

## SEISMIC VULNERABILITY OF ELEVATED WATER TANKS USING PERFORMANCE BASED-DESIGN

F. Omidinasab<sup>1</sup> and H. Shakib<sup>2</sup>

<sup>1</sup>Structures Engineering Ph.D. Candidate, Dept. of Civil Engineering, Tarbiat Modarres Uni., Tehran, Iran

<sup>2</sup>Structure and Earthquake Faculty Professor, Dept. of Civil Engineering, Tarbiat Modarres Uni., Tehran, Iran  
Email: [omidi@modares.ac.ir](mailto:omidi@modares.ac.ir), [shakib@modares.ac.ir](mailto:shakib@modares.ac.ir)

### ABSTRACT :

Liquid tanks and especially the elevated tanks are structures of high importance which are considered as the main lifeline elements that should be capable of keeping the expected performance. i.e. operation during and after earthquakes. Thus, researchers, in recent years, have focused on studying the seismic behavior of these tanks. Many researches have been done on the behavior, analysis, and design of seismic tanks, particularly ground tanks, while only a few of these researches have concerned with the elevated tanks and even less with the reinforced concrete elevated tanks. In this research, a sample of a reinforced concrete elevated water tank, with 900 cube meters under seven earthquake records have been studied and analyzed in dynamic time history and the tank's responses including base shear, overturning moment, tank displacement, and sloshing displacement under these seven record have been calculated, and then the results have been compared and contrasted.

**KEYWORDS:** Elevated water tank, Vulnerability, Performance-based design

### 1. INTRODUCTION

Elevated liquid tanks and especially the elevated water tanks are considered as important city services in many cities. Their safety performance during strong earthquakes is of critical concern. They should not fail after earthquake, so that they can be used in meeting essential needs like preparing drinking water and putting out fires. The failure of these structures and the subsiding of water may cause some hazards for the health of city due to the shortage of water or difficulty in putting out fire during critical conditions. Many studies concentrated on the seismic behavior, analysis, and design of tanks, particularly ground tanks. In the past decade most of these studies have focused on the elevated tanks. In the past earthquakes elevated tanks have been of the vulnerable structures and their seismic behavior has not been convenient being damaged. Thus, past earthquakes have shown that due to failure of lifeline structures, such as elevated tanks with insufficient seismic resistance, fire fighting and other emergency response efforts can be hindered (e.g., experiences from Chile 1960, 1978 Izu-Oshima and Miyagi, 1971 San Fernando, and 1987 Whittier earthquakes). There have been numerous studies analyzing and investigating the dynamic behavior of fluid storage tanks, however, most of these studies have focused on the ground level cylindrical tanks. Very few studies have concentrated on the behavior of elevated tanks. Therefore, the attention is generally focused on the dynamic behavior of the fluid and/or the support structure. Most studies investigating the behavior of elevated tanks are summarized below. Haroun and Ellaithy developed a model including an analysis of a variety of elevated rigid tanks undergoing translation and rotation. The model considers fluid sloshing modes and it assesses the effect of tank wall flexibility on the earthquake response of the elevated tanks [1]. Resheidat and Sunna investigated the behavior of a rectangular elevated tank considering the soil-foundation structure interaction during earthquakes. They neglected the sloshing effects on the seismic behavior of the elevated tanks and the radiation damping effect of soil. Haroun and Temraz analyzed models of two-dimensional X-braced elevated tanks supported on the isolated footings to investigate the effects of dynamic interaction between the tower and the supporting soil-foundation system but they also neglected the sloshing effects [2]. Marashi and Shakib carried out an ambient vibration test for the evaluation of the dynamic characteristics of elevated tanks [3]. Dutta et al. studied the supporting system of elevated tanks with reduced torsional vulnerability and they suggested approximate empirical equations for the lateral, horizontal and torsional

stiffness for different frame supporting systems. Dutta et al. also investigated how the inelastic torsional behavior of the tank system with accidental eccentricity varies with increasing number of panels. Subsequently, Dutta et al. showed that soil-structure interaction (SSI) could cause an increase in base shear particularly for elevated tanks with low structural periods. This study also concluded that ignoring the effect of SSI could result in potential large tensile forces in some of staging columns due to seismic loads. Livaoglu and Dogangun proposed a simple analytical procedure for seismic analysis of fluid-elevated tank-foundation-soil systems, and they used this approximation in selected tanks [4]. Livaoglu conducted a comparative study of seismic behavior of the elevated tanks considering both fluid-structure and soil-structure interaction effects on elevated tanks [5]. Seismic designs of these tanks are done on the basis of different countries well-known creditable codes like IBC, UBC and ACI. There is no certainty about the convenient performance of these structures during earthquakes due to their complexities and therefore more studies are needed in this regard.

