EARTHQUAKE RESISTANT DESIGN OF AN ELEVATED WATER TOWER

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SYNOPSIS

This paper deals with the design of Water Towers in seismic zones with reference to a specific problem. The criterion for design suggested is that the system remains elastic under the effects of moderately strong ground motion which are expected to be more frequent in the particular zone, and under the effect of a severe ground motion, which may be expected there once in the life-time of the structure, the system may undergo plastic deformations.

INTRODUCTION

Liquid containing tanks and towers form a very important part of the life of a community as well as industrial undertakings. Many such structures have failed during earthquakes in the past resulting in considerable hardship and damage. The Earthquake Engineering Research School has consequently undertaken a programme to study the most economical methods of strengthening such structures in seismic zones of different intensities. A study has recently been made for a 50 ft. high, 50,000 gallons water tower proposed to be built at Rishikesh, a region which is not far from the main boundary fault running along the length of the Himalayas. The method employed in arriving at a suitable seismic coefficient and that of analysing the tower for earthquake forces have been explained in this paper. In estimating the future ground and the structural response, use has been made of the principles underlying the recommendations made by Jai Krishna (1960) and Housner (1959).

The design procedure recommended assures that under the vertical and horizontal loads the columns will deform as springs between two braces with the point of contraflexure at the mid-height of the panel. Further, the structure will stand the small earthquakes, which occur in the region quite often, remaining "elastic". During a major earthquake occurring not very far from the epicentre of 1905 earthquake, the structure will go "plastic", permit rotation at the junctions so that the entire resistance will come from the diagonal braces. It has been estimated that the structure will not deflect more than 6″ at the top even during such intense vibrations. The provision of diagonal braces in addition to the horizontal ones has been recommeded in view of the experience gained in Chile on account of 1960 shock. The tower at Rishikesh is founded on river deposits and, therefore, an eccentric reinforced concrete raft, that will resist torsion, small unequal settlements and excess pressures during earthquakes, has been recommended.

SEISMICITY OF THE REGION:

Rishikesh lies in the foot-hills of the Himalayas and is located within a few miles of the main Himalayan boundary fault which is seismically active. One of the major earthquakes that had occurred in the region was in 1905 with epicentres in Mussoorie and Kangra. The Mussoorie epicentre was about 25 miles from Rishikesh. There have been a number of earthquakes in the south east near about Almora and Kotdwar regions. The smaller earthquakes had their magnitude of about 6 and the major earthquake of 1905 had a magnitude of about 7.9. It is, therefore, essential that structures in this region should be designed for earthquakes. As far as a water tower is concerned, its safety is even more important because quite often an earthquake is followed by fire caused either by electrical short-circuiting or from the kitchen fires. The water storage, therefore, must

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remain in tact so that a fire could be dealt with. Keeping this in view, the ISI code (1893—1962) recommends somewhat higher values for design of water towers than for other structures. Both these factors have been kept in view in the design that follows :

DESIGN CRITERION

(a) According to Seismic Zoning Map [Fig. 1 page 7 of ISI Code (1893—1962)], Rishikesh is in zone IV. For soft soil, the horizontal seismic coefficient is equal to 0.08. As per clause 5.2.7.1. this factor is to be multiplied by 1.20 due to the tallness of the structure. The modified seismic coefficient is 0.096. The proximity of the boundary fault, however, necessitates special study.

(b) It is assumed that the boundary fault might be responsible for causing an earthquake of magnitude corresponding to N = 4 (The factor 'N' defines the intensity of earthquakes; N = 4 approximately corresponds to an earthquake of magnitude 8.0 at a distance of 25 miles or so from the epicentre).

(c) The structure is assumed to have a damping of 5% of critical.

(d) The design would permit plastic deformations for the strongest ground motion (N = 4) envisaged. This strong ground motion is likely to occur at least once during the life of the structure. Ground motions of lesser intensity are likely to occur more often. The deformation, say, for N=1 will be in the elastic range.

DYNAMIC BEHAVIOUR OF WATER TOWERS.

