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Seismic Design of Frame Staging for Elevated Water Tanks

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SYNOPSIS

I.S. code provisions for aseismic design of elevated water tanks have been reviewed. It is seen that, due to absence of a suitable value of performance factor for tanks, the code provides for rather low seismic design force for these structures. Simple expressions are given which allow calculation of staging stiffness, and hence the time period, while incorporating beam flexibility.

1. INTRODUCTION

In India the seismic design criteria is provided by IS:1893-1984 (10) which gives minimum loading standards and IS:4326-1976 (11) which contains design and detailing requirements for construction of buildings. IS:11682-1985 (12) gives the criteria for design of RCC staging of such structures. In this paper provisions pertaining to aseismic design of elevated water towers are reviewed and several suggestions given for making these more rational. The design seismic force for the water tank depends on its flexibility and hence on the time period. Many engineers in the country tend to evaluate the time period by taking the column stiffness as $12EI/L^3$ which is based on the assumption of bracing beams being infinitely rigid. In practice these beams are quite flexible and therefore the above assumption very much overestimates the staging stiffness. An approximate procedure is presented for calculation of staging stiffness, and hence time period, giving due consideration to the beam flexibility.

2. REVIEW OF INDIAN CODAL PROVISIONS

2.1 Lateral Design Force

IS:1893 requires that the design lateral force shall be taken as $F = \alpha_h W$; where $\alpha_h = \beta I F_o S_a/g$; β = coefficient depending upon the soil-foundation system; I= importance factor; F_o = seismic zone factor; and S_a/g = average acceleration coefficient obtained from acceleration spectra given in the code. Here, performance factor, K, which the code uses for computing base shear for buildings, is absent. This implies that K is 1.0 for the elevated water towers which is also the value used for buildings with ductile moment resisting frames. This is unreasonable as the elevated tank type structures have lower energy absorbing capacity and poor ductility as compared to those in ductile moment resisting frame buildings. The seismic codes all over the world prescribe a performance factor which is 2.5 to 4.0 times higher for elevated water tanks than that for the ductile buildings. Thus while the lateral design force prescribed for buildings in Indian codes is of about the same order as that in the other codes for zones of comparable seismicity; the design lateral force provided by Indian code for elevated tanks is far below that by seismic codes in other countries. Hence it is necessary to introduce a suitable value of performance factor, say 3.0, for elevated tanks in IS:1893.

2.2 Single Degree of Freedom Idealisation

IS:1893 suggests a single degree of freedom idealisation of elevated tanks which is reasonable only for closed tanks completely full of water. For tanks with a free water surface two-mass idealisation (7) is preferred. This view has been supported by several investigators (1, 2, 3, 6, 9, 15, 17) based on experimental and computational observations. Calculations for a few tanks indicate that the single degree of freedom representation overestimates the lateral design force; the difference in value depends on the geometrical properties of the tank and relative stiffness of the staging. Hence, there is a good case for incorporating two degree of freedom idealisation in IS:1893 also.

2.3 Staging Stiffness and Time Period

In both single- and two-degree of freedom idealisations stiffness of the staging needs to be obtained for calculating the time period. It has been emphasised that a design criteria must also include the procedure for time period calculation (8). This is because there can be large variation in time period calculated by different designers based on different assumptions and/or different procedures. The Handbook SP: 22-1982 (18) takes the column stiffness as $12EI/L^3$ which assumes horizontal bracings to be infinitely rigid. However the bracing beams are clearly not rigid and this assumption overestimates the stiffness substantially. It is suggested that IS:1893 must specify that in time period calculation bracing girders are not to be treated as rigid.

2.4 Hydrodynamic Pressure on Tank Walls

IS:1893-1984 gives formulae for determining this pressure distribution due to impulsive pressure (and not that due to convective pressure) which are based on Ref (7) However, the code must give complete details of the equivalent mechanical analog model proposed by Housner in the same publication. Further, clause 5.2.7.1 of the code states that the convective pressures can be ignored in comparison with the impulsive pressures. However, the convective pressures can be a dominant factor for certain proportions of the tank and the structure. Moreover, IS:11682-1985 clearly states that "whereever required the effect of surge due to wave formation of the water may be considered." Hence IS:1893 must provide complete details of Housner's model and omit the clause 5.2.7.1.

It is known that the seismic effects on flexible tanks are substantially greater than those induced in similarly excited rigid tanks (5, 20). Simple procedures for evaluating hydrodynamic force in flexible tanks have been developed (5). It is desireable to caution the designer about larger forces induced in flexible tanks and to provide procedure for their analysis in the IS code or in SP:22.

