Performance of Elevated Tanks During Recent California Seismic Events

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This paper tells of experiences concerning how steel elevated water storage tanks performed during the Whittier, Loma Prieta, and Northridge earthquakes. In general, the steel elevated tanks evaluated performed very well in that none of them fell or lost water.

The tanks reviewed in this paper were all multiple column, strut and diagonal bracing type tanks. Modes of failure noted were all readily explainable, and are classified as:

- Failure to design ductility into the structure
    - No enlarged portion of diagonal bracing where threaded (no upsets).
    - Discontinuities in diagonal bracing.
    - Inability of horizontal struts to accommodate yield loading in diagonal bracing.
    - Inadequately designed connections
        - single shear
        - unbalanced double-shear
        - cotter key retainers
        - failure to design for yield loads
    - Improper anchorage
        - Anchor bolts
        - Anchor chairs
        - Shear keys

Tank 1

This 100,000 riveted elevated tank was constructed in 1928 with no seismic design criteria. After the February 9, 1971 earthquake, it was found that one of the diagonal rods had broken, the forged eye for the pin on the other end of that rod cracked, and all rods had stretched. The AWWA D100 Standard in 1971 (D100-67 and subsequently D100-73) called for a 10% lateral load, and indicated that the area of California affected was in Zone 3. This was also based on the Uniform Building Code (UBC). Engineers at that time recommended a 16% seismic factor, but felt that they were limited by the strength of the original columns. Today's standards would be for a Zone 4 design criteria, which would yield a larger factor. The modifications included replacing the diagonal bracing with larger diameter steel bars, thicker wing plates, pins using a tight grip, and placing a horizontal strut at grade level. Tank 1 was evaluated in October 1987, after experiencing an approximately 6.1 followed by a 5.6 Richter event centered in Whittier, about 10 miles away. No damage or signs of disturbance were observed at that time.

The next major activity it experienced was in 1994 during the Northridge quake, about 22 miles away. It was again evaluated, and no damage due to seismic loads was observed. However, in the meantime, neighborhood resistance to the presence of the tank had emerged. The water utility had scheduled to remove the tank after certain piping and control modifications had been accomplished. The Northridge quake and its after shocks did cause the diagonal bracing to rattle. The family living adjacent to the tank was sure that the tank was going to fall down any minute. The neighbor confronted the engineers evaluating the tank. He obviously had little faith in engineers and made the remark that none of them would want to sleep under that tank. About eleven p.m. that night, one of the tank evaluation engineers appeared at the irate neighbor's door with pillow and blankets in hand and asked where the sofa was that he was to sleep on. He stayed there all night, calming the neighbor somewhat, but the neighbor continued to call the water utility daily to check on when the tank was going to be dismantled. About 10 months after the Northridge quake, the tank was finally taken out of service and removed from the site.

Tank 2

This 60,000 gallon riveted elevated tank was constructed in 1921 with no seismic design criteria. Like Tank 1, it was affected by the 1971 earthquake and was upgraded shortly thereafter to what would be an approximate Zone 3 design criteria. During the 1987 Whittier quake (about 12 miles away), two of the upper pins attaching the diagonal bracing to the columns in the upper panel of the west bay of the tank were dislodged, allowing those two diagonal braces to become detached and fall. The fallen rods were supported by the strut. One end of one strut was damaged. The rods in the easterly upper panel remained in place; however, the cotter keys installed as pin retainers had partially sheared, and the pins were in danger of becoming dislodged. The 1971 "upgrade" had utilized single-shear connections on the upper end of the top panel. These single-shear connections placed an eccentric loading on the pins, resulting in a classic failure mode. (Cotter keys have not been recommended for retaining the pins in double-shear connections in seismic designs for many years, as they can become dislodged due to the vibrations. Similar dislodging has been observed in tanks subject to high winds.) Cotter keys should also not be used because of the tendency for the clevis plates to spread. Bolts or pins with thick welded washers are required to retain the clevis position during loadings exceeding the working load.

There did not appear to have been any movement in the anchor bolts or base plates. Likewise, the struts showed no signs of over stress, appeared intuitively to be designed to take the yield loading of the diagonal bracing. The pins were initially replaced in-kind by the owner. Upon our recommendation, the pins were replaced with structural bolts which would keep the single-shear connection from developing eccentricities which would cause the connection to separate.

Tank 2 was evaluated again in January 1994 following the Northridge tank, and no disturbance due to seismic activity was observed. This tank was removed from the site in 1994 when tie-ins to a larger system served by ground storage tanks on the outlying mountains was completed. Unlike Tank 1, the neighbors in this affluent neighborhood did not want the landmark tank torn down.

Tank 3

Tank 3 is a 100,000 gallon four column shop welded and field riveted double ellipsoidal elevated tank constructed in 1939. This tank was taller than the first two, being 102 feet to the high water level. This tank and tower was designed for some seismic loading, estimated to be 5% (Zone 2). Following the 1987 Whittier earthquake, the tank was evaluated for damage. No structural damage was observed. The tower did show signs of movement at the diagonal bracing clevis-to-pin-to-wing plate connection. This movement was evidenced by disturbance of the heavy coating of paint that cracked under movement. No evidence of stretching of the diagonal bracing or bending of the struts was observed. No lateral movement at the tower-to-foundation interface was observed. The paint was cracked under one anchor bolt nut, indicating that an upward strain, likely exceeding yield stress had been experienced.