Water Towers consist of a heavy mass (tank proper and water) supported on top of columns, which act as springs. If the columns are braced only horizontally, the column and brace system will act as a Vierendeel girder to resist the horizontal forces. In this system, the lat ral force is resisted by moments and shears. If diagonal bracing is provided and if sufficient rotation is permitted at the joints, the system would act as a hinged truss system where the forces in the members would be axial.

A water tower resting on columns could be represented as a single degree of freedom system (fig. 1).

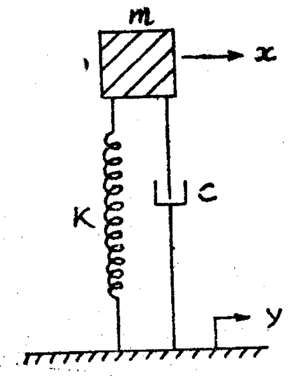

Fig. 1.

The equation of motion 'x' of such a system subjected to a ground motion represented by 'y' is given by

$$m\frac{d^2x}{dt^2} + C\left(\frac{dx}{dt} - \frac{dy}{dt}\right) + k(x-y) = 0 \qquad (1)$$

where 'm' is the mass, 'c' is the coefficient of damping and 'k' the spring constant.

The solution of equation is given by

$$x - y = \frac{1}{p} S_v \qquad \ldots (2)$$

where 'p' is the natural frequency of the system in radians/sec. and equal to $\sqrt{k/m}$.

$$\text{and} \quad S_v = \int_0^t \ddot{y}(\tau) e^{-p(C/C_c)(t-\tau)} \sin p(t-\tau) d\tau \qquad (3)$$

and known as the "Velocity Response Spectrum" of the ground motion. An average velocity spectrum curve is shown in Fig. 7 (IS : 1893—1962). Calculating the period of the sturcture and assuming a suitable value of N, it is possible to evaluate the relative displacement (x—y) which the system would undergo during a corresponding ground motion.

The behaviour in the elastic range could be reasonably predicted as above. For a severe earthquake a design would be possible only if plastic deformations are permitted. The energy input to the structure due to the earthquake would be stored as plastic strain energy in the diagonal braces. The forces in the members may be evaluated on the basis that diagonal braces are subjected to yield stresses. This method of design will localise the failure, if any.

ANALYSIS OF FORCES.

(For details of analysis and design refer Krishna and Chandrasekaran 1963).

(A) Estimation of Weight of the System.

A part of the weight of the column and braces is assumed to be added to the weight at top, to take care of the effect of the weight of column and braces on the vibration characteristics of the system on the basis of Rayleigh's method.

i.	(a) Weight of tank	=	301,500
	(b) Weight of water	=	500,000
			801,500
ii.	(a) Columns	=	130,100 lbs.
	(b) Horizontal Braces	=	61,150 lbs.
	(c) Diagonal Braces	=	2,750 lbs.
			194,000 lbs.
iii.	Weight at top	=	801,500 lbs.
	$\frac{1}{3}$ weight of columns and braces	=	64,700 lbs.
	Total equivalent weight		866,200 lbs.
			Say 870 kips.

Without water, equivalent weight = 370 kips.

(B) Estimation of the Stiffness of the Structure.

In the elastic stage, bulk of the force would be resisted by the columns which are considerably more stiff than the tie rods.

The supporting system could be assumed to be made of springs in series. The stiffness of the spring (one bay) is made of stiffness of columns acting as parallel systems with diagonal braces.

(a) Stiffness of one bay:

i. Stiffness of column k_c is given by

$$k_c = \frac{12EI}{L^3} = 70.0 \text{ kips/in.}$$

Stiffness for 8 columns acting together will be

$$= 560.0 \text{ kips/in}$$

ii. Stiffness of the Diagonals.

$$k_b = F/\Delta = \frac{AE}{L}$$

where k_b = stiffness of one diagonal (refer fig. 2)

F = horizontal force

A = Area of Cross-section of member

Δ = horizontal deflection due to F

E = modulus of elasticity of member

L = equivalent length of member

k_{b_1} = 30.0 kips/in

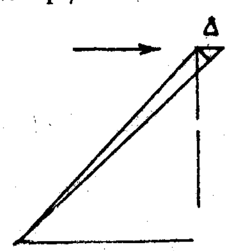

Fig. 2.

