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SEISMIC DESIGN OF CONCRETE PEDESTAL SUPPORTED TANKS

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SUMMARY

Reinforced concrete pedestal (circular, hollow shaft type supports) are popular choice for elevated tanks for the ease of construction and the more solid form it provides compared to framed construction. In the recent past Indian earthquakes, Gujarat (2001) and Jabalpur (1997), thin shells (~150 to 200 mm) of concrete pedestals have performed unsatisfactorily when great many developed circumferential tension-flexural cracks in the pedestal near the base and a few collapsed. These observations partially fill the void that exists about the actual performance of such structures in earthquakes of significant magnitude.

The shaft support of elevated tanks should have adequate strength to resist axial loads, and moment and shear forces due to lateral loads. The observed damage pattern shows that, for tanks of large aspect ratio and falling in long time period range, flexural behavior is more critical than shear under seismic loads. Therefore, the concrete pedestal should be adequately designed and detailed for flexural deformations and actions as well as for shear strength and deformations. However, for very large tanks designed as per ACI 371 provisions, shear strength frequently controls design of the cylindrical pedestal wall and it is partly due to the fact that Chapter 21 provisions of ACI 318 don't consider the beneficial effects of axial compression.

Currently codes recognize that thin shaft shells of pedestal not only possess a very low flexural ductility but also lack redundancy of alternate load paths that are present in framed structures. As a result, for seismic design the response reduction factors for such structures are kept lower than those structures with higher capacity for ductility and energy dissipation, such as building frames, which results in about 3 times large design forces. However, it is not adequate as the design and detailing of the support structure should conform to the expected behavior and to the controlling failure mode. This paper will review the existing code design procedures in the light of actual performance data and will suggest modifications for safer designs.

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INTRODUCTION

Water tanks supported on Reinforced Concrete (RC) shafts are popular in many parts of the world. RC shafts are designed to sustain vertical gravity loads and typically moderate lateral loads. RC shaft stagings lack redundancy and ductility and hence should be designed to behave elastically in the event of an earthquake. However, the current designs are vulnerable in case of significant lateral loads as evidenced in a number of past earthquakes. In the earthquake that occurred in Bhuj, India on January 26, 2001 ($M_w = 7.7$), many tanks supported on RC shafts were affected [1]. Tanks as far as 150km away from the epicenter were damaged and at least one tank in the epicentral region collapsed (Figure 1). The tanks had undergone circumferential cracking near the base indicating failure in tension-flexure mode, where as ACI 371-R does not require tension-flexure check explicitly and emphasizes on the behavior of shaft as shear wall.

***Figure 1*: Collapsed 265 kL water tank in Chobari village about 20 km from the epicenter. The tank was approximately half full during the earthquake.**

This paper deals with the design forces for the construction of RC shaft stagings. A set of eight tanks affected in Bhuj earthquake, covering a wide range of possible geometry for RC shafts, was analyzed to support the study. This paper studies both tension-flexure mode and shear mode of failure of shaft supports.

REVIEW OF DAMAGE OBSERVED TO SHAFT SUPPORTS

The shafts supporting the tanks are thin circular, hollow cylinders of varying heights and diameters depending on the water head required and the capacity of tanks respectively. The weight of the tank container is much greater as compared to that of the staging. The shaft supported elevated water tanks can be seen as inverted pendulum like structures. The lateral forces coming on the shaft are resisted by its flexural and shear strength. The section close to the ground is subjected to the maximum flexural demand for uniform staging. The tanks affected in the 2001 Bhuj, India earthquake, also show circumferential cracks from the level of first lift, reaching to one third height of the shaft. (Figure 2). The cracks cover the entire perimeter and are clearly visible from inside too [1].