## 2. DESCRIPTION OF THE ELEVATED TANK

A reinforced concrete elevated tank with a container capacity of 900 m<sup>3</sup> in Turkey is considered in the seismic analysis (Fig. 1 and 2) [5]. The elevated tank is supported by a frame structure in which the columns are connected by the circumferential beams at a height of 7, 14 and 20 m above ground. Since the intze type tank container has an optimal load balancing shape and it is widely preferred, it is selected here. This type of container and supporting structure has been extensively used in Turkey until recent years. Supporting system of the tank is elastic and it contains beams and columns located on a truncated cone. Radius of the cone's bottom base is 6.375 m and radius of the upper base is 4.30 m. The horizontal section of the tank's base is a regular octagon with 8 columns per level located on the vertex of this octagon. Its support bending frame has three stories. The first and second stories have 7 m and the third storey has 6m height. The dimensions of square column sections are 120 cm and the dimensions of beam sections of the first and second storey are 120x60 cm, the circumferential beams of the third storey under the container are 80x120 cm. The details and the elevation of the tank are shown in fig.1. Arrangement of the columns and beams under the tank container, and also their arrangement on the foundation are respectively illustrated in fig. 2(a) and 2(b). The tank's container consists of a truncated cone with a height of 23.5 m and a cylinder with a 6m radius and a 7.05 m height with 40 cm thickness, and an arch-slab roof of 1.7 m height. Tank's roof is a cone with 12.81m radius, 170 cm height and 20 cm thickness.

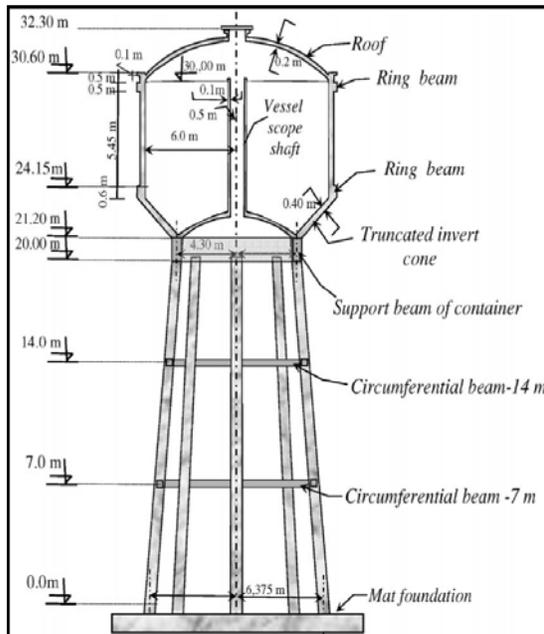


Figure 1 Details and elevation of the tank

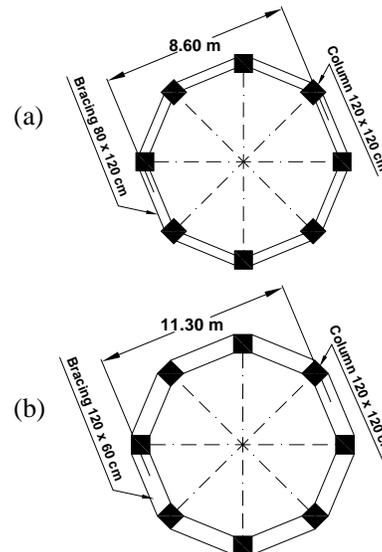


Figure 2 (a) Arrangement of the columns and beams under the tank container; (b) Arrangement of the columns and beam on the first storey

## 3. MODELING

A finite element model (FEM) is used to model the elevated tank system. Columns and beams in the support system are modeled as beam elements (with six degrees-of-freedom per node) and the truncated cone and container walls are modeled with quadrilateral shell elements (with four nodes and six degrees-of-freedom per node). The fluid elements are defined by eight nodes with three translational degrees-of-freedom at each node. Fluid-structure interaction problems can be investigated by using different techniques such as added mass (AM), Lagrangian (LM), Eulerian (EM), and Lagrangian-Eulerian (L-E M) approaches in the finite element method (FEM) or by the analytical methods like Housner's two-mass representation or multi-mass representations of Bauer and EC-8 [6].

