Seismic Behaviour of Tunnel Form Building Structures: An Experimental Study

S. Eshghi, A. Tavafoghi
Int. Inst. of Earthquake Engineering and Seismology (IIEES), Tehran, Iran.

15 WCEE
LISBOA 2012

SUMMARY:
Multi-story reinforced concrete (RC) tunnel form buildings have been increasingly employed for mass construction industry in many countries. This system is very attractive for the medium to high-rise buildings with repetitive plans due to satisfactory performance during past earthquake, industrialized modular construction technique, low cast and also saving in construction time. Recent studies show that the current seismic codes and guidelines do not provide sufficient requirements for seismic design of these structures. An experimental study was carried out to decrease the uncertainty of linear and nonlinear behaviour of the tunnel form buildings. The experimental study included the testing of two three-story 1/5-scale of the tunnel form building. Some of forced vibration tests were done and both specimens were subjected to quasi-static cyclic lateral loading. The results were used to estimate the period of mode shape and to evaluate the fundamental period of cracked structures. The nonlinear behaviour of specimens and failure mechanisms were captured and compared to a FE model.

Keywords: Tunnel Form Buildings, Seismic Behaviour, Forced Vibration, Shear Wall, Concrete

1. INTRODUCTION

Tunnel forms came about as the result of a need for affordable housing dating back to the years following World War II. Demand for affordable single family residences and apartments caused Guy Blonde, technical director for Outinord, a small start-up manufacturer based in France, to come up with the idea of tunnel forms in the early 1950s. The system saved money and reduced the time to build structures because workers formed both walls and decks in one operation.

With today's refinements, tunnel forming systems are ideal for projects that offer repetitive forming opportunities, the more repetitive steps there are, the greater the benefits. These systems are ideally suited for the construction of multi-unit housing, single-family residences, hotels, townhouses, military housing, prisons, and some warehouse applications.

Tunnel form is a modular construction method that utilizes in-situ concrete poured into two steel half-tunnel forms that together form the walls, ceilings and floors of the building. Reinforcement and service conduits can be placed within the moulds as necessary, prior to pouring the concrete and openings for stairwells and interconnecting doors can also be formed. Each 24 hours, the formwork is moved so that another tunnel can be formed. When a story has been completed, the process is repeated on the next floor.

The main components of this system are walls and flat plate slabs. Walls in tunnel form buildings have two functions: resisting lateral loads as well as carrying vertical loads. Tunnel form buildings diverge from other conventional reinforced concrete (RC) structures due to lack of beams and columns in their structural components. In these buildings, all of the vertical load carrying members are made of shear walls and floor system. In addition, in tunnel form buildings, walls and slabs are made of thin concrete plates and rebars should be placed in one layer. So the confinements of concrete and ductility level cannot be defined like other conventional structures.

The seismic codes usually recommend the traditional force based design methodology. The fundamental period of structure, mode shapes and behaviour factor (R factor) are used to compute the design base shear of conventional building structures. Recent studies showed that current seismic codes did not address fundamental period and R factor for tunnel form buildings clearly, in spite of the fact that these parameters are directly used to compute the design base shear.

Also, the dynamic behaviour in linear or nonlinear state of the structures under the seismic excitation is of major interest to structural engineers. Prior to occurrence of an earthquake, it is important to determine the structural characteristics such as natural frequencies, mode shapes and failure mechanism.

In addition, there is a lack of experimental work to understand the three-dimensional (3D) response of tunnel form buildings under extreme lateral loading conditions. Previous experimental studies conducted on shear-wall systems were generally limited to two-dimensional (2D) investigations. However, it was analytically proven that the 2D approach is not adequate to capture important behaviour of tunnel form buildings under seismic action due to significant slab-wall interaction and global tension and compression (T/C) coupling effects (Balkaya and Kalkan 2003, 2004).

2. RECENT STUDIES

Studies about the tunnel form buildings mainly include two design parameters; a) Accurate estimation of the fundamental period of this type of building and b) Reliable response modification factor.

Different authors studied estimation of the fundamental period of the tunnel form buildings. Goel and Chopra (1998) compared the database information with the code formulation. The database on the fundamental period of buildings measured from their motion recorded during several California earthquakes. They showed that code formulas were inadequate.

