1. INTRODUCTION

1.1 Introduction

Water tanks are lifeline items in the aftermath of earthquakes. The current practice of designing elevated water tanks (Figure 1.1) is resulting in tanks that are extremely vulnerable under lateral forces due to earthquake shaking, as observed in the recent earthquakes in India. In addition, seismic design is not being performed in many seismic areas in India, supervision during construction is inadequate, and critical items are not being implemented, like providing 135° degree hook ends in transverse reinforcement. This makes it necessary to assess safety of existing elevated water tanks of that are largely gravity designed and poorly constructed. Based on the above common deficiencies, a procedure is described below for *Rapid Assessment* of *earthquake safety* of existing Elevated Water Tanks with Reinforced Concrete Frame Staging (Circular Columns) in India, considering primarily the shear capacity of beams and columns, even though flexure capacities of beams and columns also are critical parameters governing earthquake safety of such tank.

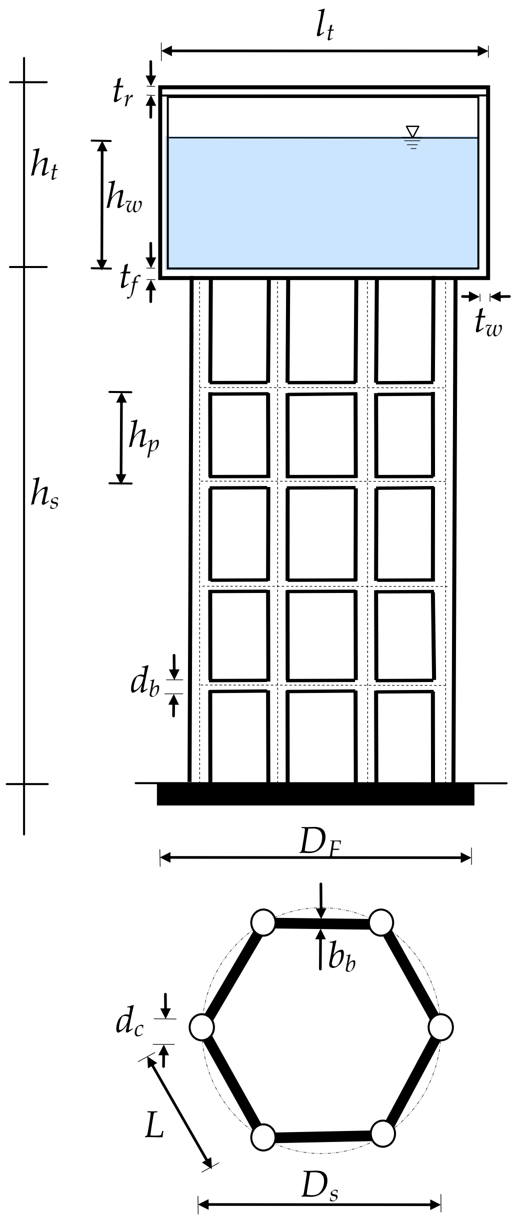


Figure 1.1: Elevated Water Tanks with Frame Staging

1.2 Assumptions

The following assumptions are made in the *Rapid Assessment of Seismic Safety of Elevated Water Tanks with Frame Staging*:

(1) *Elevated water tank is a single degree of freedom system*: When a tank containing fluid with a free surface is subjected to earthquake ground motion, the fluid exerts impulsive and convective pressure on the tank. Convective pressures during earthquakes are considerably less in magnitude as compared to impulsive pressures, and therefore usually neglected. Hence, the assumption is justified, that elevated water tanks can be regarded as single degree of freedom systems with their mass concentrated at their centers of gravity.
(2) Columns and beams have points of contra-flexure at their mid-spans.
(3) Axial deformations in the bracing beams are small.
(4) Earthquake lateral forces are shared in proportion to lateral stiffness of the columns, which are rotationally restrained by horizontal beams.
(5) Frame staging has uniform panel height, identical brace beams and identical columns.
(6) Columns are located on the periphery of a circle.
(7) Mass of the tank is greater than that of the staging.
(8) Damping is assumed to be 5% in the tank-staging system.
(9) Dimensions considered are centerline measurements.

