Magic R: Seismic Design of Water Tanks

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ABSTRACT

Seismic design of water tanks relies upon a number of rules issued by various code-setting groups. The AWWA code [1] includes a factor "R" that is used to establish forces for the seismic design of water tanks (circular welded steel, circular bolted steel, circular prestressed concrete, rectangular reinforced concrete, circular wood, and open cut lined with roof systems). The "R" factor is sometimes called a "ductility factor" or "response modification factor", and is often in the range of 3.5 to 4.5. Essentially, the R factor is used to adjust the elastically-computed seismic forces, $V = \frac{ZIC}{R} W$, where V = seismic base shear, Z = local site specific peak ground acceleration, I = importance factor, C = normalized response spectra ordinate, W = weight, with adjustment to suitably combine the effects of the structure, water impulsive and water convective (sloshing) components of the total load.

This paper examines the technical basis of "R". Is it from test? empirical data? experience? a desire to keep the cost of construction low? The evidence in this paper shows that the "R" factors in the code are based on "magic", that is to say, without factual evidence. When the empirical evidence is examined for more than 500 tanks and reservoirs, we find that the use of R has led to poor performance of water tanks under moderate to strong ground motions, often leading to loss of water contents.

This paper provides recommendations as to how to adjust code R values, as well as refinements in detailing for side entry pipes, bottom entry pipes, and the roof. These recommendations are made in reflection of the observed empirical evidence of actual damage of tanks in past earthquakes, tempered with findings from shake table test data. By adopting these refinements, it is hoped to achieve cost effective seismic design of water tanks that also provides high confidence of suitably reliable performance in large earthquakes.

SEISMIC DESIGN CODES FOR BUILDINGS

Ductility plays an important role in the response of structures due to earthquake motions. Prior to the mid-1980s, the common code approach to seismic design for regular buildings (not tanks) in high seismic areas of California) was as follows:

1933 to 1943 (Los Angeles)

V = 0.02W to V = 0.10W, with the base shear coefficient (0.02 to 0.10) chosen depending on the type of building.

1943 to 1957 (Los Angeles)

Taller buildings, being more flexible, were allowed to be designed with lower base shears.

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$$V = \frac{0.6}{N + 4.5} W,$$

where N = number of floors.

Sample: N = 1, then V = 0.133W, or if N = 5, then V = 0.063W

1956 to 1974 (San Francisco)

$$V = \frac{K}{T} W,$$

where K = 0.035 for non-building structures and T = period of the structure in seconds, and K/T (max) = 0.10.

1975 to 2009 (Modern Era)

Since about 1975, almost all building codes in the USA have been reformulated to compute required seismic base shear as follows:

$$V = \frac{PGA * I * C}{R} W$$

where PGA = design level horizontal peak ground acceleration, set at the 475 year motion, or 2/3 of 2,475 year motion, I = importance factor (I= 1 for regular buildings, 1.25 for important buildings or 1.5 for critical buildings), C = response spectral coefficient for 5% damped spectra (usually about 2.75 for structures at the peak of the spectra), and W = dead weight of the building, sometimes including a percentage of live load). In this formulation, R includes the effects of hysteretic energy from yielding, increased damping over 5%, and all other factors of safety embedded into the code design approach. For working stress design approaches, R is replaced with Rw; for ultimate strength design approaches, R is often set at R = Rw / 1.4, just enough to offset the load factors used in the design approach. Depending on which code is considered, Rw values have ranged from 1.5 (for unreinforced masonry construction, where allowed) to as high as 12 (for presumed ductile steel moment frame buildings). The 1997 UBC provided R = 8.5 (same as Rw = 12) for special moment frame steel buildings.

