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Behavior of reinforced concrete water towers during Manjil-Roudbar earthquake of June 1990

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ABSTRACT: The behavior of two reinforced concrete elevated water tanks during the June 21, 1990 earthquake of Manjil-Roudbar is investigated. The level of seismic design forces based on standard 519 according to which the structures were designed are compared with the standard 2800 which has superceded the former. Also level of seismic forces according to LEDRS for several PGA's are indicated. The effects of sloshing and secondary moment is also investigated.

1 INTRODUCTION

The June 20, 1990 earthquake in Iran inflicted enormous damage to an area well over 10,000 Km^2. This magnitude 7.3 to 7.7 earthquake caused extensive destruction of structures within a radius of less than 40 Km from the epicenter, where it was estimated to be located near the town of Manjil. The nearest accelerograph to the epicenter (40 Km) recorded a high ground acceleration of 0.65 g at Abbar (Moinfar and Naderzadeh (1990)). In Rasht, located about 60 km north of Manjil, the damage was minor to most buildings. However, several midrise buildings with five to eight stories collapsed. Another type of structure that collapsed was a 1500 m^3 reinforced concrete water tower. Two other similar and new towers with 2500 m^3 capacity that were empty at the time of the earthquake received only some perimeter cracking in the shaft. Figure 1 shows a picture of one of the 2500 m^3 towers (tower 2) and Figure 2 shows the collapsed 1500 m^3 tower (tower 1).

It is the purpose of this paper to investigate the behavior of these towers during the earthquake and offer an explanation to the reasons and mode of failures. In addition, some imprtant factors regarding modeling and other effects are explored. To this end, first a simple single degree of freedom and then a two degree of freedom model for sloshing effect were studied. Later a more refined beam model was investigated, and finally a well refined finite element model was examined. The seismic loading is in the form of spectral acceleration. The seismic lateral forces according to the older Iranian loading code (standard 519) (ISIRI (1971)) and the recent Iranian code for seismic resistant design of buildings (standard 2800) (BHRC (1988)) are computed and the adequacy of designs with such forces explored.

Figure 1. Picture of cracked tank 2

2 DESCRIPTION OF STRUCTURES

The collapsed structure (tower 1), which was built some 20 years ago, was a reinforced concrete elevated water tank with a height of 47m and a capacity of 1500 m^3 . This tower rested on a mat foundation. The shaft with the inside diameter of 6m had a height of 25.5m and a thickness of 0.3m . The other two towers (tower 2), that received only

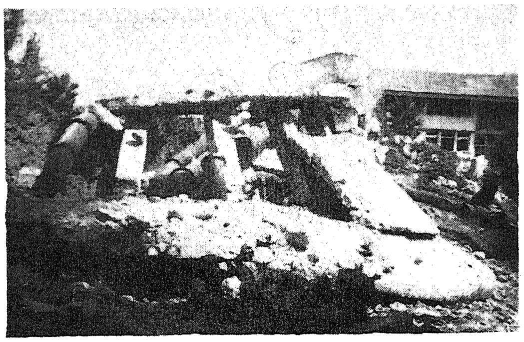

Figure 2. Picture of collapsed tank 1

minor damage in the form of perimeter cracking in the shaft above the door opening, have a height of 50m with a capacity of 2500 m^3. The shafts for these towers have an inner diameter of 7m, height of 25m, and thickness of 0.5m. A schematic diagram of the exterior elevation of tower 2 is shown in Figure 3 and the dimensions of various parts are illustrated in Figure 4.