Lee et al (2000) studied another database of fifty apartment buildings, which were recorded from 1998 to 1999. They have introduced another formula for the fundamental period.

Balkaya and Kalkan (2002) suggested a new formula for estimation of the fundamental period of tunnel form building structures. They developed the formula using three dimensional finite element analysis of 16 different plans for five different building heights (2, 5, 10, 12, 15 stories). Two years later, they modified the suggested formula by using new database. New database was consisting of 20 different buildings with 7 different heights (5, 10, 12, 15, 18, 20 and 25 stories). Shear wall thickness was taken 12 cm for buildings up to 15 stories, 15 cm for 18 story and 20 cm for 20 and 25 story buildings.

Another study was reported by Tavafoghi and Eshghi in (2008) showed that the formulas based on building height according to ASCE/SEI 7-05 are simpler and more accurate than the suggested formulas based on the other parameters.

Second important parameter about the tunnel form buildings is the seismic behaviour and response modification factor of these buildings. Previous studies showed that there is no full scale numerical analysis or experimental test to estimate the R factor of the tunnel form buildings.

Balkaya and Kalkan (2003) have studied the differences between 2D and 3D modeling in capacity–demand curve and effect of transverse walls and slab-wall interaction in the 3D behaviour. The static push over analysis has been carried out on a typical plan in 2 and 5 stories. In 2004, they have proposed the R factor by some analysis on this plan. The value of the response modification factor evaluated by the period-dependent ductility factor (R_μ), period- dependent overstrength factor (R_S) and redundancy factor (R_R).

An experimental study was done by Yüksel and Kalkan (2006) to obtain a better understanding of the 3D behavior of tunnel-form buildings. Four-story 1/5 scale specimens were tested under quasi-static reversed cyclic loading. A minimum amount of reinforcement was utilized without boundary reinforcement in the shear walls. The experimental results showed that structural walls of tunnel-form buildings with low axial stress might exhibit brittle flexural failure under cyclic loading. This failure took place due to rupture in longitudinal reinforcement with no crushing of concrete. Nonlinear FE models of test specimens were created by using the general purpose FE program, TNO DIANA. The results of these analyses generally showed a reasonable matching with the results of experiments. Load displacement curves which were developed by analytical and experimental results, had good fitting. In addition, analysis could successfully predict cracking pattern.

Again, Yüksel and Kalkan (2007) developed the same 3D FE models of test specimens. They studied the effects of the vertical reinforcement ratio of shear walls on response of the models and the failure mechanism. A parametric study was done including variation of the vertical reinforcement ratio from 0·0015 to 0·01 by pushover analysis on an 1/5 scale test specimens.

A primary study to evaluate more accurate R factor of the tunnel form building by Tavafoghi and Eshghi in 2010 was done. In this study, a typical building with 5 stories has modeled and the value of R factor was evaluated for this building according to ATC-63.

3. DESCRIPTION OF EXPERIMENTAL STUDIES

Based on the reviewed literature, the only experimental study was carried out by Yüksel and Kalkan (2007). So, an experimental study carried out to study the nonlinear behaviour of the tunnel form buildings, crack patterns in the slabs and shear walls and failure mechanisms. The results can be used for further numerical studies. Also some forced vibration tests were done to estimate the period of mode shapes and to evaluate the fundamental period of cracked and uncracked structures.

Our experimental study included the testing of two three-story 1/5-scale of the tunnel form building as shown in 'Fig. 3.1'. The specimens were the representatives of a typical tunnel form building. Both specimens had identical dimensions, reinforcement detailing and material properties. The material properties of steel reinforcements and concrete were not scaled. The size of aggregate; diameter and spacing of reinforcements were reduced for considering the scaling effects. The specimens were constructed on separate foundations that connected to the strong floor by 24 high strength bolts. The reinforcements detailing of foundation, shear walls and slabs are shown in 'Fig 3.2'.