SEISMIC DESIGN FOR WATER TANKS

Two of the earliest "codes" or manuals or practice for fluid-filled containers are work by Housner [2, 1954] and TID 7024 [3, 1963]. These approaches assumed that R = 1, and assumed the tank is rigid (responds at PGA) for the impulsive mode. The net result was that V = 0.25*W for small tanks (for radius of tank = 13 feet, height of water = 15 feet). The convective mode was calculated elastically (R=1) and combined with the impulsive mode by absolute sum. The long period of the convective mode (commonly T = 3 to 8 seconds) as compared to the high frequency of the impulsive mode (f = 3 to 8 hertz) strongly suggested that the maximum impulsive forces could occur at (or nearly at) the time of maximum convective forces, and hence an absolute sum of the two terms seemed reasonable. TID 7024 required that a ring girder be placed at the top level of the tank shell "to provide stability against excessive distortion due to the lateral forces generated by the accelerated fluid". TID 7024 specifically allowed that sloshing forces need not be accommodated in the design if damage to the rood was considered acceptable. TID 7024 recognized that uplift of tanks shells promoted higher stressed in compression, leading to increased chance of damage due to wall buckling (elephant foot).

By about 1970, it was recognized that the impulsive mode of most common-sized water tanks was in the range of f = 3 to 8 hertz, and so the seismic base shear from the impulsive mode should be computed using the amplified spectral coordinate. For design of water tanks outside of the nuclear industry, it was also decided (largely for convenience) to base the design spectra using horizontal 5%-damped spectra, as that was the default set in regular building codes. In the nuclear industry, it was commonly set that the impulsive mode for steel tanks had 2% damping, and the convective (sloshing) mode had 0.5% damping.

By the mid-1990s, various AWWA code committees diverged on R values. The D100 code (for steel tanks) allowed that the base shear and slosh height in the convective mode could be computed buy dividing by "R"; whereas the D110 code (for concrete tanks) the R value for the convective mode is 1. In some codes, the impulsive mode and convective mode base shears could be combined by square-root of the sum or the squares (although there is little technical basis to support this). Some practitioners further divided the slosh height by R, a practice that could be interpreted as acceptable by code, but that has no technical basis (in other words, the wave heights are not affected in any appreciable manner by any local yielding in the steel shell).

In 1978, a non-mandatory seismic design code was issued for water storage tanks. By non-mandatory, the code was optional for seismic zones 1, 2 and 3, but require in seismic zone 4. By "seismic zones", zone 4 was limited to areas of the USA with PGA = 0.4g (or higher); zone 3 was for areas with PGA = 0.3g, zone 2 with PGA = 0.15g, zone 1 with PGA = 0.075g, and zone 0 was for non-seismic areas.

$$V = ZK\left(0.14\left(W_{Shell} + W_{Roof} + W_{Water-Impulsive}\right) + C_1 S W_{Water-Sloshing}\right)$$

with

Z = 1 (zone 4), 0.75 (zone 3), 0.375 (zone 2), 0.1875 (zone 1)

K = 2.00 (anchored flat bottom tank) or 2.50 (unanchored flat bottom tank)

S = 1.0 (rock site), 1.2 (stiff soil site), 1.5 (soft sol site), and CS $\leq$0.14

For an anchored tank on rock (D = 140 feet, H = 40 feet) with T (impulsive) = 0.2 seconds and T (sloshing) = 7.7 seconds and located in zone 4 on a rock site, then

V = (1.0)(2.0)(0.14W(steel + water impulsive) + 0.013W(sloshing))

For a moderately large 4.6 MG tank with D = 140 feet and H = 40 feet, built with mild steel (Fy = 30 ksi) with average wall t = 0.45 inches, average roof t = 0.1875 inches, then the weight of the steel is 441,000 pounds, the weight of water (when full) is 38,423,000 pounds. The weight of the contents (water) is 87 times more than the weight of the steel in this tank. For this tank, the weight of water in the sloshing (convective) mode is about 23,438,000 pounds, and the weight of water in the impulsive mode is about 12,700,000 pounds. Thus, for this tank, the total base shear is V = 3,679,000 pounds (impulsive) + 610,000 pounds (sloshing) = 4,289,000 pounds (total), or V = 0.110W. If the tank where unanchored, V = 0.138W.

In contrast, if one were to assume that the tank were to respond elastically, for a horizontal PGA = 0.40g, and assuming about 2% damping in the impulsive mode, then the elastically computed base shear would be about SA(2%, 0.2 seconds) = 1.20g, SA (2%, 7.9 seconds) = 0.08g, then V = 1.2(441,000) + 1.2(12,700,000) + 0.08(23,438,000) = 529,000 + 15,240,000 + 1,875,000 = 17,644,000 pounds, or V = 0.454W.