As can be seen in Figure 4, the tank is divided into two cylindrical tanks, each with a capacity of 1250 m^3, the inner tank with a radius of 7m and the other with a radius of 10m. The walls of the tanks are prestressed concrete. The cylindrical tank is supported on the shaft through a transition funnel with interior columns. The foundation consist of a double walled transition section which rests on a 20m

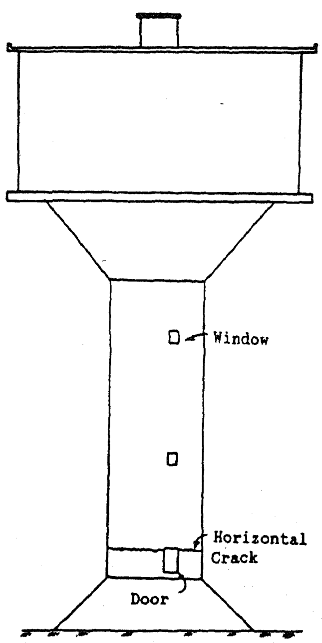

Figure 3. Elevation of tank 2

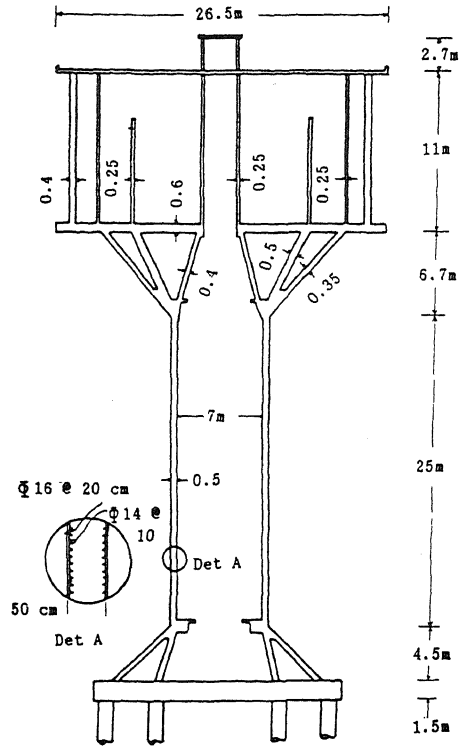

Figure 4. Dimensions of tank 2

diameter mat supported on piles. There are 24 piles each with a height of 30m and diameter of 1.2m, where 10 of them are located on a circle of radius 5m and the rest on a circle of radius 9m.

The towers were designed according to the standard 519. The level of seismic design forces will be discussed later. A new building code, standard 2800, has superseded the old code and the seismic load requirements for this new standard will also be studied.

3 GROUND MOTION

There is no record of the ground motion in the epicenteral region, and the nearest accelerograph was 40 Km away with a peak ground acceleration (PGA) of the 0.65 g. It is expected that the mountaineous area of Manjil and Roudbar experienced at least that acceleration. Rasht is located on an overburden of medium fine and granular soil. This overburden apparently amplified the base rock motion in such a way that the predominant period of excitation of the ground surface nearly matched the period of midrise buildings and other tall structures such as the water towers. The evidence of the collapse of some five to eight story buildings and relatively minor damage to short buildings confirms this assertion.

In order to estimate the ground motion in Rasht several attenuation relations were used with the results shown in Table 1. The distance to the epicenter was 60 Km with a focal depth of 15 Km and the magnitude used was 7.3 . Although there is some scatter among the results, the average values suggest the base rock motion in Rasht to be 165 cm/s^2 (0.17g), 18 cm/s, and 15.5 cm for acceleration, velocity, and displacement, respectively. When several other equations for estimating acceleration were used the value of 0.17g turned out to be reasonable. Using a soil amplification factor of 1.5 we obtain the maximum ground motion to be 248 cm/s^2 (0.25g), 27 cm/s, and 23 cm for acceleration, velocity and displacement, respectively. This maximum ground acceleration is a little less than the value 0.29g obtained by using several of the instrumentally measured peak ground acceleration in the area (IIEES (1991)).