In this research, displacement based Lagrangian approach is selected to model the fluid-elevated tank interaction. The fluid elements are defined by eight nodes with three translational degrees-of-freedom at each node. It should be noted that, because of lack of a geometrical capability in the Lagrangian FEM with brick shaped elements considered here, intze-type is idealized as a cylindrical vessel that has same capacity with the intze type. The brick fluid element also includes special surface effects, which may be thought of as gravity springs used to hold the surface in place. This is performed by adding springs to each node, with the spring constants being positive on the top of the element. Gravity effects must be included if a free surface exists. For an interior node, the positive and negative effects cancel out [5]. The positive spring stiffness can be expressed as:

$$K_s = \rho A_f (g_x C_x + g_y C_y + g_z C_z) \quad (3.1)$$

where  $\rho$  is the mass density,  $A_f$  is the area of the face of the element,  $g_i$  and  $C_i$  acceleration in the  $i$  direction and  $i$ th component of the normal to the face of the element, respectively. Expressions for mass ( $M_f$ ) and rigidity matrices ( $K_f$ ) of fluid are given below :

$$M_f = \rho \int_v Q^T Q dV = \rho \sum_i \sum_j \sum_k \eta_i \eta_j \eta_k Q_{ijk}^T Q_{ijk} J_{ijk} \quad (3.2)$$

$$K_f = \int_v B^T E B dV = \sum_i \sum_j \sum_k \eta_i \eta_j \eta_k B_{ijk}^T E B_{ijk} \det J_{ijk} \quad (3.3)$$

where  $J$  is the Jacobian matrix,  $Q_{ijk}$  is the interpolation function,  $\eta_i, \eta_j, \eta_k$  are the weight functions,  $B$  is the strain-displacement matrix obtained from  $\varepsilon = Bu$ , where kinetic (T) and potential energy equations (U) can be written as :

$$U = \Pi_\varepsilon \rightarrow U = \frac{1}{2} u^T K_f u \quad (3.4)$$

$$T = \frac{1}{2} v^T M_f v \quad (3.5)$$

If the expressions for the kinetic and potential energies are substituted into Lagrange equation, then :

$$\frac{d}{dt} \left( \frac{\partial T}{\partial \dot{u}_j} \right) - \frac{\partial T}{\partial u_j} + \frac{\partial U}{\partial u_j} = F_j \quad (3.6)$$

where  $u_j$  is the  $j$ th displacement component and  $F_j$  is the applied external load, the governing equation can be written as:

$$M_f \ddot{u} + (K_f + K_s) u = R \quad (3.7)$$

where  $\ddot{u}$  is the acceleration and  $R$  is a general time varying load vector.

Mechanical properties considered for the steel and concrete are given in table 3.1. Performing the linear modal analysis, the tank's dynamic properties consisting the period and the mode mass participation ratio are obtained and illustrated in table 3.2. Sum of the structure's first six modes partnership is more than 90 percent. First to third modes are related to convective and forth to sixth modes are associated with impulsive modes.

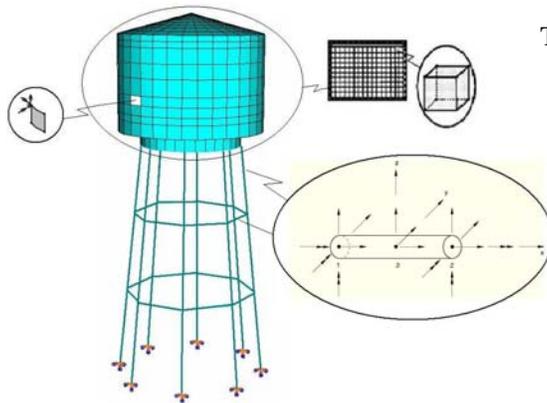


Fig 3 Finite element model of the fluid-elevated tank system considered in this study

Table 3.1 Steel, concrete, and water properties

Steel	parameters	Dimension
$E_{Steel}$	$2.03 \cdot 10^6$	$kg/cm^2$
$f_y$	4000	$kg/cm^2$
$f_u$	5000	$kg/cm^2$
Weight Volume unit	7850	$kg/m^3$
Concrete	parameters	Dimension
$E_{Concrete}$	$2.53 \cdot 10^5$	$kg/cm^2$
$f'_c$	300	$kg/cm^2$
Weight Volume unit	2400	$kg/m^3$
Water	parameters	Dimension
Density	1000	$kg/m^3$
Bulk Module	2.10	$GPa$