1.3 Shear Demand on Beams and Columns

The shear demand on beams and columns is estimated by

(1) Obtaining the design lateral earthquake force on the tank as per the expression given in IS:1893(Part 1)-2002, and
(2) Obtaining the design shear force on the columns and beams of the frame staging through approximate structural analysis method.

1.3.1 Design Horizontal Seismic Force V_B

The design seismic base shear V_B shall be determined by

$$V_B = A_h W_s, \tag{1}$$

V_B is estimated for both tank empty and tank full conditions, and the higher used. In Eq.(1), design horizontal seismic coefficient, A_h shall be obtained by:

$$A_h = \frac{Z_{SS} I}{R}\left(\frac{S_a}{g}\right), \tag{2}$$

where

Z_{SS} = Site-specific horizontal acceleration expected at the location of the Tank (expressed as a fraction of Acceleration due to gravity)

I = Importance factor = 1.5 for water storage tanks (because of post-earthquake importance);

R = Response reduction factor

= 1.8 for frames without ductile detailing: *Ordinary Moment Resisting Frame (OMRF)*

= 2.5 for frames with ductile detailing: *Special Moment Resisting Frame (SMRF);* and

S_a/g= Average response acceleration coefficient depending on soil type

$$= \begin{cases} 2.5 & 0 \le T \le 0.4 \\ 1.00/T & 0.4 \le T \le 4.0 \end{cases}$$ for Soil Type I **(N>30)**

$$= \begin{cases} 2.5 & 0 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.0 \end{cases}$$ for Soil Type II **(15<N<30)**

$$= \begin{cases} 2.5 & 0 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.0 \end{cases}$$ for Soil Type III **(N<10)**

in which N is the Standard Penetration Test value.

1.3.2 Natural Period T of Tank

In Eq.(2), the natural period T of the tank shall be obtained by:

$$T = 2\pi\sqrt{\frac{M}{K}}\,, \tag{3}$$

where M is the mass of the empty/filled tank, and K the lateral stiffness of the staging.

1.3.3 Seismic Mass M and Weight W_s

Research showed that when mass M of the tank is much greater than the mass of the staging $m_{staging,}$ the equivalent mass may be taken as:

$$M = m + \frac{m_{staging}}{3}\,. \tag{4}$$

In Eq.(4), only 1/3 of the staging mass is found to be effective at the centroid of the tank from energy considerations; 2/3 of the staging mass is found to be effective at the bottom of the tank (at the ground level) and does not contribute to the vibration of the system.

1.3.4 Lateral Stiffness K of Frame Staging

The design seismic force for the water tank depends on its flexibility and hence on the time period. Often, column stiffness is considered as $12EI/L^3$, which is based on the assumption that bracing beams are infinitely rigid. In practice, these beams are flexible and therefore the assumption overestimates the staging stiffness.

Portal Method has been used to estimate the bracing flexibility considering three dimensional behavior of frame staging. The assumption of point of contra-flexure at the centre of the bracing girders implies equal rotation of all joints at any bracing level. Hence, rotational stiffness of the staging at any bracing level is approximated by adding the rotational stiffness of all the joints at the bracing level. Thus, the staging is modeled as a column connected to a rotational springs at each bracing level; the rotational spring stiffness at each joint is equal to the sum of the rotational stiffness contributed by beams and columns meeting at that joint.

Most tank stagings have identical bracing girders and equal panel heights. Moreover, the top end of column in topmost panel and bottom end of column in bottommost panel are fixed against rotation. For the most commonly used stagings, having all the columns along the periphery of a circle, panel stiffness is obtained as below:

$$k_{panel} = \frac{12E_cI_cN_c}{h^3}\left[\frac{\frac{E_bI_b}{L}}{\frac{E_bI_b}{L} + \frac{2E_cI_c}{h}}\right] \text{ for intermediate panels, and} \tag{5}$$

$$k_{panel} = \frac{12E_cI_cN_c}{h^3}\left[\frac{\frac{E_bI_b}{L}}{\frac{E_bI_b}{L} + \frac{E_cI_c}{h}}\right] \text{ for top and bottom panels.} \tag{6}$$

The stiffness of individual columns in a panel is summed to obtain panel stiffness. The overall stiffness of the staging can then be determined by treating the panels as springs in series along the height.

When the tank structure is located on soft soil, the support is not rigid and hence bottommost panel is no more fixed against rotation. Under these conditions, the panel stiffness is calculated using Eq.(5), which accounts for end rotation of the columns.

