A REVIEW OF REQUIREMENTS IN INDIAN CODES FOR ASEISMIC DESIGN OF ELEVATED WATER TANKS

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SYNOPSIS

In this paper I.S. code provisions for aseismic design of elevated water tanks have been reviewed based on comparison with seismic codes of other countries and reports of several investigations in recent years. It is pointed out that the seismic design force in IS:1893-1984 is rather low due to the absence of suitable performance factor which must be in the range of 3.0 - 4.5. On the other hand code is conservative (in most cases) in proposing that the tank be modelled as a single degree of freedom system. Other proposals for modifications include consideration of torsion in design, explicit ductile detailing requirements for staging, separate treatment for tanks with flexible walls, etc.

KEY WORDS

Design Code; Earthquake; Seismic Design; Water Tanks.

1. INTRODUCTION

Water supply is a life-line facility that must remain functional following an earthquake. Most municipalities in our country have water supply system which depends on overhead tanks for storage. In major cities the main supply scheme is augmented by individual supply systems of institutions and industrial estates for which overhead tanks are an integral part. These structures have a configuration that is specially vulnerable to earthquakes due to the large total mass concentrated at the top of a slender supporting structure. The reports of past earthquakes have indicated that often damage of the elevated water tanks was due to the failure of the supporting structure. The patterns of damage have been identical in many cases (e.g., Steinbrugge and Flores, 1963). The bracings failed under shear causing increase in moment in columns leading to formation of hinges in columns.

One tank of capacity 1 lakh British gallons on staging height of 18.3 m was damaged in Bihar-Nepal earthquake (magnitude 6.7) of August 1988, Figs. 1 and 2. The tank was located at Khagaria in Bihar which is at about 100 km from the epicentre. Due to the large epicentral distance, the ground motion must have been rather weak. The tank was built by a large reputed construction company in about 1960. This was before the first Indian code on earthquake forces was published (IS: 1893-1962). Fig. 3 shows the crack locations obtained from Public Health Engineering Department of the Government of Bihar. The damage has been described as:

"Almost all the members of the cross beams and bracings of first stage and second stage have cracked in central portion. In the second stage the cross bracings and bracings have cracked at the joints as well. In the third stage, two members of the cross beams have cracked in their centres.

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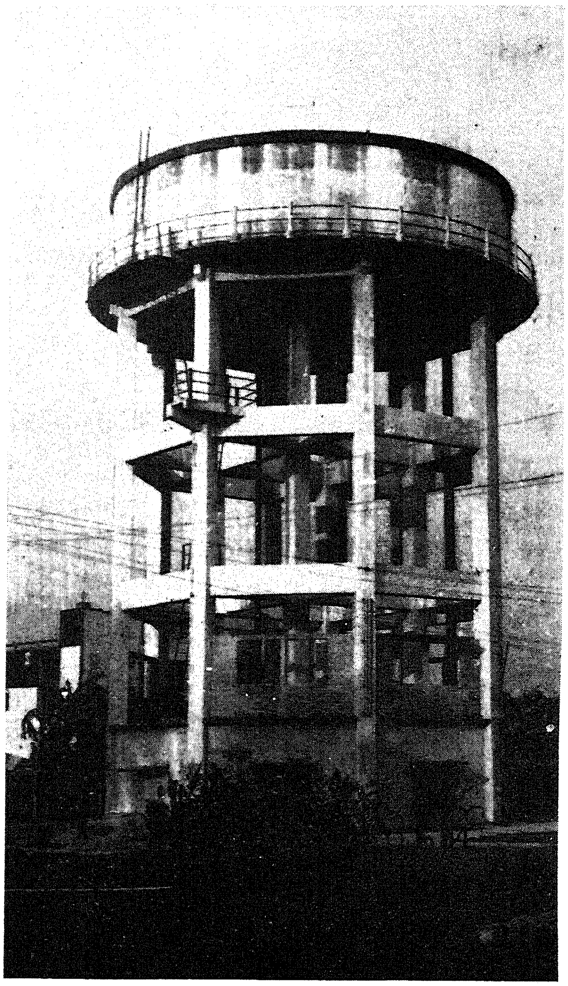

Fig. 1. Water tank damaged at Khagaria during Bihar - Nepal earthquake of August 1988

In addition to this, eight members have cracked at both ends and eight members have cracked at one end. In top staging, seven members of the cross beams and bracings have cracks in their central portions. In the tank portion there are two circumferential hair cracks at an interval of 3 feet. In columns, quite a large number of fine hair cracks were observed. At a new junctions, significant joint cracks were also seen. Verticality test conducted on all the eight external columns gave no indication of any tilting. A few cracked sections were chiseled out to access the nature and extent of the cracks. In some of these, concrete portions were also found affected."

There appears some contradiction between the above description and the damage marked in Fig. 3. However, it is clear that in this case also the bracing girders sustained damage in shear. This is in line with the earlier observations based on the Chilean earthquake.

