Seismic Retrofitting of R/C Shaft Support of Elevated Tanks

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The circular, reinforced concrete (R/C) shaft-type support for elevated tanks lacks redundancy, damping and additional strength typically present in building framing systems and, therefore, should be designed for larger seismic resistance. However, the Indian seismic code *IS:1893-1984* prescribes the same basic seismic force as that for the most ductile building framing system for which the design force is the least. Furthermore, the code-specified one-mass idealization of elevated water tanks is not appropriate for large (large width to depth ratio) and partially filled tanks. The low design forces lead to a weak and slender support—a very unfavorable feature in high seismic areas, as evidenced in the failure of two water tanks in the 1997 Jabalpur earthquake and a great many in the 2001 Bhuj earthquake. It is rather difficult to enhance the ductility and energy dissipation capacity of thin-walled, R/C shaft supports. Concrete jacketing is used as a retrofit measure to enhance the lateral strength and ductility by changing the failure mode of concrete crushing to a more ductile tension yielding. This scheme requires substantial strengthening of the existing foundation.

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INTRODUCTION

The stimulus of this paper was the failed support structure of two 10- to 12-year-old, 0.5-million-gallon (2270 m^3) capacity, elevated water tanks in the Jabalpur earthquake of 22 May 1997. The cylindrical shaft-type staging developed circumferential flexural-tension cracks near the base. Similar damages to support structures had been observed in past earthquakes and recently in the Bhuj earthquake of 26 January 2001, as shown in Figure 1a, which is typical of the damage sustained to a large number of water tanks of capacities ranging from 80 m^3 to 1,000 m^3 and as far away as 125 km from the epicenter (Rai 2001). Figure 1b shows a collapsed water tank in the epicentral tract of the Bhuj earthquake. Such a performance from essential facilities like water tanks is not acceptable, as they are expected to remain functional and safe to operate even after the occurrence of a design-level earthquake (i.e., the largest conceivable earthquake). The supply of safe water is required to prevent outbreaks of disease and to keep fires under control after an earthquake. The performance of existing elevated tanks during severe earthquakes is questionable, especially of those located in high seismic regions, as evidenced in the 2001 Bhuj earthquake.

This study identifies the seismic deficiencies of the shaft supports and how they can be retrofitted or upgraded for future earthquakes. It also highlights the weaknesses of the

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(a) (b)

Figure 1. (a) Flexural-tension cracks in support shaft of 500 m^3 tank being repaired by injecting epoxy in Morbi, 80 km away from epicenter of the Bhuj earthquake; and (b) collapsed 265 m^3 water tank in Chobari village, about 20 km from the epicenter.

current Indian practice of seismic design and analysis of such structures and factors that affect the ductility of reinforced concrete (R/C) hollow cylindrical shell sections, which are used for bridge piers as well as shafts of elevated tanks.

DAMAGE OBSERVED IN JABALPUR WATER TANKS

Two R/C water tanks supported on 20-m-tall shafts developed cracks near the base. There were five such tanks in the city, and those damaged were located in areas that suffered heavy damage in the earthquake (Rai et al. 1997). The tank containers were Intze type, (i.e., below the cylindrical container is a conical shell with a dome shaped tank floor that provides an economical substitute for otherwise thick floor slabs in elevated tanks). The dimensions of the conical walls and the spherical bottom domes are such that the outward thrust from the spherical dome is balanced by the inward thrust from the conical shell. Because of its optimal load balancing shape, the Intze-type containers are widely used.

The Gulaotal water tank (Figure 2) was more seriously affected because it was nearly full when the earthquake struck, whereas the other one was only 60% full. The Gulaotal tank developed flexural-tension cracks along half its perimeter, as shown in Figure 3, on diametrically opposite sides. Some diagonal cracks of shear-flexure origin and some around corners of the window openings were also observed. The flexure-tension cracks in shafts appeared at the level of the first "lift," a plane of weakness, at 1.4 m above the ground level. These tanks were founded on the compaction-type bored under-reamed piles, and no visible distress to surrounding soil or foundation was noted.

Jabalpur elevated water tanks are *inverted pendulum*-type structures, which resist lateral forces by the flexural strength and stiffness of their circular, hollow shafts. The section close to the ground is subjected to the maximum flexural demand for a uniform shaft. Any damage to the shaft at this critical section should be considered serious as it can significantly undermine its lateral load carrying capacity. In fact, the water tank was

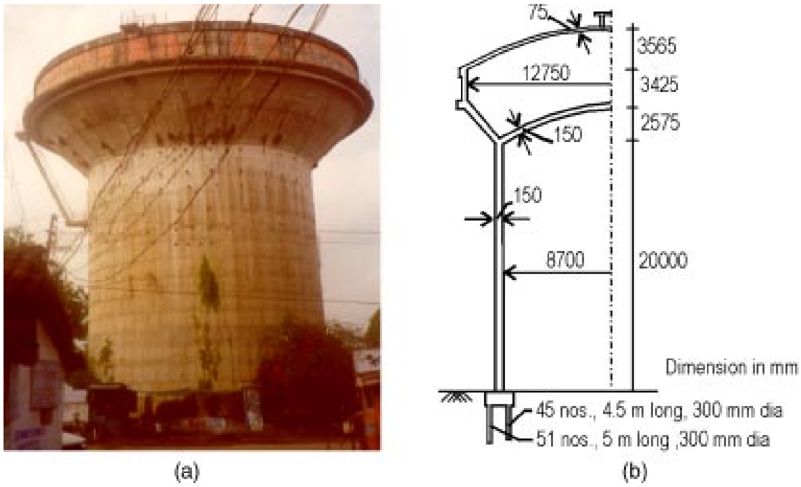


Figure 2. (a) Damaged shaft of Gulaotal water tank in Garha area of Jabalpur city, and (b) a schematic showing various dimensions of the tank structure.

taken out of the city water distribution system, causing severe hardship to neighboring residents particularly in the summer months. The observed damage pattern is consistent with the expected response of these structures under lateral loads.

