

Seismic retrofit of liquid storage tanks

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ABSTRACT: We used the results of recent experimental and numerical studies on the dynamic response of above ground liquid storage steel tanks to develop an improved numerical model for two 1,000,000 gallon (3.8 million liter) water tanks in a highly seismic area. Detailed numerical analyses were used to evaluate alternatives to upgrade the seismic capacity of these tanks. The selected scheme included new steel straps, welded to the bottom of the tank shell and anchored in a new reinforced concrete ringwall, to prevent uplift and shell buckling. This paper presents practical information related to design and construction and the lessons learned during this project.

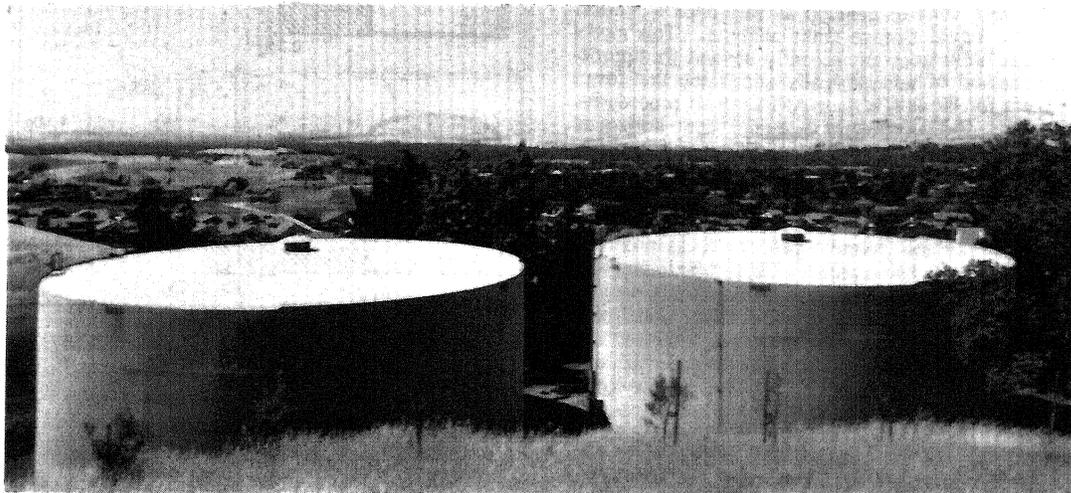


Fig. 1 General view of Clayton Valley tanks

1 INTRODUCTION

The Clayton Valley tanks, shown on Figure 1, are located near Concord, California. These two above-ground steel reservoirs, 76 feet (23.2 m) in diameter and 30 foot (9.1 m) high, were built in 1958 about 4 km away from the Concord Fault, a historically active member of the California Coast Range fault system. Although assigned an upper bound of magnitude of only 6.0 on the Richter scale, the Concord Fault was concluded to be capable of generating a mean peak ground acceleration (PGA) of about 0.32g at the

Clayton site. Other larger, but more distant features, such as the Hayward, Calaveras or San Andreas faults, could induce significant motion at the site, of longer duration but lesser intensity than the Concord Fault.

The tanks represent an essential supply of potable water and are used as a source of emergency water by the City's Fire Department in case of a major earthquake. The owner, the Contra Costa Water District, decided that the Clayton Valley tanks should be capable of withstanding the most severe level of ground motion estimated for the site.

2 ANALYSIS

Observed damage during recent earthquakes has shown that flat-bottom liquid storage tanks, constructed according to modern U.S. design guidelines, such as those of the American Water Works Association (AWWA-D100, 1980) or the American Petroleum Institute (API-650, 1985), can be highly vulnerable to severe ground motion (Manos, Clough, 1985). Shortcomings of current industry standards for above-ground steel storage tanks include: the limit placed on the spectral response in acceleration of the fundamental mode of vibration; under-estimation of the sloshing wave height; and the fact that these standards do not address the uplift mechanism of unanchored tanks adequately (Manos and Clough, 1985).

Seismic performance of liquid storage tanks reveals a more complex behavior than is implied in standard design assumptions. Yet, essential storage tanks in the vicinity of recognized active faults must maintain full operating capacity. For such tanks, actual earthquake loads may exceed the minimum Code requirements by a factor of two or three.

We used the results of recent experimental and numerical studies on the dynamic response of steel tanks (Haroun and Housner, 1982; Veletsos and Tang, 1986; etc.) to develop an improved numerical model for liquid storage tanks analysis. This model (Bureau, 1989) goes significantly beyond the approximate method of seismic loads calculation that forms the basis for the AWWA or API analysis procedures. It includes specification of site-dependent horizontal and vertical response spectra, wall and roof inertia, impulsive and convective dynamic water pressure loads and possible soil-structure interaction between the tank and its foundation. Tank uplift is simulated by an approximate iterative procedure, as proposed by Kennedy and Kassawara (1988). Maximum base shear, overturning moment, sloshing wave height, foundation toe pressure, roof column loads, and peak hoop, compressive, tensile and shear stresses are computed for each component of the specified motion. Stresses are then combined by the Square-Root-of-the-Sum-of-the-Squares (SRSS) method.