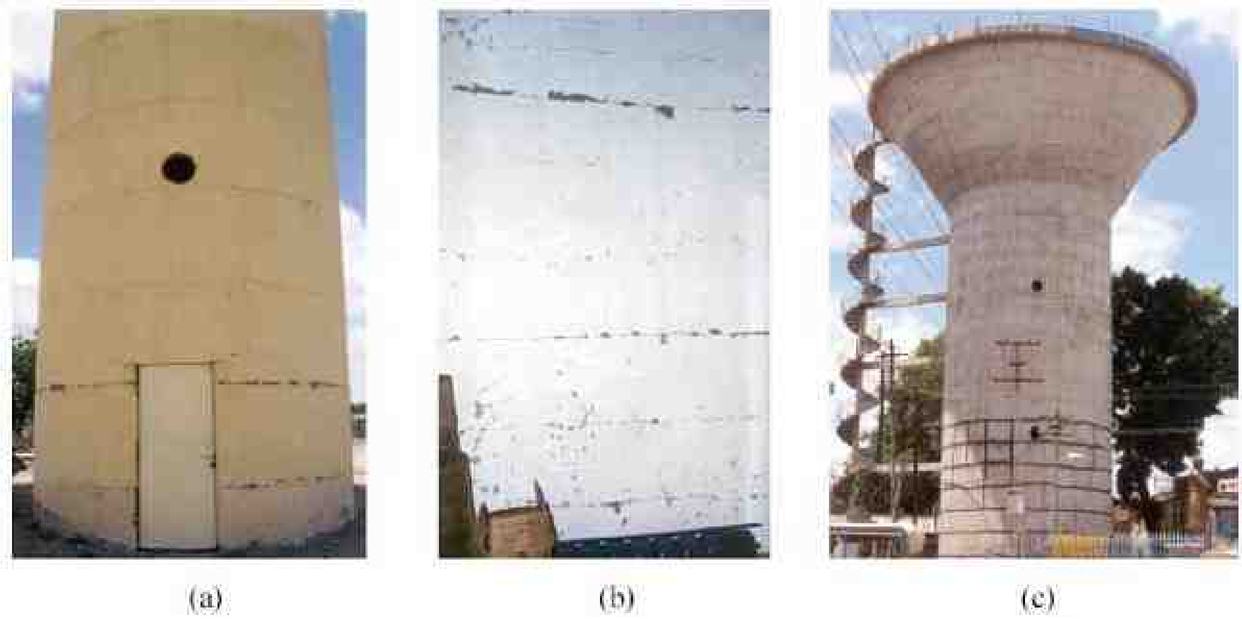

***Figure 2*: (a) 200 kL Bhachau water tank developed tension-flexural cracks up to one-third height of the staging. Severe cracking at the junctions of the first two 'lifts', (b) Cracks are 'through' the shell thickness as seen from inside the shaft of 1000 kL Anjar Nagar Palika Tank, and (c) Cracks in staging of 500 kL tank repaired by injecting epoxy. This tank in Morbi, 80 km away from the epicenter, was empty at the time of the earthquake**

Of the many tanks affected in the Bhuj earthquake, a set of eight tanks was selected to cover a wide range of possible geometry of RC shafts. The height of the shaft varies from 10m to 20m depending on the water head required. Thickness of the cylinder varies from 175mm to 225mm and the diameter of staging depends on the capacity of the water tanks. Capacity of the tanks varies from 80kL to 1000kL and the diameter of staging varies from 2.75m to 8m. The characteristics of the tanks studied are shown in detail in Table 1 [1]. It also shows the fundamental time periods of the structures under various conditions as discussed in the next section.

Table 1 Characteristics of water tanks analyzed

Name and Location	capacity (kL)	Geometry of Shaft Support				Time Period (impulsive mode) sec			
						Tank Empty		Tank Full	
		Dia. d (m)	Thick. t (mm)	Height H (m)	Slenderness H/d	Fixed Base	With SSI	Fixed Base	With SSI
Gandhidham (T-1)	1000	8.00	250	14.60	1.85	0.164	0.25	0.313	0.449
Anjar (T-2)	1000	7.60	225	16.00	2.11	0.187	0.315	0.421	0.622
Chobari (T-3)	265	4.50	160	10.50	2.35	0.166	0.248	0.314	0.424
Morbi (T-4)	500	6.60	200	16.00	2.42	0.209	0.286	0.366	0.475
Bhachau (T-5)	200	4.00	150	11.00	2.75	0.195	0.274	0.348	0.533
Sapeda (T-6)	100	3.00	150	12.50	4.15	0.285	0.393	0.468	0.593
Samakhiali (T-7)	80	2.75	175	11.50	4.18	0.248	0.362	0.427	0.55
Gala (T-8)	300	3.66	125	20.00	5.45	0.698	0.736	1.171	1.221

DYNAMIC BEHAVIOR OF WATER TANKS

The forces on the tanks during earthquake depend on the dynamics of the structure, fluid stored in the tank and of the founding soil. For dynamic behavior characterization of the shaft supported elevated water tanks two level of interactions need to be studied. First being the interaction between the structure and water stored in the tank container. All the water stored in the container does not always moves with it. The sloshing motion of water inside the tank container can not be modeled by a single degree of freedom model. Housner [2] proposed a simple two degree of freedom model to include the sloshing motion of water and considers two modes of vibrations, namely, impulsive mode and sloshing (convective) mode. Housner gives a simple method to characterize these modes. However, according to ACI 371R [3] a single mass idealization can be a reasonable approximation for an elevated water tank where the water weight is typically 80 percent of the total structure weight.

