SEISMIC DESIGN OF A COMBINED STANDPIPE AND AN ELEVATED TANK IN SAN DIEGO COUNTY, CALIFORNIA

BY
MARK R. PIEREPIEKARZ[1], JOHN G. SHIPP[2], DON A. SMITH[3] AND KEVIN A. BINDER[4]

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ABSTRACT:

This paper presents the seismic analysis and design of the combined tank structure consisting of a 1.1 MG elevated tank (Fletcher Hills Reservoir) supported on a 2.5 MG standpipe (Grossmont Reservoir). The diameters of the elevated tank and the standpipe will be 100 feet and 70 feet. The over all tank height will be 132 feet. The structure will be supported on a reinforced concrete mat foundation. The overturning seismic loads will be resisted by a system of rock anchors in tension and the soil in bearing. The combined structure will replace two separate existing 2.5 MG and 1.4 MG standpipes.

KEYWORDS:

Welded steel tanks, water reservoirs, AWWA, tanks, rock anchors, concrete mat foundations

AUTHOR INFORMATION:

1 EQE International, Inc., Seattle, WA, USA
2 EQE International, Inc., Irvine, CA, USA
3 John Powell and Assoc. Inc., Encinitas, CA, USA
4 Pitt-Des Moines, Inc., Des Moines, IA, USA

INTRODUCTION:

Two water districts located in San Diego County, California requested engineering services for the relocation of two existing water tanks located on adjacent sites and in the way of the proposed State Route 125 Freeway. The feasibility of the replacement structures was strongly influenced by seismic design considerations.

The existing Grossmont Standpipe is a ground-supported flat-bottom tank with the overall dimensions of approximately 70 feet in diameter and 93 feet in height. The tank roof, shell, and floor consist of welded steel plates of varying thicknesses. The tank holds approximately 2.5 million gallons (MG) of water. The adjacent Fletcher Hills Tank is approximately 50 feet in diameter and 100 feet in height with a maximum volume of 1.4 MG of water. The height-to diameter (H/D) ratios of the existing tanks are driven by high hydrostatic water pressures required to service the surrounding area. The existing two tanks are shown in Figure 3.

A number of replacement tank alternatives were considered and included combined tanks as well as in-kind replacement of the existing structures. All alternatives exhibited high H/D ratios; a vary unfavorable feature for a high seismic area. The selected combined tank scheme (an elevated tank stacked on top of a standpipe) was favored by the water districts due to both economics and a desire to develop a common solution.

Since the design basis seismic ground motions are very high, a combined tank scheme represented a challenging problem requiring creative approaches. Base isolation was considered, however, the solution was unsuitable for a combined tank scheme due to the concern that that the long-period sloshing may be amplified in a base-isolated structure. Other solutions focused on achieving seismic stability utilizing a rock anchor system. One scheme included steel buttresses welded along the perimeter of the shell to enlarge the tank base and provide a more desirable H/D ratio. This solution involved substantial additional material costs and was eliminated. Con-centric tanks were also proposed, but did not satisfy the system hydraulic requirements.

Finally, an innovative combined tank scheme meeting the clients' needs and feasible from the seismic stand point was developed. The innovation was a compression dome in the lower reservoir (Figure 1). In conventional flat-bottom tanks, since the contained liquid weight is transferred directly to the foundation, only a portion of the tank contents can be utilized in reducing the net uplift force on the anchors which are located along the perimeter shell. Consequently, a dome-shaped bottom plate was proposed to utilize the liquid weight more efficiently in stabilizing the structure against overturning. The gravity loads from the liquid contained in the lower tank are thus transferred in compression through the dome plate and are concentrated at the tank perimeter (rather than distributed over the entire area of the bottom plate as in a flat bottom tank). The entire lower reservoir liquid is therefore utilized in reducing the net uplift loads on the rock anchors. The scheme required a tension ring to be located at the tank base to resist radial thrust forces from the dome plate. The overall stability of the tank-mat system would be provided by the underlying rock substrata in bearing and rock anchors in tension.

PRELIMINARY DESIGN

A preliminary feasibility study focused on the foundation system was performed prior to soliciting tank fabricator construction proposals. The proposed configuration consisted of a 1.1 MG elevated tank (Fletcher Hills) and a 2.5 MG standpipe (Grossmont). The diameter of the elevated tank and standpipe are 100 feet and 70 feet, respectively. The total tank height is approximately 132 feet.

Both AWWA and the Maximum Credible Earthquake (MCE) analyses were performed. The MCE analysis is based on a 475 year return event horizontal ground motions response spectra developed for the site. The horizontal ground motions are relatively strong with a peak ground acceleration of 0.31g. The ground motions are amplified by the structure and the horizontal spectral accelerations are in the range of 0.7g to 1.0g. No vertical accelerations were considered.