The calculated response was significantly larger than used in AWWA-D100. Figure 2 schematically compares the lateral loads of standard AWWA provisions for Seismic Zone 4 (maximum hazard in California), the loads calculated in the detailed model for the Clayton Valley tanks, and loads estimated by Manos and Clough (1985) for oil tanks of same size during the May 2, 1983 Coalinga, California earthquake (M 6.7). Significant tank damage occurred in Coalinga at short distance from the epicenter. Although the Clayton tanks comply with basic AWWA's design standards, rigorously calculated earthquake loads were substantially larger than those established through the code formulas. Figure 2 underscores possible shortcomings of a simple application of the code formulas.

The analyses indicated that, at maximum water level, the tanks would amplify ground motion and experience significant uplift. Based on the resulting compressive stresses, the lower part of the tank walls was concluded likely to fail through excessive buckling. We calculated, however, that the tanks would

perform satisfactorily for the specified earthquake loads if their maximum water level was kept at about seven feet (2.1 m) below current overflow level. Such a restriction (76 percent of capacity) would not be consistent with the District's operational requirements and the analysis results for the full tanks were used to evaluate alternatives to upgrade their seismic capacity.

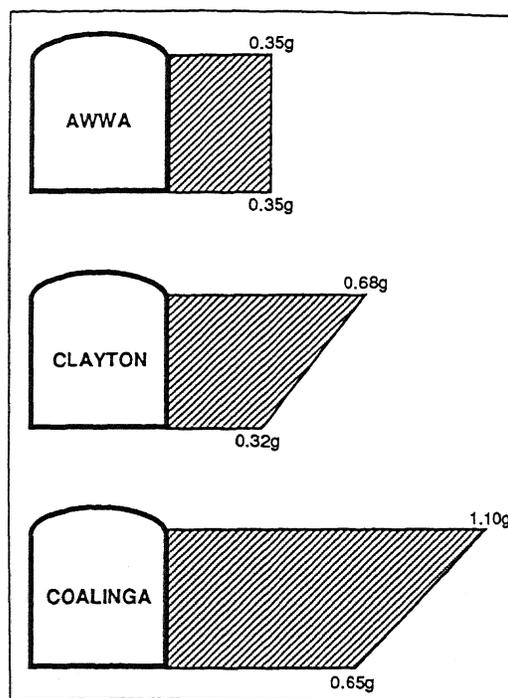


Fig. 2 Comparison of seismic loads

3 SEISMIC RETROFIT

We considered several alternatives for structural improvements. These included: adding a thicker outer annular ring (sketch plate) to the existing bottom plates; anchoring the tanks to their foundation; and/or isolating the tank shells from rigidly attached valves and pipes through the use of flexible couplings.

AWWA-D100 imposes restrictions regarding thickness, maximum width (which depends on the tank diameter), and minimum width of the bottom annular ring (which depends on the thickness of the lower course of the tank wall and the specified yield strength of the plate). Considering those limitations, we determined that a sketch plate would only allow the maximum safe water level to be raised by about two feet (0.61 m), and this solution was rejected. We concluded anchoring the tank to be viable, consistent with the fact that, despite mediocre field performance of some poorly detailed anchor systems, no anchored tank is known to have lost its content as a result of an earthquake (EPRI, 1989).

We calculated the anchorage forces required to resist seismic overturning moment and base shear. Our approach was conservative, as it ignored contributions to the hold-down capacity due to membrane forces in the tank bottom and limited uplift of the tank, should the anchorage be stretched. Conventional anchor bolt chairs, such as detailed in the Design Manual of the Steel Plate Fabricators, are known to introduce a large moment in the tank shell. This moment results from the eccentricity between the anchorage center of resistance and the tank shell. Instead, we decided to use steel anchor straps, regularly spaced, and welded to the tank shell. The use of straps considerably reduces the eccentricity and applies restraining forces around the tank perimeter more evenly than large bracket restrainers. To provide additional ductility, a specified length of the straps above foundation level was left unwelded to the tank wall. Dimensions and materials requirements for the anchorage system were established to provide stability against uplift and prevent tank buckling.