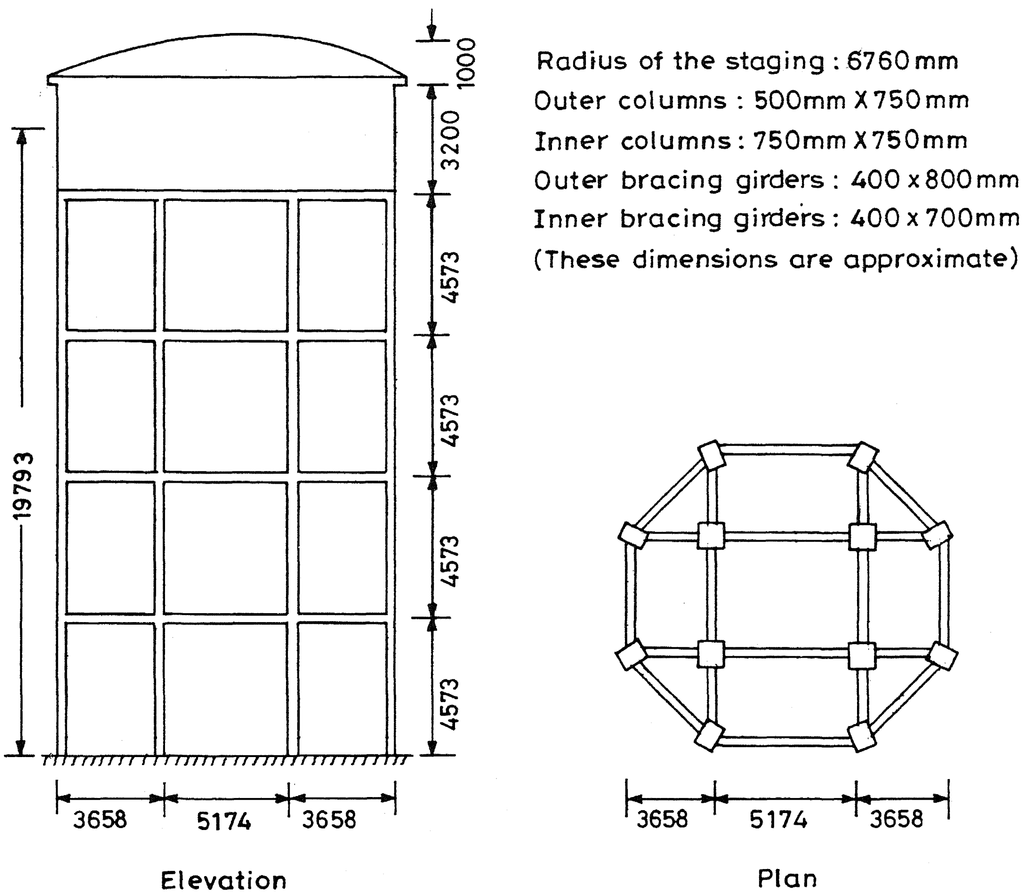

Fig. 2. Details of the tank at Khagaria

It is thus seen that elevated water tanks are quite vulnerable to damage in earthquakes. Due to smaller exposed area and large mass specially when it is full, earthquake forces almost always govern the lateral force design criteria for these structures in zones of intense seismic activity. Thus out of the total cost of the supporting structure, a major share goes towards making the supporting structure earthquake resistant. Hence, the aseismic design criteria for these structures requires a careful consideration and compromise between economy and importance for ensuring safety of these structures.

In our country the seismic design criteria is provided by IS:1893-1984 which gives minimum loading standards and IS:4326-1976 which contains design and detailing requirements for construction of building. IS:11682-1985 gives the criteria for design of RCC staging of such structures. In this