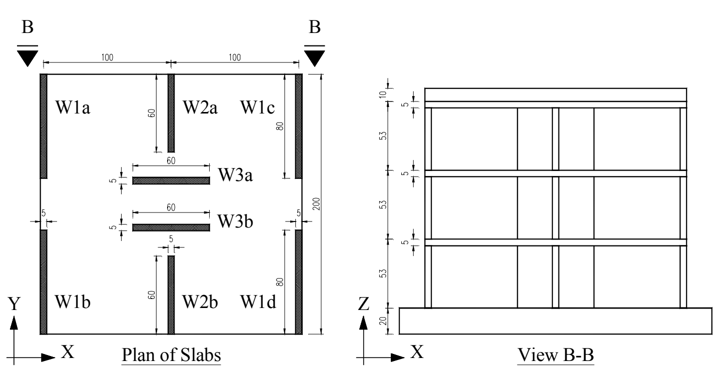

Figure 3.1. Plan and elevation view of experimental studies. (Units are in cm)

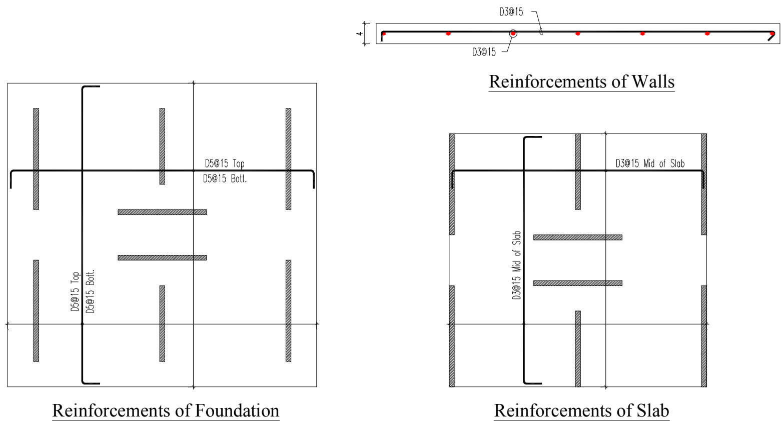

Figure 3.2. The reinforcements detailing of foundation, shear walls and slabs.

At the first the instruments of forced vibration tests were mounted on the specimen No. 1. The experiments have been carried out using the shakers and accelerators. The two shakers were mounted on 3rd floor of the building in a position to deliver horizontal force to the structure. The shakers and accelerometers can be located on the structure where natural vibration of modes gives maximum response. So, three accelerometers were mounted on each floor to capture horizontal and rotational response of building in each story (nine accelerometers totally). Three accelerometers were used on the top of the foundation to record the response of strong floor. The locations of shakers and accelerometers and data acquisition system are shown in 'Fig. 3.3' and 'Fig. 3.4'.

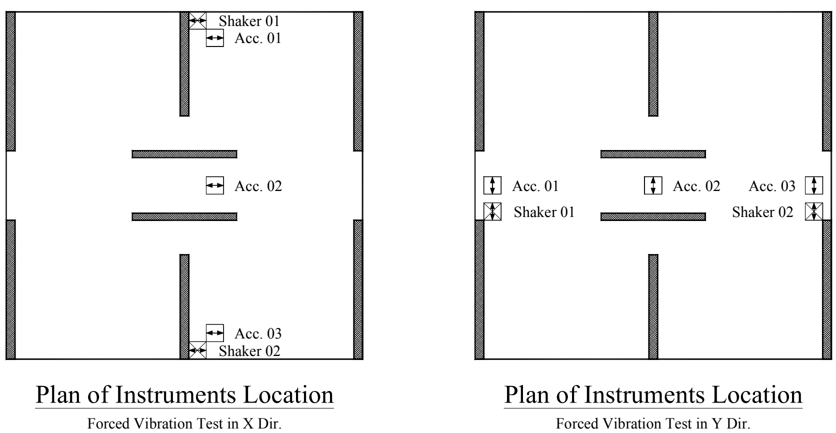

Figure 3.3. Locations of instruments mounted on the 3rd floor of specimens.

Figure 3.4. 3D view of test setup for forced vibrations of specimens.