(a) Shear V_{bj} in beams

As most of the mass is concentrated at the top, the tank staging is subjected to seismic force only at its top, and hence, the total shear is same in all the panels. The force distribution in the beams and columns of a typical panel section is shown in Figure 1.2. The shear in those beams will be maximum, for which the vertical plane will contain the bending axis of the whole staging with half the number of columns on either side of the plane as shown in Figure 1.3a(1) and 1.3b(1). So, the largest shear, V_{bj}, in beams at beam level j, is

$$V_{bj} = \sum_{i=1}^{N_c/2} \frac{F_{i,j} - F_{i,j+1}}{2}, \tag{7}$$

where $F_{i,j}$ and $F_{i,j+1}$ are the axial forces in column i of panels j and $(j+1)$ respectively. By Cantilever Method, axial force in column i of panel j is given by

$$F_{i,j} = \frac{2V_b H_j x_j}{N_c R_s^2}, \tag{8}$$

where x_i is the distance of column i measured along the direction of lateral force , V_b, from the centre of the staging, R_s, radius of the frame staging and H_j is the height of the point of application of lateral load from point of contraflexure in panel j. Simplified equation

$$V_{bj} = \frac{V_b Y_j}{N_c R_s} \operatorname{cosec}\left(\frac{\pi}{N_c}\right), \tag{9}$$

where Y_j, is the distance between point of contraflexure of panel j and ($j+1$) which is the panel height h_p, result of assuming the point of contraflexure to be at the beam midspan. An overstrength factor of 1.4 is used to account for the reserve strength of the beams due to partial safety factors and material overstrength, as specified in IS 1893 (Part 1): 2002. Hence, Eq. (9) is modified as

$$V_{bj} = \frac{1.4 V_b h_p}{N_c R_s} \operatorname{cosec}\left(\frac{\pi}{N_c}\right). \tag{10}$$