Table 3.2 Modal properties of the tank in filled, half filled, and empty states

	Mode	1	2	3	4	5	6
<b>Filled</b>	T (sec)	3.68	2.16	1.87	1.03	0.74	0.15
	P (%) <sup>†</sup>	<b>5.10</b>	<b>3.8</b>	<b>1.6</b>	<b>51.7</b>	<b>22.20</b>	<b>10.50</b>
<b>Half-filled</b>	T (sec)	4.26	2.34	1.43	0.95	0.09	0.12
	P (%)	<b>8.11</b>	<b>6.1</b>	<b>2.00</b>	<b>45.30</b>	<b>5.60</b>	<b>25.80</b>
<b>Empty</b>	T (sec)	0.00	0.00	0.00	0.14	0.11	0.08
	P (%)	<b>0.00</b>	<b>0.00</b>	<b>0.00</b>	<b>76.20</b>	<b>13.20</b>	<b>5.30</b>

<sup>†</sup> Modal partnership mass ratio in percent

#### 4. MODELING NONLINEAR BEHAVIOR OF MATERIAL

Concrete is a material that its behavior has a significant difference. Many researchers attempted to present a mathematical model of this type of materials on the basis of experimental results. Many concrete behavioral models have been obtained by the researchers. Their most famous and applicable ones are Park and Kent models [7]. In 1972 Kent and Park presented a mathematical relation on stress-strain behavior of reinforced concrete (square cross sections confinement by strip) and in 1982 Scott et al. revised it. Due to the precedence of the revised Kent and Park relation by Scott in 1982, many researchers used this relation in their bending frames [7]. Considering the properties of the sections of beams and columns and the concrete resistance of that equals  $300 \text{ kg/cm}^2$  in this research, and also considering Kent and Park model, the curve and concrete behavior model under stress and strain are obtained as in fig. 4. Since the concrete covering is not confined in the reinforced concrete section, it will have a different strain-stress curve that is considered for the current concrete curve that is not surrounded and given in fig. 4.

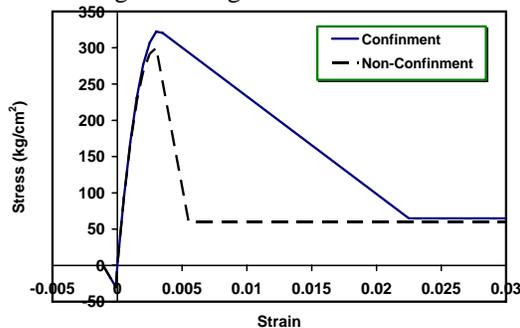


Figure 4 Concrete strain-stress curve in the obtained strain and stress behaviors based on Kent & Park relation

Table 5.1 Features of the used records

Number	Record	Year	Component	PGA (g)	PGV (cm/s)	PGD (cm)	Duration (sec)	M
1	Duzce, Turkey	1999	BOL - 00	0.728	56.4	23.07	55.90	7.1
			BOL - 90	0.822	62.1	13.55	55.90	7.1
2	Landers	1992	YER - 270	0.245	51.5	43.81	44.98	7.3
			YER - 360	0.152	29.7	24.69	44.98	7.3
3	Loma Prieta	1989	G04 - 00	0.417	38.8	7.09	40.95	6.9
			G04 - 90	0.212	37.9	10.08	40.95	6.9
4	Morgan Hill	1984	G02 - 00	0.162	5.1	1.42	30.98	6.2
			G02 - 90	0.212	12.6	2.1	30.98	6.2
5	Northridge	1994	CNP - 106	0.356	32.1	9.13	25.98	6.7
			CNP - 196	0.420	60.8	20.17	25.98	6.7
6	Supersitt Hills	1987	B-CAL - 225	0.18	15.5	3.3	23.24	6.7
			B-CAL - 315	0.247	14.6	3.1	23.24	6.7
7	Whittier Narrows	1987	A-CAS - 00	0.332	27.1	5.04	32.16	6.0
			A-CAS - 270	0.333	14.1	1.48	32.16	6.0

## 5. EVALUATION OF CAPACITIES AND DEMANDS

In this part the demands of structure elements in hazard level 1 and 2 and in full, half full and empty modes of the tank using linear static methods, linear dynamic and nonlinear dynamic has been calculated and obtained. In the following, the expected capacities of the structural elements have been calculated and finally the criteria of acceptance of the structural members including control parameters by force and variation have been studied.