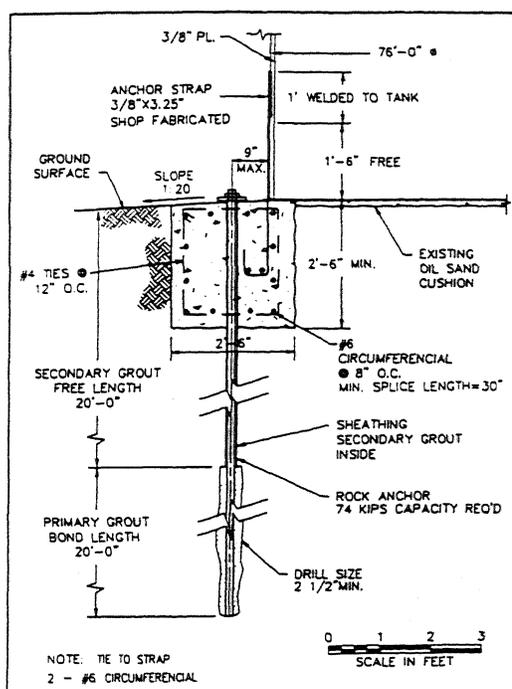


Fig. 3 Anchorage system cross-section

While foundation conditions at the Clayton site were excellent to support flat bottom reservoirs, there was no reinforced concrete mat or ringwall. A new reinforced concrete ringwall, in which the lower part of the anchor straps would be embedded, was required. The ringwall was designed to extend a minimum of six inches (0.15 m) under the tank bottom. Reinforcement was detailed to resist applicable forces and moments when the tank tries to uplift. It was necessary to hold-down the reinforced concrete ringwall with prestressed rock anchors.

We developed plans and technical specifications for the recommended retrofitting scheme. As an extra safety measure, the design included isolation of the main inlet/outlet piping and valve from the tank wall with a double-ball flexible coupling. The drain piping was extended beyond the tank bottom plate to provide additional flexibility and the lower part of the overflow piping was re-routed to avoid interference with the ringwall. Figure 3 shows a cross section of the anchorage system, which included 60 anchor straps around the tank wall and 30 rock anchors along the ringwall perimeter.

Detailed design included verifying anchors straps weld, yield, tensile, and pullout capacities; checking the concrete ringwall against shear, torsion, bending and punching loads; sizing the rock anchors (diameter, primary bond length, secondary grout length) to prevent pullout or bond failure; and checking secondary stresses in the tank shell to prevent tearing along the welded edges of the straps. The anchorage system was designed so that the lowest factor of safety would correspond to failure by strap yield or pullout. In this manner, an energy-absorbing mechanism is provided if unexpected earthquake loads exceed the specified criteria.

4 CONSTRUCTION OF IMPROVEMENTS

Field implementation of the improvements began late July 1991. They were completed in November. The tanks were dewatered one at a time to construct the new foundation. A critical task was to excavate the ringwall volume without overloading the tank bottom plate by the unsupported shell. The Design Engineer, Dames & Moore, had originally considered an excavation plan for diametrically opposed sectors along the tank perimeter to control the unsupported extent at any given time. The Contractor, Soil Engineering Construction, Inc. of Redwood City, California opted to carry the edge of the tank on thin intermediary metal shoring supports at 10-foot (3 m) centers, left in place after pouring the concrete ringwall. This solution did not affect the way ringwall and anchors are expected to perform and reduced the number of splices in the circumferential steel reinforcement. It also allowed the ringwall volume to be excavated all at once, as shown on Figure 4. The excavation resulted in an unexpectedly clean cut on the inner side of the ringwall and did not disturb the tank bottom. Therefore, releveling of the tank bottom was not required.

Following placement of the reinforcing steel in the ringwall excavation, the anchor straps were positioned along the tank perimeter by tack welds (see Figure 5) and the new foundation concrete was poured. The anchor straps were dip-galvanized and their hooked extremities shaped in the shop, prior to installation.

The design pullout capacity of each ringwall anchor was 74 Kips (329 kN). One-inch (2.54 cm) diameter DYWIDAG Threadbars[®] Grade 150, protected against corrosion with PVC sheathing, were used for such anchors. PVC guide tubes had been placed prior to pouring the ringwall to facilitate installation. A construction requirement was to center anchor heads at 9 inches (23 cm) or less from



Fig. 4 Ringwall perimeter excavation



Fig. 5 Detail of tank anchorage

the tank shell, to limit torsion of the ringwall when the straps are stressed under earthquake loading. Nine anchors were tested. Six proof tests (fast test to 1.3 times the design dynamic load) and three performance tests (long test to verify capacity, established free length, and residual movement) were performed on selected units. These tests confirmed that the required capacity would be achieved under earthquake loading. After testing, the anchors were post-tensioned to 20 percent of ultimate pullout capacity in a prescribed sequence to distribute loads uniformly around the ringwall perimeter. Lastly, the upper part of the anchor straps was final welded to the tank wall along the specified length.