A series of tests were done on the specimen No. 1. The Shakers forced the structure in X direction to excite translational responses. The two shakers were placed within the edges of the structure, close to the shear walls, and the forcing frequencies were ranging from 1.0 Hz to 100 Hz with the step of 1.0 Hz. For the first series of tests, both of shakers forced the structure with the same phase. Also, in the second series of tests, the shakers forced the specimen with the 90 degrees phase delay to mobilize the torsional modes in the structures. These two series of tests were done on the specimen No. 1 to estimate the fundamental period of longitude and torsional mode shapes.

After the forced vibration tests, the specimen No. 1 was subjected to quasi-static cyclic lateral loading. The loading history was in terms of the number of cycles versus roof displacement and applied in X direction as shown in 'Fig 3.5'.

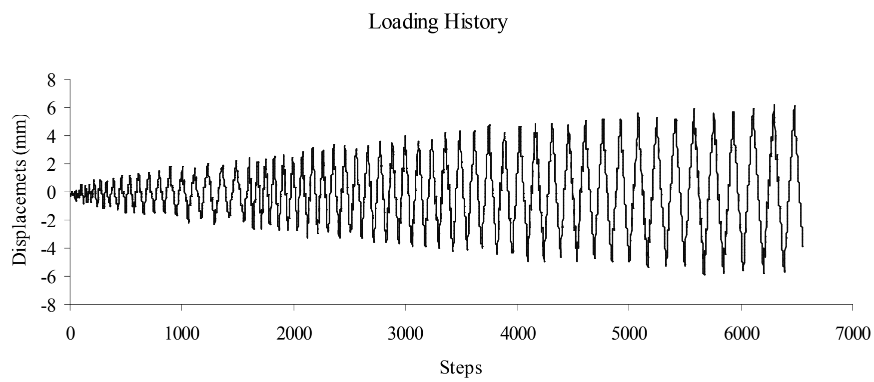

Figure 3.5. Loading history in cyclic test.

The same sets of tests were done on the specimen No. 2. To Study the effects of the additional mass, each floors of specimen were subjected to 50Kg per meter by using cast iron pieces. The forced vibration tests were done with and without additional mass like specimen No.1. Before complete cyclic lateral loading on specimen No.2, the structure was subjected to quasi-static cyclic lateral loading until 5mm roof displacement. In this state some cracks was appears on the wall-slab connections and on the walls near the foundation. Again, some forced vibration tests were done to obtain the dynamic characteristics of cracked structure. The complete cyclic lateral loading on specimen No.2 was done after this series of forced vibration tests.

4. THE RESULTS OF CYCLIC LOADING TESTS

After carrying out the experiments, the following results were obtained. The crushing of concrete was not observed and the damage was occurred on the shear walls, shear wall – slab connections and slabs. As soon as the tensile stress in the concrete exceeded the tensile strength, the cracking took place and the concrete immediately released the tensile force to reinforcements. For both specimens, longitudinal steel was unable to carry the tensile force. Therefore concrete cracked, longitudinal reinforcements yielded and ruptured suddenly. The damage was concentrated on the bottom of shear walls near the foundation, all shear wall – slab connections and first story slab (slab punching happened around the W3a, W3b shear walls) as shown in 'Fig. 4.1'.
The behaviour of the specimens was simulated by 3D finite element model using TNO DIANA program to verify numerical model responses against experimental results.

For Concrete, a crack model was employed which had capabilities of opening, closing and rotating. The stress–strain relationships were evaluated in the principal directions of the strain vector. The hardening-softening curve that modified by Thorenfeldt et al (1987) for unconfined concrete, was used in compression behaviour. The tension stiffening of concrete was considered as a linear curve up to cracking limit and the tension softening curve was based on the model proposed by Hordijk et al

(1986). Softening behavior is related to mode-I fracture energy (G_f), ultimate tensile strength (f_t) and crack bandwidth (h_{cr}).

Figure 4.1. Damages at the walls above the foundation, wall – slab connections and first story slab.

The behavior of the reinforcing steel was modeled using the Von-Mises plasticity model with an associated flow law and isotropic strain hardening. A perfect bond was assumed, and steel nodes were rigidly attached to concrete element nodes. The reinforcement strains are computed from the displacement field of the mother elements. Stress–strain behavior of the steel was modeled using a bilinear relationship.
The material properties using in numerical simulation is described through Table 4.1 to Table 4.2 and four FE models are selected to study overall behaviour of the specimens.