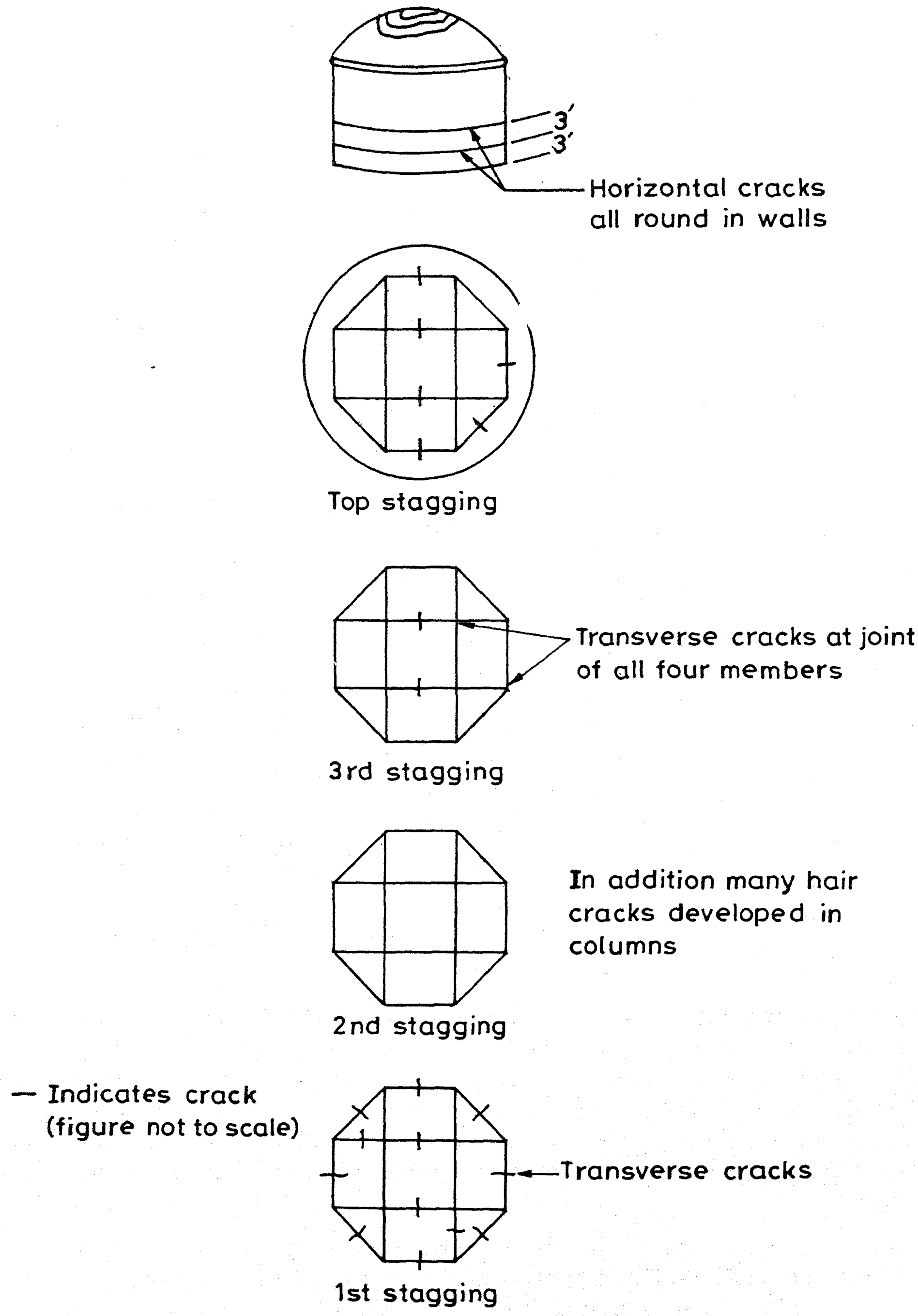

Fig. 3. Details of damage to the tank at Khagaria (as per PHE Dept., Govt. of Bihar).

paper provisions pertaining to aseismic design of elevated water towers are closely examined and proposals that may be useful in future editions of the codes are put forward. Also areas that need further investigation are pointed out.

2. PROVISIONS FOR ELEVATED TANKS IN INDIAN CODES

IS:1893-1984 requires that the design lateral force shall be taken as

$$F = \alpha_h W \tag{1}$$

where α_h is equal to $\beta I F_o S_a/g$ in which β = coefficient depending upon the soil-foundation system; I=importance factor; F_o =seismic zone factor; and S_a/g = average acceleration coefficient obtained from acceleration spectra given in the code. The tank needs to be designed for tank full as well as empty condition. When empty, W used in Eq. 1 consists of dead load of the tank plus one third the weight of the staging. For tank full condition weight of the contents is to be added to that at the empty condition. Time period of the tank is to be calculated as

$$T = 2\pi \sqrt{\Delta/g} \tag{2}$$

where Δ is the static deflection at the top of the tank under a static horizontal force equal to W. Damping is to be assumed as 2 percent for steel structures and 5% for reinforced concrete structures.

The code prescribes expression for calculation of impulsive hydrodynamic pressure at any point in the tank. The code specifically suggests that convective pressures (due to sloshing of the liquid) need not be considered. On the contrary, IS:1168201985 clearly states that "wherever required the effect of surge due to wave formation of the water may be considered."

Even though the title itself of IS:4326-1976 (Code of Practice for Earthquake Resistant Construction of Buildings) suggests that it is applicable only for buildings and not for tanks, IS:11682-1985 indicates that if design seismic coefficient, αh, exceeds 0.05, detailing should follow IS:4326-1976.

3. DISCUSSION ON CODAL PROVISIONS

3.1 Lateral Design Force

In Eq. 1 for determination of lateral design force for elevated tower, performance factor, K. Which the code uses for computing base shear for buildings, is absent. This implies that K is 1.0 for the elevated water towers which is the same as that for buildings with ductile moment resisting frames. This appears unreasonable as the elevated tank type structures have single load path, have lower energy absorbing capacity and poor ductility as compared to that in ductile moment resisting frame buildings. The seismic codes all over the world prescribe a factor, similar to the performance factor in IS:1893-1984, which is 2.8 to 4.5 time higher for elevated water tanks than that for buildings with ductile moment resisting frames. Commentary to SEAOC-1980 justifies larger value of K for elevated tanks as follows: "cross-braced towers supporting elevated tanks and their contents require K = 2.5 ... considering relatively poor performance of this type of structure and the importance in maintaining their integrity following an earthquake, this highest value of K is justified."

Table 1 gives comparison of performance factor (or equivalent) prescribed by various seismic codes for ductile moment resisting frames and for elevated tanks. It is observed that except the IS code, all other codes give performance factor for elevated towers 2.8 to 4.5 times that for ductile buildings. Fig. 4(a) gives a plot of ratio of lateral force to the weight of the tank and contents against time period for a few of the seismic codes for the highest seismic zone and hard soil condition. This assumes a single-mass representation of the tank. A similar plot for buildings is given in Fig. 4(b). It is noted that lateral force level adopted by Indian code for elevated tanks is far below that by other