Similarly k_{b_2} for the adjoining one is given by

$$k_{b_2} = 44.0 \text{ kips/in}$$

for all braces

$$k_b = 4\,k_{b_1} + 2k_{b_2}$$
$$= 208 \text{ kips/in}$$

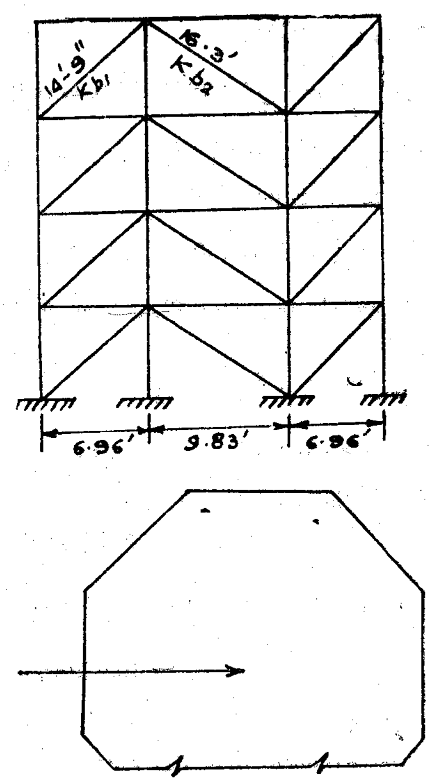

Fig. 3.

iii. Total 'K' per bay
$= 560 + 208$
$= 768$ kips/in

(b) Stiffness for entire structure
$= \frac{1}{4}$ of K
$= 192$ kips/in,

Period of the Structure

$$T = \frac{2\pi}{\sqrt{g}}\sqrt{w/k}$$

$$=0.32\sqrt{870/192} = 0.666 \text{ sec,}$$

(C) Lateral Force due to Earthquake,

i. Elastic Range

Let us assume that $N = 1$

and the damping factor $C/C_c = 5\%$ of the critical for deformations in the elastic range.

From the average velocity spectrum curves (Fig. 7, IS : 1893—1962) for $T = 0.666$ sec, $C/C_c = 0.05$ and $N=1$, we have

$$S_V = 0.46 \text{ ft/sec:}$$

$$(x-y) = \frac{S_V}{p} = \frac{0.666}{2\pi} \times (0.46 \times 12) = 0.58 \text{ inch}$$

This is the total horizontal displacement, and therefore, total shear 'V' is given by

$$V = K(x-y) = 200 \times 0.58 = 116 \text{ kips}$$

The horizontal seismic coefficient α_h is thus given by

$$= \frac{2000 \times 0.58}{870} = 0.133$$

Displacement per bay $\triangle = \frac{1}{4}$ of total $= 0.145''$

(a) Moment in the columns $= k_c \times \triangle \times \frac{1}{2}$
$= 55.2$ kips/ft.

(b) Axial forces in diagonals $= K_{b2} \times \triangle \times \frac{16.3}{9.83}$
$= 10.6$ kips.

ii. Plastic Range

It will be assumed that the energy input to the structure would correspond to a spectral velocity for $N = 4$ and $c/c_c = 5\%$ and S_V calculated on the basis of elastic behaviour. The spectral velocity in the plastic range would be $4 \times 0.45 = 1.84$ ft/sec.

The lateral force coefficient would be calculated on the basis that diagonal braces are subjected to yield stresses. On this basis the lateral force coefficient works out to be 0.20.

DESIGN DETAILS

(A) Columns

Elastic Range

The columns are subjected to direct load, upward and downward forces due to moment of lateral force and bending moment due to shear in the columns.

(i) Direct load on the columns $=125.0$ kips. (adding together the tank and water load as well as self weight of columns and braces we get 1000 kips. approximately, Each column will thus have 125 kips.)