Table 4.1. Reinforcements and concrete properties used in FE numerical models of experimental program. (Units are in SI).

Reinforcements		**Concrete**	
Young's Modulus	1.87E+11	Young's Modulus	2.3E+10
Poisson's Ratio	0.30	Poisson's Ratio	0.20
F_y	2.8E+8	Tensile Stress	1.8E+6
F_u	4.3E+8	Compressive Stress	3.0E+7
ε_u	0.1	Mode I Tensile Fracture Energy	Table 4.3 (G_f)
		Crack Band Width	Table 4.3 (h)

Table 4.2. Mode I tensile fracture energy and Crack band width used in four FE numerical models. (Units are in SI).

FE Model	G_f	h
N0. 1	25	1
N0. 2	50	1
N0. 3	75	1
N0. 4	100	1

The specimens were modeled using 8 nodes shell elements. A 2x2 Gauss integration scheme in elements was used in FE models. The envelope curves contain the maximum load at each displacement level. 'Fig. 4.2' shows the cyclic test results for specimen No. 2 and the envelope curve. Envelope load-deflection curves were obtained from FE models and compared to the experimental results in 'Fig 4.3'. The computed responses of FE models were somewhat stiffer and stronger than the experimental results. Comparison of results showed that the analytical No.2 model reasonably captured the ultimate force and displacement of the test specimens.

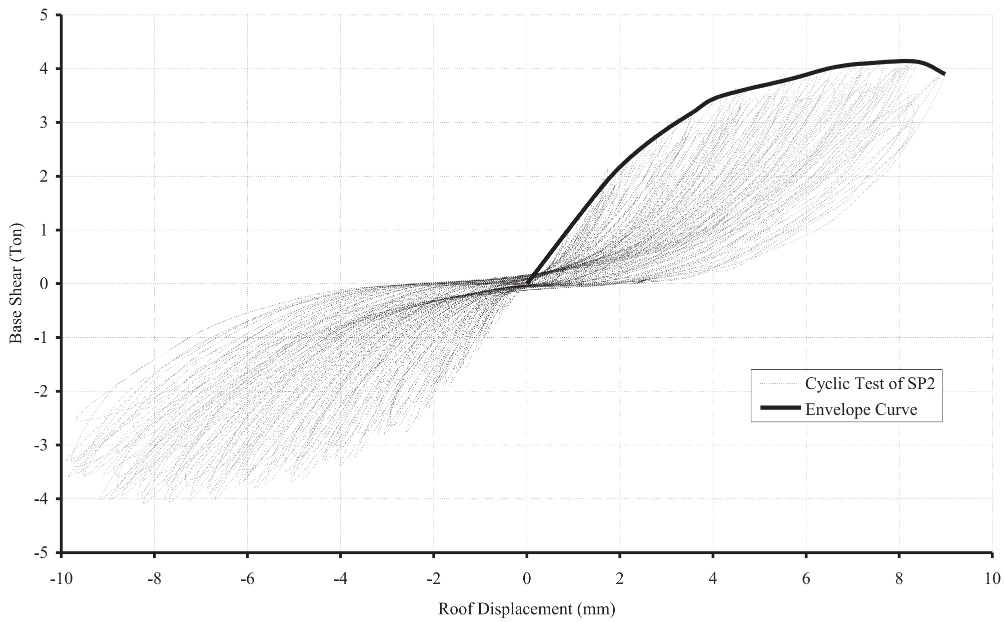

Figure 4.2. Load-deflection curves of cyclic test (SP2).

