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Review of Code Design Forces for Shaft Supports of Elevated Water Tanks

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ABSTRACT

The current designs of circular shaft-type supporting structures of elevated water tanks are extremely vulnerable under lateral forces due to earthquakes. The 2001 Bhuj earthquake provided another illustration of this vulnerability when a great many water tank stagings suffered damage as far as 100 km from the epicentral tract. Shaft type stagings suffer from poor ductility of thin shell sections in addition to lack of redundancy of load paths and toughness. Lateral strength analyses of a few damaged shaft type stagings clearly show that all of them either met or exceeded the requirements of IS:1893-1984, however, they were all found deficient when compared with requirements of International Building Code in similar seismic exposure conditions. IS:1893-1984 design forces are unjustifiably low for these systems which do not have advantage of ductility and redundancy. The code's much higher degree of reliance on ductility to reduce design forces does not yield satisfactory performance; these forces are currently being grossly underestimated. A response reduction factor equal to 2 is proposed to be used with the revised code IS:1893-2002 for such structures, which provides reasonably safe design forces.

INTRODUCTION

Reinforced concrete (RC) circular shaft type supports (staging) is widely used for elevated tanks of low (~ 100 kL) to very high (~2000 kL) capacity for its ease of construction and more solid form it provides compared to frame construction. In recent past earthquakes Bhuj, Gujarat (2001) and Jabalpur (1997), thin shells of circular shafts have performed unsatisfactorily (Rai 1997, Rai 2002a). Thin shaft shell when used as column (or pedestal) are vulnerable because they not only possess a very low ductility but also lack redundancy of alternate load paths that are present in framed structures. As a result, 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.

The paper reports a comparison of lateral strength capacity of damaged shaft supports with the code expected seismic demands. It was observed that tanks in seismic zones V (Kachchh, Gujarat) and III (Jabalpur) were damaged despite they possessed lateral strength equal or greater than code required design forces. The tanks with short periods falling in the acceleration sensitive region of design spectrum were especially vulnerable despite large overstrength. A comparison with seismic demands stipulated in International Building Code for such structures is also presented, which clearly shows that the Indian code assumes a much higher degree of reliance on ductility to reduce design forces which does not yield satisfactory performance.

REVIEW OF DAMAGE OBSERVED TO SHAFTS SUPPORTS

Shaft supported elevated tanks are inverted pendulum type structures, which resist lateral forces by the flexural strength and stiffness of their circular, hollow shaft type staging. The section close to the ground is subjected to the maximum flexural demand for uniform staging. Any damage to the staging at this critical section should be considered alarming as it can seriously undermine its lateral load carrying capacity. The observed damage pattern as described below in the recent 2001 Bhuj

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earthquake is consistent with the expected response of these structures under lateral loads. Shaft supports are thin circular cylinders of height that typically varies from a minimum of about 10 m to a maximum of 20 m whereas the shape and size of the tank container largely depends on the storage capacity and the required head for the water supply. The storage capacity the affected tanks varied from 80 kL to 1 000 kL in Gujarat. The diameter of the staging generally increases with increase in the tank capacity, however, the thickness of the shaft shell is usually kept between 150 to 200 mm.

The tension-flexure cracks in stagings were observed from the level of the first "lift" to several lifts reaching one-third the height of the staging, as shown in Fig. 1(a). These cracks are mostly in circumferential direction and cover the entire perimeter of the shaft. They usually appear near the edges of the form used during casting of the shaft, which appear to form planes of weaknesses along the shaft's length. These cracks pass through the thin section of the staging and are clearly visible from inside too (Fig. 1(b)). This damage to the staging should have seriously reduced their lateral load carrying capacity, increasing their susceptibility to a greater damage or collapse in a repeat occurrence of such an event. However, most of these tanks are being used as before. In a few cases, for example, the water tank in Morbi was inadequately repaired by injecting epoxies in the cracks, as shown in Fig. 1(c). Where many water tanks escaped the earthquake with minor to severe flexural cracks, the water tank in the village Chobari in the epicentral tract did collapse (Fig. 2).