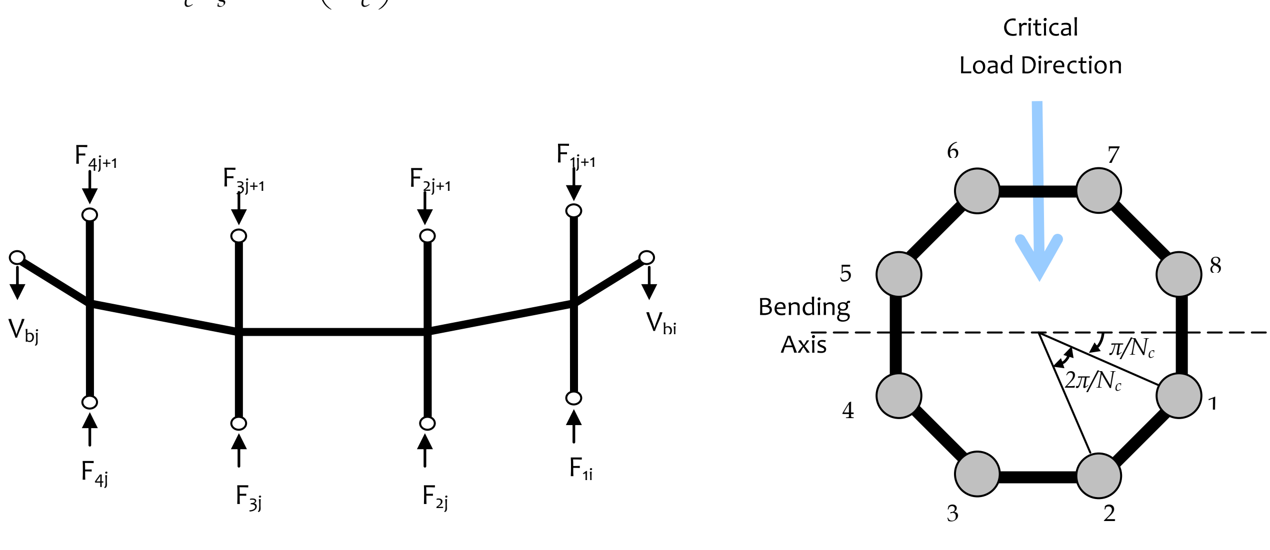
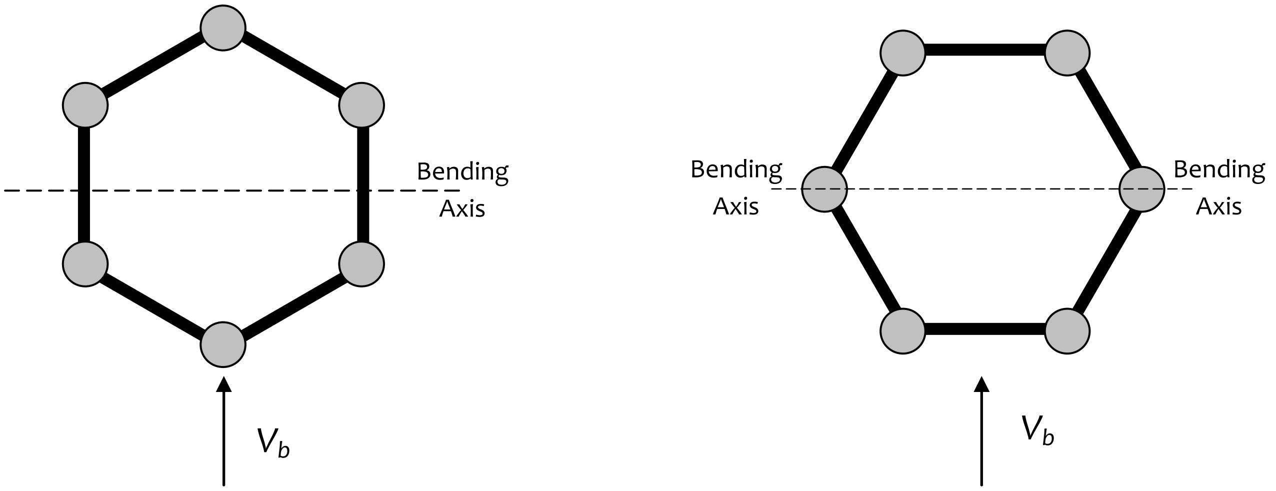
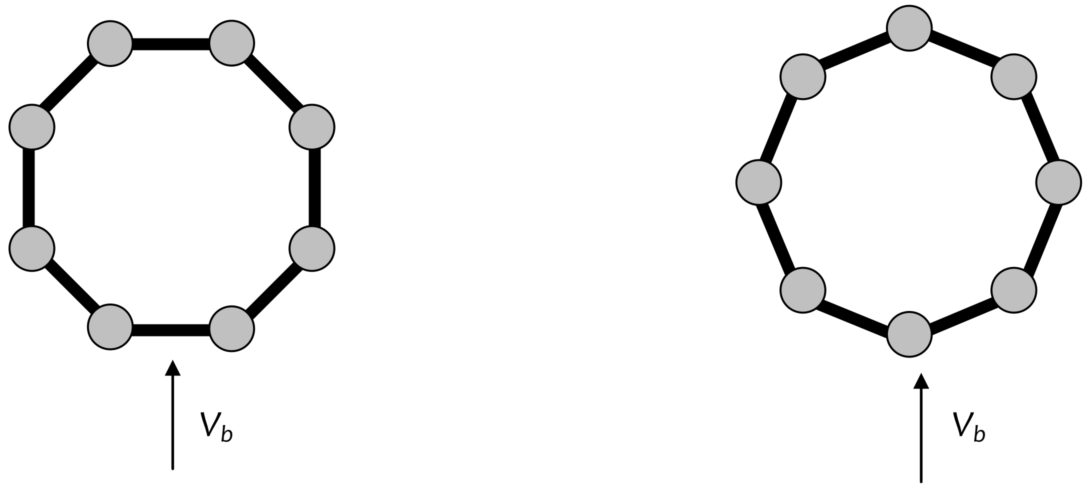


Figure 1.2: Determination of shear forces in beams of Frame Staging of Elevated Water Tanks



(1) Shear force in beams *(2) Shear force in columns*

(a) $N_c/2$ is odd



(1) Shear force in beams *(2) Shear force in columns*

(b) $N_c/2$ is even

Figure 1.3: Critical direction for forces in beams and columns of Frame Staging of Elevated Water Tanks