TABLE 1 : COMPARISON OF PERFORMANCE FACTOR USED BY VARIOUS SEISMIC CODES

Code	Description of structure	Performance factor or equivalent	$\frac{K_{tank}}{K_{building}}$
UBC-1985 and SEAOC-	Buildings with ductile moment resisting space frame	0.67	
	Elevated tanks plus full conbraced legs	2.5	3.73
UBC-1988	Special moment-resisting frames of concrete for buildings	$\frac{1}{12}$	
	Tanks, vessels or pressurised spheres on braced or unbraced legs	$\frac{1}{3}$	4.00
ATC-3-06	Special moment resisting frames of R.C.C.	$\frac{1}{7}$	
	Inverted pendulum structures (R.C.C.)	$\frac{1}{2.5}$	2.80
A.S.: 2121-1979 (Australia)	Dutile moment-resisting frames (R.C.C.)	0.67	
	Free standing elevated tank, plus full contents on four or more cross braced legs	2.50	3.73
NBC of Canada 1980	Ductile moment-resisting frame	0.7	
	Elevated tanks plus full contents	3.0	4.29
SI 413-1975 Isreal	Ductile frames of reinforced conc.	0.67	
	Water reservoirs and structures, with their mass concentrated at the top of a column	3.0	4.48
IS:189 -1984	Moment resisting frame with appropriate ductility details given in IS: 4326-1976	1.00	
	Elevated tanks	Absent (Implies = 1)	1.00

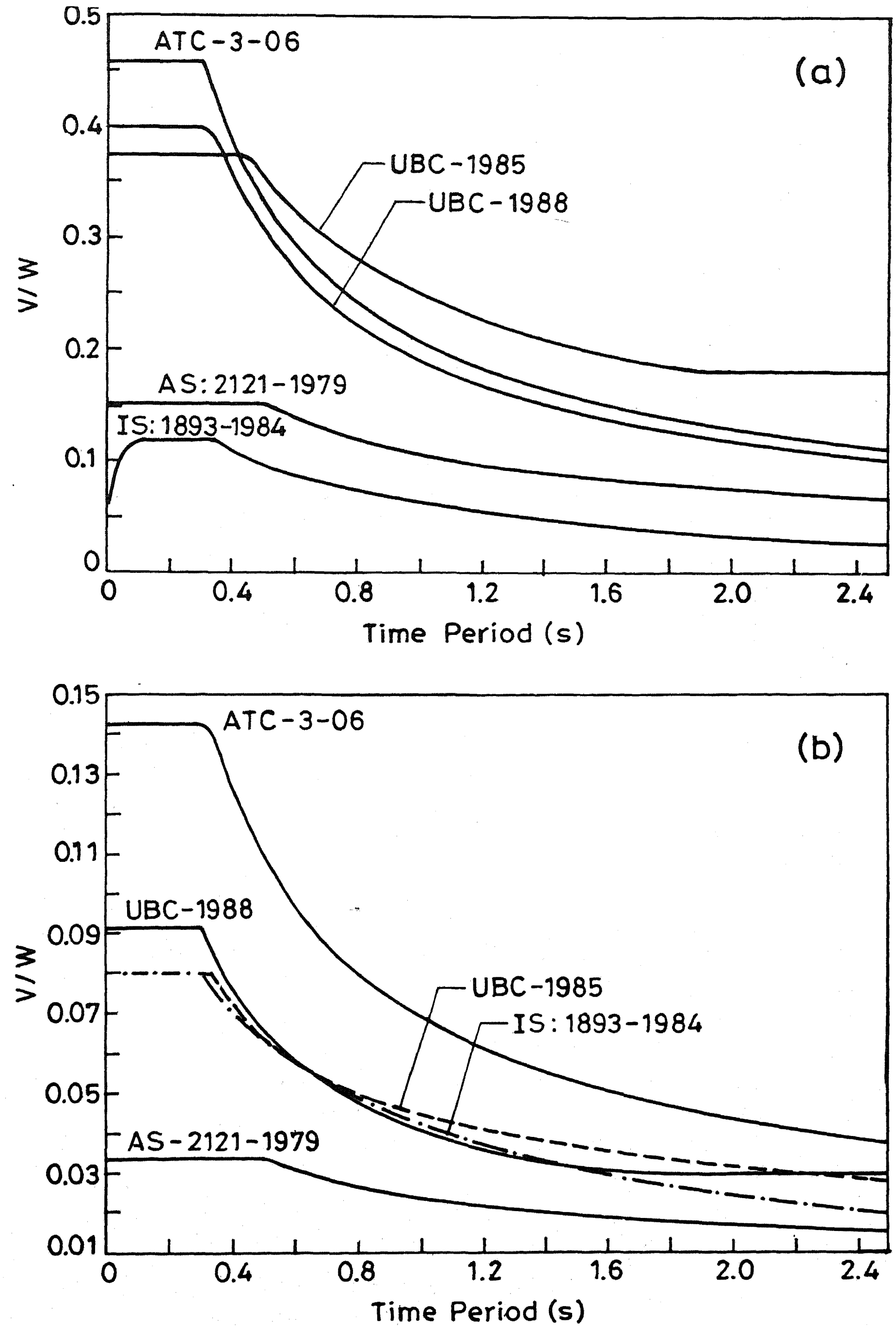

Fig. 4. Lateral force coefficient (a) for elevated water tanks (assuming one-mass system), and (b) for ductile buildings

seismic codes. On the other hand, the lateral design force presribed for buildings in Indian codes is not much different from that in the other codes. This means that the lower level of lateral design force for elevated tanks is not due to low seismic risk, but has resulted from not giving due consideration to the poor performance of elevated tanks as compared to buildings under dynamic loads. Hence it is necessary to introduce a suitable value of performance factor for elevated tanks in our code taking into account the type of supporting structure used. For instance shaft supported towers are now becoming popular due to their better aesthetics. The ductility and performance of these structures require detailed investigation as there are reports of contradicting behaviour of this type of structures in the past earthquakes. During the 1957 Mexico earthquake many inverted pendulum type structure collapsed, whereas during the 1960 Chilean earthquake most shaft supported strucutres performed well in contrast to severe damage to the elevated tanks supported on R.C.C. moment resisting frames (Steinbrugge and Flores, 1963). However, until such issues are resolved the code may prescribe a performance factor of 3.0 for all elevated liquid retaining structures.