(a) (b) (c)

Figure 1. *(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* 'lift', *(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*

Figure 2. *Collapsed 265 kL water tank in Chobari village about 20 km from the epicenter. The tank was approx. half full during the earthquake.*

LATERAL STRENGTH OF SHAFT TYPE STAGINGS

The shaft support of elevated tanks should have adequate strength to resist axial loads, and moment and shear forces due to lateral loads. These forces depend on the total weight of the structure which varies with the amount of contents present in the tank container. Typically, the seismic load analysis is performed for tank-full and tank-empty conditions, while analysis at other contents levels may be desirable if convective fluid motions are of significance. A set of lateral strength analysis was performed for a sample of eight tanks of capacities varying from 80 kL to 1 000 kL as given in Table 1 and these were compared against the code expected seismic demands.

The geometrical dimensions given in Table 1 are approximate only, as they are derived from a few easy to make field measurements. Further, the fundamental time period is based on single-degree-of-freedom model of tank structure ignoring the convective vibration modes of water and assuming the shaft to act as a cantilever beam with a concentrated mass of tank container at its tip. 2. The seismic weight is taken as weight of the tank container plus one-third the weight of the staging. For tank-full condition the entire weight of the water is added to the weight of the container.

Table 1. Characteristics water tanks used in the study

No.	Name and Location	Capacity (kL)	Geometry of Shaft Support				Natural Time Period , ***T*** (s)	
			Dia. ***d*** (m)	Thick. ***t*** (mm)	Height ***H*** (m)	Slenderness ***H/d***	Empty Tank	Full Tank
1	Anjar	1000	7.60	225	16.0	2.11	0.26	0.44
2	Gandhidham	1000	8.00	250	14.6	1.85	0.24	0.34
3	Morbi	500	6.60	200	16.0	2.42	0.31	0.40
4	Gala	300	3.66	125	20.0	5.45	1.02	1.30
5	Chobari	265	4.50	160	10.5	2.35	0.25	0.34
6	Bhachau	200	4.00	150	11.0	2.75	0.28	0.38
7	Sapeda	100	3.00	150	12.5	4.15	0.39	0.51
8	Samakhiali	80	2.75	175	11.5	4.18	0.36	0.45

Tension-flexural Cracking Strength Analysis

As shown in Fig. 3 due to lateral seismic forces on tank structures, the maximum moment occurs at the base of the staging and 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.,

$$-\frac{\gamma P}{2\pi r t}+\frac{M_{cr}}{\pi r^2 t}=f_{cr} \tag{1}$$

where γ is the appropriate 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}. Taking $f_{cr}=0.7\sqrt{f_{ck}}$ MPa, where f_{ck} is characteristic strength of concrete, the above relation can be used to provide M_{cr} of the staging section. An estimate of lateral shear strength V_{cr} corresponding to the flexural tension cracking can be obtained by dividing M_{cr} by height of shaft support using a simplified single degree of freedom representation for the elevated tank structures.

For the sample of eight tanks given in Table 1, the lateral shear strength corresponding to flexural tension cracking was computed using Eqn. (1) and is shown in Table 2. A typical concrete grade M 20 was assumed for the staging, i.e., f_{cr} = 3.1 MPa. As expected, smaller shear strengths to seismic weight ratios were observed for tank-full condition indicating greater vulnerability when tanks are full. In Fig. 4, the lateral shear strength provided against flexural-tension cracking in stagings of the affected tanks is compared against the lateral strength demand required by IS:1893-1984 in the Seismic Zone V, the highest seismic zone in which most of the affected tanks are located. The code forces were multiplied by a typical load factor of 1.5 to raise them to ultimate stress level from the working stress level. It is clear that the shaft supports' tensile cracking ctrength staging is either equal or larger than the code required strength. In other words, the stagings do meet or exceed the strength requirements of IS:1893-1984. However, they will be considered seismically deficient by International Building Code (IBC 2000) under a similar seismic exposure conditions due to inadequate lateral strength capacity.