(b) Shear V_c in columns

(i) Intermediate Panel

When the direction of lateral force is such that the two diametrically opposite columns are situated on the bending axis of the whole staging, these columns, with beams meeting at the angle of π/N_c with the direction of the lateral force, carry the maximum shear force. The critical directions of lateral force are depicted in Figure 1.3a(2) and 1.3b(2). From the equilibrium of joints on these critical columns, the largest shear V_c in the columns of intermediate panels is

$$V_{bj} = \frac{2\overline{V_{bj}}L}{h_j + h_{j+1}} \cos\left(\frac{\pi}{N_c}\right), \qquad (11)$$

where h_j and h_{j+1} are the heights of panels j and $j+1$ respectively, and $\overline{V_{bj}}$ is the shear force in the beams connected to the columns carrying largest shear. Incorporating the overstrength factor and deriving expressions for $\overline{V_{bj}}$ in a similar manner as for V_{bj}, and substituting in Eq. (11),

$$V_{bj} = \frac{1.4 V_p L}{N_c R_s} \cos\left(\frac{\pi}{N_c}\right) \cot\left(\frac{\pi}{N_c}\right). \tag{12}$$

(b) End Panels

Columns are assumed to be completely fixed against rotation at top ring beam level, and against rotation and translation at foundation level. All columns have equal restraint at the fixed end. At the other ends, fixity against rotation depends on beam configurations. Because of this difference in end conditions of the columns in the end panels, a redistribution of shear takes place among the columns in the end panels. The largest moment M_{cr} of the column at the restrained end of the end panel consists of the fixed-end moment due to lateral displacement of the braced-end of the end-panel column and the released moment due to rotation of the braced end. The final expression for the largest moment M_{cr} of the column is

$$M_{cr} = \frac{V_b}{N_c}\left[2\bar{y}\cos^2\left(\frac{\pi}{N_c}\right) + \frac{Y_j}{3} K_r\right], \tag{13}$$

where $\bar{y}$ is the distance of point of inflection of end panel from the braced end, which in this case is panel height $h_p/2$ and K_r being the ratio of Moment of Inertia of column and beam. Hence Eq.(13) becomes,

$$M_{cr} = \frac{V_b h_p}{N_c}\left[\cos^2\left(\frac{\pi}{N_c}\right) + \frac{K_r}{3}\right]. \tag{14}$$

The shear force V_c in a column (Figure 1.4), including the overstrength factor, is given by

$$V_c = \frac{1.4 \times 2 M_{cr}}{h_p} = \frac{2.8 V_b}{N_c}\left[\cos^2\left(\frac{\pi}{N_c}\right) + \frac{K_r}{3}\right]. \tag{15}$$

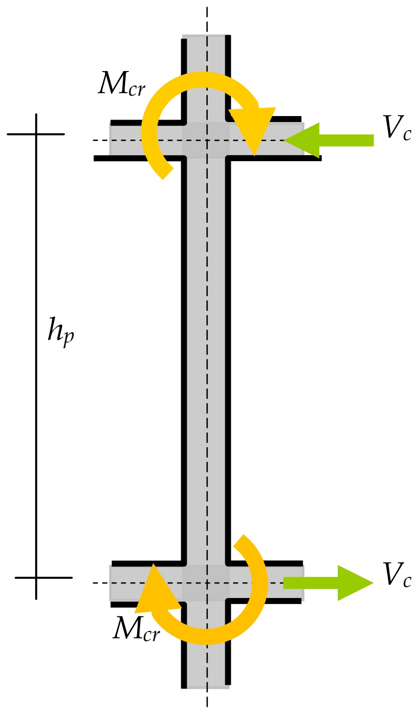


Figure 1.4: Shear Force in Columns in the intermediate panel of Frame Staging of Elevated Water Tanks

1.4 Shear Capacity and Shear Checks in Beams and Columns

1.4.1 Shear Capacity of Beams and Columns

The Shear Capacity of Beams and Columns are calculated as sum of the shears resisted by concrete V_{uc} and the transverse steel reinforcement V_{us}, as

$$V_{u,member} = V_{uc} + V_{us} \tag{16}$$

For a percentage of longitudinal reinforcement ρ in staging members and the given grade of concrete, from Table 19 of IS:456-2000, the *design shear strength of concrete* τ_c is obtained.