DYNAMIC BEHAVIOR OF ELEVATED TANKS

The basic dynamics of elevated tanks is somewhat complex, especially those related to the movement of fluids in the tank. However, the estimation of design forces for the supports is relatively simpler. Under lateral accelerations, the fluids in the upper regions of the tank do not move with the tank wall, thus generating seismic waves or sloshing motion of fluids (*convective behavior*). On the contrary, fluids nearer the base of the tank move with the tank structure and, therefore, add to the inertial mass of the tank

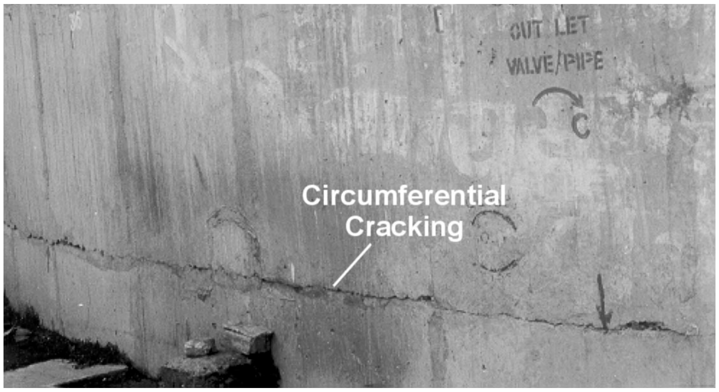


Figure 3. Horizontal flexural-tension cracking near the base of the shaft.

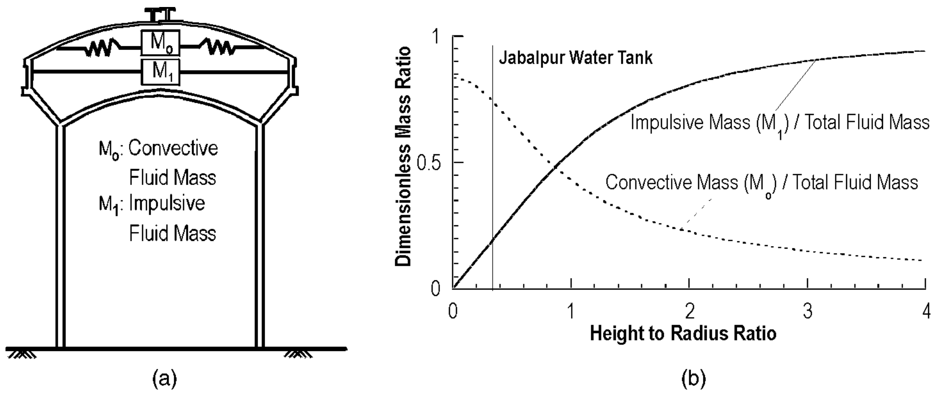


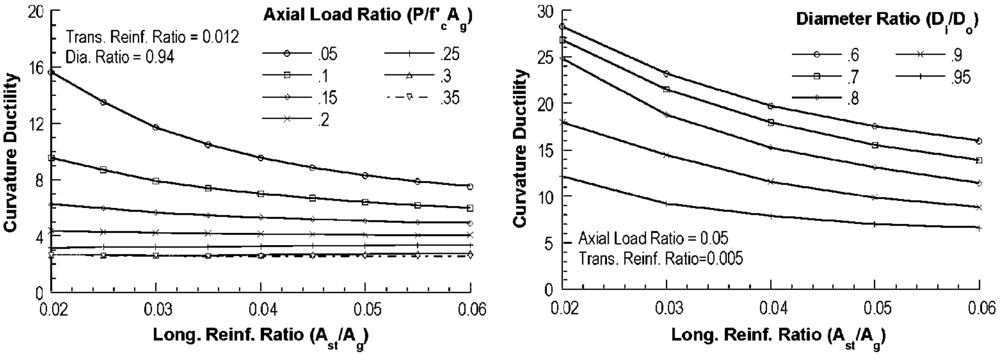
Figure 4. Housner's two-mass model for computing convective and impulsive fluid masses and its variation with aspect ratio of the tank container.

structure (*impulsive behavior*). The portion of the tank fluid that acts in the impulsive mode depends largely on the aspect ratio (height/diameter) of the tank. For tanks of very-low-aspect ratio, very little tank fluid acts in the impulsive mode. The period of sloshing motions are typically long (up to 10 s) and are influenced by the ground displacement rather than the ground acceleration that typically affects impulsive modes of vibration (Priestley et al. 1986).

Several mechanical analogues involving spring-mass systems have been proposed to simulate the dynamic response of elevated tanks (Jacobsen 1949, Housner 1963). The inertial masses are connected to tank walls by rigid links, whereas the convective masses are connected by springs. The flexibility of these springs and attached masses represents various anti-symmetric sloshing frequencies of fluids in the tank. Recently Malhotra et al. (2000) have developed a simplified procedure for seismic analysis of tanks taking account of impulsive and convective actions of the fluid, which has been adopted in *Eurocode 8* (1998). The procedure is developed for cylindrical ground supported tanks but can be easily adapted to elevated tanks.