The impulsive and convective fluid masses and centroids were based an TID 7024, Nuclear Reactors and Earthquakes, Chapter 6 " Dynamic Pressure on Fluid Containers". In general, this approach utilizes an idealized tank-liquid model in which the contained liquid pressures are caused by (1) an impulsive portion of the liquid accelerating with the tank and (2) a convective portion of the liquid sloshing in the tank.

The liquid below the apex of each dome was assumed to be constrained. The remainder of the liquid was modeled assuming an equivalent cylindrical volume. The bottom pressure effects were considered in both reservoirs in calculating the shell and foundation loads. Material damping of 2% for the shell and impulsive response and 0.5% for the convective response was assumed. Various liquid fill conditions were assumed.

The analysis confirmed that the proposed structure is feasible and will require a rock bolt system and a reinforced concrete mat foundation 100 feet in diameter.

FINAL DESIGN

The final combined tank configuration is the same as the preliminary design and is supported on a reinforced concrete mat foundation 100 feet in diameter and 10 feet thick. The final shell thicknesses are shown in Figure 1. The outer shell will be constructed of ASTM A588 (50 ksi) steel. This material was chosen for the exterior shells due to low maintenance for a uniformly corroding core ten steel.

Even with the above innovations the uplift and shear forces are formidable for both MCE event as well as the AWWA seismic zone 4 loads. The final anchorage system involves 150 2.25 inch diameter bolts (75 ksi steel). Fifty (50) of these are rock anchors embedded 50 feet into the underlying rock. One hundred (100) shear anchors are embedded 8 feet into the concrete mat.

Final Design Criteria:

Both AWWA and the Maximum Credible Earthquake (MCE) analyses were performed. The AWWA seismic forces are based in the draft AWWA D100 1994 provisions. No vertical accelerations were considered. A reduction factor of 1.5 was used for MCE analysis. This factor was deemed conservative, yet appropriate given the tank-anchor system ductility. Each set of calculations included three loading conditions: both tanks full, top full/bottom empty, and bottom empty/top full. A wind force check was also made when both tanks are empty. AWWA allowable stresses with 1/3 increase were used for the code analysis. The MCE analysis was based on material yield stresses.

Hydrodynamic Masses Model:

The combined tank structure was modeled using the ANSYS finite element program as a fixed cantilever with multiple degrees of freedom and lumped masses. Elastic straight pipe elements were utilized to model the cylindrical and conical shell. Structural mass elements were used to model the masses. Link elements were utilized to connect the convective masses to the shell.

As in the preliminary design, the impulsive and convective fluid masses and centroids were based an TID 7024, Nuclear Reactors and Earthquakes, Chapter 6 " Dynamic Pressure on Fluid Containers". For the impulsive water mass, both mass and mass moment of inertia were required. The mass moment of inertia for the convective water mass is not required because the sloshing water has been shown to provide no rotational restraint. The lower tank liquid was represented as an equivalent cylindrical volume containing the same volume as the actual tank with a matching top-of -water line. For the upper reservoir, the water below the upper dome apex was modeled as a constrained liquid. The remainder of the upper reservoir liquid was modeled as an equivalent cylinder. Material damping coefficients of 2% (shell, impulsive liquid) and 0.5% (convective liquid) were used. Bottom pressures for both the upper and lower reservoirs were calculated according to TID 7024. The bottom pressure effects on the upper dome were considered when designing the lower shell. The lower reservoir bottom pressure effects were considered when calculating the foundation loads.

A modal analysis of the structure was performed which was followed by a response spectrum analysis. The absolute values of the significant impulsive modes were added and combined with the convective modes using SRSS method.

Tank Anchorage:

The final tank anchorage consists of 150 2.25 inch diameter Grade 75 anchors. Of these, 100 anchors are embedded 8 feet into the concrete mat and 50 anchors extend 50 feet into the underlying rock. The overturning resistance is primarily provided by the rock anchors. Shear resistance between the tank shell and concrete foundation is provided by all anchors. All 150 anchors include bearing plates in the concrete to provide reasonably "uniform" stiffness in engaging the tank shell. The anchorage details are shown in Figure 2.

The anchorage design for the shear anchors is based on ACI-349 and takes advantage of compression due to gravity loads. The shear design is governed by anchors which are at 45 degrees to the assumed earthquake direction. This is due to relatively a high shear and limited tank shell compression force at that location. The anchor bolt embedment depth into concrete is adequate to fully develop the tensile strength (100 ksi) of the shear bolts.