(ii) Maximum Moment in the columns (at the junction of the column and brace)
$= 55.2$ kips ft.

(iii) Upward and downward forces in columns due to moment of lateral force.

(a) Bottom of columns between foundation and lowermost (third) brace $= \pm 135.0$ kips.

(b) Bottom of columns between second and third brace $= \pm 106.0$ kips.

(c) Bottom of columns between first and second brace $= \pm 76.6$ kips.

(d) Bottom of columns between first brace and tank $= \pm 47.2$ kips.

Net forces in columns

(i) Maximum moment $= 55.2$ kips. ft.

(ii) Direct load in kips

(a) above foundation $= +260$ or -10.0
(b) above third brace $= +231$ or $+19.0$
(c) above second brace $= +201.6$ or $+48.4$
(d) above first brace $= +172.2$ or $+77.2$

Section Assumed

18″ × 18″ square

Longitudinal reinforcement 8 bars 7/8″ ϕ + 4 bars 3/8″ ϕ

Laterals

(i) Ties $\frac{3}{8}$″ ϕ at 8″ c/c

(ii) Spiral $\frac{3}{8}$″ ϕ at 4″ pitch for a length of 3′ 0″ on either side of brace.

Minimum ultimate cube strength of concrete 2250psi.

m = 15

Area provided = 397.4 sq. in.

I = 12,210 in^4.

Stresses

Just above foundation

(a−1) when direct load = 260 k
and Moment = 55.2 k ft.
Stresses = 1144 psi (Comp) or 166 psi (Comp)

(a−2) when direct load = − 10.0 kips
and Moment = 55.2 k. ft.

Stresses are

Concrete = 641.5 psi

Steel = 26,830 psi

These high stresses in concrete (for a−1 condition) and in steel (for a−2 condition) is permitted because these include stresses due to earthquake the force due to which has been taken to be much higher than that recommended in the code. The increase of working stresses by 50% under the circumstances is reasonable. At all other levels, stresses are lower than the above condition. In actual practice, however, there will be a redistribution of stresses due to semi-plastic rotations, and will lower the stresses below the values obtained above.

Plastic range

Loads

(i) Downward load due to self weight = 125 kips.

(ii) Upward and downward forces due to moment of lateral force of 0.2 w = ± 202.5 kips.

Net forces

(a) Downward = 327.5 K

(b) Upward = 77.5 K

Stresses

824 psi in concrete (comp)
14,800 psi in steel (tension)

(B) Diagonal Braces

Elastic Range

1″ ϕ m.s. rods are used as diagonals

Area of steel = 0.783 sq. in

Stress = 13,550 psi (tensile)

If the earthquake is very severe, the joints would become plastic and permit large deformations of diagonals. In that case, the diagonals would be fully stressed.

Plastic Range

Energy to be dissipated

$= \frac{1}{2}$ W/g. S_v^2

= 45,600 ft. lbs.

Considering half the diagonal braces acting in tension, average plastic strain = 0.0068

In extreme cases actual values of strain may be double that of the average strain and this would correspond to a maximum displacement of 6″ at the tank level which is considered as an upper limit.

(C) Horizontal Braces

Elastic Range

Maximum moment = 156.4 kft.

Maximum shear = 22,400 lbs.

Assume 12″ × 21″ section

4 bars 1$\frac{1}{2}$″ ϕ at top and bottom

$\frac{3}{8}$″ ϕ stirrups at 6″ c/c throughout. Minimum ultimate cube strength of concrete = 2250 psi.

Stresses.

Stress in tensile Steel
= 27,700 psi

Stress in concrete
= 1,180 psi

Shear stress
= 110.5 psi

This is permissible, but as a precautionery measure against small torsion that may develop $\frac{3}{4}''$ dia. stirrups at 6″ are provided.

Plastic Range

Horizontal Braces act as structs.

Axial force = 15,600 lbs.

considering steel alone

stresses in steel = 15,600/8 = 1,950 psi.

