8.55	2
4	13.34	13.55	2

Table 2 Comparison of numerical and experimental modal frequencies for front exterior wall of TANK_B

Mode Number	Modal Frequencies (Hz)		Error (%)
	Numerical Model (FEM)	TEST_1	Error (FEM)
1	4.65	4.30	8
2	7.87	7.35	7
3	10.87	10.30	5
4	13.47	12.45	8

Table 3 Comparison of numerical and experimental modal frequencies for side exterior wall of TANK_B

Mode Number	Modal Frequencies (Hz)		Error (%)
	Numerical Model (FEM)	TEST_2	Error (FEM)
1	3.40	3.65	7
2	7.39	7.75	5
3	9.23	9.35	1
4	11.56	11.30	2

encountered in the tests and approximations and drawbacks in the proposed numerical models. So, it can be easily said that in these types of structures under investigation, the calculated errors are negligible from the engineering point of view due to complex interacting phenomena and the inherent variability and uncertainties of soil properties. Moreover, it is worth emphasizing that the safe estimation of material, mechanical and geometrical properties of reinforced concrete rectangular tank under consideration are remarkably efficient on results obtained.

6. Conclusions

In this study, both the finite element modeling and vibration testing of the wall of the rectangular tank-backfill systems located in Erzincan (NE Turkey) were presented. In reality the rectangular tanks are three dimensional structures. On the other hand, there are several parts in a rectangular tank since these structures were constructed as segmental, not monolithic in practice. Therefore, each part of the structure is subjected to different loads, and exhibits different behaviours. For example, while the exterior walls are subjected to backfill interaction in one side and fluid interaction in the other side, the interior walls are subjected to fluid interactions in both sides. Thus, only the exterior walls of the rectangular tanks which interact with both the backfill and fluid were considered, as each part of the structure shows considerable differences in terms of both the load bearing mechanisms and the geometrical and positional differences. The proposed three dimensional finite element models (3D-FEM) of the considered systems were analyzed by using the ANSYS structural analysis program from which the mode frequencies and shapes were determined. The forced vibration tests were also conducted on the exterior wall-backfill and fluid-exterior wall-backfill systems in the field, and mode frequencies were determined experimentally. Consequently, based on the results of this study, the main conclusions include the following:

A total of four mode frequencies were obtained from the FEM of the systems under investigation. These modes which are extracted from among a great number of modes can be evaluated as sufficient because their contributions to total response are approximately 90% or over this value in all analyses. Therefore, one may say that only four modes can be adequate to estimate the total response of such a system investigated in this study.

The frequency results which are obtained experimentally provided a high degree of confidence in the validity of the measured data since not only satisfactory coherence between measurements for the frequency range of 0-20 Hz and the measurements for the frequency range of 0-10 Hz taken from each of three different channels but also the coincidence in measurements acquired at different excitation frequencies were always achieved.

The results of the vibration tests provide strong support for the finite element models presented in this paper. Thus, it can be easily stated that the proposed finite element models themselves are the meritorious approximations to the real problem, and this makes the models appealing for use in comprehensive investigations. Using same model approximation directly or one of the modal analysis procedures for fluid-wall-backfill systems, researchers can easily estimate the seismic response.

When comparing the numerical and experimental results, there is little differences in the frequencies of modes. Thus, the differences between theory and experiment can be evaluated as negligible from the engineering point of view. Nevertheless, it must be stated here that researchers and/or designers have to define the soil and structure material properties in their studies via sensible precautions.

Acknowledgements

The present work is supported by Grant-in-Aid for Scientific Research (Project No.105M252) from the Scientific and Technological Research Council of Turkey (TÜBİTAK).

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