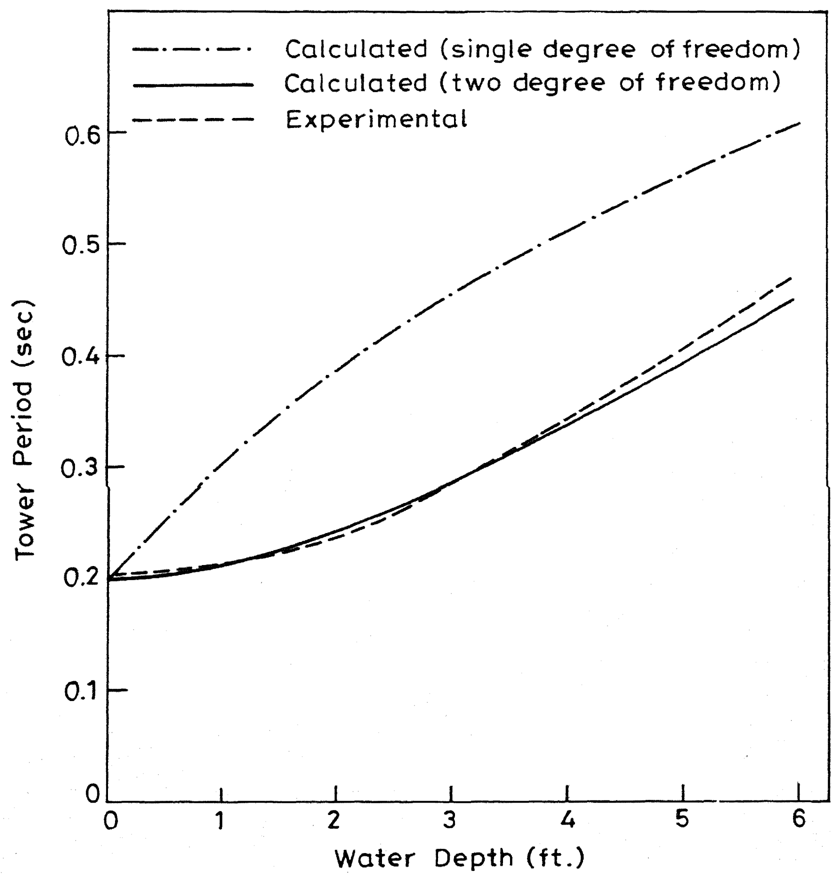

Fig. 5. Tower periods obtained from experimental results, single and two degree of freedom representation (from Boyce, 1973).

3.2 Time Period

The force for which a water tower needs to be designed depends on its time period and hence the computation of time period needs to be reasonably accurate. IS:1893-1984 suggests a single degree of freedom idealization Housner (1963) proposed a two mass idealization for tanks with a free water surface and pointed out that one-mass representation is reasonable only for closed tanks completely full of water. This view point was supported by Cloud (1963) based on measured time period of inverted pendulum type water tanks. Many investigations conducted subsequently have established that the one-mass indealization of the tank is not consistent with the actual behaviour of elevated water towers. Boyce (1973) has reported that the single degree of freedom system idealization used by IS:1893 does not agree with the experimental result obtained from forced vibration tests conducted on a 10 m high steel tower, for varying water depths. He showed that a two degree of freedom system approach based on a mechanical model proposed by Housner (1963) confirm with the experimental results. The plot obtained from his experiment is reproduced in Fig. 5. Shephered (1972) also has reported the same observation based on test conducted on a 13 m high prestressed tubular tower supporting a cylindrical reinforced concrete tank. Sonobe and Nishikawa (1969), Ifrim and Bratu (1969), Gracia (1969), and Haroun and Ellaithy (1985) have supported this view point based on experimental and computational observations.

The mechanical model proposed by Housner (1963) is based on the assumption that the fluid is inviscid, incompressible and that the fluid displacements are small. The pressure generated within the fluid due to the dynamic motion of the tank can be separated into impulsive and convective parts. The impulsive pressures are those associated with forces of inertia produced by impulsive movement of container walls. Convective pressures are those produced by oscillations of the fluid and are thus a consequence of impulsive pressures. The equivalent dynamic system of the liquid storage tank is shown in Fig. 6(a). The tank with oscillating fluid and the equivalent effect on the tank is replaced by two masses m1 and m2 acting at a distance of h_1 (h°_1) and h_2 (h°_2), respectively, from the base of the tank as shown in Fig. 6(a). m_1 is mass that represents the portion of liquid rigidly attached to the tank and simulates the effect of hydrodynamic force on tank walls due to impulsive pressure. The distance at which m_1 is attached, h_1, contributes to the moment of hydrodynamic forces due to impulsive pressures acting on the walls only whereas h1 includes the effect of overturning moment produced due to uneven pressure distribution at the base slab of the tank. m_2 is the mass that simulates the effect of oscillating fluid and is flexibly connected to the tank at a distance of h2 or h°2 from tank base and these distances have same funtions as for h1 and h°1 in the case of mass m1 but for convective pressures (Shephered, 1972). The values of these parameters are reproduced in Appendix A. For tanks having depth larger than 0.75 times the lateral dimension, the portion of fluid below 1.5 R may be considered to be completely constrained to move with the tank and the computation of impulsive and convective pressures need to be carried out only for the portion of fluid contained in the region 1.5 R below the free surface (Fig. 6a).