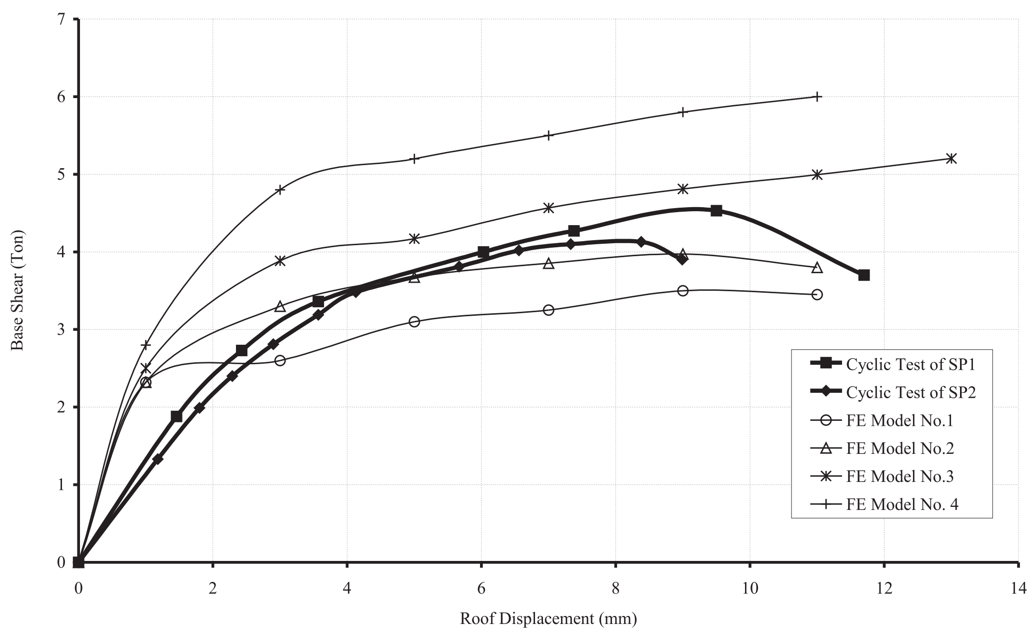

Figure 4.3. Load-deflection curves of experimental studies.

5. THE RESULTS OF FORCED VIBRATION TESTS

In the Civil Engineering field, there are two methods for system identification: Input-Output and Output-Only Modal Identification techniques. There is presently a wide variety of input-output modal identification methods which can be obtained through the inverse Fourier Transform. These methods try to perform some fitting between measured and theoretical functions and employ different optimization procedures and different levels of simplification.

Also, there are two main groups of output-only modal identification methods: nonparametric methods essentially developed in frequency domain and parametric methods in time domain. The basic frequency domain method (Peak-Picking) was systematized by Felber about twelve years ago. This

approach, which leads in fact to estimates of operational mode shapes, is based on the construction of power spectral densities (PSD). In the PSD diagram, the frequency with high PSD leads to the main frequency of the mode shapes.
The results of forced vibration tests were studied using Peak-Picking method to estimate the natural frequency of specimens and compared to the numerical simulation. For example, the results obtained from FV-SP1-X-NM-S1 test, has shown 27.9 Hz as the main natural frequency for specimen No. 1 throw ‘Fig. 5.1’.