Examining these results, the 1978 AWWA code infers R = 0.454 / 0.110 = 4.13 (anchored) or R = 0.454 / 0.138 = 3.29 (unanchored).

This simple example ignores variations such as how the impulsive and sloshing modes should be combined (absolute sum or SRSS), higher mode effects, the code damping (commonly 5%) and the observed damping (commonly 2% for the impulsive mode and 0.5% for the convective mode). While all these variations are important, their cumulative effect is secondary as compared to the magic R effect is deciding how much base shear (and corresponding overturning moment) for which to design.

The inferred R (in the 1978 code) varies whether or not the tank is anchored or unanchored. This is a direct result of the 2.5 (unanchored) and 2.0 (anchored) multipliers that the 1978 code authors used, which was geared to *penalize* unanchored tanks (i.e., require a higher base shear force for design). Once we convert the 1978 base shear formula to the mode modern V = (ZIC/R)W formulation, we end up having to assign the energy dissipation in an anchored tank to the bolts, and energy dissipation in the unanchored tank to the uplifted sketch plate, and then observe that the common detailing of anchor bolts is non-ductile (failure in the threads), and the common detailing of sketch plates welds have a large stress riser (at the fillet welds). This is nonsense, as any beneficial yielding of the anchor bolts or sketch plates results in a trivial amount of energy absorption as compared to the mass of the water versus the available hysteretic energy absorption

The AWWA code also incorporates other serious flaws.

Once the seismic overturning moment is calculated, the code then requires that the vertical stress in the shell be less than the buckling stress (this is a good provision), as calculated using the traditional σ = M/S. For a shell annulus with D(inside) = 140 feet and t = 0.60 inches,

$S = \dfrac{\pi\left(d_{outside}^4 - d_{inside}^4\right)}{32 d_{outside}}$, and substituting d(outside) = 140x12+2*0.60 and d(inside)=140*12, we get S = 1,330,499 inches^3.

This infers that the shell of the tank behaves as a long beam, with "plane sections remaining plane". Ignoring the weight of the steel shell, the code formula for vertical stress is:

$\sigma_c = \left(\dfrac{1.273M}{D^2}\right)\dfrac{1}{12t}$, where M is in pound-feet, D in inches and t in inches.

Assuming D = 140 feet, t = 0.60 inches, and making the conversions from feet to inches, then we get the same result as above, or:

$$\sigma_c = \left(\frac{1.273 * M * 12}{(140*12)^2}\right)\frac{1}{12*0.60} = \frac{M}{1,330,275}\, psi.$$

Since the selection of the bottom course shell thickness is such a critical factor in preventing buckling, we must ask: do plane-sections-remain-plane in an at grade tank? Shake table test data performed by Akira Niwa [5] shows the answer is clearly NO for unanchored tanks, and perhaps not such a bad analogy for anchored tanks (see Section 4 for details). However, the AWWA code makes no provision for calculating the true state of stress in the tank shell due to overturning moment, a severe deficiency that perhaps is compounded by the rather arbitrary selection of R.

Another twist in the AWWA code is how the code treats the allowable stress in compression against buckling. In the 1978 code, the allowable stress in compression in the bottom course was set at 1.333 times the allowable compressive stress under dead weight, plus a factor that reflected that the hoop tensile stress due to internal water pressure has been shown to resist the tendency to buckle the shell due to vertical compression (only half this effect is allowed): $\sigma_{eq} = 1.333\left(\sigma_{allow} + \frac{\Delta\sigma_{cr}}{2}\right)$, and $\Delta\sigma_{cr} = \frac{\Delta C_c E t}{R}$. Say for our example tank, water pressure at the bottom of the tank is 40 feet * 62.4 pcf / 144 = 17.33 psi. Say E = 29,600,000 psi. R = 70 feet (radius), t = 0.60 inches, then the ΔC_c = 0.21 (based on code nomograph), and $\Delta\sigma_{cr}$ = (0.21)(29,600,000)(0.60)/(70*12) = 4,440 psi, or the total allowable compressive stress is increased by 2,220 * 1.333 = 2,959 psi, over and above the stress to safely prevent buckling (=1.333*1,395 = 1,860 psi), due to vertical stress alone (limited to yield), or a total of 4,819 psi. In the 1978 code, a warning is provided that there is controversy over this factor, stemming from the idea that the simultaneous effects of vertical earthquake could be decreasing (or increasing) the beneficial hoop tensile stress at the same time as the maximum vertical stress from overturning moment is applied.

