WATER TOWERS IN SEISMIC ZONES

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SYNOPSIS

This paper discusses special problems concerning elevated water towers in seismic zones. Results of experimental study on prototype reinforced concrete towers are reported and corresponding analytical solutions are discussed.

INTRODUCTION

Elevated water towers are important public utility structures. Since there is a possibility of fire breaking out immediately after an earthquake, it is necessary to design these structures with special care.

Water towers are top heavy structures, in the sense that a greater portion of their weight is concentrated at an elevation above its base. Critical parts of this structure are the columns and braces through which the loads are transmitted to the foundations. The design of foundations also poses special problems in such structures.

BEHAVIOUR UNDER LOADS

a) Tank

The tank essentially behaves like a rigid body, as, if it is considered as a separate unit, its natural frequency is high. The tank walls are to be designed for hydrodynamic pressures due to impulsive and convective motions of water generated at the time of earthquake. However, the magnitude of these pressures is a small fraction of the hydrostatic pressures for which the tank is otherwise designed. The additional stresses caused due to hydrodynamic pressures are taken into account by the permissible reduction in the factor of safety when dynamic forces are considered. Therefore, no special precaution is necessary as far as the design of tank is concerned.

b) Columns and Braces

There are two structural systems used as a supporting structure depending upon whether steel or reinforced concrete is used as construction material. In the case of steel towers, the members are essentially subjected to axial forces, vertical load being transferred through the columns in compres-sion and the horizontal loads transferred as tensile forces in the diagonals.

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In the case of reinforced concrete towers, diagonal braces are not usually provided. The columns and braces act as a vierendeel system transmitting loads axially and by bending, twisting and shearing action. The columns are essentially subjected to compressive forces due to vertical loads and moments and shears due to horizontal loads. Braces, which are usually horizontal, are subjected to moments, torques and shears due to horizontal loads.

The columns of a reinforced concrete tower behave as springs in parallel between two successive braces, but, as a whole, each column behaves as springs in series connected at brace levels. Because water towers are top heavy, the entire system could be approximated to a single degree of freedom system without appreciable loss of accuracy. Certain fraction (usually one third) of the weight of columns and braces may be assumed to be added to the weight at top and the columns may be treated as weightless springs. [Addition of a fraction of the self weight to the weight at the tip of a cantilever is based on energy considerations (1)]. Periods calculated on this basis approximates very closely to those calculated by sophisticated methods.

c) Foundations

The foundations are subjected to vertical loads due to self weight and moments and shears due to horizontal loads. For integral action, it is advisable to tie the columns at the bases. A raft foundation, which may be annular, may be adopted. If the soil is soft, the raft might rest on piles to avoid unequal settlement under the columns. A tall structure which is top heavy cannot stand much slope in the foundation base without the possibilities of instability. It would be of advantage to design the base with long projection outside the line of columns to help stability.

DESIGN FEATURES

One of the important features of design is to decide about the state of stress that is considered safe with earthquake forces superimposed upon other static forces. It is generally presumed that wind and earthquake will not occur simultaneously. In some very tall water towers wind may be more dominant a factor but each of the two occasional forces could be considered separately without much risk. Further, a decision has to be taken regarding the size of a shock for testing the safety of the structure.

One of the criterion of design could be that the tank and tower, under the effects of moderately strong ground motion which are expected to be more frequent in the particular zone may not have stresses beyond the elastic limit. On the other hand, under the effect of a severe ground motion, which may be expected there once in the lifetime of the structure, the system may undergo plastic deformations.

Housner [2] has given an example of an earthquake resistant design of steel water tower. The authors [3] have given an example of an earthquake resistant design of reinforced concrete water tower. Salient design features of the design of concrete towers are summarised below (Fig. 1 gives details of the water tower).

It is suggested that diagonal braces of steel be also provided in the case of concrete towers. This provision is recommended in view of the experience gained in Chile in 1960 shock.