Shear resistance offered by concrete is calculated as

$$V_{uc} = \tau A_c, \tag{17}$$

where

A_c = Area of concrete ; and

τ = Design Shear Strength of the member,

And, shear resistance offered by transverse steel is calculated as

$$V_{us} = 0.87 f_y A_{t\_st} \frac{d}{s_v}, \tag{18}$$

where

f_y = Yield Stress of Steel reinforcement

A_{t_st} = Area of cross section of transverse reinforcement

d = Depth of Member

s_v = Spacing of transverse reinforcement

If the *Shear Capacity* $V_{u,member}$ of the frame staging is greater than the *Shear Demand* $V_{d,member}$ on the member, *i.e.*,

$$V_{u,member} > V_{d,member}, \tag{19}$$

then the staging is considered to be safe.

1.5 Checks for Stability of Shaft Staging

For the water tank to be safe against overturning, the *Overturning Moment* M_{OT} generated due to the lateral earthquake shaking should be smaller than the Restoring Moment M_R due the self weight of the tank system (*i.e.*, tank, staging and foundation). But, Restoring Moment is reduced when the effects due to vertical motion of earthquakes are considered. The design vertical acceleration spectrum is taken as two-thirds of the design horizontal acceleration spectrum as specified in Clause 6.4.2. of IS 1893 (Part 1): 2002. *i.e.*, $V_v = \left(2/3\right) V_b$. To ensure safety, a factor of safety of 1.5 is considered to be a minimum value against overturning, *i.e.*,

$$M_R\left(1 - \frac{2}{3}A_h\right) \geq 1.5 M_{OT} \tag{20}$$

or

$$\left(W_{\text{tank}} + W_{\text{staging}} + W_{\text{foundation}}\right)\left(1 - \frac{2}{3}A_h\right)\frac{D_F}{2} \geq V_B\left(h_s + \frac{h_t}{2}\right) \tag{21}$$

where

V_b = Base Shear;

h_s, h_t = height of staging and tank, respectively;

W_{tank}, $W_{staging}$ = Weight of tank and staging respectively;

$W_{foundation}$ = Weight of foundation; and

D_F = Diameter of foundation.

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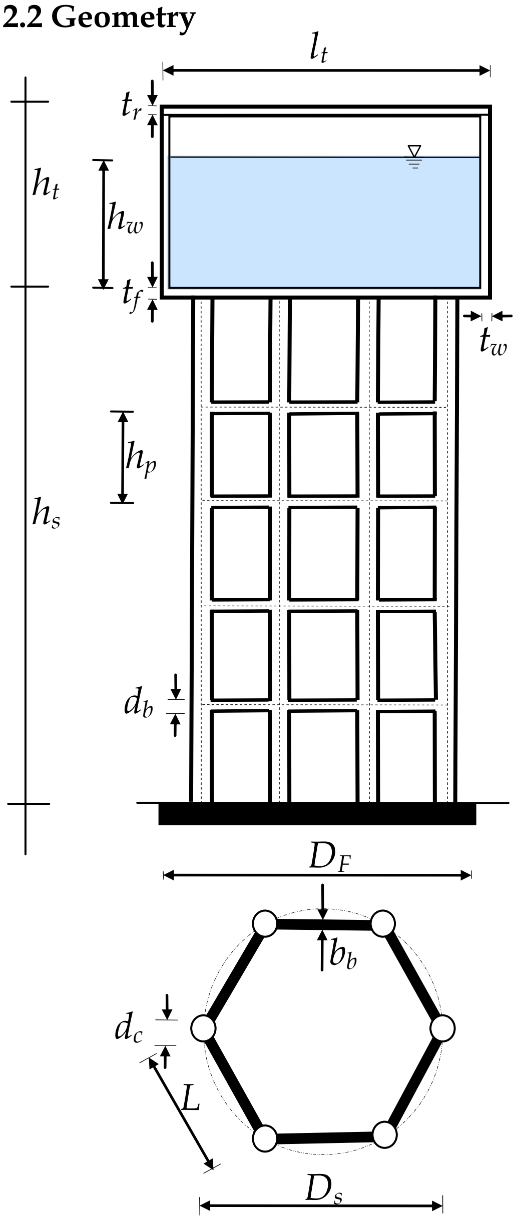
Rapid Assessment of Seismic Safety of
Elevated Water Tanks with Frame Staging