As per this model an elevated tank can be treated as a two degree of freedom system as shown in Fig. (6 b). The lateral forces obtained for three existing tanks using the one-mass as well as two-mass representation are compared in Table 2. It can be observed that the single degree of freedom in the lateral design force obtained using the two approaches depends on the geometrical properties of the tank and relative stiffness of the staging.

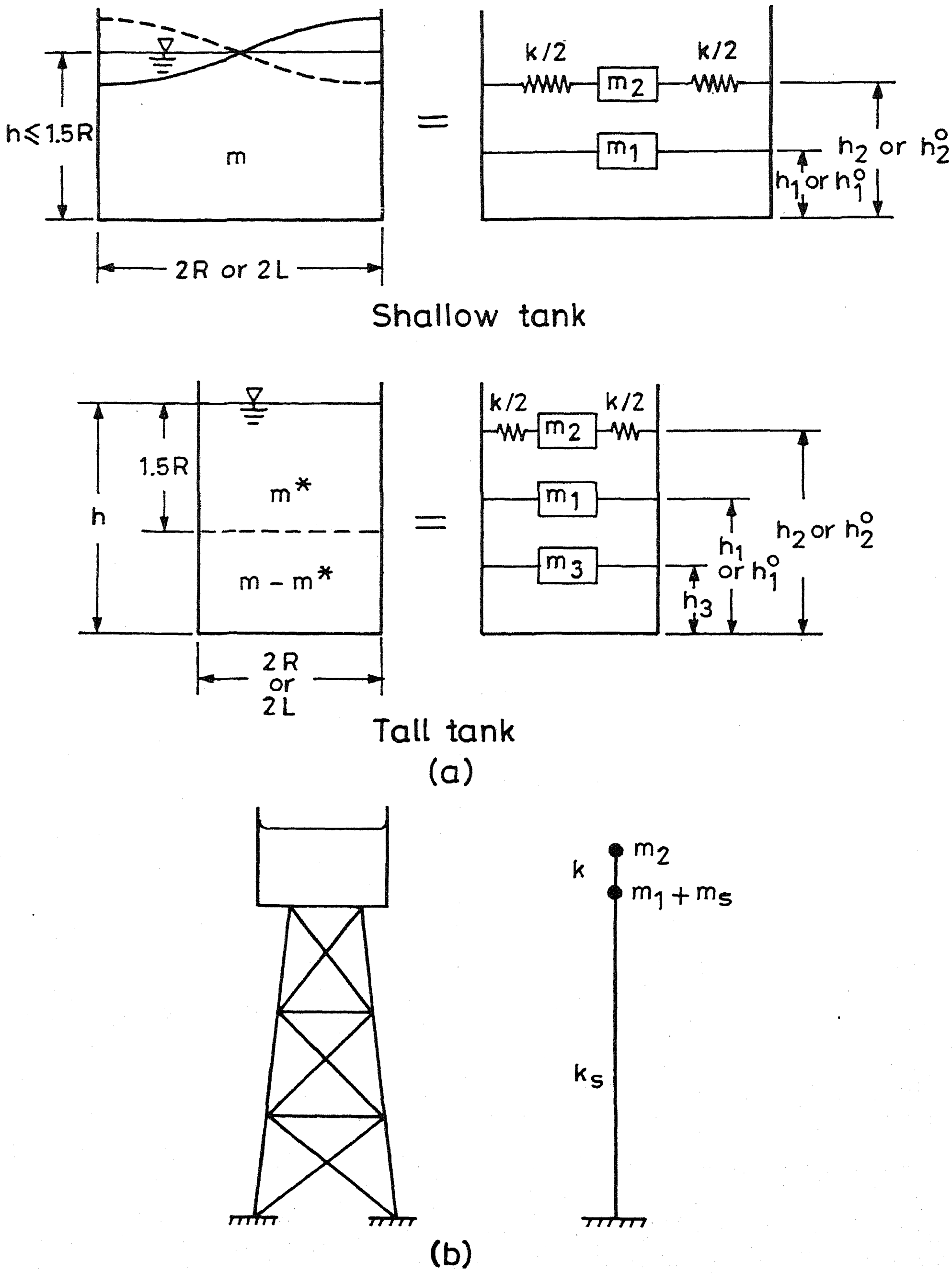

Fig. 6. (a) Mechanical model for hydrodynamic pressures in tanks (after Housner, 1963), (b) Two mass representation for typical elevated water towers (after Shepherd, 1972).

TABLE 2: COMPARISON OF LATERAL FORCE ON STAGING USING SINGLE-MASS AND TWO-MASS REPRESENTATION FOR ELEVATED WATER TANKS

(A) DETAILS OF TANKS

Tank	Capacity (kl)	No. of Columns	Height of Container (m)	Container radius (m)	Staging Stiffness (kN/m)	Mass ($x10^3$KG) * Structure (ms)	Mass ($x10^3$KG) * water
1	455.0	8	4.0	6.00	9100.0	241.7	455.0
2	568.8	10	5.4	5.79	18696.5	348.7	568.8
3	2200.0	30	5.0	11.84	72500.0	1056.8	2200.0

* Mass of structure consists of mass of tank and one-third mass of staging.