During the elastic stage, due to small deformations, the diagonals are stretched very little and the horizontal loads are essentially shared by the columns and horizontal braces by developing moments. During plastic stage, when deformations are large, the horizontal loads are essentially borne as tensile forces by the diagonals. In order to permit large deformations of the diagonals, the junction between columns and diagonal braces should be capable of plastic rotations. It is recommended that reinforcement in the form of spiral be provided in the column and horizontal brace for a distance of about one fourth the length to permit plastic deformations.

It is found that a design would be economical when the columns are tied at the foundation level by means of a beam and the foundations are in the form of an annular raft.

EXPERIMENTS ON PROTOTYPE WATER TOWERS

For thirteen modern reinforced concrete water towers, wind excited vibrations were recorded. Out of these, for four towers, free vibration caused by pull test were also recorded. At the time of testing, six of the tanks were empty, five of them were partly full and one of them was full. For one of the towers, (the one at Roorkee-Fig. 2) measurements were made with various levels of water in the tank. (Details of above are given in Reference 4).

The assumptions made for theoretical calculations of period, with respect to mass and stiffness, are discussed below.

Evaluation of Mass

The weights of various parts of the structure were estimated. For calculating periods, it was assumed that a mass equivalent to the weight of the tank including water and one-third the weight of columns and braces is concentrated at the top of the columns. For any particular distribution of stiffness, it was observed that this method of evaluating equivalent mass gave periods which were very close to the ones calculated more sophistically. Thus the above simple approximation for finding equivalent mass gives satisfactory results for all the towers worked out.

Evaluation of Stiffness

For horizontal vibrations, the columns are essentially subjected to bending deformations. The stiffness then depends upon, the modulus of elasticity of the material (E), net moment of inertia of the section (I) and the effective length of the column (L). The construction material being of reinforced concrete, E and I could change depending upon the vertical stresses. L depends upon the rigidity of joints at the junction of columns and braces and for convenience it could be taken equal to the distance between centre to centre of braces.

The various towers had different levels of water. Therefore, the intensities of vertical stresses are different. Hence the effective flexural rigidity, EI, which depends upon the magnitude of vertical stresses would be different in various cases. Table 1 gives the factor α which is the ratio of effective flexural rigidity and nominal flexural rigidity. The nominal EI, has been worked out assuming E equal to 2×10^6 pounds per square inch and I evaluated on the basis of gross s-ection of concrete. For serial numbers 1 to 5 (tanks either full or partly full), the effective EI is very nearly equal to nominal EI. Serial numbers 7 to 12 are tanks which were empty. For these cases the factor α is very much less than one indicating effective EI to be smaller than nominal EI. This is due to reduced vertical stresses (tanks being empty). There is a scatter in the value of α which is due to different qualities of concrete and due to possible variations in effective length. (Joints in various towers do not have the same rigidity due to variation in relative rigidity between columns and braces).

Table 2 gives the variation of period of a water tower for various levels of water in the tank. It appears that the stiffness of the structure decreases with decrease in vertical load. The factor α decreases as vertical load is reduced corroborating the earlier observation. This aspect is being examined further.

Damping

Damping as calculated from free vibration records(pull tests) are given in Table 3 and a typical record is given in Fig. 3. It is observed damping is very small. This is because the vibrations for which measurements were taken were also very small. (Maximum amplitude about 7 mils.) Damping would increase with greater amplitudes of vibration. A test [5] on the model of the Roorkee water tower indicated a damping of 0.06 of critical value for large amplitudes of vibration.

CONCLUSIONS

Provisions should be made in the design for energy absorbtion during severe earthquakes. Measurements of period of concrete water towers indicate that effective flexural rigidity changes considerably with the vertical load. For tanks that are full, the nominal values suggested for E, I and L give periods which are a very good approximation to the measured values. It is significant that the most critical condition for design of water towers against earthquake forces is usually the one with tank full and therefore the approach suggested in this paper is adequate. It is, however, worth examining the relative effects of the decrease in period when tank is empty or partially full on the spectrum value in the relevent range of periods and damping and the decrease in the inertia force due to decrease of mass of water. This work is in hand and will be reported later.