The second interaction is that between the structure and the foundation soil. The assumption of the base of the shaft of an elevated tank being fixed is valid if it is founded on hard rock. For structures supported on soft soil, the foundation motion is generally different from the free-field motion. The motion at foundation level includes both translational and rocking component. The rocking motion is significant for tall structures like elevated water tanks and the structure can be assumed to be flexibly supported with a translational and rotational spring at its base. Soil also causes dissipation of significant amount of vibrational energy due to its inelastic behavior, called material or internal damping, and due to radiation of elastic stress waves, called radiation damping. Thus, the effect of soil structure interaction (SSI) can be summed up as a longer time period and increased damping. (Figure 3). A simple procedure to account for soil structure interaction was proposed by Velestos [4]. This procedure is also accepted and recommended by NEHRP 2001 [5] which has been summarized in Appendix 1. The accuracy of this procedure has also been studied and is found to be providing reasonably accurate and unbiased predictions of the SSI effects of period lengthening and foundation damping [6].

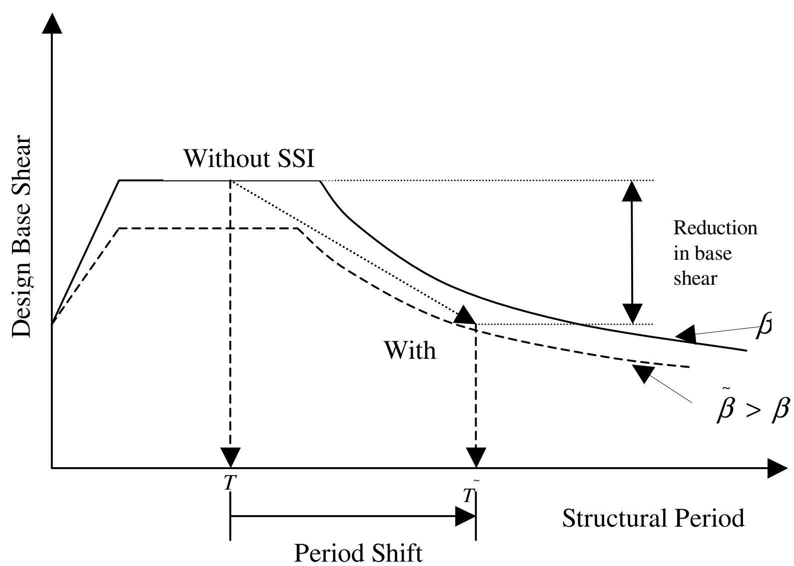

***Figure 3*: Effect of soil structure interaction**

EVALUATION OF LATERAL STRENGTH OF SHAFT TYPE STAGINGS

Housner's two mass model, with soil structure interaction consideration as recommended in NEHRP 2001 [5] provisions, has been used for modeling the tanks and evaluating their lateral strength. Force to be resisted by the shaft during an earthquake depends on the weight of water present in the tank container. Seismic load analysis has been carried out for both tank empty and tank full conditions. The characteristics of the tanks analyzed are shown in Table 1.

Tension-Flexural Cracking Strength Analysis

As shown in Figure 4, due to lateral seismic forces on tank structures, the maximum moment occurs at the base of the staging. For circular shaft type staging, the points on the outer fibers of the staging section are subjected to maximum bending stress. The critical stress for design is obtained by combining this maximum bending stress with the uniform axial compression stress due to the weight of the tank structure. For the section to crack, it is necessary that the combined stress at outer fibers exceed the tensile strength of the concrete, f_{cr}. Assuming thickness of staging t to be much smaller in comparison to the radius of staging r, and ignoring the small percentage of shell reinforcement, the expression for the moment which will cause cracking, M_{cr}, can be obtained by equating combined stress at outer fiber to the tensile strength of concrete, i.e.,

$$-\frac{\gamma \times P}{2\pi \times rt} + \frac{M_{cr}}{\pi \times r^2 t} = f_{cr} \qquad (1)$$

Where γ is the approximate load factor for axial load P and is taken as 0.9 to give a lower bound estimate of cracking moment of resistance M_{cr}. $f_{cr} = 0.7 \cdot \sqrt{f_c}$ MPa, where f_c is the characteristic cylinder strength of concrete. The critical shear force is hence calculated as

$$V_{cr} = \frac{M_{cr}}{L} \qquad (2)$$

Where L is the height of staging.