HOUSNER'S TWO-MASS MECHANICAL ANALOGUE

A satisfactory spring-mass analogue (Figure 4a) to characterize the basic dynamics of elevated tanks was suggested by Housner (1963) after the 1960 Chilean earthquake, which saw damage to a large number of tank supports. This simple two-mass model is more appropriate for elevated tanks than the one-mass model. The two-mass model adequately represents the impulsive and convective modes of vibration, as observed in many experimental studies (Boyce 1973, Shepherd 1972, Gracia 1969, Haroun and Ellaithy 1985). Figure 4b shows the variation in the ratio of impulsive and convective masses to total mass, with respect to the height-to-radius ratio of the tank as given by Housner. For the Jabalpur tanks, only 20.1% of the total water in the tank participated in the impulsive mode. The model parameters were obtained for the Intze-type tank container by considering it an equivalent cylindrical tank having radius and volume equal to the Intze tank. Joshi (2000) has shown that that errors associated with such an approxi-



(a) (b)

Figure 5. Effect of (a) axial load ratio and (b) shell thickness on the curvature ductility of hollow circular sections (Rao 2000).

mation are small and model parameters corresponding to equivalent cylindrical tank for Intze tanks can be used for design purposes. Although the flexibility of the tank shell is usually neglected for the design of support, it should be accounted for in designing the tank walls. The damping value for the impulsive mode of R/C structures is usually taken as 5%; for convective mode, a value of 0.5% is recommended.

INELASTIC BEHAVIOR OF SUPPORT SHAFTS

Conventional earthquake-resistance design is based on the premise that structures can undergo large plastic deformations without collapse. This concept allows the structure to be designed for seismic forces significantly less than those required if the structure had to remain elastic. The seismic performance of such structures rests heavily on the ductility, the energy-absorbing capacity of the detailed structural components, and the redundancy due to alternative load paths. The factor used to reduce the elastic seismic forces to arrive at design forces is, therefore, a function of these properties. The design forces for less ductile systems would be larger than those for more ductile systems. It is expected that the supporting structure of elevated tanks would experience inelastic deformations and, as a consequence, the acceleration response can be reduced by using an appropriate ductility factor. However, this reduction is applied to only impulsive forces, and no reduction is permissible for convective forces as a result of ductility.

The shaft support of the Jabalpur tanks is a shell structure with a thickness of 150 mm and a diameter of 17.4 m. The shell has reinforcement of about 0.2% in each direction. Studies have shown that thin, hollow, circular, R/C sections with high axial load behave in a brittle manner at the flexural strength (Zahn et al. 1990, Rao 2000). The available curvature ductility of hollow circular sections is largely controlled by the level of axial stress, the thickness of the wall, and the longitudinal steel ratio, as shown in Figure 5. Zahn et al. (1990) have shown that an appreciable ductility can be achieved with low axial load, small longitudinal ratio, and a wall thickness not less than 15% of the overall section diameter. Comparing these with the section properties of the Jabalpur tank shaft, which has a large axial load ratio ($P/f'_c A_g$) of 0.26 and a wall thickness of

0.86% of overall diameter, it is obvious that the shaft section of the Jabalpur tanks cannot withstand even moderate inelastic deformations and, therefore, cannot be considered ductile. As a result, design forces cannot be reduced significantly on account of ductility and energy dissipation capacity of the shaft.

CODE PROVISIONS FOR SUPPORTS

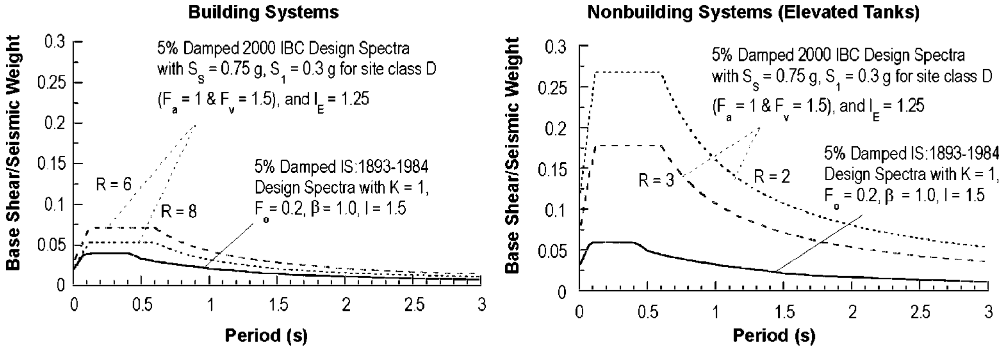
MODELING FOR ANALYSIS

The Indian seismic code, *IS:1893-1984* (BIS 1984) requires elevated tanks to be analyzed as a single-degree-of-freedom system, i.e., a one-mass system, suggesting that all fluid mass participates in the impulsive mode of vibration and moves with the tank wall. This can be a realistic assumption for very long, slender, tank containers with a height-to-radius ratio exceeding 4 (i.e., a stand pipe), as shown in Figure 4b. Although the *2000 International Building Code (IBC)* (ICC 2000) does not specify how elevated tanks should be analyzed, a multi-mass model is generally used in practice. The *ACI 371R-98* (ACI 1998) allows one-mass approximation in cases where the water weight is about 80% of total seismic weight of the tank structure. A two-mass model was recommended for the analysis of elevated tanks by a study group of the New Zealand National Society of Earthquake Engineering (Priestley et al. 1986). An estimate of design base shears, using *IS:1893-1984* with the one-mass and the two-mass model approach for the Jabalpur tanks, indicates that by using the two-mass model approach, the design base shear and the base overturning moment are 0.56 and 0.74 times, respectively, of those obtained by using the one-mass model (Rai 1998). It should be noted that this discrepancy will be different for tanks with different geometric proportions.