2. INPUTS

2.1 Basic Information

(1) Location : Chandigarh — Seismic Zone as per Indian Seismic Code:
(2) Type of Staging : Frame — Site-specific horizontal acceleration Z_{SS} :
(3) Importance Factor I : 1.5 — Detailing Type: Ordinary / Special R = 1.8/2.5
(4) Capacity : m³
(5) Shape of Water Tank : Rectangular — Stirrups : 90˚/135˚ hooks

2.2 Geometry



Inputs	*Units*
N_p	=
h_s	= *m*
h_t	= *m*
D_t^i	= *m*
t_w	= *m*
t_f	= *m*
t_r	= *m*
h_w	= *m*
L	= *m*
N_c	=
b_b	= *m*
d_b	= *m*
d_c	= *m*
D_F	= *m*
t_{RF}	= *m*

Beam size: *Longitudinal rebar diameter:* $d_{b,eff}$ = *m*	*Cover:* *mm* *Transverse rebar diameter:*
Column size: *Longitudinal rebar diameter:* $d_{c,eff}$ = *m*	*Cover:* *mm* *Transverse rebar diameter:*

Figure 2.1: Elevated Water Tanks with Frame Staging

2.3 Materials and Structural System

(1) Grade of Concrete f_{ck} = MPa Modulus of Elasticity, $E = 5000\sqrt{f_{ck}}$ = MPa

(2) Type of Soil **(Tick ONE)**

(i)	*Rocky and Hard Soil*	**N>30**	: Type I
(ii)	*Medium Soil*	**30>N>10**	: Type II
(iii)	*Soft Soil*	**10<N**	: Type III

Rapid Assessment of Seismic Safety of
Elevated Water Tanks with Frame Staging

3. BASIC SAFETY CHECKS

3.1 Section Properties

Derived Quantities	*Units*
$h_p = h_s / N_p =$	= *m*
$D_t^e = D_t^i + t_w$	= *m*
$W_{T\_empty} = \left[\pi D_t^e t_w h_t + \pi \left(D_t^e/2\right)^2 \left(t_f + t_r\right)\right]\gamma_{concrete}$	= kN
$W_{water} = \left[\pi \left(D_t^i/2\right)^2 h_w\right]\gamma_{water}$	= kN
$W_{T\_full} = W_{T\_empty} + W_{water}$	= kN
$D_s = L \big/ \sin(\pi/N_c)$	= m
$I_b = \dfrac{b_b d_b^{\ 3}}{12}$	= m^4
$I_c = \dfrac{\pi d_c^{\ 4}}{64}$	= m^4
$K_r = \left(\dfrac{I_c}{I_b}\right)$	=
$W_{staging} = N_c\left(\pi\left(d_c/2\right)^2 h_s + N_p b_b d_b L\right)\rho_{concrete} g$	= kN
$W_{s\_full} = W_{T\_full} + \left(1/3\right) W_{staging}$	= kN
$W_{\_empty} = W_{T\_empty} + \left(1/3\right) W_{staging}$	= kN
$W_{foundation} = \left(\pi/4\right) D_F^{\ 2} t_{RF}\, \rho_{concrete} g$	= kN

3.2 Lateral Stiffness of the Staging

Soft Soil	*Intermediate and Hard Soil*	*Calculation*
Top panel $k_{panel} = \dfrac{12 E_c I_c N_c}{h^3}\left[\dfrac{1}{1 + K_r\left(\dfrac{L}{h}\right)}\right]$	Top and bottom panels $k_{panel} = \dfrac{12 E_c I_c N_c}{h^3}\left[\dfrac{1}{1 + K_r\left(\dfrac{L}{h}\right)}\right]$	= kN/m
All other panels $k_{panel} = \dfrac{12 E_c I_c N_c}{h^3}\left[\dfrac{1}{1 + 2K_r\left(\dfrac{L}{h}\right)}\right]$	Intermediate panels $k_{panel} = \dfrac{12 E_c I_c N_c}{h^3}\left[\dfrac{1}{1 + 2K_r\left(\dfrac{L}{h}\right)}\right]$	= kN/m
Lateral Stiffness of staging $K_{staging} = 1 \Big/ \sum_{j=1}^{N_{panels}} \left(\dfrac{1}{K_{panel}}\right)$		= kN/m

3.3 Natural Period of the Tank

Tank Full $T_{full} = 2\pi\sqrt{\dfrac{W_{s\_full}}{gK_{staging}}}$; Tank Empty $T_{empty} = 2\pi\sqrt{\dfrac{W_{s\_empty}}{gK_{staging}}}$	T_{full} = s	T_{empty}= s