(B) PARAMETERS OF MECHANICAL MODEL

Tank	Mass (x 10 Kg) Implusive (ml)	Mass (x 10 Kg) Convective (m2)	Mass (x 10 Kg) Mass rigidly attached (m_i+m_s)	Convective Spring Stiffness (kN/m)
1	173.3	264.2	415.0	669.0
2	291.7	262.9	640.4	768.5
3	536.3	1558.6	1593.3	1546.8

(C) RESULTS OF COMPUTATION

Tank	Time Period Model	Time Period Mode2	Staging Shear (kN) One mass model	Staging Shear (kN) Two mass model	Force on tank wall (kN) Impulsive	Force on tank wall (kN) Convective	Sloshing Height (mm)
1	4.10	1.29	246.1	240.8	168.3	72.3	137.0
2	3.76	1.14	432.0	393.8	334.3	59.5	131.3
3	6.38	0.92	1629.4	1322.0	1113.0	209.0	228.0

In either approach discussed above stiffness of the staging needs to be obtained before calculating the time period. SP:22-1982 assumes horizontal bracings to be infinitely rigid in stiffness calculation and this has led to this approach being widely used by design engineers. It has been shown (Sameer, 1990) that this approach overestimates the stiffness substantially and simple formulae have been proposed to include the effect of bracing flexibility.

Considering the above facts it is recommended that the time period computation proposed by the IS code be rationalised by replacing single degree of freedom idealisation by two-mass representation based on Housner's model. As has been emphasised by Housner and Jannings (1986), a seismic design specification must also specify the procedure to obtain time period. Hence IS:1893 must specify that in time period calculation bracing girders are not to be treated as rigid. Further, the solved example in SP:22-1982 for computing stiffness of the staging needs to consider the girder flexibility to obtain more realistic results. The handbook may adopt one of the simple approximate methods proposed by Sameer (1990) for this. These modifications would give results consistent with the tank behaviour and would partially offset the increase in cost that would be caused due to introduction of an appropriate value of performance factor for water tanks. For instance, Jain and Sameer (1990) show for an example tank that consideration of K = 3.0 and appropriate beam flexibility leads to an increase in design seismic force by only 64 percent over that based on K = 1.0 and rigid beams.

3.3 Hydrodynamic Pressure on Tank Walls

The hydrodynamic pressure on walls and on the base slab of the tank are not axisymmetric. IS:1893-1984 gives formulae for determining this pressure distribution due to impulsive pressure (and not that due to convective pressure) which are based on Housner's work mentioned earlier. However, the code does not give further details of the equivalent dynamic model proposed by Housner in the same publication. As a result the design engineer does not get an idea of the magnitude of the forces and moments acting on the tank wall due to convective and intertial pressures. Further, the clause 5.2.7.1 of the code statues that the convective pressures can be ignored in comparison with the impulsive pressures. This statement is misleading since the convective pressures can be a dominant factor for certain proportions of the tank and the structure (Housner, 1963). Table 2(c) gives relative magnitude of convective and impulsive forces acting on tank walls for three typical tanks. It can be seen that the force on the tank wall due to convective pressures is substantial for tank 1. Hence it is recommended that the code provide complete details of Housner's model and omit the clause 5.2.7.1.

The stresses in the tank wall due to asymmetric hydrodynamic pressure distribution may be considerably different from those due to hydrostatic pressure. The bending moments are developed in the horizontal plane adding to the tensile stress due to hoop force. The overturning moment on the tank may cause vertical tensile stresses in the cross section of the wall. For Intze type of tanks the stresses in the bottom dome will be significantly altered due to uneven hydrodynamic pressure distribution. However determination of these internal forces requires rigorous mathematical analysis or use of finite element procedure. This has led to a practice among the design engineers to design only the supporting tower for seismic effects and to ignore the effect of hydrodynamic pressure on the walls in design of the container. Chandrasekaran and Krishna (1965) have indicated that this is justified since for the tank wall design, increase in permissible stress of 33.3 percent used for other structural components, is not admissible and hence this reserve strength would take care of additional hydrodynamic pressure (Ramaiah and Gupta, 1966). This view is true from the strength point of view; however tanks become non-functional if cracks develop. Though most damages reported for elevated towers during the past earthquakes occurred for supporting structure, Hill and Biggs (1974) point out that some were due to structural inadequacies of the storage tank itself (Lee and Reddy, 1979). Moreover horizontal cracks along the circumference were observed on the wall of the elevated tank damaged in Bihar-Nepal earthquake of August 1988 and described earlier. Though methods have been developed for analysis of the container due to hydrodynamic pressure distribution, these are not so useful in the design office due to their complexity (Lee and Reddy, 1979). Hence it is recommended that tables similar to those given in IS:3370 (Part 4)-1967 be prepared for computing internal forces in the tank wall and the base slab due to hydrodynamic pressures and published in the form of a design aid. Solved examples may also be provided explaining the design procedure. As Intze type tanks are very popular in our country it is proposed that the tables cover design and detailing aspects of this type of tanks also.

Further, the code does not make any distinction between the hydrodynamic pressure developed during earthquakes in rigid and deformable tank walls. Veletsos (1973) and Haroun and Housner (1981) have indicated that the seismic effects on flexible tanks are substantially greater than those induced in similarly excited rigid tanks. These investigators have developed simple procedures for evaluating hydrodynamic force developed in flexible tanks. Hence it is necessary to caution the designer about larger forces induced in flexible tanks and to provide procedure for their analysis in the IS code or in the Explanatory Handbook. This may be based on one of the above approximate procedures.