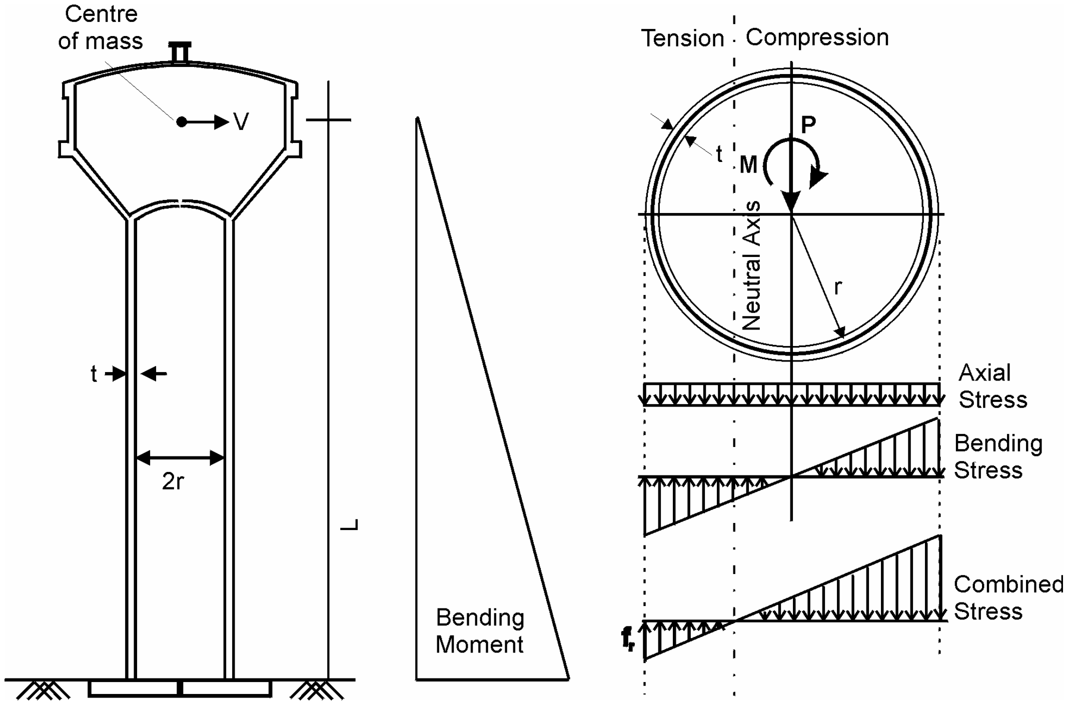


Figure 3. *Stresses developed in shell staging.*

Table 2. Flexural tensile strengths of shaft stagings

No.	Name and Location	Tank-Empty Condition		Tank-Full Condition	
		M_{cr} (MNm)	V_{cr}/W_s	M_{cr} (MNm)	V_{cr}/W_s
1	Anjar	44.0	0.45	60.8	0.20
2	Gandhidham	54.3	0.46	72.0	0.25
3	Morbi	30.1	0.37	37.4	0.23
4	Gala	7.76	0.11	10.2	0.08
5	Chobari	10.6	0.38	13.3	0.22
6	Bhachau	8.0	0.33	9.76	0.20
7	Sapeda	4.50	0.25	5.16	0.16
8	Samakhiali	4.25	0.28	4.73	0.19

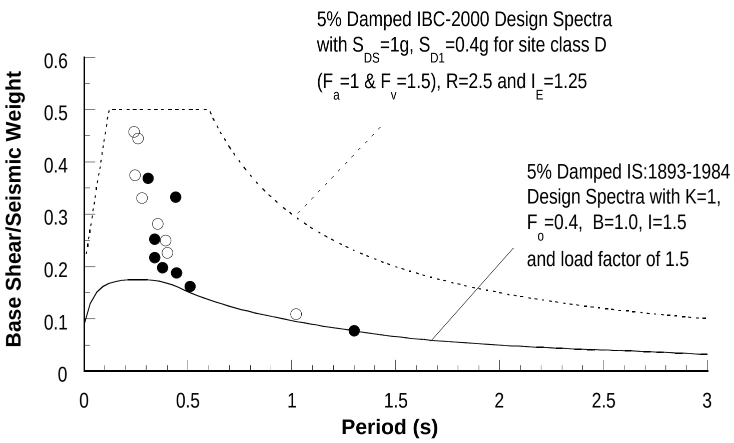


Figure 4 . *Comparison of provided shear strength against tensile cracking of eight tanks with base shear strengths required by IS:1893-1984 and IBC 2000 codes. 'Open' and 'filled' circles correspond to tank-empty and tank-full conditions.*