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TABLE 1

Sl. No.	Location of Water Tower	Capacity in gallons	Height of Staging in feet	Number of intermediate levels at which braces are located	Number of columns	Weight of Tank in kips (without water)	Weight of column and braces in kips	Weight of water in kips	Details of column section	Experimental value of fundamental period in seconds	Effective EI / Nominal EI α
1	2	3	4	5	6	7	8	9	10	11	12
1.	Maulana Azad Medical College, New Delhi	50,000	78	5	8	276	300	384^{+}	18"x18" square 4 bars $\frac{7}{8}$"ϕ	1.06	0.95
2.	Municipal Office Muzzafarnagar	200,000	60	3	12	920	392	805^{+}	24" dia. 10 bars $\frac{3}{4}$"ϕ	1.00	0.90
3.	Bulandshahar	150,000	55	3	16	776	465	1190^{+}	18"x18" square	1.00	1.00
4.	Hapur	150,000	65	3	16	776	519	1012^{+}	18"x18" square	1.19	1.11
5.	Commissioner's Office, Meerut	100,000	60	3	10	352	258	1000^{++}	21" dia 10 bars $\frac{7}{8}$"ϕ	1.30	1.06

Contd -

1	2	3	4	5	6	7	8	9	10	11	12
6.	Jheel Khuranja Shahdara, Delhi	50,000	50	2	8	276	164	337+	21" dia.	1.34	0.46
7.	Khatauli	50,000	45	2	8	276	144	-	21" dia. 5 bars $\frac{3}{4}$"φ	0.71	0.60
8.	Sardhana	50,000	45	2	8	276	144	-	21" dia. 5 bars $\frac{3}{4}$"φ	0.74	0.55
9.	Jhilmili, Shahdara Delhi	50,000	55	3	8	276	186	-	18"x18" square 4 bars $\frac{7}{8}$"φ	1.00	0.37
10.	Jhilmil 2, Shahdara, Delhi	50,000	55	3	8	276	186	-	18"x18 square 4 bars $\frac{7}{8}$"φ	1.04	0.35
11.	Lady Hardinge Medical College Delhi	50,000	100	7	8	276	398	-	18"x18 square 4 bars $\frac{7}{8}$"φ	1.40	0.34
12.	Kishen Nagar, Shahdara, Delhi	50,000	55	2	8	276	164	-	21" dia	1.13	0.32

+ Tank partly full
++Tank full

Note ; All towers except serial Nos 3 and 4 are circular in plan and the columns are equally spaced along a circumference of a circle. 3 and 4 are rectangular in plan and the columns and braces are in the form of a grid work (12 column on the periphery and 4 inside).

TABLE 2

Water Tower at Roorkee

Capacity	- -	100,000 gallons
Height of Staging	-	60 feet
Number of intermediate levels at which braces are located	-	3
Number of columns	-	12

Tank circular in plan 8 columns are located equally spaced on the circumference of a circle and 4 at the intersection of main chords. The columns and braces form a gridwork.

Weight of Tank	-	454 kips
Weight of columns and braces	-	465 kips

Amount of water in the tank	Experimental value of fundamental period in seconds	α' *
Full	0.785	1.00
$\frac{3}{4}$ Full	0.75	0.92
$\frac{1}{2}$ Full	0.73	0.79
$\frac{1}{4}$ Full	0.715	0.64
Empty	0.705	0.47

* Represents relative values.

TABLE 3

Damping as Obtained From Free Vibration Records of Pull Tests

Sl No.	Location of Water Tower	Average value of Damping expressed as a fraction of critical value
1.	Sardhana, Meerut	0.013
2.	Jhilmil 1, Shahdara Delhi	0.016
3.	Jhilmil 2, Shahdara Delhi	0.015
4.	Roorkee	0.03*

* Relatively, this tower was subjected to a larger initial displacement. An average value of damping, as obtained from the ratio of initial maximum amplitude to that of the next successive maximum amplitude, was 0.05.