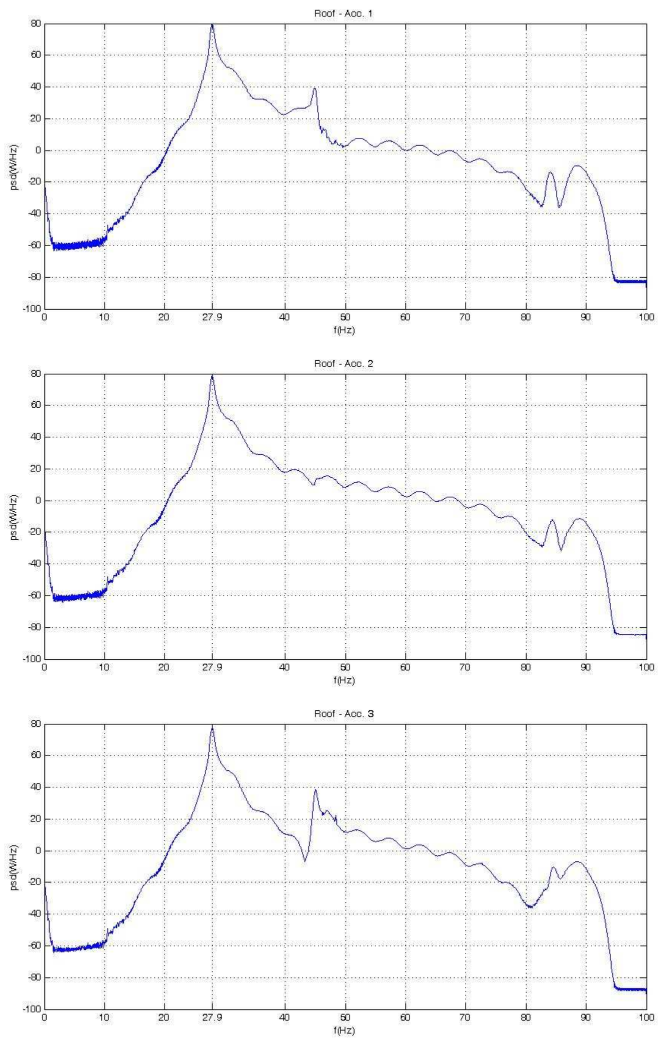

Figure 5.1. Power spectral density obtained from accelerometers in FV-SP1-X-NM-S1 test.

The comparison between experimental results and numerical simulation were shown in Table 5.1 to Table 5.2.

Table 5.1. Natural frequency of specimens in X direction (No added Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP1-X-NM-S1	Frequency sweep (1 to 100 Hz), SP1	27.9
FV-SP1-X-NM-S2	Frequency sweep (1 to 100 Hz), SP1	28.0
FV-SP2-X-NM-S1	Frequency sweep (1 to 100 Hz), SP2	29.1
FV-SP2-X-NM-S2	Frequency sweep (1 to 100 Hz), SP2	29.1
FV-SP1-X-NM-H1	Impulse Hummer , SP1	28.7
FV-SP1-X-NM-H2	Impulse Hummer , SP1	28.7

Test Name	Descriptions	Hz
FV-SP2-X-NM-H1	Impulse Hummer , SP2	29.8
FV-SP2-X-NM-H2	Impulse Hummer , SP2	29.8
Numerical Simulation		**28.5**

Table 5.2. Natural frequency of specimens in torsional direction (No added Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP1-X-NM-ST1	Frequency sweep (1 to 100 Hz), SP1	44.1
FV-SP1-X-NM-ST2	Frequency sweep (1 to 100 Hz), SP1	44.0
FV-SP2-X-NM-ST1	Frequency sweep (1 to 100 Hz), SP2	38.4
FV-SP2-X-NM-ST2	Frequency sweep (1 to 100 Hz), SP2	38.3
FV-SP1-X-NM-H3	Impulse Hummer , SP1	44.7
FV-SP1-X-NM-H4	Impulse Hummer , SP1	44.6
FV-SP2-X-NM-H3	Impulse Hummer , SP2	39.3
FV-SP2-X-NM-H4	Impulse Hummer , SP2	39.3
Numerical Simulation		**47.1**

Table 5.3. Natural frequency of specimens in Y direction (No added Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP2-Y-NM-S1	Frequency sweep (1 to 100 Hz), SP2	39.7
FV-SP2-Y-NM-S2	Frequency sweep (1 to 100 Hz), SP2	39.8
FV-SP2-Y-NM-H1	Impulse Hummer , SP2	39.5
FV-SP2-Y-NM-H2	Impulse Hummer , SP2	39.6
Numerical Simulation		**50.0**

Table 5.4. Natural frequency of specimens in X direction (Additional Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP2-X-AM-S1	Frequency sweep (1 to 100 Hz), SP2	26.5
FV-SP2-X-AM-S2	Frequency sweep (1 to 100 Hz), SP2	26.7
FV-SP2-X-AM-H1	Impulse Hummer , SP2	27.2
FV-SP2-X-AM-H2	Impulse Hummer , SP2	27.2
Numerical Simulation		**26.0**

Table 5.5. Natural frequency of specimens in torsional direction (Additional Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP2-X-NM-ST1	Frequency sweep (1 to 100 Hz), SP2	35.1
FV-SP2-X-NM-ST2	Frequency sweep (1 to 100 Hz), SP2	35.1
FV-SP2-Y-NM-ST1	Frequency sweep (1 to 100 Hz), SP2	36.1
FV-SP2-Y-NM-ST2	Frequency sweep (1 to 100 Hz), SP2	36.0
FV-SP2-X-NM-H3	Impulse Hummer , SP2	35.5
FV-SP2-X-NM-H4	Impulse Hummer , SP2	35.2
Numerical Simulation		**42.8**