In the 1996 and 2005 AWWA codes, this factor is further confused by the requirement that the $\Delta\sigma_{cr}$ can only be credited for unanchored tanks, but not anchored tanks. The net effect is that for the AWWA 1996 and 2005 codes, unanchored tanks are allowed to have thinner bottom course shells than for anchored tanks. The empirical evidence in Section 5 shows this to be a dubious practice. In contrast, the US NRC never allows credit for $\Delta\sigma_{cr}$, whether anchored or unanchored, as a safety precaution for commercial nuclear power plants.

The API [4] also provides for seismic design of storage tanks. The API 650 standard of 1990 is essentially identical to the AWWA code of 1978, except that an importance factor, I, is introduced. I is set to 1.0 for regular tanks, and up to 1.5 for important tanks that must provide emergency service to the public; and (K)(0.14) is replaced with 0.24 (about 15% lower than AWWA) and the API long period sloshing spectra constant is similarly about 15% lower than the AWWA value. In other words, API would allow about a 15% lower seismic load when I = 1.0; but when the engineer selects I = 1.25 (or 1.50), the API code would ultimately require a higher base shear. The API code allows for an increase in allowable shell compressive stress to account for hoop tension, but limits the total allowable vertical stress to no more than 50% Fy. For a tank with D = 140 feet and t = 0.60 inches, the allowable seismic vertical compressive stress would be 4,286 psi. In other words, the API code provides a somewhat larger factor of safety on buckling than the comparable AWWA D100 code. Of course, the true stress needed to initiate a buckle in the shell is generally much higher than either the AWWA- or API-computed values, and the ratio of the true stress (computed from elastic-plastic theory) versus the allowables provided by AWWA code is a large reason that tanks do not fail under earthquakes any more often than they already do.

TEST EVIDENCE

In 1976, Niwa [5] performed shake table tests of water-filled tanks at Berkeley. Niwa clearly showed that there is dynamic amplification of the impulsive mode: i.e., it responds according to amplified spectral acceleration, and not the PGA. In part based on these tests, the codes and practices used up to that time (Housner 1957, TID 7024 1963) had to be changed.

Niwa showed that for anchored tanks, the elastic shell stresses could be reasonably estimated using a cantilever beam model with a combination of amplified impulsive and convective modes. Niwa also showed that for unanchored tanks subject to uplift, that the rocking response is highly nonlinear, and no appropriate single-degree-of-freedom oscillator model can be used to accurately predict the response.

Niwa showed that the Housner slosh-height analog model (H = 0.42 * Sac * D, where H = slosh height, Sac = 5% damped spectra at the sloshing period, D = tank diameter) under predicts actual unrestricted slosh heights by 15 to 32 percent or so; in part, this may be due to neglecting higher mode effects of waves. The Niwa tests do not justify applying a "R" factor to reduce slosh heights or convective-induced shell stresses.

Niwa showed that the code-computed compressive stresses due to overturning moments on an anchored tank were under-or over-predicted by -18%, -7% or +64% for three different seismic input motions adjusted to achieve PGA = 0.5g input. This finding partially justifies use of an R factor of perhaps 1.13 (on average) * other factors of safety.

In Niwa's tests, the D100 allowable for buckling stress (excluding hoop effects) was 1,560 psi. Actual measured compressive stresses from several tests were as high as 3,698 psi, yet no buckling was observed. This shows at least a factor of safety of 2 on buckling if hoop pressure effects are excluded.