Concrete Mat and Rock Anchorage Analysis

The rock bolt forces and soil bearing pressures were based on a 180 degree ANSYS finite element model of the mat foundation. The forces were benchmarked against hand-calculations for a circular foundation in partial bearing. The maximum uplift force in the rock bolts occurs on the tank axis parallel to the earthquake direction. The governing load case is with top tank empty for the MCE. The rock bolt embedment and chair designs were based on full-development of the steel anchors.

A 180 degree model of the mat foundation using ANSYS 5.0A was developed to analyze the foundation stresses. The concrete mat was modeled with 9200 eight-node solid elements. Elastic shell elements were utilized beneath the mat to model the elastic supporting substrata. On the compression side the interface between the mat and the soil was modeled with gap elements which allowed the foundation to lift from the soil as required. Contact-type elements were utilized to model the rock anchors. These elements were assigned no compression stiffness and a tension stiffness based on the bolt area, modulus of elasticity, and rock bolt length.

Top and bottom grid bars are provided to resist flexure in the concrete mat. Additional "hoop" reinforcement is provided to resist circumferential flexure and provide additional reinforcement where top and bottom flexural bars may not be fully developed. Additional horizontal and vertical bars are provided along the perimeter of the mat. Shear stress checks were performed and vertical steel is not required. In additional check for crack control in mass concrete based on ACI-207 was performed. The ACI 207 requirements controlled the design of top grid bars.

Tank shell analysis

The vertical shell thickness requirements were evaluated at 14 discrete points along the tank height (see Figure 1 for summary). The shell analysis included various loading conditions and included consideration of longitudinal compressive and tensile stresses, hoop stresses and local buckling (elephant's foot) stresses. Both hydrostatic and hydrodynamic effects were considered.

The roof dome design includes: roof plate, tension and compression rings, typical rafters, and rafters adjacent to openings. The roof plate design followed a conventional procedure driven by roof dead and live loads.

Upper Dome Analysis

The upper dome plate and upper perimeter tension ring is sized based on ASME procedure for spherical plates with external pressure. Conservatively, maximum hydrostatic pressure was used. The upper dome consists of 1.75" thick unstiffened steel shell (ASTM A36). At the apex, the dome includes a 5 foot diameter opening for a drywell.

A 180 degree ANSYS finite element model of the upper dome, lower shell, upper shell cone, and center drywell is considered appropriate. The seismic analysis considered the MCE case with upper tank full. AWWA seismic loads are bound by this load case. The allowable stresses are based on ASME Section VIII Division 2, Boiler and Pressure Vessel Design Code. The stress limits are based on stress intensities associated with maximum shear failure theory. The reported stresses are within the above stress limits.

In addition, a buckling analysis of the upper unstiffened dome was performed. The analysis considered the three loading combinations: uniform pressure, hydrostatic pressure, and hydrostatic pressure plus seismic pressure (when both tanks are full of water). The buckling analysis also considered material and geometric non-linearities, initial imperfections and penetrations.

Lower Dome Analysis

The proposed dome is to consist of a 1. 50" to 1.75" thick steel shell (ASTM A588) stiffened with 36 steel stiffeners (ASTM A36). The ANSYS finite element model of the lower dome, base ring, and lower shell considered two MCE cases: both tanks full, and lower tank full. AWWA seismic loads were bound by these two load cases. The interface of the ring with the foundation was modeled using gap and contact elements to allow the base plate rotation under the loads and to model the anchor bolt resisting uplift. The allowable stresses were based on ASME Section VIII Division 2, Boiler and Pressure Vessel Design Code. The stress limits were based on stress intensities associated with maximum shear failure theory.

A finite element analysis was also performed to analyze the base ring stresses. A three-dimensional 10 degree section of the lower dome, base ring and shell was developed.

CONSTRUCTION ISSUES:

While many aspects of this project present formidable construction challenges, two issues are of particular note with respect to their importance to the structural and seismic integrity of the project.

Controlling the heat of hydration and, in particular, the differential cooling of the 10-foot thick concrete foundation slab proved to be a significant construction issue. As previously described, particular attention was paid to the concrete mix design per American Concrete Institute and Portland Cement Association guidelines. The design utilized fly ash and water reducing admixtures to control heat and shrinkage. In order to avoid shrinkage cracking exacerbated by differential cooling, the slab was tented, heated, and insulated during the curing period, and the actual slab temperature at various locations was monitored using thermocouples placed in the slab during the pour.