DESIGN FORCE LEVELS

IS:1893-1984 prescribes design forces for elevated tanks at 1.5 times of the most ductile building frame system. This increase is due to the importance factor and is not due to the structure performance factor K, which is assumed to be 1 (a value specified for the most ductile building framing system). The factor K represents the acceptable level of inelastic deformation demand for a given material and system. In comparison, the *2000 IBC* prescribes an importance factor of 1.25 for water tanks but specifies different values for the structure performance factor R (equivalent to K) from building systems to indicate their lack of redundancy, lesser damping, and strength due to the absence of nonstructural and nonconsidered resisting elements typically present in building systems. On average, building systems have a value of $R=6$, which is reduced to one-third for elevated tank systems, i.e., $R=2$. Similarly, *ACI 371R-98* also specifies a value of 2 for R for the support of elevated tanks for *1997 Uniform Building Code (UBC)* formulas for design base shear, which is not significantly different from the *2000 IBC*. This means the design forces for elevated tank systems are about three times as large for a building system with similar dynamic properties. Though these R values are judgmental, larger R values are assigned to systems with excellent energy dissipation capacity and stability, as ensured by very specific design and detailing procedures.

In conclusion, *IS:1893-1984* prescribes forces that are too small for elevated tanks, compared to those prescribed by advanced standards, such as *2000 IBC*. The damage



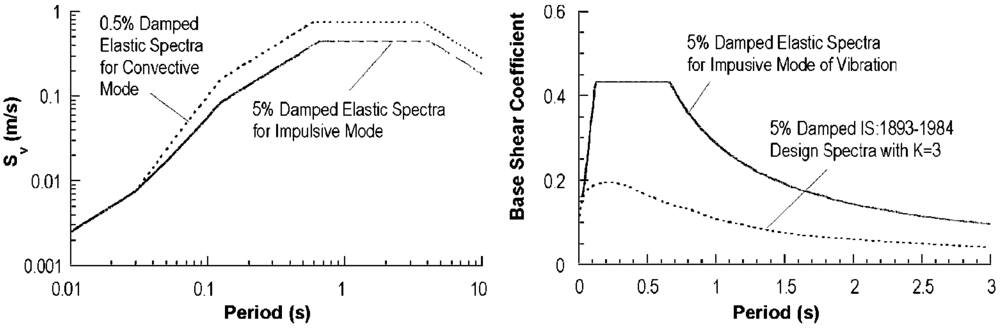
(a) (b)

Figure 6. Comparison of design base shear spectra of *IS:1893-1984* and *2000 IBC* for building and non-building (elevated tank) systems. (Note that *2000 IBC* forces were reduced by a factor of 1.4 for comparison with working stress level forces of *IS:1893-1984*)

observed in the Jabalpur and Bhuj earthquake illustrates that the design forces are currently being underestimated. The slender and weak support that results from the low design forces is a highly unfavorable, feature for high seismic areas. For seismic zone III, Figure 6 compares the *IS:1893-1984* design spectrum curves for a building system (e.g., moment-resisting frame) and a non-building system (e.g., a water tank) to those of *2000 IBC* in a similar seismic environment (i.e., the short period and 1 s spectral response acceleration for maximum considered earthquake ground motion were taken as 0.75 *g* and 0.3 *g*, respectively). It is apparent that the *IS:1893-1984* design force levels are very low and unrealistic. These spectra, if used with the two-mass model of elevated tanks, will result in further lowering of the design forces. Clearly, the provisions of *IS:1893-1984* do not truly reflect the state of knowledge and result in questionable design parameters for elevated tanks.

DUCTILE DETAILING OF SHAFT SUPPORTS

There are no provisions in the IS codes for ductile detailing of shaft supports, though they are expected to undergo large inelastic deformations during a maximum credible earthquake. The *Eurocode 8*, Part 4 (CEN 1998), *2000 IBC*, and *1997 UBC* expect a ductility value ranging from 1.7 to 2.5, which can only be achieved by proper proportions of the shaft and reinforcing details. In contrast, framed supports of elevated tanks can be detailed in accordance with *IS:13920-1993* (BIS 1993) and *IS:11682-1985* (BIS 1985), which refer to the ductility requirements of *IS:4326-1976* (BIS 1976). There is very limited literature available on the ductile detailing of thin shell sections, and they are generally considered nonductile.



(a) (b)

Figure 7. (a) Newmark-Hall design spectra for impulsive and convective modes (*PGA* =0.16 *g*) and (b) comparison of impulsive mode spectra with *IS:1893* spectra with *K*=3.