### 5.1. Evaluation Of Demands

In order to meet the demands, linear static methods, linear dynamic and nonlinear dynamic methods were used. The purpose of the evaluation of the elevated water tanks is that since they are located as effective and important members of the critical lifelines of the society. Thus, they are selected specifically so that they can represent functional level of immediate occupancy (IO) against hazard level 1 earthquake (with a return period of 475 years) and the functional level of finite against hazard level 2 (with a return period of 2475 years). UBC-9 range for four regions and soil type c are considered for hazard level 1. Whereas, for the hazard level 2 ranges, it is assumed 1.5 times more than hazard level 1.

For evaluation of dynamic response of elevated tanks, three modes of filled, half-filled, and empty have been considered. History analysis has been done using the above said equations. Moreover, Rayleigh attenuation has been applied in this analysis. In dynamic analyses, earthquake records have been inserted simultaneously and in 100 percent in two horizontal directions horizontally located on the tanks. For performing a historical nonlinear analysis considering that, the studied tank is located in soil type c according to UBC-97 divisions, 7 types of records are used in this type of soil. The features of these records have been stated on table 5.1. According UBC-97 code, in order to scale the records, it has been done on the basis of the structure's period between 0.2T and 1.5T.

For example, horizontal components Duzce earthquake acceleration are presented in figure 5. Besides, response ranges of horizontal components of Loma Prieta earthquake have been illustrated in figure 6 with scaled response range. According to table 5.1, the maximum PGA by gravity acceleration for records related to Duzce, which equals to 0.822. The maximum PGV by m/s belongs to Duzce record that equals to 62.1 m/s.

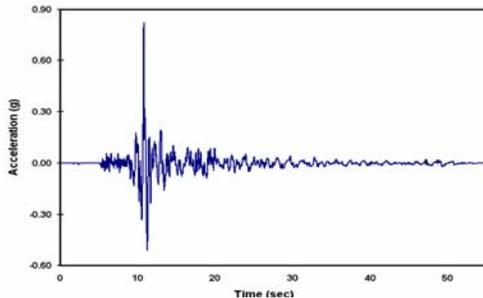


Figure 5 Horizontal component of Duzce-90 earthquake acceleration

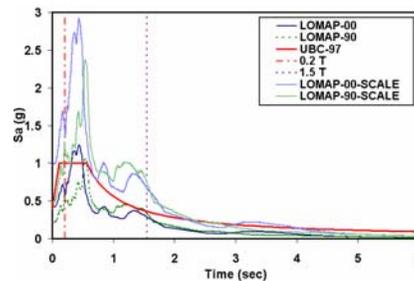


Figure 6 Response spectrum of Loma Prieta earthquake horizontal components in of 5% damping, Scaling and UBC spectrum

In figure 7 to 9 show some of the results of dynamic analysis of 7 records, average results, and results plus and minus standard deviation.

In linear static analysis method of the base shear force in each extension of the structure is calculated through the following equation as a coefficient of the total weight of the structure:

$$V = C_1 \cdot C_2 \cdot C_3 \cdot C_m \cdot S_a \cdot W \quad (5.1)$$

Where w as the total weight of the structure includes dead load and a percentage of the live load or snow load (100% of this load for tanks) and Spectral Acceleration ( $S_a$ ) in lieu of the structure's fundamental period (T).  $C_1$ ,  $C_2$ ,  $C_3$ , and  $C_m$  have been considered according to FEMA regulations. Structure responses for hazard level 1 and 2 for filled, half filled and empty modes of the tank are shown in figure 10 to 14[8,9].

According to FEMA, since the structure has one or more common columns between 2 or more lateral loading system frames in different directions. Therefore, the simultaneous effect of the earthquake components should be considered. In order to carry this effect into an account, the analysis method being linear, earthquake effect in all direction is summed with 30% of earthquake effect in vertical direction[8].

According to FEMA-356, the behavior of the efforts in members are divided into two categories of behaviors

controlled by deformation and behaviors controlled by force. This guideline presents different load combinations for these two behaviors including 16 combinations of dead, live and earthquake loadings. As well as for columns in order to control the axial forces in this element due to the lateral and gravitational loads. Vertical members of the structure's subjected lateral load-bearing system should be controlled considering effects of the overturning moment. In such a case, the actuator force in the element rises from the earthquake loading in due to the concentrated mass on the top of structures. Restoring forces against overturning are the structure's dead load and tensile forces in columns. In the linear dynamic analysis method, of the structures subjected to Duzce, Landers and Loma Preita the maximum response are presented in figures 10 to 14. In order to evaluate the vulnerability of the system and control the response acceptance criteria, the mean response of the ensemble record are considered and the results are compared with the linear as well as the nonlinear dynamic analysis as illustrated shown in figures 10 to 14.