(D) Design of Foundation.

Design coefficient α_h	=	0.20
Depth of foundation	=	8′ 0″
Inner Radius of annular raft	=	6′ 0″
Outer Radius —do—	=	24′ 0″
Area of the footings A	=	177, 500 in^4
Downward load P	=	2342 kips.
Overturning moment M	=	12,200 kips ft.
Direct Stress	=	1536 psf.
Due to Moment	=	±1650 psf.
Total pressure	=	3125 psf. or—114 psf.

For the extreme case of a severe earthquake these values of stresses in soil may be permitted.

Width of footing = 10.5′

Max. Moment = 1394,000 lbs. ft.

Depth reqd. = 32″

Overall depth provided is 42″ with clear cover to bars 3″

Steel required = 2.24 sq. in.

Provide 1″ ϕ at 4″ c/c.

At top provide $\frac{3}{8}''\phi$ at 4″ c/c to take care of negative moment.

Check for Torsion and shear.

Taking moment of the net forces.

M = 758,000 lbs. ft.

Torsion = 379,000 lbs. ft.

Torsional Resistance.

Consider a rectangular beam 30″ deep and 102″ wide

$$f_s = \frac{T}{q\,b^2 d} = 54 \text{ psi where } q = 0.27$$

Toal Load = 230 kips

f_s = 42 psi

Direct Shear

Total Max. Shear = 54 + 42 = 96 psi

Beam—30″ deep and 102″ wide.

Moment = 3190 kips in.

Depth required = 15.2″ wide beam

A_s=5.04 sq. in

HYDRODYNAMIC PRESSURES IN THE TANK

Suppose the tank to be given a maximum acceleration of 0.20 g and subjected to a maximum displacement of 6″.

For the purpose of analysis let us assume that the tank is a cylinder of radius 13.5 ft. and height 14.0 ft.

(Formulae from reference 1).

Impulsive Pressures on the Tank.

$$P_i = W.\alpha_h.\frac{\tanh\sqrt{3}\,R/h}{\sqrt{3}\,R/h}$$

$$= 0.11\ W$$

acting at a height h_o from bottom

$$\text{where } h_o = \tfrac{3}{8}h\left\{1+(4/3\frac{\sqrt{3}\,R/h}{\tanh\sqrt{3}\,R/h}-1)\right\}$$

$$= 0.77\ h$$

Convective Pressures on the tank.

$$P_c = 1.2\ M_1\ g\ \theta_h$$

$$M_1\ g = W \times \tfrac{1}{4} \times \tfrac{25}{34} \times \sqrt{\frac{27}{8}}\frac{R}{h}\tanh\sqrt{\frac{27}{8}}\ h/R$$

$$\theta_h = A\sqrt{\frac{27}{8}}\,\frac{1}{R}\tanh\sqrt{\frac{27}{8}}\,\frac{h}{R} \times 5/6$$

$$P_c = 0.019\ W$$

Acting at a height of h_o from bottom

$$\text{where } h_o = h\left(1-\frac{\text{Cosh}\sqrt{\frac{27}{8}}\cdot\frac{h}{R}-2.0}{\sqrt{\frac{27}{8}}\frac{h}{R}\,\text{Sinh}\sqrt{\frac{27}{8}}\frac{h}{R}}\right)$$

$$= 0.77\ h$$

Hydro – static pressure

$$P_h = \frac{wh}{2} \times \frac{\pi \times 2R \times h}{w \times \pi R^2 h}\ W$$

$$= Wh/R = 1.07\ W$$

Acting at a height of 0.33 h from bottom

The sum of impulsive and convective pressures, if assumed to attain maximum values simultaneously will be equal to 0.129 W which is equal to 12% of that of hydrostatic pressure. The sum of the moment of impulsive and convective pressures will be equal to 0.099 Wh which is 27.8% of that of hydrostatic pressure.

The above increase in pressure and moment due to hydrodynamic effect is accounted for by the normal increase in the working stresses. However, care should be exercisd ethat all reinforcements are anchored properly.

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