Table 5.6. Natural frequency of specimens in Y direction (Additional Mass – Uncracked Structures)

Test Name	Descriptions	Hz
FV-SP2-Y-NM-ST1	Frequency sweep (1 to 100 Hz), SP2	36.4
FV-SP2-Y-NM-ST2	Frequency sweep (1 to 100 Hz), SP2	36.5
FV-SP2-Y-NM-H1	Impulse Hummer , SP2	37.4
FV-SP2-Y-NM-H2	Impulse Hummer , SP2	37.5
Numerical Simulation		**45.4**

As the results showed, the natural frequency of specimens captured in X direction, have good fitting with the numerical results. But in other direction and torsional frequency, the numerical results have some differences that may cause by construction tolerance.

6. CONCLUSION

In this paper, an attempt is made to evaluate the seismic behavior of the tunnel form buildings based on experimental studies. An experimental study included two three-story 1/5-scale of the tunnel form building, was done. Some of forced vibration tests were done and both specimens were subjected to quasi-static cyclic lateral loading. The results were used to estimate the period of mode shapes and to evaluate the fundamental period of cracked structures. The numerical simulation results have good fitting in evaluation of first mode in X Direction but estimated period in Y direction and torsional mode had some differences with experimental results. The nonlinear behaviour of specimens and failure mechanisms were captured and compared to a FE model. Numerical model showed good adaptation with test results in failure mechanism. The specimens had brittle failure mechanism that occurred in the shear walls above the foundation connections. The rebars of shear walls ruptured suddenly and crushing of the concrete were not observed.

REFERENCES

ACI. (2005). Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary, (ACI 318R-05), American Concrete Institute, Farmington Hills, Michigan.

ASCE. (2005). Minimum Design Load for Buildings and other Structures, ASCE Standard ASCE/SEI 7-05, American Society of Civil Engineers.

ATC. (2009). Quantification of Building Seismic Performance Factor, Report No. ATC-63, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C.

Balkaya, C., Kalkan, E. (2003). Estimation of fundamental periods of shear wall dominant building structures. *Earthquake Engineering and Structural Dynamics.* **32,** 985–998.

Balkaya, C., Kalkan, E. (2004). Seismic vulnerability, behavior and design of tunnel form buildings. *Engineering Structures.* **26:14**, 2081–2099.

Cornelissen, H. A. W., Hordijk, D. A., and Reinhardt, H. W. (1986). Experimental determination of crack softening characteristics of normal weight and lightweight concrete. *Heron* **31:2**, 45-56.

Eshghi, S., Tavafoghi, A. (2010). Estimating a Response Modification Factor (R) for Tunnel Form Concrete Building Structures. *14ECEE*. Paper ID:733.

Goel, RK., Chopra, AK. (1998). Period formulas for concrete shear wall building. *Journal of Structural Engineering. ASCE* **124(ST4)**, 426-433.

Hordijk, D.A. (1991). Local Approach to Fatigue of Concrete, PhD thesis, Delft University of Technology.

Kalkan, E., Yuksel, S.B. (2007). Pros and cons of multistory RC tunnel form Buildings. *Structural Design of Tall Buildings*. **9:4**, 295-307.

Lee, L., Chang, K., Chun, Y. (2000). Experimental formula for the fundamental period of RC buildings with shear wall dominated systems. *Structural Design of Tall Buildings*. **9:4**, 295-307.

Tavafoghi, A., Eshghi, S. (2008). Seismic behavior of tunnel form buildings. *14WCEE*. Paper ID:14_05-03-0175.

Thorenfeldt, E., Tomaszewicz, A. and Jensen, J. J. (1987). Mechanical properties of high-strength concrete and applications in design. *In Proc. Symp. Utilization of High-Strength Concrete, Stavanger, Norway.*

TNO DIANA. (2007). Finite Element Analysis User Manuals Version 9.2, Delft, The Netherlands,.