3.4 Ductility Requirements

As its title itself suggests, IS:4326-1976 gives design requirements for buildings only. As per clause 1.2 of the code "the provisions of this standard are applicable for building construction in seismic zones III to V". Thus may design engineers tend to think that ductile detailing is not required for R.C.C. elevated tanks even in zones IV and V. The design philosophy adopted by most seismic codes is to depend on inelastic deformation and consequent energy absorption for avoiding collapse under severe earthquakes and to carry out elastic design for resisting earthquakes of moderate intensity. This approach is necessary as a structure having resistance to most severe earthquakes which may visit only once in its life time may be too expensive to build. This approach is for all types of structures and not confined only to buildings. Hence it is recommended that the scope of IS:4326-1976 be enlarged to also include structures other than buildings.

IS:4326-1976 gives ductile detailing criteria for only framed structures. As mentioned earlier shaft supported tanks are now very common in our country and hence the ductile detailing criteria for such structures need to be included in the code.

A figure in IS:11682-1985 states that "where design seismic coefficient is 0.05 or more reference to clause 7.2 to 7.4 of IS:4326-1976 shall be made to cater for ductility requirement". This is in right spirit. However, the term "design seismic coefficient" in its present form is very confusing. IS:1893 defines the horizontal seismic coefficient, h, in two ways which mean two different things. For "Seismic Coefficient Method" it is equal to which is not dependent on the flexibility of the structure (clause 3.4.2.3 a). On the other hand, for "Response Spectrum Method" it is defined as IF S/g which depends on the time period of the structure (clause 3.4.2.3 b). Clause 2.7 of IS:432-1976 takes the first definition (i.e. clauses 3.4.2.3 (a) of IS:1893). Clause 7.1.5 of IS:4326-1976 indicates that the ductility detailing requirements are to be adopted when this design seismic coefficient is 0.05 or more. To confuse matters further, IS:1893-1984 prescribes the "Response Spectrum Method" for elevated tanks and hence uses the second definition of h. This entire thing is quite confusing to the design engineers and the future editions of these codes must be very clear and specific on these points.

3.5 Miscellaneous Aspects

Steinbrugge and Moran (1954) have reported that the modes of failure of some of the elevated tanks during the 1952 Kern Country, California, earthquake indicated occurrence of torsional motion. Shephered (1973) has also pointed out that it is impossible to prevent torsional response in elevated water towers. SEAOC-1980 makes it mandatory to design elevated water towers for shear stress developed due to horizontal torsion resulting from an accidental eccentricity equal to 5 percent of the largest lateral dimension. The elevated towers constructed are seldom truly symmetric due to presence of staircase and other structural components. Further, with failure of one or two bracings, the structure will go into the torsion mode. Hence it is desirable for the code to require design for accidental torsion.

The code does not provide expression for determination of sloshing height of water. In large capacity tanks, having small height to radius ratio, the rise of free surface of water may be significant to cause pressure on the roof which is not designed to resist this pressure. Table 2(c) gives sloshing height for three typical tanks. It may be noted that for Tank 3 with smaller height to radius ration rise of free surface is quire significant. Hence it is recommended that the code may provide Housner's expression for calculating sloshing height.

The code does not provide design guidelines for ground supported tanks. These tanks face multiplicity of problems during earthquake such as uplifting, buckling of the tank wall, failure due to pressure on tank base slab due to vertical acceleration, etc. It is therefore desirable to have a separate section in the code giving guidelines for this type of tank.

4. CONCLUSIONS

The provisions of Indian seismic codes pertaining to evevated water tanks have been reviewed based on the reports of many recent investigations. Comparison has been made with the reviewed based on the reports of many recent investigations. Comparison has been made with the requirements in seismic codes of other countries. Proposals that may be adopted in the future revision of the code have been presented. Importance of these structures as an integral part of the essential facility has been given due consideration in arriving at the proposals. Major recommendations are listed below.

(i) An appropriate value of performance factor, say 3.0, may be introduced in the expression for determination of design lateral force for elevated tanks. Absence of performance factor for elevated towers has resulted in the lateral design force prescribed by IS: 1893-1984 being substantially low.

(ii) One mass model prescribed by the IS code may be replaced by the two-mass representation.

(iii) The code must specify that the flexibility of bracing girders be accounted for in computation of staging stiffness. Similarly, the solved example in SP: 22-1982 for computation of time period of stagings may be revised to include the effect of girder flexibility.

(iv) The expressions for hydrodynamic pressure distribution in the tank may be supplemented by details of Housner's mechanical model.

(v) The clause 5.2.7.1 of IS: 1893-1984 stating that convective pressures are not important be deleted from the code as it is not applicable to all tanks.

(vi) Table similar to those given in IS 3370 (Part IV)-1967 may be prepared to calculate the stresses in the tank walls and the bottom slab due to hydrodynamic pressure and published in the form of a design aid. A solved example may be provided in SP:22 to explain the design procedure.

(vii) A clause may be introduced to distinguish hydrodynamic pressure in flexible tanks from the rigid ones as forces in the former may be considerably higher than those in the similarly excited rigid tanks.

(viii) The scope of IS:4326-1976 may be expanded to cover structures other than buildings also. The ductile detailing requirements for tank stagings must be made very explicit and specific. Detailing criteria for shafts supporting the elevated tanks may be included in the code.

(ix) A clause must be introduced in IS:1893 as well as in IS:11682 to make it mandatory to design the tanks for accidential torsion.

(x) An expression for calculating sloshing height of water may be introduced in the code.

(xi) A separate section giving guidelines for designing ground supported tanks may be introduced in the code.

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