SEISMIC DEFICIENCIES OF JABALPUR WATER TANKS

ESTIMATE OF SEISMIC DEMAND: RESPONSE SPECTRUM ANALYSIS

In view of the above-mentioned deficiencies of *IS:1893-1984*, an estimate of seismic demand is obtained by an alternative procedure for the Jabalpur tank structure. A response spectrum analysis is carried out using Housner's two-mass idealization for the Jabalpur tank, with spectral values from a response spectra developed from the procedure outlined by Newmark and Hall (1982). The basic input parameter in this method is the probable peak ground acceleration (PGA) at the site. A value of 0.16 *g* for mean horizontal *PGA* is considered adequate. The proposed draft code of *IS:1893* (BIS 1998) also specifies a value of 0.16 *g* for the city of Jabalpur. The values for peak ground velocity (PGV) and peak ground displacement (PGD) were obtained from the typical relations: $PGV/PGA = 1.22$ m/s/g and $PGA \times PGD/PGV^2 = 6$, recommended for firm ground. For $PGA = 0.16$ g, these relations give, $PGV = 195$ mm/s and $PGD = 146$ mm.

The spectral amplification factors correspond to the 84th percentile, which represents the mean plus one standard deviation of the scattered data points. These values near the upper bound of the scatter are desirable, considering the critical importance of the water tank structure. A 5% damped elastic spectra was developed for the impulsive mode, whereas a 0.5% damped elastic spectra was computed for the convective mode of vibration, as shown in Figure 7a. The derived design response spectra for impulsive mode is compared with *IS:1893-1984* design spectra in Figure 7b. It should be noted that in Figure 7b, the *IS:1893-1984* spectra that was based on a performance factor *K* =3, as suggested by Jain and Sameer (1993) for water tank structures, has smaller spectral ordinates at all periods; this means that a significant level of inelastic deformation capacity (ductility) is expected from the shaft support. The 0.5% damped spectra for convective modes is not available in the *IS:1893-1984*. Table 1 shows the seismic demand as obtained from the response spectrum analysis in terms of unfactored base shear, base overturning moment, and axial load using two-mass models of Housner (1963) and

Table 1. Summary of dynamic properties of Jabalpur tank and estimates of seismic demand using models of Housner (1963) and Malhotra et al. (2000)

Item	Housner (1963)		Malhotra et al. (2000)	
	Impulsive Mode	Convective Mode	Impulsive Mode	Convective Mode
Water weight (MN)	4.48	16.4	4.59	17.7
Heights of weights (m)	1.67	2.30	1.78	2.34
total weight (MN)	14.5@	16.4	14.6@	17.7
Spring stiffness (MN/m)	1880	0.87	-	-
Time period (s)	0.19*	7.0	0.19*	7.2
Spectral Acceleration (g)	0.43	0.024	0.43	0.024
Base Shear (MN)	8.15	0.39	8.23	0.41
	SRSS value	*8.15*	*ABS value*	*8.64*
Heights of weights for moments in the support (m)	10.5	12.4	10.5	13.1
Base overturning moment (MN m)	203.0	12.8	205.3	13.5
	SRSS value	*203.4*	*ABS value*	*218.8*
Height of sloshing wave (m)	-		-	0.3

@ Includes weight of tank proper, one-third weight of support and weight of impulsive portion of water.

* $T=2\pi\sqrt{W/(gK)}$, where W is total impulsive weight and K is structure lateral stiffness determined from the deflection of the shaft acting as a cantilever beam subjected to a concentrated end load.

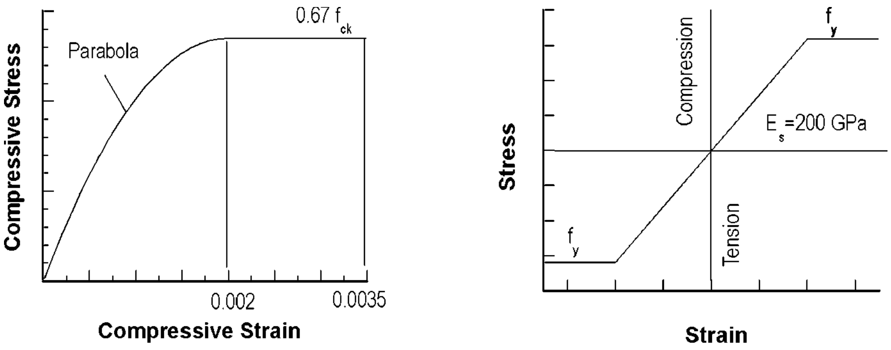
Malhotra et al. (2000). It is clear that similar values were obtained from both approaches, however, the method by Malhotra et al. (2000) is easier to use and especially suitable for design offices.

ULTIMATE STRENGTH (CAPACITY) OF SHAFT SECTION

The ultimate strength analysis of the shaft section involves the calculation of the ultimate direct force, P_u, and the ultimate bending moment, M_u, that can be resisted by the resulting stress envelope. The calculations are essentially the same as outlined by Pinfold (1989), with different material properties of concrete and steel as given in *IS:456-2000* (BIS 2000) without partial safety factors (Figure 8). For the shaft section, the envelope of ultimate resistance is presented in the form of an interaction plot M_u as the abscissa and P_u as the ordinate, as shown in Figure 9.

ASSESSMENT OF SEISMIC VULNERABILITY: DEMAND CAPACITY RATIOS

To assess the seismic vulnerability of a tank's support, the available ultimate capacity at the critical section is compared to the probable demands shown in Table 1. The critical combinations of demands, the factored axial load P, and the base moment M, are plotted on the capacity plot (interaction diagram) as shown in Figure 9. It is obvious that the structure is not safe since demands lie outside the interaction curve, implying compression or tension failure of the section depending on the level of axial compression. The failure by crushing of the concrete is not regarded as a ductile mode of failure and, therefore, is not preferred.