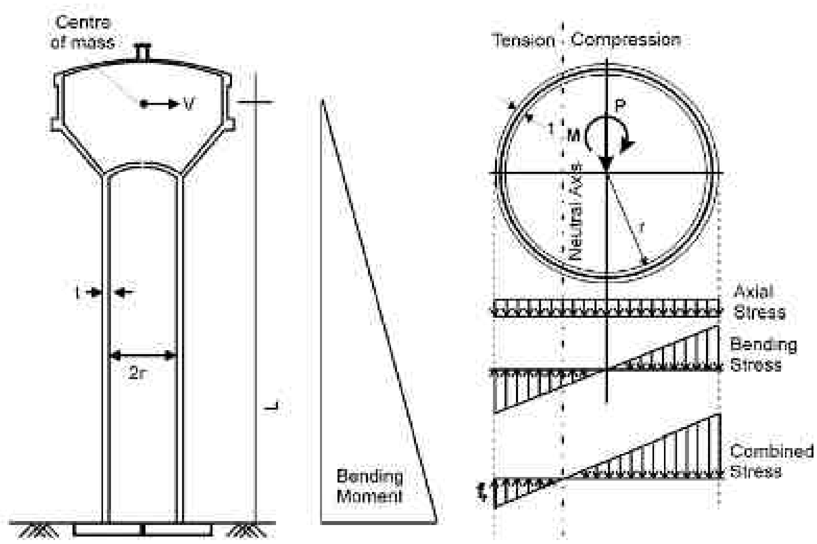

***Figure 4*: Distribution of stresses and calculation of M_{cr} and V_{cr}**

The critical shear force calculated above is based on the assumption of fixed base. With SSI the force required to cause failure would be larger than those for fixed end condition. The procedure for consideration of soil structure interaction, as recommended by NEHRP 2001, is given in Appendix 1. The critical moments and base shear with and without soil structure interaction are shown in Table 2 and the base shear ratios are also shown in Figure 5(a) and 5(b) for tank empty and tank full conditions, respectively.

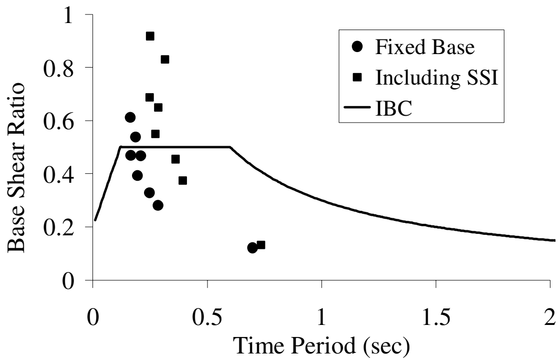

Figure 5(a) Base shear ratios for tank empty

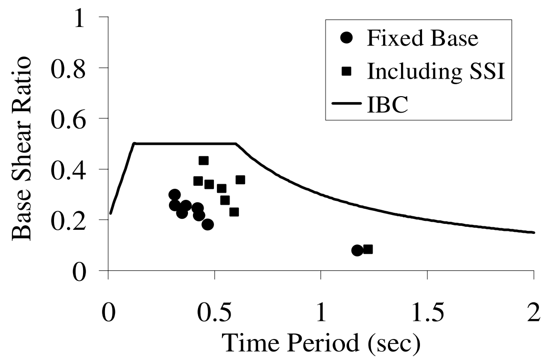

Figure 5(b) Base shear ratios for tank full

Base shear ratio is the ratio of base shear and the seismic weight of tank. 5% damped IBC 2000 [7] design spectra with S_{DS}=1g, S_{DI} =0.4g for site class D, R=2.5 and I =1.25 is also plotted. It can be inferred from the plotted data that the design forces recommended by IBC 2000 [7] are adequate for tank full condition but they may fall short for tanks in short period range, when they are empty

Table 2 Critical moment and shear that caused failure of respective tanks

Name and Location	Tank Empty			Tank Full		
	M_{cr} (MNm)	V_{cr}/W_s		M_{cr} (MNm)	V_{cr}/W_s	
		Fixed Base	With SSI		Fixed Base	With SSI
Gandhidham (T-1)	42.63	0.612	0.918	60.29	0.299	0.340
Anjar (T-2)	34.82	0.538	0.830	51.59	0.246	0.358
Chobari (T-3)	8.49	0.469	0.687	11.12	0.256	0.353
Morbi (T-4)	23.30	0.468	0.649	30.59	0.256	0.323
Bhachau (T-5)	6.32	0.393	0.549	8.09	0.226	0.434
Sapeda (T-6)	3.52	0.280	0.374	4.18	0.182	0.230
Samakhiali (T-7)	3.31	0.328	0.455	3.80	0.218	0.277
Gala (T-8)	5.57	0.121	0.132	7.99	0.080	0.084