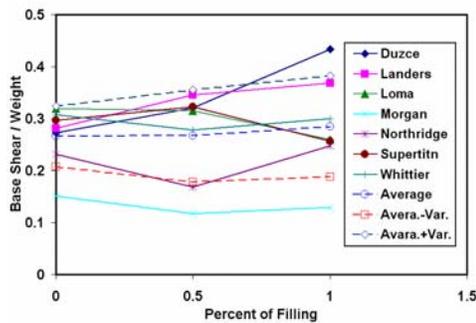


Figure 7 Base shear changes based on the percent of filling

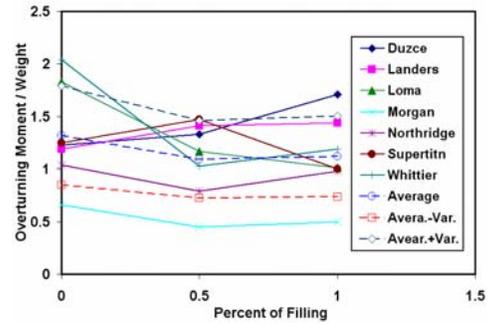


Figure 8 Overturning moment changes based on filling percent

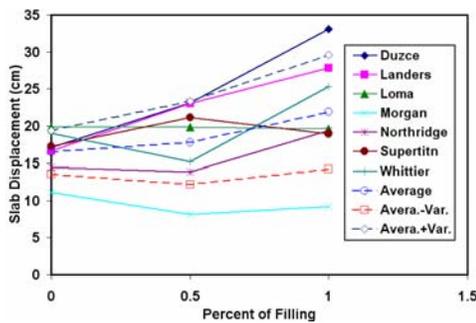


Figure 9 Slab displacement changes based on filling percent

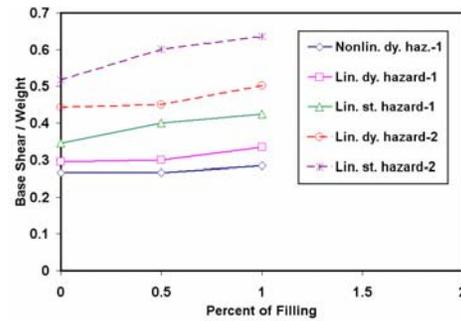


Figure 10 Base shear changes based on the percent of filling

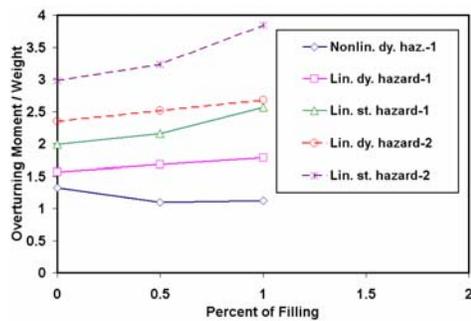


Figure 11 Overturning moment changes based on filling percent

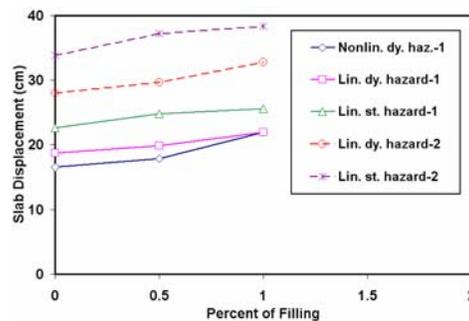


Figure 12 Slab displacement changes based on filling percent

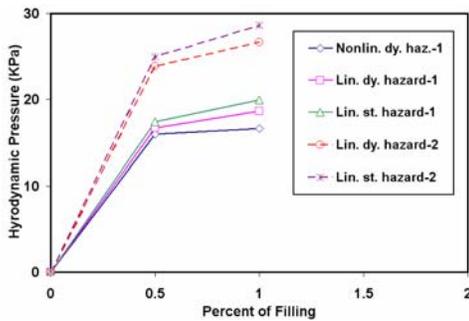


Figure 13 The hydrodynamic pressure changes in cylindrical wall based on filling percent