(a) (b)

Figure 8. Stress-strain curves for concrete and steel materials used in the strength analysis as per *IS:456-2000*. f_{ck}, compressive strength of 150 mm concrete cubes ($\sim 1.25 f'_c$, the cylinder strength)=20 MPa and f_y, specified yield strength of steel=415 MPa.

The largest demand capacity ratio (DCR) is 1.2, which is obtained by dividing the factored loads (in this case, base moment *M*) by the expected capacity (moment capacity M_u) at the corresponding level of axial load *P*. DCR larger than unity represents unsafe behavior of the structure in design-level earthquakes. Moreover, the above computation of capacity assumes that the section was not damaged and that the materials were in "new" condition. If suitable allowances are made for reduction in the material strength,

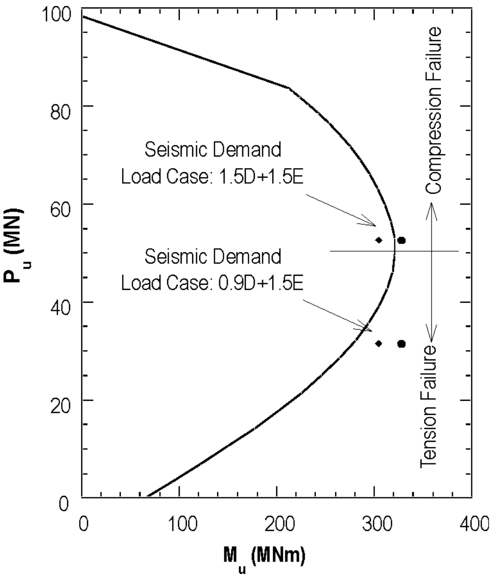


Figure 9. Interaction diagram for the shaft section and seismic demands corresponding to critical load cases using models of Housner (1963) and Malhotra et al. (2000).

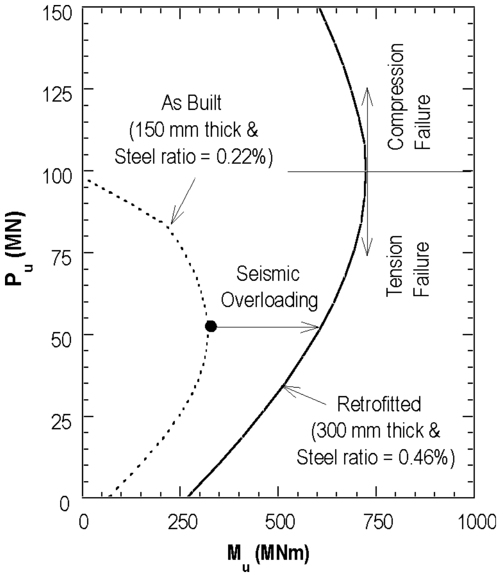


Figure 10. The nominal strength envelope of retrofitted shaft section.

the section properties, and other aspects of capacity affected by sustained damage, then it is highly likely that without retrofitting, the DCR will increase further for the damaged structure, indicating its vulnerability in future earthquakes.

SEISMIC STRENGTHENING FOR DEFICIENT SHAFT SUPPORT

The preceding sections demonstrated the need for retrofitting the support, given the deficiency in the flexural strength of the shaft section even for design level earthquakes. A number of retrofitting techniques have been developed for increasing flexural ductility and strength of R/C members, such as confining by pre-stressing and jacketing with steel, R/C, and other composite materials. Steel jacketing, which enhances the deformability of the section through passive confinement, has been the method of choice for circular bridge columns. Alternatively, a thick layer of R/C concrete can be used to increase the flexural strength, ductility, and shear strength of R/C columns. Longitudinal bars in the jacket should be well anchored into the footing so that reinforcement strength can be developed. Moreover, increase in flexural strength generally requires retrofitting the footing to avoid flexural and shear cracking in the footing.

RETROFITTING OF SHAFT SUPPORT

For the shaft of the Jabalpur tanks, a R/C jacketing was found to be a convenient retrofit method because of its ease of construction. Since the ductility of the section depends heavily on the thickness of the shell wall, a thick layer of reinforced concrete would enhance its seismic response. The thickness of the R/C jacket and the amount of longitudinal reinforcement was determined so that the tension yielding occurs in the case of seismic overload for an extreme earthquake event. As shown in Figure 10, a R/C

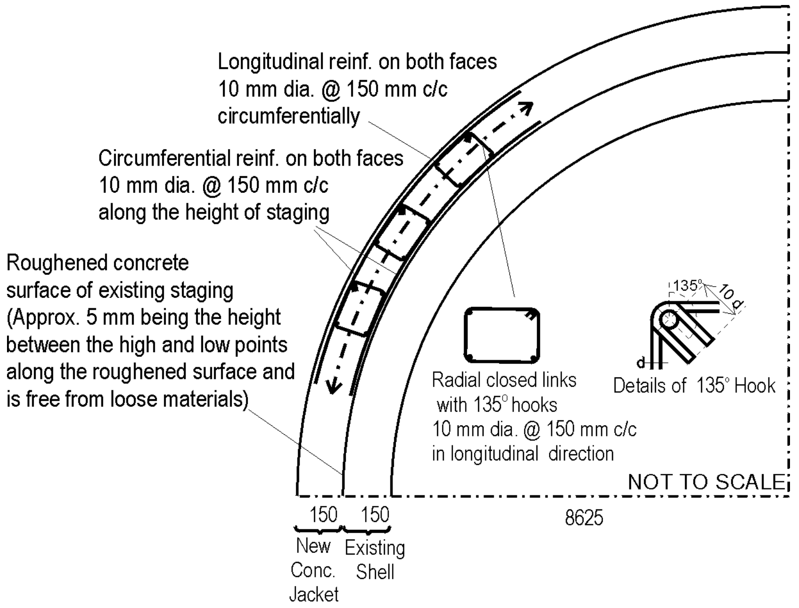


Figure 11. Quarter plan of the retrofitted shaft section and arrangement of reinforcement.