Ultimate Flexural Strength Analysis

An ultimate strength analysis of the staging section of the collapsed Chobari water tank is carried out which involved the calculation of ultimate direct force and ultimate bending moment that can be resisted by the resulting stress envelope. The envelope of resistance is presented in the form of an interaction plot with the moment as the abscissa and axial load as the ordinate. The strength interaction curves were developed corresponding to factored strengths and nominal strengths. Geometrical and material parameters used to derive the resistance envelope were: Mean radius of section r=2.25 m, Shell thickness t=160 mm, Ratio of longitudinal steel to gross section=0.00283, Angle subtended by door opening at the center of the section=0.44 rad, Cube strength of concrete f_{ck}=20 MPa, and Yield strength of reinforcement, f_y=415 MPa.

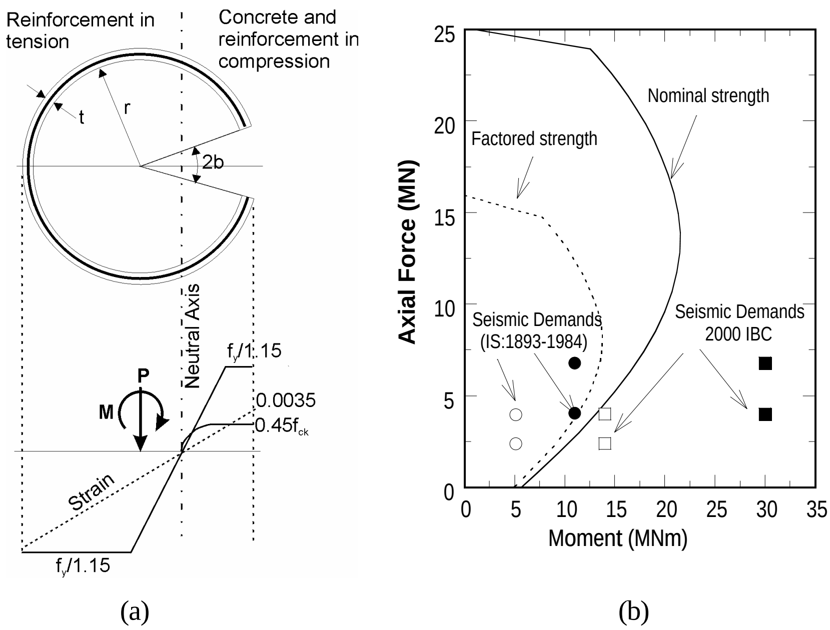
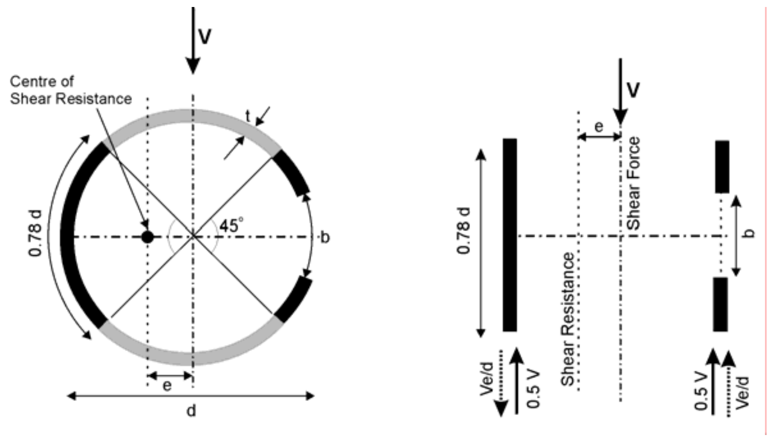


Figure 5. *(a) Distribution of stress at failure for the critical section of the staging, and (b) Strength envelopes (interaction diagrams) for staging section near the base of the Chobari tank and demands expected for Tank Empty and Tank Full conditions. Factored strength is computed with usual partial safety factors as specified by IS:456-2000. Nominal strength corresponds to partial safety factors being unity.*

To assess the safety of the structure, the available capacity at the critical section is compared with probable demands specified by IS:1893-1984. The ‘open’ and ‘filled’ circles in Fig. 5, represent factored seismic demand for empty and full tank cases according to the two critical load combinations. It is clear that the staging of Chobari water tank was probably safe for seismic forces specified by IS:1893-1984, if we ignore the possibility of poor quality of construction which can not be ruled out considering its remote location. In other words, the seismic forces were indeed larger than code specified forces on the morning of the earthquake when the tank was about half full. A low seismic design forces results in low flexural demand from the staging section which encourages use of slender stagings with thin shell sections. It should be further noted that the ultimate flexural strengths are nearly same as the tensile cracking strength for this tank, indicating a very little reserve strength beyond elastic behaviour.