Key conclusions form the Niwa tests are as follows:

- Computation of overturning moments and resulting compressive stresses, using AWWA D100 simple beam analogies, is reasonably correct for anchored tanks, but entirely speculative for unanchored tanks.
- Actual shell buckling occurs at substantially higher stresses than the allowables in the D100 code.

Extrapolating the Niwa findings towards R, we observe the following:

- Unanchored tanks respond in a highly nonlinear way once they begin to uplift.
- Computation of vertical membrane stresses in unanchored tanks (subject to uplift) using simplified code formula is highly speculative. The computed stresses may be off by several hundred percent. A three-dimensional model that captures both uplift and hoop breathing modes can better predict these stresses. Computation of

vertical membrane stresses in anchored tanks is reasonably predicted using modern (post 1975) codes that include amplification of the impulsive mode.

- For anchored tanks, R = 2 (or so) would be justified to avoid initiation of tank wall buckling, using AWWA D100-2005. This R value is better described as a Factor of Safety against shell buckling, than as an energy absorption factor. Proper detailing of side entry (accommodate at least 4 inches of uplift) and bottom entry pipes (at least 2 feet from shell) is required to accommodate uplift, as uplift may still occur using the factored R loads.
- For unanchored tanks, the actual shell buckling stresses will vary based on rigid (concrete ring beam) or flexible (tank directly on grade) conditions. Higher membrane stresses are possible for tanks on concrete ring beams; lower on flexible foundations. Actual membrane stresses cannot be accurately predicted using code formulations. If an R value is to be rationally set, then it should vary based on whether or not the tank shell sits on a concrete ring beam or on a flexible asphalt-on-soil condition.

EMPIRICAL EVIDENCE

We collected actual performance data for 542 at-grade welded steel tanks for 20 earthquakes from 1933 (Long Beach) through 2003 (Paso Robles). We developed statistics for damage / no damage, sorted by anchored and unanchored steel tanks, and degree of fill at the time of the earthquake. The complete database for 532 tanks with PGA ≥ 0.10g and reduction of the data to fragilities is provided in [6]. In this report, we describe the performance of 10 steel tanks from the 2003 Paso Robles earthquake.
The damage states are as follows:

- DS 1. No damage.
- DS 2. Damage to a pipe creates only slight leaks or minor repairs (such as damage to an overflow pipe). The tank roof might be damaged. The tank remains in service after the earthquake, with relatively minor cost repairs. Leaks do not represent a credible life-safety threat due to erosion or inundation.
- DS 3. Tank wall buckling has occurred, but without leak of tank contents. The tank remains in service immediately after the earthquake. Relatively expensive repairs (or tank replacement) are performed some time after the earthquake.
- DS 4. Tank wall buckling has occurred, or side / bottom entry pipes have broken, with loss of tank contents. The tank is out of service immediately after the earthquake. Tank replacement or expensive repairs are needed to restore the tank to service. The leaking contents could present erosion or inundation risks under certain (mostly infrequent) circumstances.
- DS 5. Tank has structurally collapsed and lost all its contents. The leaking contents present erosion or inundation risks under certain (mostly infrequent) circumstances.

Table 1 provides the breakdown of the number of tanks with various damage states. (Note: one tank was in DS 5 collapsed due to collapse of an adjacent tank – this tank was removed from the database used for developing fragilities).

Table 1. Tank Database

PGA (g)	All Tanks	DS = 1	DS = 2	DS = 3	DS = 4	DS = 5
0.10	8	4	4	0	0	0
0.16	263	196	42	13	8	4
0.26	65	32	18	11	4	0
0.36	56	22	19	8	6	1
0.47	47	32	11	3	1	0
0.56	53	26	15	7	3	2
0.67	25	9	5	5	3	3
0.87	14	10	0	1	3	0
1.18	10	1	3	0	0	6
Total (542)	532	331	112	47	25	16

Effect of Fill Level

Table 2 presents fragility curves that were calculated for a variety of fill levels in the tank database.