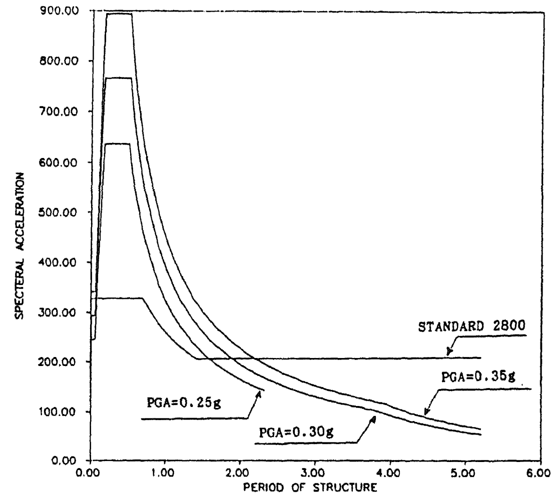

Figure 5. Spectral accelerations

Table 1. Estimated base rock motion based on attenuation relations.

Equation by	Acc. (cm/sec^2)	Vel. (cm/sec)	Displ. (cm)
Esteva & Rosenblueth (1963)	176	21	21
Mickey(1971)	163	12	25
Orphal & Lahoud (1974)	171	18	8
Mc Guire(1977)	150	22	11

In order to obtain the response of the structure to the ground motion, a linear elastic design response spectrum (LEDRS) based on Newmark method was prepared. The amplification factors were obtained for a damping ratio of 5% for the reinforced concrete tower. Due to uncertainty of the actual ground motion in Rasht LEDRS were also made for PGA of 0.3g and 0.35g for comparison purposes. Since only acceleration spectra are to be used for modal superposition analysis, the logarithmic scale acceleration spectra were converted to arithmetic scale as shown in Figure 5 . This Figure shows the acceleration spectra for PGA of 0.25g, 0.3g, and 0.35g. To study the response of the structure were it designed according to standard 2800, the acceleration spectrum according to this code is also plotted in Figure 5.

4 ANALYSIS USING A SINGLE DEGREE OF FREEDOM (SDOF) MODEL

The first model for the structure consists of a SDOF oscillator. This system is used to compare the level of design forces suggested by standards 519 and 2800 to the capacity of the structure, and also to see the level of forces created in the structure due to various possible ground motion in the area. Consider the simple oscillator shown in Figure 6. In this model the weight of the tank and water is concentrated at the cantilever end of the stick, and the weight of the shaft is assumed to be uniformly distributed throughout its height. It is of interest to study the behavior of the towers at various water levels in tanks. It is believed that tower 1 was 1/3 full at the time of the earthquake, while tower 2 was empty.

According to standard 519, the seismic coefficient is calculated from the following

$$C = 0.05/T \geqslant 0.05 \qquad (1)$$

Where T is period of building. Although this standard gives an equation for the period of the buildings, it does not have provisions for inverted pendulum type structures. Standard 2800 defines the seismic coefficient as

$$C = ABI/R \qquad (2)$$

Where A is the ground acceleration coefficient based on the seismic map of the area, which in this case it will be 0.35, I is the importance factor which may be taken as 1.2 ; R is the behavior coefficient or a measure of ductility which according to the code for inverted pendulum may be assumed 2.5; and B is the response coefficient calculated according to the following

$$B = 2\ (T_0/T)^{2/3} \qquad (3)$$

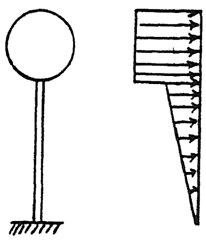

Figure 6. Single degree of freedom model

In this equation, To is the period of the soil in the area which according to the code may be assumed 0.5 to 0.7 . For the area under consideration it is assumed 0.7. The period of the structure may be estimated for a inverted pendulum with distributed mass in the shaft by

$$T = 2\pi(P'L^3/3EIg)^{\frac{1}{2}} \qquad (4)$$
$$P' = P + 0.236\,gL$$

In this relation P is the concentrated whight, g is the uniformly distributed weight of the shaft, E is the modulus of elasticity (200,000 Kg/cm^2 for concrete), and I is the moment of inertia of the shaft. The height of the shaft L is taken according to Figure 6. The B/R ratio according to the code should not be less than 0.5 . The estimated period of vibration and the base shear for both towers according to both standards and three different PGA's are listed in Table 2.