Shear Strength Analysis

An analysis of available shear strength at the critical section of shaft supports is performed using a method described in *ACI 371R-98* (ACI 1998). In this method, the circular shear wall is idealized as two parallel shear wall of length *0.78d*, where *d* is the diameter of the circular wall, as shown in Fig. 6. This idealization is based on the fact that the shear force per unit length of the equivalent shear wall is equal to the maximum in-plane shear in cylindrical wall, i.e., $2V/(\pi d)$, where V is the applied shear force and d is the diameter. For symmetrical sections, the load is equally shared between the two equivalent walls, however, in the presence of unsymmetrical openings, the induced torsional moment changes the shear distribution pattern. As shown in Fig. 6, the shear force is increased by an amount equal to *Ve/d*, where *e* is the eccentricity of the shear resistance. It can be shown that the torsional shear increase the symmetrical shear force by a factor S equal to $\{1+\psi/(2-\psi)\}$, where ψ is the ratio of the width of opening to length of equivalent shear wall *(0.78d)*.



(a) (b)

***Figure 6**. (a) Circular section resisting lateral shear and (b) equivalent shear wall model for computing lateral shear strength*

The shear strength of shafts at a section near the ground was calculated for all eight tanks using the concept of equivalent shear wall with the shear wall procedure of IS:13920-1993. The critical section was chosen near the base where a door opening of width not less than 1.2 m was always provided for an access to inside of the shaft. A typical transverse and longitudinal reinforcement in shell wall was assumed as 0.25% for the calculations. The smaller shear strength calculated for the equivalent wall with the opening was further reduced by the factor S to account for an increase in shear force demand due to torsional effects. Similarly, the shear strength of the other equivalent wall was increased by the same factor S to account for reduction in the shear force due to torsional effects. The calculations are summarized in Table 3. The net shear strength thus obtained was compared with the expected seismic demand tank-empty and tank-full conditions in Fig. 7.

Table 3. Shear strengths of shaft stagings

No.	Name and Location	Shear Strength of Equivalent Walls		Factor for Torsional Effects	V_{sh}/W_s	
		Wall 1	Wall 2		Tank-Full	Tank-Empty
1	Anjar	1.11	0.90	1.11	0.46	0.14
2	Gandhidham	0.30	0.15	1.32	0.44	0.24
3	Morbi	0.39	0.25	1.23	0.42	0.19
4	Gala	1.31	1.06	1.10	0.44	0.16
5	Chobari	0.30	0.18	1.26	0.18	0.09
6	Bhachau	0.32	0.15	1.36	0.54	0.31
7	Sapeda	0.86	0.67	1.13	0.43	0.18
8	Samakhiali	0.47	0.32	1.20	0.44	0.18

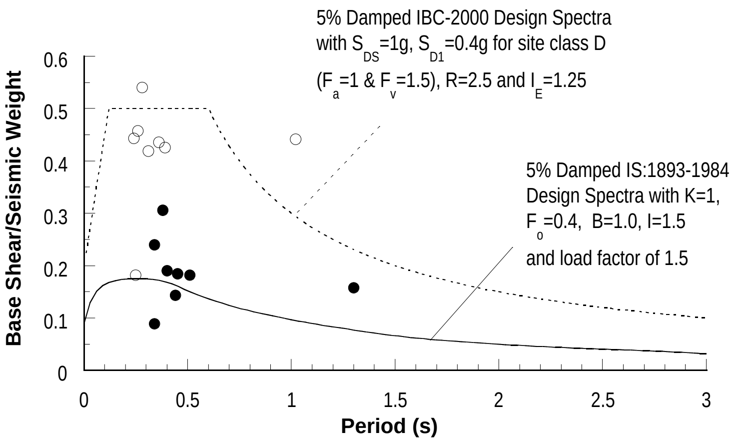


Figure 7 . *Comparison of provided shearing strength of eight tanks with the base shear required by IS:1893-1984 and IBC 2000 codes. 'Open' and 'filled' circles correspond to tank-empty and tank-full conditions.*