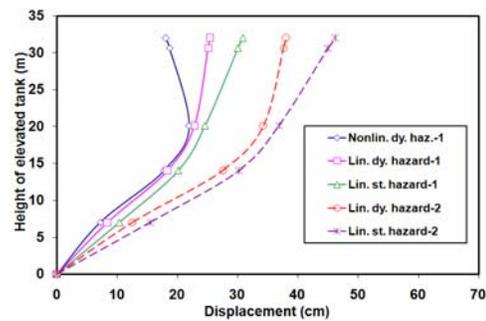


Figure 14 Maximum displacement calculated in elevated tank height in filled mode

### 5.2. Evaluation Of Capacities

The capacity of the each system elements contain columns, beams, cylindrical wall and bottom slab are estimated base on the there behavior mode of falure and given in table 5.2.

Table 5.2 Evaluation of capacities

Number of Story	Element	Q <sub>CE</sub>	
		V (KN)	M (KN.m)
1	Beam-120x80	652.9	1651.3
	Column-120x120	-----	1861.8
2	Beam-120x80	652.9	1721.9
	Column-120x120	-----	45.78
3	Beam-80x120	831.2	1731.6
	Column-120x120	-----	1861.8
Vessel	Cylindrical Wall	1521.1	851.5
	Bottom Slab	-----	170.4

Table 6.1 Acceptance criterion control of *m* in linear static method in hazard level 2

Element	Acceptable Criteria			Analysis		
	IO	LS	CP	IO	LS	CP
Beam-1	2	3	4	2.1	2.8	4.3
Beam-2	2	3	4	1.95	2.5	3.8
Beam-3	2	3	4	1.83	2.41	3.53
Column-1	1.75	1.73	2.8	1.71	1.75	2.35
Column-2	1.72	1.7	2.6	1.66	1.69	2.67
Column-3	1.75	1.73	2.8	1.81	1.93	2.95
Bottom Slab	2	1.73	3	0.76	0.94	1.34

## 6. ACCEPTANCE CRITERIA CONTROL

Since linear and nonlinear methods have been used for assessing the system vulnerability, acceptance criteria have been controlled and evaluated. In the linear methods, *m* is considered as the controlling parameter. Considering system responses for the elements in linear static and dynamic analyses, the ratio of demand to the capacity has been compared with the parameter *m* as given by the FEMA guideline. For example, the above said for the linear static analysis and hazard level 2 is given in table 6.1. Results of the acceptance criterion control in the linear method in hazard levels 1 and 2 show that in linear static method, some of the tank elements are vulnerable; as for beams and in hazard level two 6, 8 and 19 percents, are not satisfy the acceptance criterion for immediate occupancy (IO), life safety (LS) and collapse prevention (CP) functional levels respectively. However, in other cases and in the nonlinear dynamic analysis method acceptance criterion satisfied. For the columns, it has been observed that only in linear static method and in hazard level 1 and 2, this element are vulnerable; as in hazard level 1, 10, 16 and 25 percents and in the respective 18, 23, and 35 percents of hazard level 2, IO, LS, and CP functional levels respectively were not satisfied. For the bottom slab of the tank, the acceptance criterion is satisfied in all the cases. In nonlinear dynamic method, plastic rotation angle has been considered as a controlling parameter. Considering system response for this elements in nonlinear dynamic analysis, this parameter has been obtained and the available quantities in FEMA guideline tables have been compared. The results of acceptance criterion control for the supporting frame members have been shown in table 6.2. Table 6.2 indicates that for beam, column and joints member, only in beams and columns of the third story, the acceptance criterion parameter is not satisfied and in other members and elements, this criterion is confirmed. The story's proportional displacement parameter (Drift) for the supporting frame has been shown in figure 15. Since in FEMA-356 for the drift of both permanent and transient modes, where in reinforced concrete frames, the quantity of this parameter for immediate occupancy, LS, and CP functional levels are respectively 1,2 and 4 percent and for the permanent mode, they are defined as insignificant, 2%, and 4% respectively. The results as illustrate in this figure, it can be seen that in acceptance criterion analysis, it is not satisfied only in immediate occupancy (IO) functional level but in other levels, this parameter are the acceptance satisfied.