As mentioned earlier, tower 1 has been designed according to standard 519. The moment capacity of the shaft is approximately 3000 - 3500 ton-m which is based on the modulus of rupture of concrete. The reinforcement in the shaft is so small that only the strength of the concrete can be counted on to resist the applied moments (see Figure 4 as an example). For the condition of the tank 1/3 full, the overturning moment due to seismic forces based on standard 519 (seismic coef.= 0.054) is 4260 ton-m which when combined with gravity loads results in a maximum compressive stress of 90 Kg/cm^2 and a tensile stress of 6 Kg/cm^2 . Assuming the compressive and tensile strength of concrete to be respectively 210 Kg/cm^2 and 29 Kg/cm^2, we come to the conclusion that, according to standard 519 design force levels, the tower has sufficient strength even if the tank were full.

However, looking at the requirements of standard 2800 for the same condition (1/3 full), we find the seismic coefficient to be 5.18 times that due to standard 519 . That is, the required compressive and tensile capacities of the section are 330 Kg/cm^2 and 205 Kg/cm^2, respectively. Compared to the available capacity of the shaft, it is concluded that the tower was underdesigned for the force level now required by standard 2800.

The forces that may actually have been created in the structure (tower 1) are obtained by using the design response spectra. Assuming a PGA of 0.25g, the seismic coefficient will turn out to be 6.67 times the force level required by standard 519. Accordingly, the stresses created in the shaft are of the order of 360 Kg/cm^2 in compression and 275 Kg/cm^2 in tension. That is the section was stressed much beyond its tensile capacity. The overturning moments during the earthquake has caused failure of the shaft in tension and has brought about the collapse of this structure. Inspection of the foundation by the author after the earthquake showed no sign of foundation failure.

A similar calculation was done for the empty tower 2. The pure moment capacity of the section above the door openning is about 7800 ton-m, again based on modulus of rupture. The moment capacity based on the steel reinforcement in the shaft is only 4000 ton-m, which indicates the steel used is only for temprature and shrinkage purposes. The moment capacity of the section at the level of the door openning based on weight above, the section and modulus of rupture is 13000 ton-m.

The overturning moment at the level of the door opening in the shaft due to seismic load suggested by standard 519 is 8000 ton-m, based on standard 2800 is 31,500 ton-m, and created by PGA of 0.25g is 52,700 ton-m.

Table 2. period and seismic coefficients for various conditions of SDOF

Tank	period (sec)	seismic coefficient				
		STD 519	STD 2800	PGA		
				0.25g	0.3g	0.35g
			Tower 1			
Empty	0.8	0.062	0.31	0.41	0.5	0.56
$\frac{1}{3}$full	0.92	0.054	0.28	0.36	0.43	0.49
$\frac{1}{2}$full	0.97	0.05	0.27	0.34	0.41	0.46
$\frac{2}{3}$full	1.02	0.05	0.26	0.33	0.40	0.44
full	1.1	0.05	0.25	0.30	0.37	0.42
			Tower 2			
Empty	0.59	0.085	0.34	0.56	0.7	0.78
$\frac{1}{3}$full	0.67	0.075	0.34	0.50	0.61	0.68
$\frac{1}{2}$full	0.71	0.07	0.33	0.46	0.58	0.62
$\frac{2}{3}$full	0.75	0.067	0.32	0.44	0.55	0.58
full	0.82	0.061	0.30	0.40	0.50	0.55

Therefore it is concluded that according to standard 519 the capacity is sufficient, according standard 2800, the section is 2.5 times underdesigned and for PGA of 0.25g the section is 4.1 times underdesigned.

The shear capacity of shaft according to ACI [318-89] turns out to be 1075 tons. The shear force based on standard 519, and standard 2800 is respectively 260 ton and 1030 ton. Therefore, it is concluded that shear is not the governing critical factor.