Shear Strength Analysis

The shear strength of a RC shaft wall is contributed by the shear strength of the concrete and the shear strength contributed by the shear reinforcement provided. Typically in the shaft shell, about 0.18 to 0.25% reinforcement ratio is provided in two layers in both longitudinal and circumferential directions. The vertical reinforcing bars are spliced at staggered locations and not more than one third of total bars are spliced at a particular level. The bar diameter is usually 10 mm and a minimum lap length of 470 mm, equal to development length in tension, is provided. A typical transverse and longitudinal reinforcement in shell wall was assumed as 0.25% for the calculations.

The shear strength V_c provided by concrete, depends on the compressive strength of concrete f_c and the cross-sectional dimensions b_w, d and is calculated as

$$V_c = 2\sqrt{f_c^{'}}b_w d \tag{3}$$

And, the shear strength provided by shear reinforcement is given by

$$V_s = \frac{A_v f_y d}{s} \tag{4}$$

Where A_v is the area of shear reinforcement, of specified yield strength f_y within a distance s. Hence, the total nominal strength of the shell wall is the sum of the shear strength provided by the concrete and the shear reinforcement.

$$V_n = V_c + V_s \tag{5}$$

The factored shear strength of the shell wall is hence given by φV_n [8]. Stagings are generally provided with a door opening for inspection purposes. Because of this opening, the shear force distribution is no longer uniform, and eccentricity is introduced due to non uniformity of section of staging. ACI 371 R [3] gives an approximate method of estimating the shear force in such unsymmetrical section. The document suggests that the circular section with opening can be represented by two parallel walls of length $0.78 \times d_w$, one solid wall and other wall with an opening of length equal to that of the actual opening. It uses a ratio $\psi = \frac{b}{0.78 \times d_w}$ to calculate the eccentricity as $e = 0.5 \times d_w \times \frac{\psi}{2-\psi}$. d_w is the mean diameter of the concrete support wall. The shear force V is then distributed as $0.5 \times V - \frac{V \times e}{d_w}$ and $0.5 \times V + \frac{V \times e}{d_w}$ in the solid wall and the wall with opening, respectively. (Figure 6).

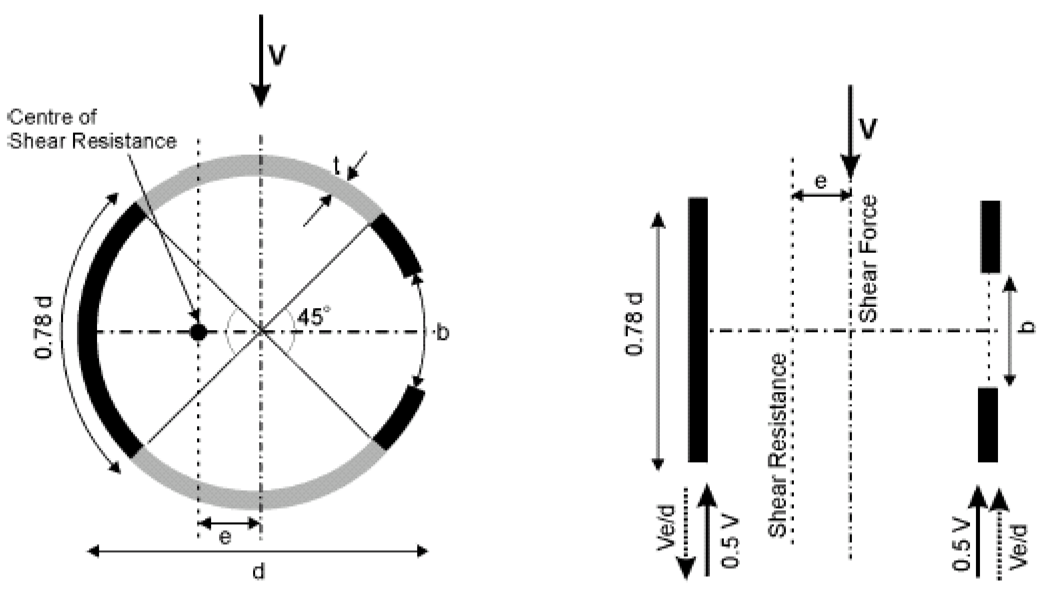

***Figure 6*: Distribution of shear in an unsymmetrical circular section**

The total shear force on equivalent wall $0.5 \times V + \frac{V \times e}{d_w}$ when equated with the shear strength φV_n gives the shear demand V on the shaft support during the earthquake. V is then corrected for soil structure interaction. The shear strength for the tanks, considering soil structure interaction, are shown in table 3 and figure 7(a) and 7(b).