Table 2. Fragility Curves, Tanks, As a Function of Fill Level

DS	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.38	0.80	0.56	0.80	0.18	0.80	0.22	0.80	0.13	0.07
DS≥3	0.86	0.80	>2.00	0.40	0.73	0.80	0.70	0.80	0.67	0.80
DS≥4	1.18	0.61			1.14	0.80	1.09	0.80	1.01	0.80
DS=5	1.16	0.07			1.16	0.40	1.16	0.41	1.15	0.10
	All Tanks N=531		Fill < 50% N=95		Fill ≥ 50% N=251		Fill ≥ 60% N=209		Fill ≥ 90% N=120	

In Table 2, "A" represents the median PGA value (in g) value to reach or exceed a particular damage state, and Beta is the lognormal standard deviation. N is the number of tanks in the particular analysis.

Effect of Anchorage

Table 3 shows that anchored tanks have performed much better than unanchored tanks.

Table 3. Fragility Curves, Tanks, As a Function of Fill Level and Anchorage (through 1994)

DS	A, g	Beta	A, g	Beta	A, g	Beta
DS≥2	0.18	0.80	0.71	0.80	0.15	0.12
DS≥3	0.73	0.80	2.36	0.80	0.62	0.80
DS≥4	1.14	0.80	3.72	0.80	1.06	0.80
DS=5	1.16	0.80	4.26	0.80	1.13	0.10
	Fill ≥ 50% All N=251		Fill ≥ 50% Anchored N=46		Fill ≥ 50% Unanchored N=205	

The performance of 10 at-grade steel tanks in the 2003 Paso Robles earthquake was as follows:

- Morro Beach. 4 tanks were unanchored bolted steel tanks, all located at one site that had PGA ~0.10g. All four of these tanks experienced slight yielding of the holes around the bolt holes at the base of the tank, resulting in very slight leaks (drips).
- Paso Robles. 1 tank was welded steel, ~0.1 MG (~25 feet diameter), unanchored without a concrete anchor ring. This tank experience PGA ~0.35 to 0.40g. The lowest course of the shell buckled severely, but did not leak; the buckle extended 360° around the tank, with permanent bulge of ~3 inches outwards on one side and ~1 inch outwards on the opposite side. During the earthquake, the buckle was wider and deeper, but rebounded somewhat after the end of shaking. Four of four side entry pipes broke (none had flexible connectors). The tank owner repaired the tank by fixing all the side entry pipes, but left the buckled lower course in place.
- Paso Robles. 2 tanks were welded steel, unanchored on concrete ring beams, each 4 MG (132 feet diameter, 41.6 feet high). The newer tank was designed per AWWA D100 in 2001. The older tank was built circa 1970, with uncertain design basis. Site motion was about PGA = 0.30g to 0.35g. Both tanks had uplift, likely of comparable amounts (2 to 4 inches) based on observations of damaged pipes. The older uplifted tank yielded its bottom course and bottom plate (as evidenced by flaked-off paint). Two side entry pipes in the newer tank broke (one had a Dresser Coupling hat exceeded its limit and pulled its gaskets; the other broke underground at a megalug joint, as the uplifted tank put excessive tension forces on the megalug causing it to slip. On the older tank, a similar megalug connection did not break The interior bottom-entry pipe for the older tank tore where connected to the bottom plate (it was only about 18 inches from the exterior wall that uplifted), and the tank leaked all its contents. Many of the roof-level steel channels holding up the steel roof on the older tank twisted when the tank uplifted; the channels disengaged from the roof; comparable roof-level damage was not observed in the newer tank.
- Templeton. 3 steel tanks were located at the same site, likely exposed to PGA between 0.25g to 0.30g. One was at-grade bolted steel, unanchored. One was at-grade welded steel, unanchored. One was at-grade welded steel, anchored and with all side entry pipes having flexible connectors. All three tanks were 0.1MG to 1.0 MG. The at-grade bolted steel tank had elephant foot buckling with minor leaks. The at-grade unanchored welded tank uplifted several inches and damaged a bottom level drain line, and had severely damaged exterior attached electrical cables. The anchored steel tank had no observable damage.