Table 6.2 Plastic rotation angle control in nonlinear dynamic method

Element	Acceptable Criteria			Analysis		
	IO	LS	CP	IO	LS	CP
Joint- 1	0.0	0.0	0.0	0.0011	0.0015	0.0018
Joint- 2	0.0	0.0	0.0	0.0012	0.0016	0.0020
Joint- 3	0.0	0.0	0.0	0.0014	0.0017	0.0023
Beam- 1	0.01	0.02	0.025	0.005	0.017	0.021
Beam- 2	0.007	0.014	0.022	0.007	0.013	0.019
Beam- 3	0.008	0.017	0.023	0.009	0.020	0.025
Column- 1	0.005	0.012	0.016	0.003	0.010	0.014
Column- 2	0.005	0.015	0.020	0.004	0.013	0.018
Column- 3	0.005	0.013	0.017	0.005	0.016	0.019

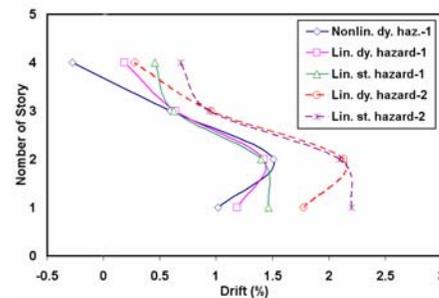


Figure 15 Drift in the elevated water tank's height in filled mode of the tank

## 7. CONCLUSION

The seismic vulnerability of elevated water tanks using performance based-design is study. A reinforced concrete elevated water tanks with a container capacity is considered. The demand of the system is evaluated to an ensemble of earthquake records by using linear and nonlinear analysis. In the other hands, the capacity of the system also evaluated. The ratio of demand to capacity for each element of the system are also estimated. The following conclusions are drawn and presented as follows :

- Critical response of the elevated tank does not always occur in full condition and it may happen in low percentage of filling and even in empty condition of the tank. The reason depends on the accordance of the frequency content and the earthquake characteristics in reduction or amplification of system responses. Thus, structure responses for each record depend upon not only the structure's dynamic features, but also the frequency content and the earthquake characteristics.
- Maximum displacement in the height of the structure in nonlinear dynamic analysis, considering the soil condition, happens in the joint of the supporting system to the container. In stiff and relatively soft soils, system's maximum displacement occurs in the joining place of the supporting system to the container and the softer the soil, the system's maximum displacement happens in the system's roof level.
- In nonlinear analysis, higher stories are more vulnerable than lower stories. The reason refers to the system maximum displacement, which occurs in the joining place of the frame to the tank's container. The other reason, probably relates to the difference in the tank's sloshing mode period with the structure's main period.
- Drift variation trend in lower stories is different from the higher stories.
- In linear analysis, lower stories are more vulnerable than upper stories.

## REFERENCES

- [1]-Haroun, M.A., Ellaithy, M.H. (1985). Seismically induced fluid forces on elevated tanks. *J. Tech. Top Civil Eng.*, **111**: 1, 1-15.
- [2]-Haroun, M.A., Termaz, M.K. (1992). Effects of soil-structure interaction effects on seismic response of elevated tanks, *Soil Dynamics Earthquake Engineering*, **11**: 2, 37-86.
- [3]-Marashi, E.S., Shakib. H. (1997). Evaluations of dynamic characteristics of elevated water tanks by ambient vibration tests. *Proceedings of the 4<sup>th</sup> International Conference on Civil Engineering*, Tehran, Iran,I, 367-73.
- [4]-Dutta, S., Mandal, A., Dutta, S.C. (2004). Soil-structure interaction in dynamic behavior of elevated tanks with alternate frame staging configurations. *Journal of Sound and Vibration*, **227**: 4-5, 825-853.
- [5]-Livaoglu, R., Dogangun, A. (2005). Seismic evaluation of fluid-elevated tank-foundation/soil systems in frequency domain. *Structural Engineering and Mechanics*, **21**, 101-119.
- [6]-Donea, J., Giuliani, S., Halleux, J.P., (1982). An arbitrary Lagrangian-Eulerian Finite Element method for transient dynamic fluid-structure interaction. *Computer Methods in Applied Mechanics and Engineering*, **33**, 689-723.
- [7]-Park R, Kent DC, Sampton RA., (1972). Reinforced concrete members with cyclic loading. *Journal of the Structural Division ASCE*, **98**:7, 1341-60.
- [8]- FEMA 273, (1996). NEHRP guideline for the seismic rehabilitation of building, *Federal Emergency Management Agency, Washington D.C., USA*.
- [9]- FEMA 368/369, (2000). The 2000 NEHRP recommended provisions for new buildings and other structures, part 1: provision and part 2: commentary, *Federal Emergency Management Agency, Washington D.C., USA*