5 EFFECT OF SLOSHING

In order to study the effect of sloshing, a two degree of freedom (TDOF) model was considered, as shown in Figure 7. The impulsive mass of water in the tank, that is the mass of water that moves with the tank, is calculated from (Newmark and Rosenblueth (1971))

$$M_i = [\tanh(1.7\ R/H)]M/(1.7R/H) \quad (5)$$

Where R is the radius of the tank, H is the height of water in the tank, and m is the total mass of the tank. The convective mass of water is given by

$$M_c = 0.71\ [\tanh(1.8\ H/R)]M/(1.8H/R) \quad (6)$$

The positions where M_i and M_c act are obtained from

$$H_i = 0.38\ H[1+\alpha(M/M_i-1)] \quad (7)$$

$$H_c = H[1-0.21(M/M_c)(R/H)^2 + 0.55\beta(R/H)[0.15(RM/HM_c)^2-1]^{1/2}] \quad (8)$$

Where, α=1.33 and β=2.0 for consideration of hydrodynamic moment on the bottom of the tank and α=0 and β=1 for consideration of hydrodynamic pressures on the walls. In this case, the latter condition will be used. The stiffness of the springs for convective mass is obtained from

$$K = 4.75g\ (M_c)^2H/MR^2 \quad (9)$$

Using the above equations for tower 1 with the condition 1/3 full, we obtain

$M_i = 47/g$ $M_c = 345/g$

$H_i = 0.6$ $H_c = 1.06$

K = 18 ton/m

Now adding the mass of the entire tower to M_i we obtain M_i = 2027/g. Taking $M_2=M_c$ and performing a vibration analysis we obtain T_1=0.87 sec and T_2=8.8 sec. These periods result in specteral accelerations of S_{a1}=0.37g and S_{a2}=0.012g. Since S_{a2} is very small, it is therefore expected that the effect of sloshing for the tank 1/3 full is

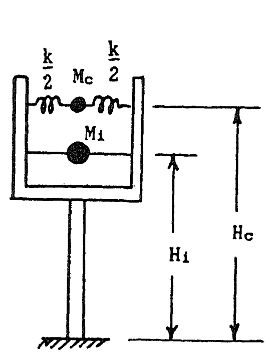

Figure 7. TDOF model

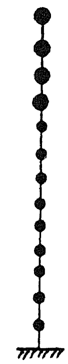

Figure 8. Beam model

small on the total base shear, and therefore it will not be considered in subsequent analysis. Moreover, it is concluded that sloshing effects could not have contributed to the collapse of tower 1.

6 REFINED BEAM MODEL OF THE STRUCTURE

The refined beam model of the structure (tower 2) consist of beam elements for the shaft and the tank (Figure 8). There are 13 elements in the model. Mode shapes and frequencies were obtained using SAP 80 finite element analysis package (Wilson and Habibullah (1980)). The periods for the first four modes for two conditions of the tank, empty and full, are listed in Table 3. For comparison, the periods for the SDOF model are shown as well. The modal superposition results according to standard 2800 and acceleration spectra are shown in Table 4. The base shear for the empty tank due to PGA of 0.25g is 940 tons which is only 54% of the SDOF model. It is therefore concluded that the single degree of freedom model is not sufficiently accurate for this type of water tower and a refined beam model or a refined finite element model using shell elements is more appropriate .

To study the P-delta effect for PGA of 0.25g, we use the maximum top deflection of 10.22 cm from table 4 . This results in a secondary moment of 250 ton-m which is only 0.6% of the primary moment. It is therefore concluded that the P-delta effect for this

Table 3 Model Periods (sec)

Mode	SDOF		MDOF		FEM	
	Empty	Full	Empty	Full	Empty	Full
1	0.59	0.82	0.92	1.33	0.85	1.24
2			0.09	0.09	0.18	0.14
3			0.02	0.02	0.05	0.05
4			0.01	0.01	0.04	0.03

Table 4 Model Superposition Results Moments (t-m) and Deflections (cm)

Loading	MDOF			
	Empty		Full	
	Mom.	Defl.	Mom.	Defl.
2800	27,700	7.81	43,900	12.51
0.25g	36,200	10.22	50,800	14.50
0.3g	43,600	12.31	60,800	17.43
0.35g	50,300	14.19	70,400	20.08

Loading	SDOF	
	Empty	Full
	Moment	Moment
2800	30,000	52,800
0.25g	51,000	70,500
0.3g	62,000	82,200
0.35g	71,500	97,900

structure is negligible. Of course, one may consider the rotation of the base as helping the P-delta effect. However, with the heavy mat on 24 deep piles, it is expected the rotation of the base, to be negligible as well.