jacket of 150 mm thickness with 0.7% longitudinal steel resulted in a section with an overstrength factor of 1.8. This relatively large overstrength is justified for a system that does not have the advantage of redundancy, and damping. The R/C jacket was provided over a height of 5 m above the top of the retrofitted pile cap, with the exception of the top 2.5 m where the thickness of the R/C layer was reduced to 100 mm and the amount of steel was also reduced to 0.35%.

A cross section of the retrofitted shaft is shown in Figure 11. *ACI 371R-98* specifies that cross ties are required in walls at locations where concentrated plastic hinge or inelastic actions are expected during seismic loading. The size and spacing of ties should conform to *ACI 318* (ACI 1999) requirements for seismic areas. The existing shell had no cross ties and the horizontal and vertical steel ratios were 0.0022 against the minimum requirement of 0.0025. The shell thickness of 150 mm is less than the minimum requirement of 200 mm. In the new concrete jacket, cross ties were provided for a height of 2.5 m above the footing top that covers the bottom one-fourth of the shaft where inelastic behavior is likely to occur (Figure 12). Also the horizontal and vertical steel ratios in the jacket were increased to 0.007. The vertical reinforcing bars were spliced at staggered locations and not more than one-third of the bars were spliced at a particular level. In the newly added jacket, closely spaced stirrups were provided over the length of splice.

RETROFITTING OF FOOTING

Retrofitting of the footing as a result of the shaft's increase in lateral strength is necessary for the success of the overall retrofit scheme. Ideally, the footing should be able

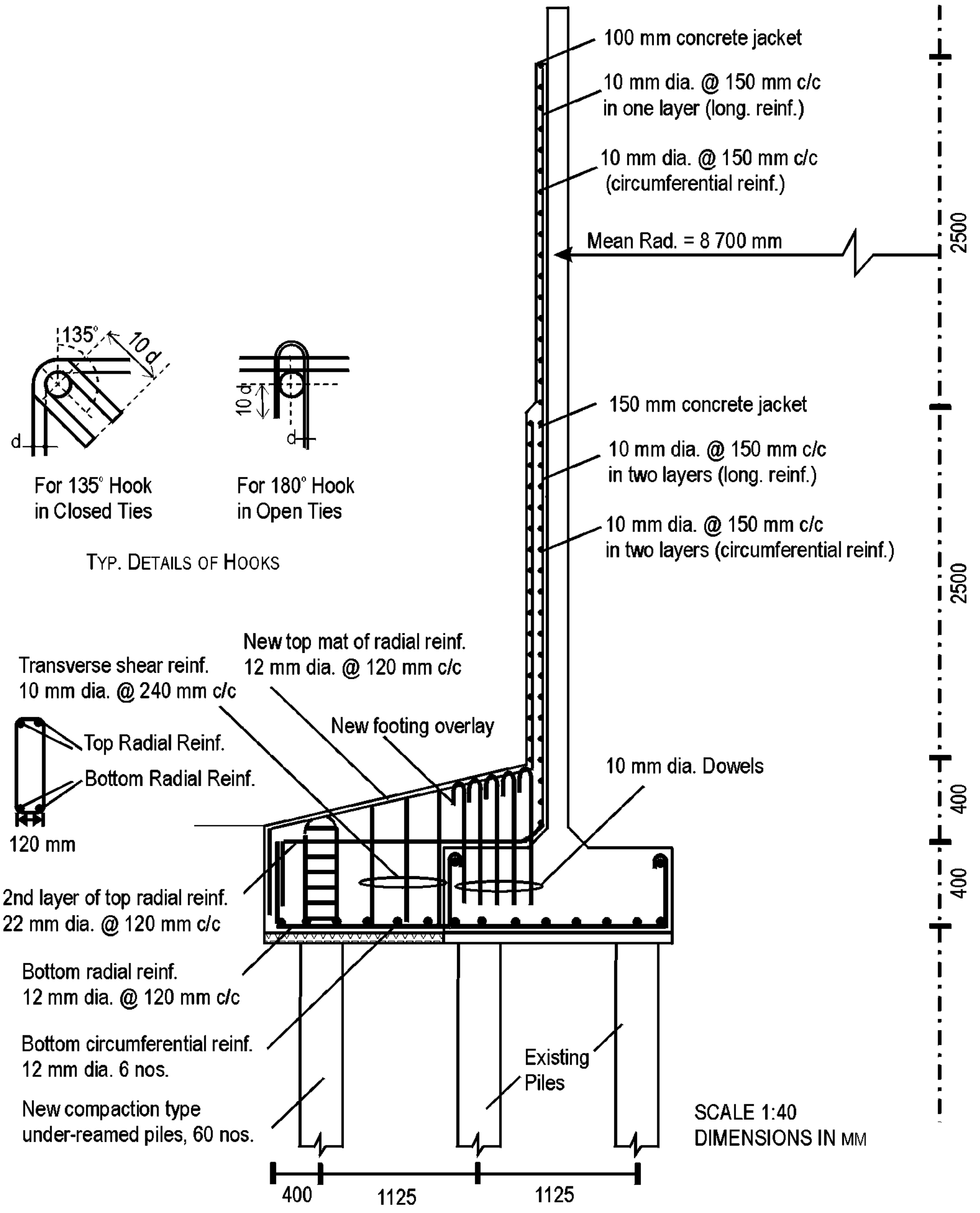


Figure 12. Schematic showing the retrofit scheme for the damaged shaft.