DISCUSSION OF RESULTS AND PROPOSED DESIGN FORCES

It is interesting to note that structural designs of eight water tanks with such large variations in their capacities (from 80 kL to 1 000 kL) are such that they are all short period structures for earthquake loads except the one at Gala subhead water works. Consequently, the overall seismic response of these structures is most directly related to accelerations of the ground motion and will not be greatly affected by yielding and ductility of the supporting structure. Therefore, providing a sufficiently large lateral strength is probably the most effective way to ensure protection against ultimate earthquake loads. Further, sections of very thin cylindrical shells do not possess any appreciable level of ductility (Zahn 1990 & Rao 2000). As a result, for such structures, on account of ductility the design forces can not be reduced below those which would be developed if it were to remain elastic during an ultimate event. Consequently, the reduction in design forces specified by various codes on account of inelastic behavior or ductility is significantly small for such structures in comparison to building structures. The small reduction in design forces is also partly due to the little redundancy present in such structures, i.e., one plastic hinge in a staging can cause collapse of the structure.

Codes of practice also realize that it is uneconomic to design structures to remain elastic during a severe earthquake, and therefore, generally allow some inelastic behavior. It is however, a somewhat unresolved issue as to how much ductility can be assigned to these structures. There appears to be a consensus, that it is not significant. Consequently, the reduction in design forces specified by various codes on account of inelastic behavior or ductility is significantly smaller for such elevated tank structures compared to building structures. As a result, most advanced codes such as 2000 IBC specify design forces for such cantilevered pendulum type structures about 2 to 3 times of those intended for building structures.

However, in contrast to 2000 IBC, the design forces prescribed by IS:1893-1984 are essentially at the same level as specified for the most ductile moment resisting frames for building structures. The resulting forces are unjustifiably low for structural systems which do not have advantages of redundancy, ductility and toughness. If the affected tanks were provided only the code level strength, the damage would have been more severe, possibly threatening the lateral stability of the entire structure. In an earlier study Jain and Sameer (1993) also have also pointed out this deficiency of IS:1893-1984 and advocated that the forces be increased by at least a factor of 3.

The slender and weak staging that results from the low design forces are a very unfavorable feature in seismic areas. If the affected tanks were provided only the IS:1893 code level strength, the damage would have been more severe, possibly threatening the lateral stability of the entire structure. In other words, the tank shafts in many cases were built *above* code as far as lateral strength is considered and if they had been built only *to* code, more would have suffered extensive damage and even collapses. As seen in Fig. 4 and 7, the margin of overstrength is more for tank-empty case than tank-full case. Relatively low grade damage to many tanks is also due to the fact they were either partially full or empty when the earthquake struck. It is clear that such deficient structures can not be made less vulnerable by repairing the cracks with epoxy grouts, etc. A comprehensive strategy of retrofitting is required which addresses their strength weakness as discussed. Such a retrofitting scheme was investigated for the tank damaged in Jabalpur and the details of retrofitting work carried out is presented elsewhere (Rai 1998 and Rai 2002b).

Proposed Design Forces and Response Reduction Factor

Recently revised version of IS:1893 (Part 1) (BIS 2002) provides the following relation for the design base shear V_b as a ratio (referred as A_h , design horizontal seismic coefficient) to seismic weight W :

$$\frac{V_b}{W} = A_h = \left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)\bigg/\left(\frac{R}{I}\right) \tag{2}$$