7 FINITE ELEMENT MODEL

To complete the study, a detailed finite element model of the structure (tower 2) by SAP 80 was made using frame and shell elements as shown in Figure 9. This model has 754 nodes with 400 shell elements, 144 solid elements and 29 frame elements. For the vibration analysis only ¼ of the model was used. The first four natural modes of vibration are shown in Figure 10. The natural periods for the empty and full tank for the first four modes are listed in table 3. The first mode period for the empty tank is 8% smaller than that for the refined beam model and for the full tank it is 5% smaller. This indicates that the refined beam model is reasonably accurate, compared to the finite element model.

The vertical stress distribution in the shaft under a PGA of 0.25g using the modal superposition method is as shown in Figure 11. It clearly indicates the critical section to be about the door openning in the shaft, and this is confirmed by existing crack above the door opening. The base moment for the empty tank for the above loading is 41,000 ton-m, which when compared to that for the refined beam model (36,200 ton-m) it is found to be 13% higher. The maximum top deflection which is 11.86 cm is about 16% higher than the result of the refined beam model in Table 4. It is therefore concluded that refined beam model can provide reasonable results. Furthermore, it is evident that seismic flexural stresses in the shaft have caused the perimeter crack in tower 2.

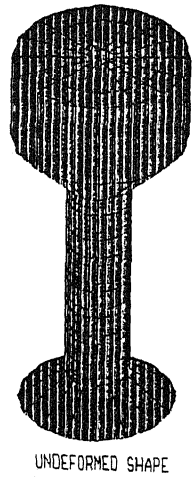

Figure 9. FEM model

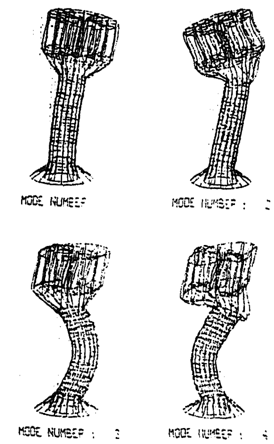

Figure 10. Mode shapes

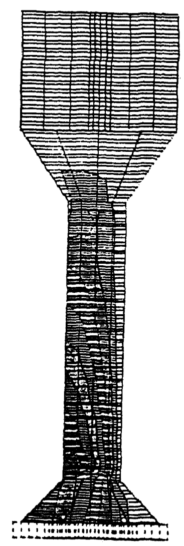

Figure 11. Flexural stress distribution

8 CONCLUSIONS

This study has resulted in several conclusions with respect to the level of design forces according to standards 519 and 2800 and also probable PGA in the area. In addition some results concerning the modeling of such elevated tanks are obtained. The conclusions are as follows:

1 The tanks were properly designed according to the older standard 519, and both had sufficient capacity for the strength level required by that standard.
2 The design forces based on standard 2800 are about five times those based on standard 519.
3 For tank 2 the base moment according to standard 2800 for the empty and full condition is respectively 2.2 and 3.4 times the available capacity.
4 The level of design forces required for PGA of 0.25g, 0.3g and 0.35g are respectively 1.2, 1.4 and 1.6 times that required by standard 2800 for a full tank. Therefore, one may conclude that the elevated tank design forces based on this standard should be rised by at least 20% , especially considering the fact that these are essential facilities and lifelines that must be operational after an earthquake.
5 The single degree of freedom model is not accurate enough for the analysis of these types of structures.
6 The refined beam model is sufficiently accurate for the analysis.
7 The sloshing effect is very minor in this type of elevated water tank.
8 The P-delta effect is very minor and may be neglected in this type of elevated water tank.
9 The predominant mode of failure has been flexural and not shear

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