to sustain forces large enough to cause plastic hinging in the shaft without any flexural or shear cracking of the footing. However, this criterion for the footing retrofit can be very expensive and even infeasible. There have been few incidents of footing failures in earthquakes, where liquefaction or failure of the ground occurred. Considering the high

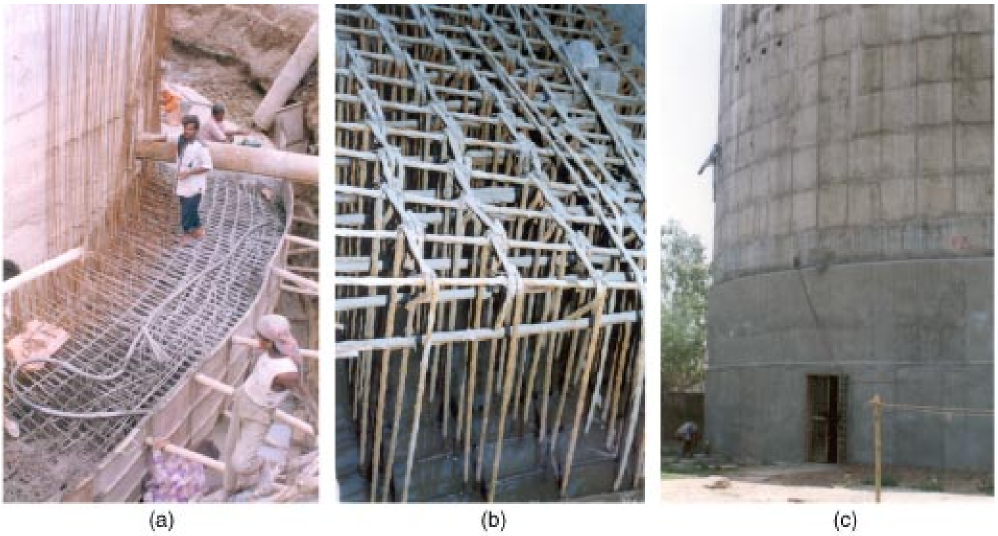


Figure 13. (a) Reinforcement layout for new pile cap and shell jacket, (b) close-up view of reinforcement in pile cap, and (c) the retrofitted shaft.

cost of retrofitting the footing and the relative absence of footing failure, a less conservative approach was adopted for the design of the footing retrofit.

In the case of the Jabalpur tanks, the footing should be ideally retrofitted to provide a flexural capacity of 605 MN m, which means an extremely extensive retrofitting and, if doable, it will be very expensive. The flexural capacity of the existing footing of Jabalpur tanks is about 75 MN m. In a less conservative approach, the footing retrofit design of the Jabalpur tanks was based on the flexural strength of the retrofitted section ignoring the increase due to axial loads from the water stored, i.e., 265 MN m corresponding to an empty tank.

A schematic of the complete retrofit scheme is shown in Figure 12. The existing footing was extended to accommodate 60 new 300-mm-diameter under-reamed piles, and the flexural strength of the footing was enhanced by increasing the depth of the footing as well as by providing three layers of reinforcement in the new footing overlay. Longitudinal reinforcements in the bottom layers were welded to the bottom reinforcements of the old pile cap across the existing footing boundary. The continuity of the tension reinforcing bars was essential to develop the full flexural capacity of the combined footing. The dowels were provided to transfer the interfacial shear, which was calculated by assuming the coefficient of friction equal to unity in addition to the shear transfer expected from the roughening of the existing surface. The closed shear stirrups provided in the footing extension would help in transmitting the shear force from the outer piles to new piles (Priestley et al. 1996). Figures 13a and 13b show the reinforcement layout in the footing overlay and in the shell jacket, respectively, and Figure 13c shows the tank shaft after the retrofitting.

CONCLUSIONS

Thin-walled, circular, shaft supports for elevated tanks behave in a brittle manner at the flexural strength. And the available ductility is also very small for thin-walled sections. The Indian seismic code, *IS:1893-1984*, specifies design forces equivalent to building framing systems and ignores the fact that shaft supports lack the redundancy, damping, and additional strength of building framing systems. Housner's two-mass idealization is a more accurate representation than the IS code-specified one-mass model. Failure of shaft supports of water tanks in the recent 2001 Bhuj and the 1997 Jabalpur earthquake illustrate the above-mentioned deficiencies of the current practice.

An appreciable ductility for hollow circular sections can be achieved with low axial load, small longitudinal steel ratio, and a thick wall. Concrete jacketing was practical to enhancing lateral strength and ductility of the sections by changing the failure mode from the concrete crushing to a more ductile tension yielding. The necessary upgrading of the footing was achieved by extending the pile cap to accommodate new piles, and an overlay of concrete enhanced the flexural strength. It is necessary that the reinforcing bars at the boundaries of the existing footing be properly joined since their continuity is a must for the section to develop the desired strength.

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