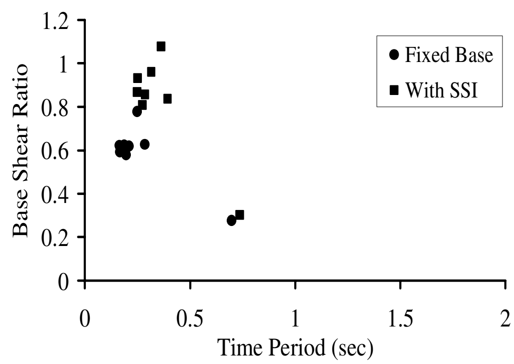

***Figure 7(a)*: Shear Strength for tank empty**

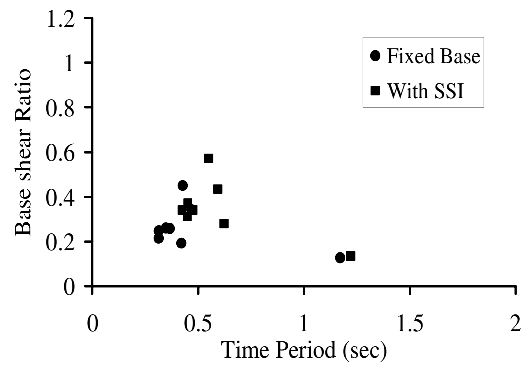

***Figure 7(b)*: Shear Strength for tank full**

COMPARISON OF SHEAR STRENGTH AND TENSION FLEXURE STRENGTH

The ratio of tension-flexure strength to the shear strength of the shafts, for both tank empty and tank full conditions are given in Table 3

Table 3 Flexural strength and Shear strength

Name and Location	Tank Empty			Tank Full		
	Flexural strength	Shear strength	Strength Ratio	Flexural strength	Shear strength	Strength Ratio
	SSI	SSI		SSI	SSI	
Gandhidham (T-1)	0.918	0.933	0.984	0.434	0.312	1.391
Anjar (T-2)	0.830	0.962	0.863	0.358	0.280	1.277
Chobari (T-3)	0.687	0.869	0.790	0.353	0.341	1.034
Morbi (T-4)	0.649	0.858	0.756	0.340	0.342	0.993
Bhachau (T-5)	0.549	0.809	0.679	0.301	0.372	0.810
Sapeda (T-6)	0.374	0.838	0.446	0.230	0.435	0.530
Samakhiali (T-7)	0.455	1.079	0.422	0.277	0.572	0.484
Gala (T-8)	0.132	0.302	0.436	0.084	0.135	0.624

The results obtained show that neither tension-flexure nor shear mode of failure always govern the failure of shaft supports. In figure 7(a) and 7(b) this strength ratio is plotted with respect to the aspect ratio. Open and filled squares represent tank empty and tank full condition, respectively.