WATER TOWER AT RISHIKESH

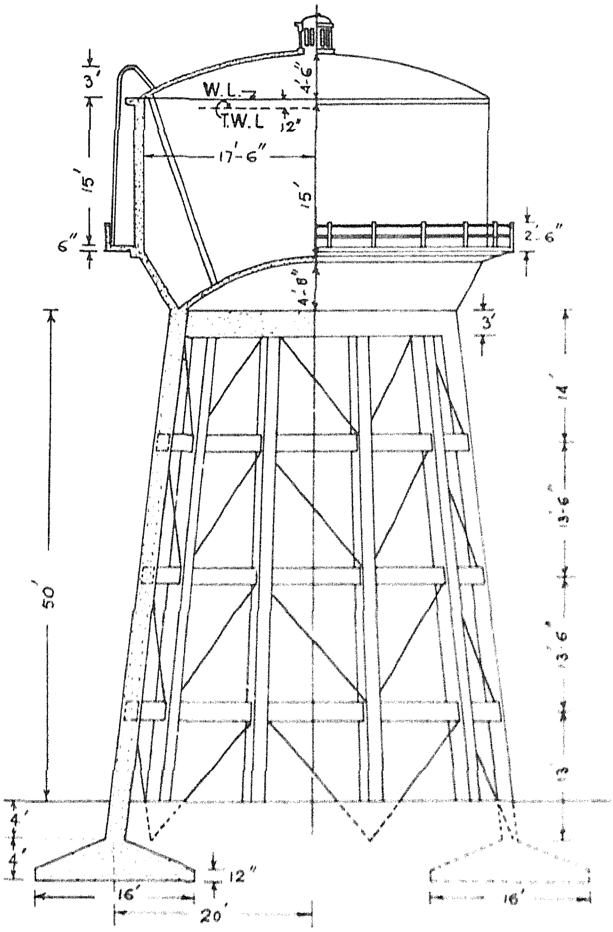

FIG. 1

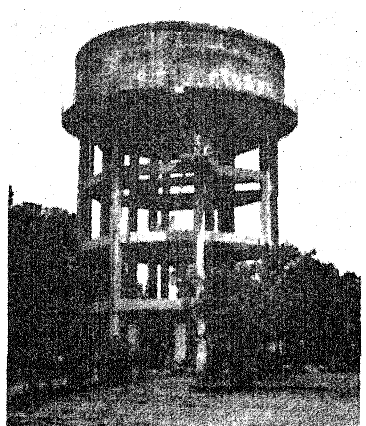

FIG. 2 - WATER TOWER AT ROORKEE.

ROORKEE WATER TOWER

FREE VIBRATION CAUSED BY PULL TEST

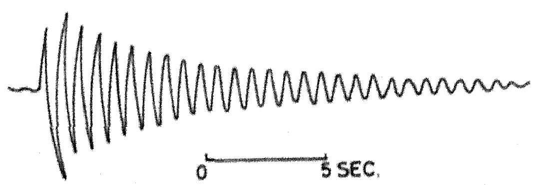

FIG. 3

WATER TOWERS IN SEISMIC ZONES

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QUESTION BY: E. ROSENBLUETH - MEXICO

Perhaps the authors would comment on the following two matters: (a) The influence of a second natural mode of vibration on earthquake stresses, even if this second mode should not affect the agreement between measured and computed periods, especially since the degree of freedom associated with water sloshing would have a very small percentage of damping. (b) The influence of axial forces on the stiffness of reinforced concrete members.

AUTHORS' REPLY: 1. The vibrations of a water tower during an earthquake are predominantly in the following modes. In one of the modes the tank proper and the water inside act as a rigid mass supported on a flexible staging. In this mode, there will be relative displacement between the ends of columns which should be taken into account in the design of staging. The other mode vibration will be essentially due to sloshing effect in the water. This would no doubt change the stresses of the tank wall and the floor system, but would not cause any appreciable deformation of the staging itself. Hence, as far as the design of staging is concerned there is ample justification in considering an elevated water tower system as a single degree of freedom system.

2. In reinforced concrete members subject to direct load and bending the modulus of elasticity as well as the effective moment of inertia very much depend on the longitudinal stresses. Therefore the stiffness of reinforced concrete members is influenced by the amount of axial force carried by them. This fact has been confirmed during the experimental determination of periods on water tower at Roorkee. We are carrying out further investigations to derive a suitable relationship between the axial forces and stiffness of reinforced concrete members.