2.5 Ductility Requirements

As its title itself suggests, IS:4326-1976 gives design requirements for buildings only. This makes many design engineers think that ductile detailing is not required for elevated tanks even in zones IV and V. Hence the scope of IS: 4326-1976 must be enlarged to also include structures other than buildings. A figure in IS:11682-1985 states that "where design seismic coefficient is 0.05 or more reference to clauses 7.2 to 7.4 of IS:4326-1976 shall be made to cater for ductility requirement" This is in right spirit. However, the term "design seismic coefficient" in its present form is very confusing. IS:1893 defines the horizontal seismic coefficient, α_h , in two ways which mean two different things. The future editions of these codes must be very clear and specific on ductile detailing requirements for tanks.

2.6 Design for Torsion

It has been recognised that it is impossible to prevent torsional response in elevated water towers (16, 19). The elevated towers constructed are seldom truly symmetric due to presence of staircase. Further, during intense shaking, with failure of one or two bracings, the structure will go into the torsion mode. SEAOC-1980 makes it mandatory to design elevated water towers for shear stress developed due to horizontal torsion resulting from an accidental eccentricity equal to 5% of the largest lateral dimension. It is very much desirable for IS:1893 to also require design for accidental torsion.

3. STAGING STIFFNESS

Well known protal method (4) has been suitably developed to account for the bracing flexibility and the three dimensional behaviour of the structure (13). The point of inflection is assumed to occur at the midspan of beams and columns. The compatibility requires that lateral deflection is same in all the columns of a panel. Hence, the columns share lateral force in proportion to their lateral stiffness. Considering a column between two bracing levels (Fig. 1), the deflection in column, Δ_c, due to shear, V, can be calculated. This gives the lateral stiffness of one column as (13)

$$K_{column} = \frac{V}{\Delta_c} = \frac{12E_cI_c}{h^3}\left[\frac{1}{1 + 0.5\,\dfrac{E_cI_c}{h^2}\left(\dfrac{h_a+h}{\Sigma k_{bt}} + \dfrac{h_b+h}{\Sigma k_{bb}}\right)}\right] \tag{1}$$

$$\Sigma k_{bt} = E_b \sum_{1}^{N_b} \frac{I_{bt}}{L} \cos^2\alpha \tag{2}$$

$$\Sigma k_{bb} = E_b \sum_{1}^{N_b} \frac{I_{bb}}{L} \cos^2\alpha \tag{3}$$

where E_b, E_c = modulus of elasticity of beam and column, respectively; I_b, I_c = moment of inertia of beam and column, respectively; h = height of panel under consideration; h_a, h_b = height of the panel just above and the panel just below the panel under consideration, respectively; N_b = number of beams meeting the column under consideration at the bracing level; L = span of bracing girder; and α = angle the bracing girder makes with the direction of lateral force. Most tank stagings have identical bracing girders and have equal panel heights. Moreover, the top end of column in topmost panel and bottom end of column in bottommost panel is fixed against rotation. Using these considerations, column stiffness can be obtained as (13)

$$K_{column} = \frac{12E_cI_c}{h^3}\left[\frac{\Sigma k_{bg}}{\Sigma k_{bg} + 2k_c}\right] \quad \text{(intermediate panels)} \qquad (4)$$

$$K_{column} = \frac{12E_cI_c}{h^3}\left[\frac{\Sigma k_{bg}}{\Sigma k_{bg} + k_c}\right] \quad \text{(top and bottom panels)} \qquad (5)$$

where $\Sigma k_{bg} = \Sigma k_{bt} = \Sigma k_{bb}$; $k_c = E_cI_c/h$; and the summation is from 1 to N_b
To evaluate the stiffness of a panel, stiffness of individual columns may be calculated from Eq. (4) or Eq. (5) and this may be summed for all the columns of that panel.

A direct expression for the determination of panel stiffness, without calculating individual column stiffness, has been obtained by further approximations (13). For tank stagings with uniform panel height, identical columns, identical bracing girders, and assuming that all the columns are located on the periphery of one circle panel stiffness is obtained as

$$K_{panel} = \frac{12E_cI_cN_c}{h^3}\left[\frac{\frac{E_bI_b}{L}}{\frac{E_bI_b}{L} + \frac{2E_cI_c}{h}}\right] \quad \text{(intermediate panels)} \qquad (6)$$

$$K_{panel} = \frac{12E_cI_cN_c}{h^3}\left[\frac{\frac{E_bI_b}{L}}{\frac{E_bI_b}{L} + \frac{E_cI_c}{h}}\right] \quad \text{(top and bottom panels)} \qquad (7)$$

4. EXAMPLE

Consider the water tank in Example (6) of Handbook (SP:22) ignoring its diagonal bracings. The tank has 4 panels of height 4 m each (total height 16 m) with 8 columns located on a circle of radius 4.5 m. Column size is 520 mm diameter. Beam size is assumed to be 200 X 500 mm. Both columns and beams are assumed to be of M20 concrete. The mass in tank full condition is 9×10^5 kg and in tank empty condition it is 3×10^5 kg.