Examination of the tank in 1994 did not indicate any new movement as there was no new coating disturbance (only the rusty areas where disturbed in 1987 were visible). One feature which this tank had which most seismic designed tanks at that time (1939) didn't was a heavy member transferring lateral loads around the three foot diameter riser pipe, keeping the four tower posts in-square, and supporting the riser pipe from bending due to the horizontal loadings placed on it.

Tank 4

This 600,000 gallon elevated tank, was constructed in 1951 using design criteria of 10% lateral acceleration (equivalent to a Zone 3 fixed percentage design). The eight wide flange columns supported the high water level to 95 feet. No records of its condition following the 1971 earthquake were found. It was the closest elevated tank to the epicenter of the 1987 Whittier quake. The tank withstood two shocks of 6.1 and 5.6 magnitude on the Richter scale without losing water or falling down. The tower did experience extensive damage, most of it predictable when the structure was analyzed using today's criteria. The industry has learned much about the need to design connections and compression members to accommodate the maximum load placed on them when the tension members reach their yield strength and become plastic. Prior design practice was to design each member based on the anticipated loading calculated based on the design acceleration of the tank and tower. We have since perceived that connections and compression members must be designed using reasonable factors of safety and taking into account any oversizing of the tension members due to the availability of standard sections and variances in the yield strength of the material as manufactured. For example, it is not unusual for steel ordered to A36 specifications (having a published yield strength of 36,000 psi) to have a yield strength of 42,000 psi indicated on the certified mill test reports.

The predictable areas of over stressing were:

Bending of the strut-to-column connections due to failure to carry the full section of the compression member to the wing plate and/or column.

Bending (bowing) of the struts in the middle due to inadequate section modulus and the eccentricities imposed by the bending end connections.

The diagonal bracing stretched as would be predicted. One striking feature of the bracing yielding was that it all appeared to take place within a few inches of the wing plate connection. The topcoats were delaminated from the primer as noted on other yielded members on this tank and others.

One component failure, which was not predictable, was the failure of two of the top diagonal bracing connections. The diagonal bracing consisted of 6" x 1-1/16" through 6" x 1-1/4" bars without any tension adjusting provisions. These bars were welded to two clevis plates that were in turn welded to the wing plates, giving a symmetrical load path on each side of the bar. Although this connection was apparently designed properly, the clevis plates at the top of the columns nearest the tank container were not in a position lending itself to easy manipulation of the manual field welding process. Two of these welded clevis connections pulled loose from the wing plate, placing an eccentric loading on the diagonal bracing in those two opposing panels, resulting in a single-shear type connection to the wing plate. The bracing did not fail, but it is predicted that any significant additional loading would have caused the two diagonal braces to become disconnected. The two diagonal members affected were in bays parallel to a radial line emanating from the epicenter.

No foundation movement was noted. One column base plate moved approximately 1/8", apparently due to a lack of soundness of the grout in the shear key on the top of the foundation which is designed to transfer the lateral load from the shear bars on the bottom of the base plates to the foundation. The paint around two of the anchor bolts was cracked, indicating that those two anchor bolts were temporarily stretched. They did not, however, appear to have been stressed to yield point.

This tank was kept in service for a few years after the October 1987 Whittier quake. The water level in it was reduced, but no repairs were made. A design was presented which would have upgraded the tank and tower to today's seismic standards (a 35% lateral loading) but an auxiliary water line was installed and additional storage on a higher elevation was developed, so the tank was eventually torn down.

Tank 5

Tank 5, a 40,000 gallon elevated tank 100 feet to overflow elevation was evaluated in 1984. As a part of a tank repainting project, seismic modifications (not a complete upgrade) we accomplished. The strut cross section area and section modulus were increased to withstand the yielding force of the diagonal bracing. Discontinuities (weld undercut) in the flat bar stock diagonal bracing were welded over and ground smooth. During the Loma Prieta earthquake it was reported that the diagonal bracing yielded as expected, but no other damage was done to the tower or tank. It is understood that since then, a more extensive reinforcement of the tower has been accomplished. It has yet to be seen if this more rigid tower will survive any better than the more simplistic "ductility assurance" approach used in 1984.

Some Upgrades are Downgrades

Sometimes we have seen tanks subjected to upgrading for seismic conditions that will likely have a greater propensity for failure than the original design, or at least a greater propensity for failure than a modified upgrade. An example, is a 100,000 gallon elevated tank 123 feet to the low water level which had been constructed in 1923. It was located near the New Madrid Fault in Mid-America. When evaluated in 1989, it was discovered that a seismic upgrade had been accomplished a year or so earlier.

Large concrete tie-struts and foundation enlargements had been installed, covering the original base plates and anchor bolts. It was assumed that increased anchorage had also been installed. The most glaring thing observed was the replacement of the originally approximately one square inch of diagonal bracing area with new double angles with an area of five square inches, attaching them to the wing plates using three 3/4 inch diameter bolts. It appears that the bolts will shear or the struts will buckle before the cross bracing will yield. Good seismic design allows yielding to take place, thus absorbing energy. Poor seismic design builds in potentially abrupt failure modes, which would cause elevated tanks to tumble instead of sway. No provision was made to transfer the load around or through the riser to the opposing column as was described earlier in this paper. Oh yes! Some more things were not considered in the "upgrading" of this tank. The tank bottom had corroded to the extent that it was leaking, the vent had no screen, and other sanitary and safety deficiencies were noted.

A sensible approach to increasing seismic survival probability includes anchoring the tank to take advantage of the maximum downward holding force of the foundation and designing the diagonal bracing to yield before the anchor bolts yield, connections fail, or the struts buckle.

In closing, lets look at this comparison: In real estate, it is said that everything is location, location, location--in seismic design for braced tower tanks it's ductility, ductility, ductility.