As seen in Table 3, the empirical evidence for the benefits of anchored tanks is clear. The median PGA value to reach various damage states is about 3 to 4 times higher for anchored tanks as for unanchored tanks. It should be noted, however, that the anchored tank database (N=46) is much smaller than the unanchored tank database (N=251), and fill levels may not have been known for all tanks in the anchored tank database. The empirical evidence strongly suggests that anchored tanks outperform unanchored tanks.

The empirical data suggests that the 1996 AWWA D100 code change to penalize anchored tanks (still reflected in the 2005 code) is probably unwarranted.

CONCLUSIONS

The current AWWA D100 (and similar) codes employ seismic demand formulae which incorporate "R" response modification factors. These R factors appear to be historically based on similar factors for ductile building structures, and are NOT based on test data or empirical data.

The test and empirical evidence shows that the R values in the modern AWWA codes are not justified. The AWWA code does not disallow non-ductile detailing for attached pipes, and arguably allows bottom entry pipes to be located too close to side walls, especially for unanchored tanks.

The evidence suggests that the main reason that most steel tanks have not failed in past earthquakes is simply that the level of shaking was too low to result in wall uplift, coupled with the fact that the true buckling stress of the steel shell is substantially higher than code allowables. The good performance of many tanks cannot be attributed to ductile yielding of the steel shell.

The recommended changes to the AWWA (and similar) codes are as follows:

- If a working stress approach is retained, then limit R to be no higher than the true buckling stress divided by the code allowable buckling stress, limited to R = 2.5 (working stress approach). For commercial nuclear power plants, R = 1.
- Unanchored steel tanks may be used when the vertical wall compressive stress due to seismic overturning moment is computed considering cantilever, lift off and hoop breathing modes, and in consideration of whether or not a concrete ring beam or compliant foundation is used under the steel shell.
- Only anchored tanks should be used for important water storage tanks (I > 1).
- The use of Importance factors of I greater than 1 should be used to increase reliability for important tanks. Wherever post-earthquake performance is deemed important, the target PGA value should be the 475-year return period but no less than PGA = 0.20g. For pressure zones with two or more seismically-designed tanks (or one tank and one reliable pumped source), use I = 1.0 (1.25 in high fire threat areas). For pressure zones with only one tank, use I = 1.25 (1.5 in high fire threat areas).
- All tanks (whether anchored or not anchored) should be designed to accommodate wall uplift whenever the design is based on R > 1. The target uplift amounts should be at least 4 inches (anchored tanks) or 12 inches (unanchored tanks) for design of flexible connections of attached side-entry pipes. Any exterior attachments (power cables, etc.) should be similarly designed.
- No bottom entry pipes into the floor of a tank should be allowed within 24 inches (clear distance) of an anchored tank shell, or 48 inches of an unanchored tank shell whenever the design is based on R > 1.
- When using working stress design, the seismic allowable stresses under seismic loading shall be AWWA D100 values, including the effect of internal water pressure, whether anchored or unanchored. When using ultimate strength

methods with R = 1, the limiting buckling stress can be based on elastic-plastic considerations, maintaining a factor of safety of 1.5.

- If unanchored tanks are used, then all roof level support beams shall be designed to safely accommodate at least 4 inches of wall uplift, while maintaining vertical load carrying capacity of at least 20 psf. The roof should be checked for loads from sloshing, but in most cases damage to the roof will be due to wall uplift unless the roof is designed to accommodate such uplift.
- When selecting the grade of steel for tanks, a low yield stress steel will provide superior resistance to buckling. Using a high-yield stress steel will result in thinner walls, and less buckling resistance. If a high-yield stress steel is used, then careful attention should be made to assure that wall buckling is avoided using elastically-computed stresses.

UNITS

1 g = 386.4 inches / sec^2 = 9.81 meters /sec^2. 1 inch = 25.4 mm. 1 feet = 12 inches = 0.3048 meter. 1 psi = 1 pound per square inch = 6.89 kiloPascal = 6.89 kN/mm^2. 1 pcf = 1 pound per cubic foot. 1 ksi = 1,000 psi = 6,890 kN / mm^2. 1 MG = 1 million gallons = 3,785,413 liters. 1 pound = 1 pound force = 4.448 newtons.

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