The main inlet/outlet piping and valve were isolated from the tank wall through installation of a double-ball "Flextend" coupling, manufactured by EBAA Iron, Inc., Eastland, Texas. This seemingly redundant protection will assure that relative movements between the tank and the surrounding ground due to seismic wave passage or unexpected anchor yield will not shear the piping off the tank and cause loss of content. The installation of the flexible coupling was straightforward (see Figure 6). A moderately sized concrete thrust block was constructed to accommodate any sudden expansion of the Flextends as a result of pressure head and water hammer when the valves are closed rapidly.

5. CONCLUSION AND LESSONS LEARNED

No problems were encountered during construction. The completed strengthening scheme is fairly



Fig. 6 Flexible coupling and thrust block

unobtrusive, see Figure 7. As is often the case with old facilities, we discovered during construction that the underground part of the inlet/outlet piping was positioned differently from the way shown on the original drawings, which resulted in a change order to relocate some pipes. This could have been avoided by indicating on the new drawings the need to verify key dimensions.

Another incident came up when the construction inspector noticed that the welding subcontractor had not stripped the tank surface, as specified in the

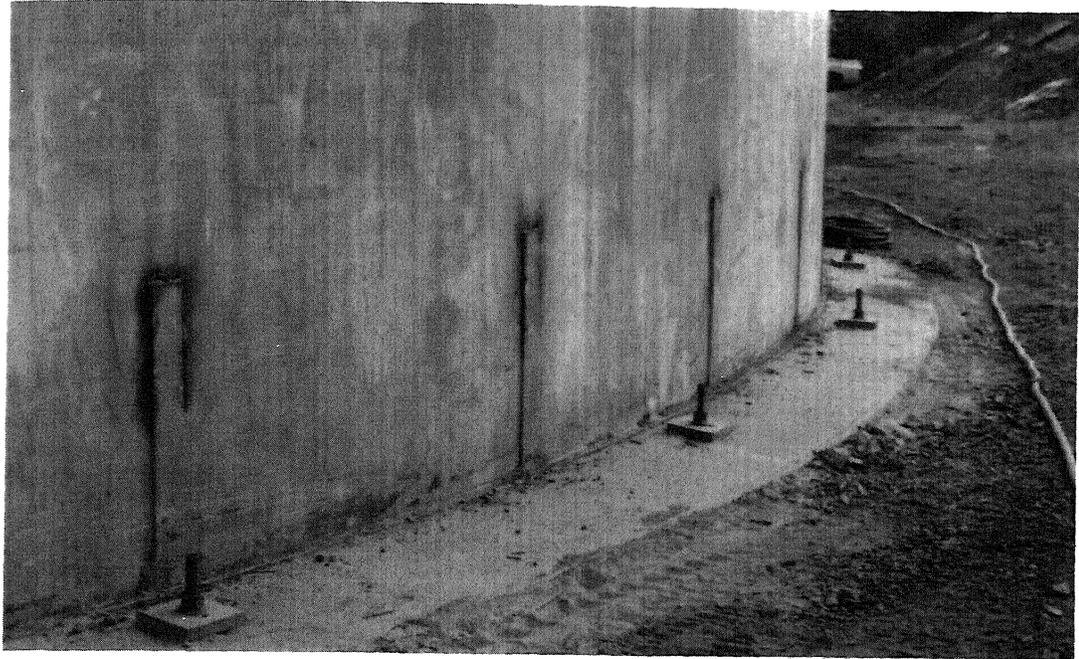


Fig. 7 Completed project prior to tank coating

construction documents, but was welding the straps directly to lead-based paint covered metal. While the resulting amount of potentially toxic vapors would be fairly small in open air, the presence of lead paint had not been anticipated. The work was continued in compliance with applicable environmental and health regulations. We learned in the process that over ninety percent of water tanks in California, if painted before 1975, have been primed with either red lead or zinc chromate paint. This issue, therefore, needs to be addressed in technical specifications for the seismic retrofit of old steel reservoirs. Lastly, in the case of the Clayton Valley tanks, cathodic protection was in place, which will reduce corrosion of the galvanized anchor straps. Cathodic protection must be part of the improvements when a similar strengthening scheme is to be implemented to steel tanks.

The construction cost for retrofitting the two tanks came to about \$115,000 each, including the installation of the new foundation ringwalls. Although such expenditures required careful planning on the owner's part, they were considerably less than the cost of dismantling the tanks and replacing them by new facilities built to accommodate the severe seismic requirements of the site. Most importantly, a similar anchoring scheme can be implemented on any new steel tank at a small fraction of its construction cost.

6 ACKNOWLEDGEMENTS

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