Now, $E_c = E_b = 5700 \sqrt{20} = 25{,}500$ MPa; $N_c = 8$; $I_b = 2.08 \times 10^9$ mm^4; $I_c = 3.59 \times 10^9$ mm^4; h = 4,000 mm; and L = 3,534 mm. The moment of inertia has been calculated assuming gross uncracked section ignoring steel. Substituting these values into Eqs. (6) and (7), the panel stiffness is obtained as 33,900 N/mm for the two intermediate panels and 54,400 N/mm for the two end panels. This gives the staging stiffness as 10,400 N/mm. On the other hand, if one were to treat the bracing girders as rigid, panel stiffness is obtained as 137,000 N/mm and the staging stiffness as 34,200 N/mm. This staging stiffness is 3.3 times what it should be. The time period, for tank full condition, is 1.85 sec considering beams flexible, and 1.02 sec otherwise. It is thus clear that the beam flexibility could not be ignored.

Let this tank be located in seismic zone V ($F_o = 0.40$). Let $\beta = 1.0$ and I = 1.5. For 5 % damping, $S_a/g = 0.11$ for T = 1.02 sec and $S_a/g = 0.06$ for T = 1.85 sec. If staging is designed treating the beams as rigid and with no performance factor, $\alpha_h = \beta\ I\ F_o\ S_a/g = 0.066$ and the design lateral load $V = \alpha_h W = 594$ kN. However, introducing performance factor K = 3.0 and accounting for the beam flexibility, $\alpha_h = K\ \beta\ I\ F_o\ S_a/g = 0.108$ and the design seismic force V = 972 kN. Thus the increase in design force for this tank is only about 64 % This increase will be reduced somewhat if two-degree of freedom idealisation is adopted.

In the above example, beam length (L) and the column height (h) have been taken on the basis of centre to centre distances. This ignores the additional stiffness provided by rigid member zones at beam-column joints. To incorporate this, one may take clear span and clear height, respectively.

5. SUMMARY AND CONCLUSIONS

The Indian code provisions for aseismic design of elevated tanks have been reviewed. The code must include an appropriate value of performance factor, say 3.0, for calculation of seismic design force for water tanks. An earthquake design criteria is incomplete unless clear specifications are included on how to calculate time period. The code is defficient on this. The Handbook (SP:22) solves an example problem treating the bracing beams as infinitely rigid which is unrealistic. The code should also give more details of the Housner's mechanical analog model for hydrodynamic forces. Provision for sloshing of liquid should be included. The requirements regarding ductile detailing of tanks are vague in the present form of codes and need to be clearly specified. A method for calculating the staging stiffness including beam flexibility and without having to resort to finite element type analysis has been presented. The method is based on the well known portal method which has been suitably developed to incorporate the beam flexibility and the three dimensional behaviour of the staging.

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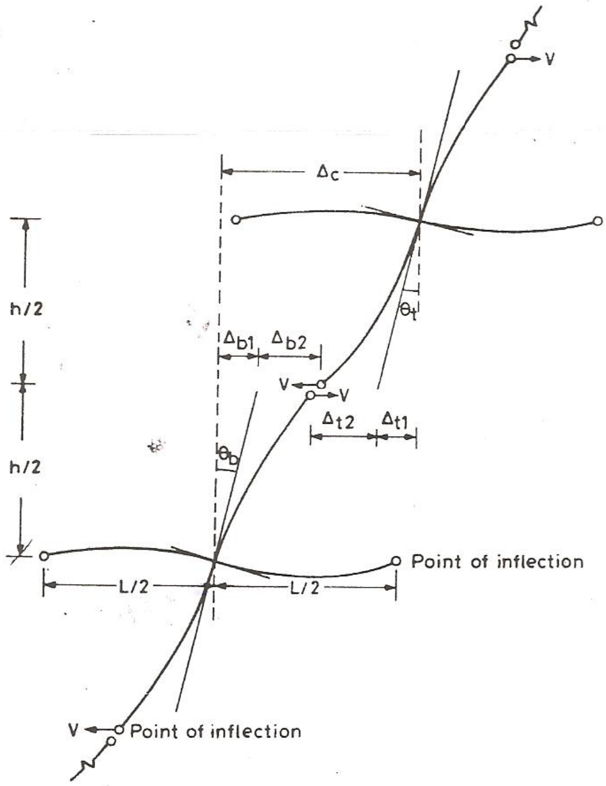

$$\Delta_{b1} = \theta_b \frac{h}{2}, \quad \Delta_{t1} = \theta_t \frac{h}{2}$$

$$\Delta_{b2} = \Delta_{t2} = \frac{V(h/2)^3}{3E_c I_c}$$

$$\Delta_c = \Delta_{b1} + \Delta_{b2} + \Delta_{t1} + \Delta_{t2}$$

Fig. (1) Deflection Between Successive Joints