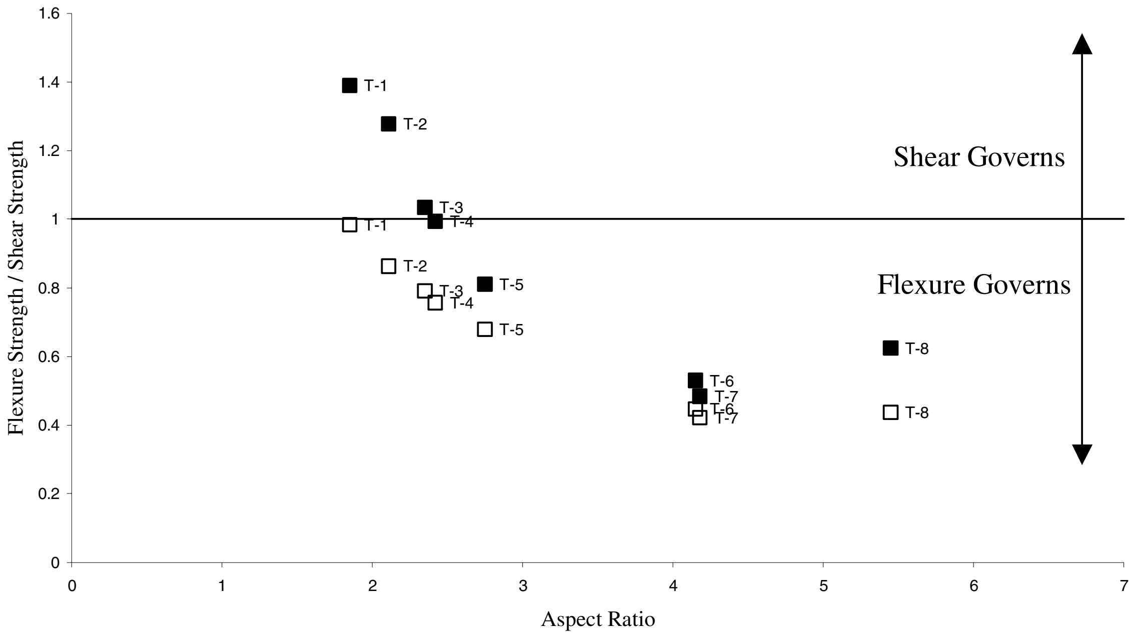

***Figure 8(a)*: Ratio of tension-flexure strength to shear strength with respect to Aspect ratio** $\left(\frac{H}{D}\right)$

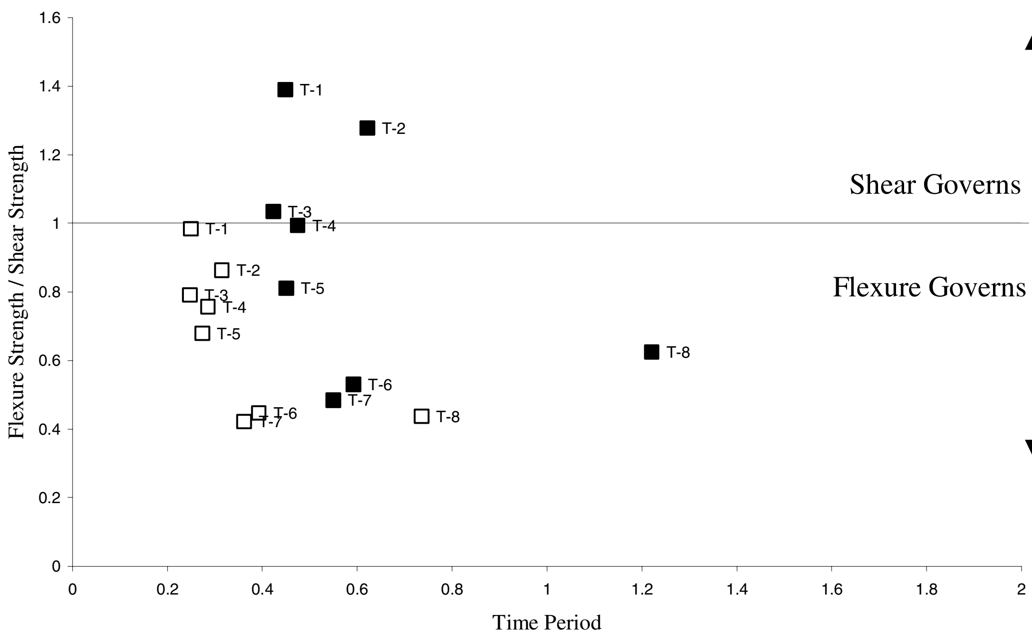

***Figure 8(b)*: Ratio of tension-flexure strength to shear strength with respect to Time Period**

On the basis of these results it can be said that, for tank empty condition, flexural strength of shafts is always less than their shear strength. And for tank full condition, shear strength governs the failure mode for shafts with low aspect ratios. In figure 7(b), flexure to shear strength ratios are plotted with respect to time period. Tension-flexural strength governs the failure mode for shafts having shorter time period. While for shafts having longer time period, shear mode governs the failure

CONCLUSIONS

For the set of tanks studied, shear demand is more when a tank is empty than when it is full. For tank empty case, even IBC 2000 [7] underestimates these forces. Although ACI 371 [3] does not require tension-flexure check explicitly, this failure mode is a strong possibility. For the tanks studied in this paper, when a tank is empty, flexure strength governs the failure mode for all aspect ratios (ratio of height to diameter) of the support shaft and time periods of the tanks. And when tank is full, shear mode is found to be governing failure of stiffer shafts having short time period and low aspect ratios.