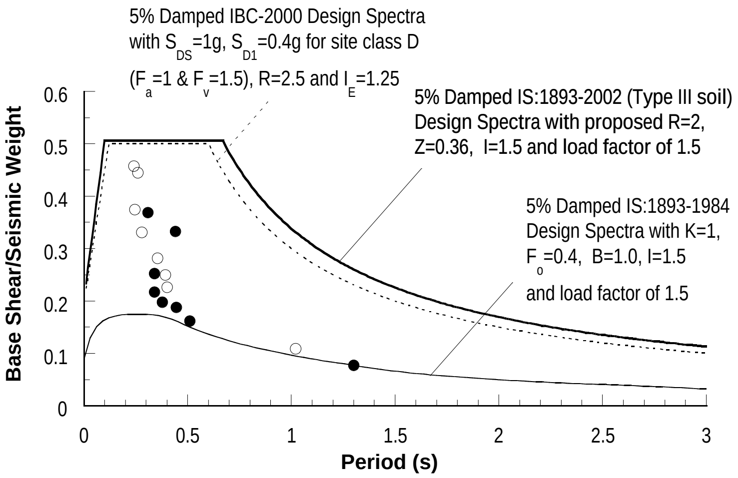
where, Z is seismic zone factor, I is an importance factor and R is response reduction factor and (S_a/g), is average response acceleration coefficient. A value of R for elevated tanks structures to be used with Eqn. (2) is currently not available as Part 2 of IS:1893 concerning with liquid tanks is still under preparation. As shown in Fig. 8, a value of R equal to 2 provides reasonably safe estimate of the expected seismic demand for the eight shaft supporting structures of the Kachchh region affected in Bhuj earthquake. Further, it compares very well with the 2000 IBC design forces in a similar seismic exposure conditions. For comparison purposes, (S_a/g) values corresponding to Type III (soft) soil is used as given below:

$$\frac{S_a}{g} = \begin{cases} 1+15T & 0 \le T \le 0.1 \\ 2.5 & 0.1 \le T \le 0.67 \\ 1.67/T & 1.67 \le T \le 4.0 \end{cases} \tag{3}$$

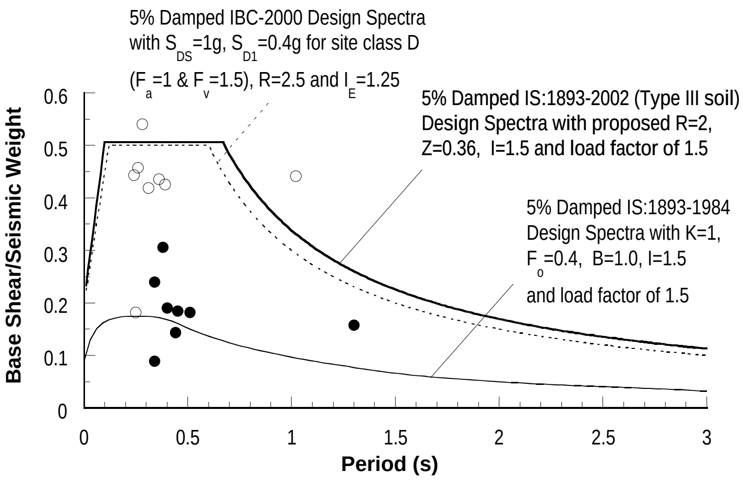
The Zone factor was taken as 0.36 as specified for the Seismic Zone V of the Kachchh region, and a value of 1.5 was assumed for the importance factor as specified in the older version of IS:1893. Since the code forces are specified at working stress level, the spectral ordinates were multiplied by typical load factor of 1.5 to bring it up to ultimate stress level.

CONCLUSIONS

The current design parameters of Indian seismic codes for elevated tanks for elevated tanks result in extremely vulnerable RC shaft-type supporting structures as evidenced in the recent Bhuj earthquake. Supporting shafts developed flexural-tension cracks were observed in tanks as far as 100 km away from the epicentral tract, despite the fact that the most had lateral strength far greater than that specified by IS:1893-1984. An analysis of the flexural tension cracking strength, ultimate flexural strength, and ultimate shear strength of a sample of damaged shaft supports clearly indicate that the code design forces are currently being heavily underestimated. A response reduction factor equal to 2 in the new base shear formula of 2002 version of IS:1893 (Part 1) appeared to provide a reasonable level of safe design forces for elevated tanks, and these forces also agreed well with the forces specified in 2000 IBC for similar seismic exposure conditions. A value of 2 for response reduction factor can be used for inverted pendulum type shaft supported elevated tank structures in the Part 2 of IS:1893, which concerns with liquid retaining tanks.



(a)



(b)

Figure 8*. Comparison of lateral strength of shaft supports with demands expected from the proposed response reduction factor of 2 in the base shear formula of new IS:1893-2002 (Part1) (a) Flexural tensile strength and (b) shearing strength.*

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