3.4 Design Horizontal Seismic Force

	Tank Full	Tank Empty
Spectral Acceleration (S_a/ g) *Soil Type* / *Spectral Acceleration (S_a/g)* Type I: $\dfrac{S_a}{g} = \begin{cases} 2.5 & 0 \le T \le 0.4 \\ 1.00/T & 0.4 \le T \le 4.0 \end{cases}$ Type II: $\dfrac{S_a}{g} = \begin{cases} 2.5 & 0 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.0 \end{cases}$ Type III: $\dfrac{S_a}{g} = \begin{cases} 2.5 & 0 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.0 \end{cases}$	*(S_a/g)* $_{full}$=	*(S_a/g)* $_{empty}$ =
Horizontal seismic coefficient $A_h = \dfrac{Z_{SS} I}{R}\left(\dfrac{S_a}{g}\right)$	=	=
Base Shear Filled $V_B = A_h W_{s\_full}$; Empty $V_B = A_h W_{s\_empty}$	= kN	= kN
Governing Shear force V_b is greatest of Full and Empty condition	= kN	

3.5 Shear Demand on Beams

Over strength shear in beams at level j, $V_{bj} = \dfrac{1.4 V_b h_p}{N_c R_s} \operatorname{cosec}\left(\dfrac{\pi}{N_c}\right)$	= kN

3.6 Shear Demand on Columns

Top panel, $V_c = \dfrac{2.8 V_b}{N_c}\left[\cos^2\left(\dfrac{\pi}{N_c}\right) + \dfrac{K_r}{3}\right]$	= kN
Other panels, $V_c = \dfrac{1.4 V_b h_p}{N_c R_s} \cos\left(\dfrac{\pi}{N_c}\right) \cot\left(\dfrac{\pi}{N_c}\right)$	= kN

3.7 Shear Capacity of Beam

Percentage of Longitudinal Reinforcement $\rho = \dfrac{100 A_{t\_st}}{A_c}$	=
For a percentage of Longitudinal Reinforcement ρ *in beam,* from Table 19 of IS:456-2000, Design Shear Stress of Concrete τ_c	= MPa
Area of concrete $A_{bc} = b_b d_{b,eff}$	= m^2
Shear Carried by Concrete $V_{uc} = \tau_b A_{bc}$	= kN
Shear Carried by Steel $V_{us} = 0.87 f_y A_{t\_st} \dfrac{d_b}{s_v}$	= kN
If hooks are 90° Total Shear Capacity of Beam $V_{u,beam} = V_{uc} + V_{us}$	= kN **> Shear Demand for Beams**
If hooks are 135° Total Shear Capacity of Beam $V_{u,beam} = V_{uc}$	= kN **> Shear Demand for Beams**

3.8 Shear Capacity of Columns

Percentage of Longitudinal Reinforcement $\rho = \frac{100 A_{t\_st}}{A_c}$	=
For a percentage of Longitudinal Reinforcement **ρ** *in column,* from Table 19 of IS:456-2000, Design Shear Stress of Concrete τ_c	= MPa
Area of concrete $A_{cc} = \frac{\pi}{4} d_{c,eff}^{\ 2}$	= m^2
Shear Carried by Concrete $V_{uc} = \tau_c A_{cc}$	= kN
Shear Carried by Steel $V_{us} = 0.87 f_y A_{t\_st} \frac{d_c}{s_v}$	= kN
If hooks are 90° Total Shear Capacity of Column $V_{u,column} = V_{uc} + V_{us}$	= kN **> Shear Demand for Columns**
If hooks are 135° Total Shear Capacity of Column $V_{u,column} = V_{uc}$	= kN **> Shear Demand for Columns**

3.9 Check for Overturning Moment

Over Turning Moment $M_{OT} = V_B \left(h_s + \frac{h_t}{2} \right)$	= kNm	= kNm
Restoring Moment $M_R = \left(W_{\text{tank}} + W_{\text{staging}} + W_{\text{foundation}}\right)\left(1 - \frac{2}{3} A_h\right)\frac{D_F}{2}$ where D_F, is diameter of the foundation	= kNm	= kNm
Factor of Safety = M_R/M_{OT}	=	=
Check	**> 1.5**	**>1.5**