APPENDIX 1: ANALYSIS FOR SOIL STRUCTURE INTERACTION

Soil structure interaction has two effects. Firstly, it increases the time period of structure because of the flexibility of soil. Secondly, the effective damping increases due to material damping and radiation damping. Effective period of vibration as modified by soil structure interaction is given by the following relation

$$\tilde{T} = T \times \sqrt{\left(1 + \frac{K}{K_x} \times \left(1 + \frac{K_x \times h_1^2}{K_\theta}\right)\right)} \quad \text{(A.1)}$$

Where, T is the time period of fixed base system. K_x and K_θ are respectively the lateral and rocking stiffness of foundation and are given by the following relations.

$$K_x = \frac{8 \times \alpha_x}{(2 - \nu)} \times Gr \quad \text{(A.2)}$$

$$K_\theta = \frac{8 \times \alpha_\theta}{3 \times (1 - \nu)} \times Gr^3 \quad \text{(A.3)}$$

Where, ν and G are poisson's ratio and shear modulus of elastic half space and r is the radius of foundation, α_x and α_θ are dynamic coefficients, as defined below:

$$\alpha_x = 1 \quad \text{(A.4)}$$

$\frac{R}{v_s T}$	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

The effective height of the structure h_1 is taken as 0.7 times the total height h except when load is effectively concentrated at a particular level, and then it is height up to that level.

The following formula combines both material and radiation damping to determine effective damping of vibration.

$$\tilde{\beta} = \beta_o + \frac{\beta}{\left(\frac{\tilde{T}}{T}\right)^3} \quad \text{(A.5)}$$

The foundation damping factor β_o incorporates energy dissipation due to radiation of waves and hysteresis of inelastic behavior of soil beneath, obtained by the following curves

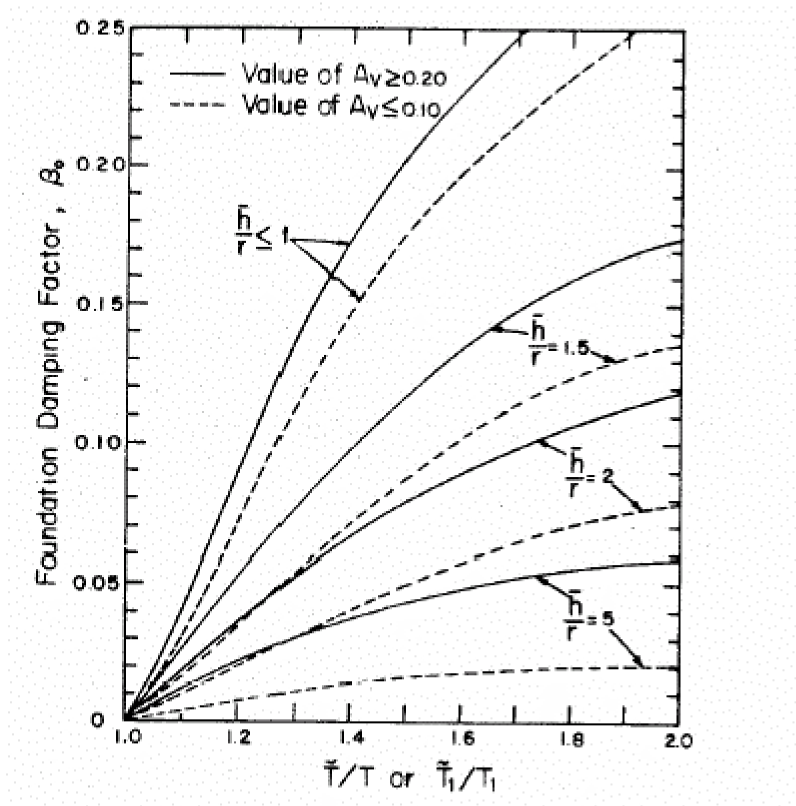

Figure 2 Curves for foundation damping factor

Due to increase in damping and elongation of time period the base shear V reduces by an amount ΔV, where

$$\Delta V = \left[1 - \left(\frac{T}{\tilde{T}}\right) \times \left(\frac{\beta}{\tilde{\beta}}\right)^{0.4}\right] \times C_s(T,\beta) \times W \tag{A.6}$$

$$V_1 = V - \Delta V \tag{A.7}$$

Where the seismic design coefficient for the fixed base system of period T, $C_s(T,\beta)$ is the ratio of fixed base shear V and the total weight of tank W.

$$C_s(T,\beta) = \frac{V}{W} \tag{A.8}$$

Using this simple procedure, soil structure interaction can be accounted for in calculations of base shear. These results obtained by considering the above method are found to be sufficiently accurate [5].

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