Due to the unusual nature of the tank design, its location in a seismically active region, and its proximity to residential properties, especially stringent welding inspection practices were adopted. While spot radiography is required by AWWA D100, this standard does not address the issues of high seismic zone activity or complicated structural systems. As a result, 100 percent ultrasonic inspection was required in both compression domes, the base ring, and the first five rings of the standpipe, in addition to the AWWA requirements.

CONCLUSION:

The design of the replacement structure for two water tanks was strongly influenced by seismic considerations. The selected combined tank structure required an innovative dome-shaped bottom to fully utilize the liquid contained in the lower tank in helping to resist uplift forces on the rock anchors. The foundation system consists a reinforced concrete mat 100' in diameter and 10' in thickness. The tank anchorage consists of 150 2.25" diameter anchors embedded 50' into the underlying rock.

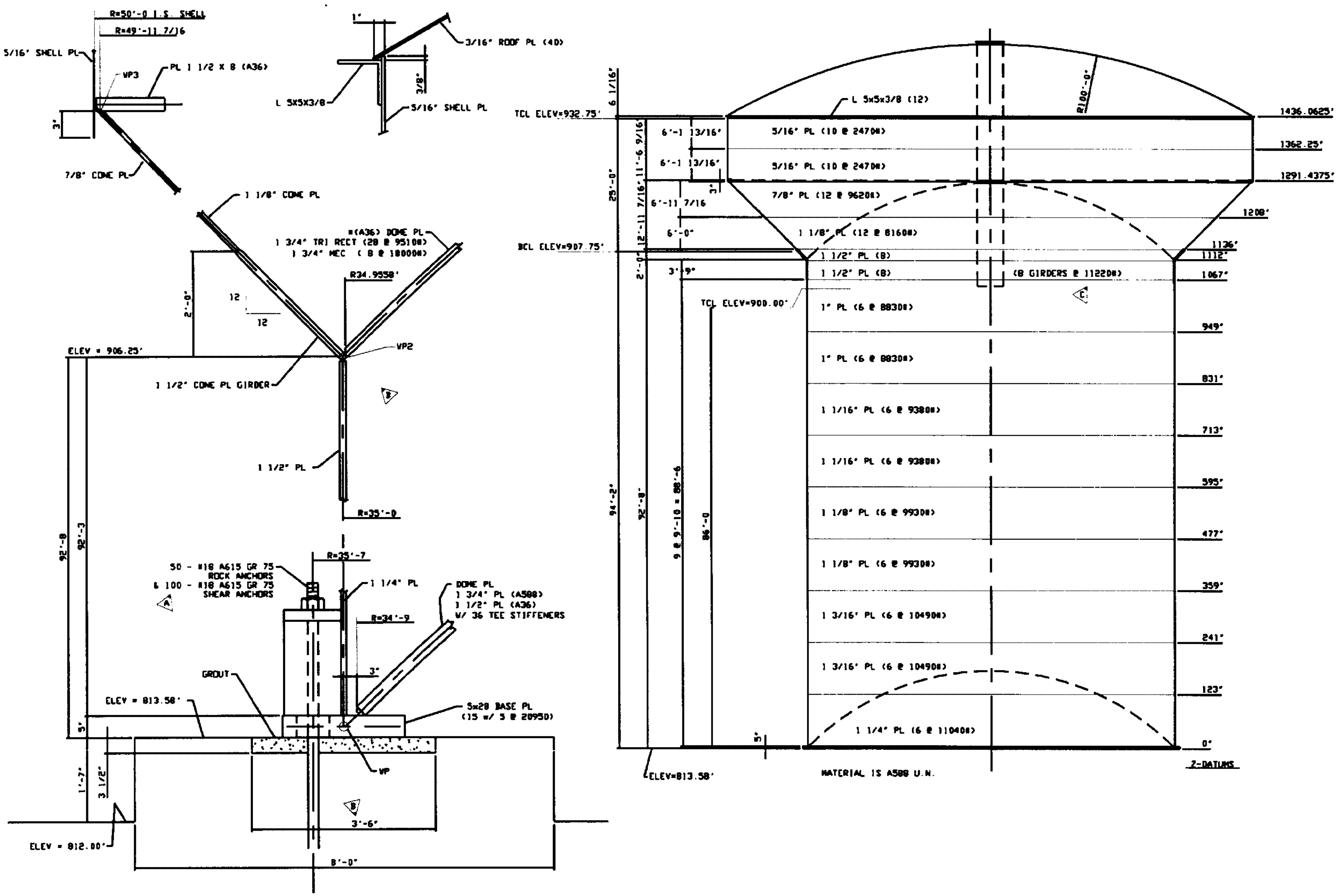

Figure 1 - New Combined Tank

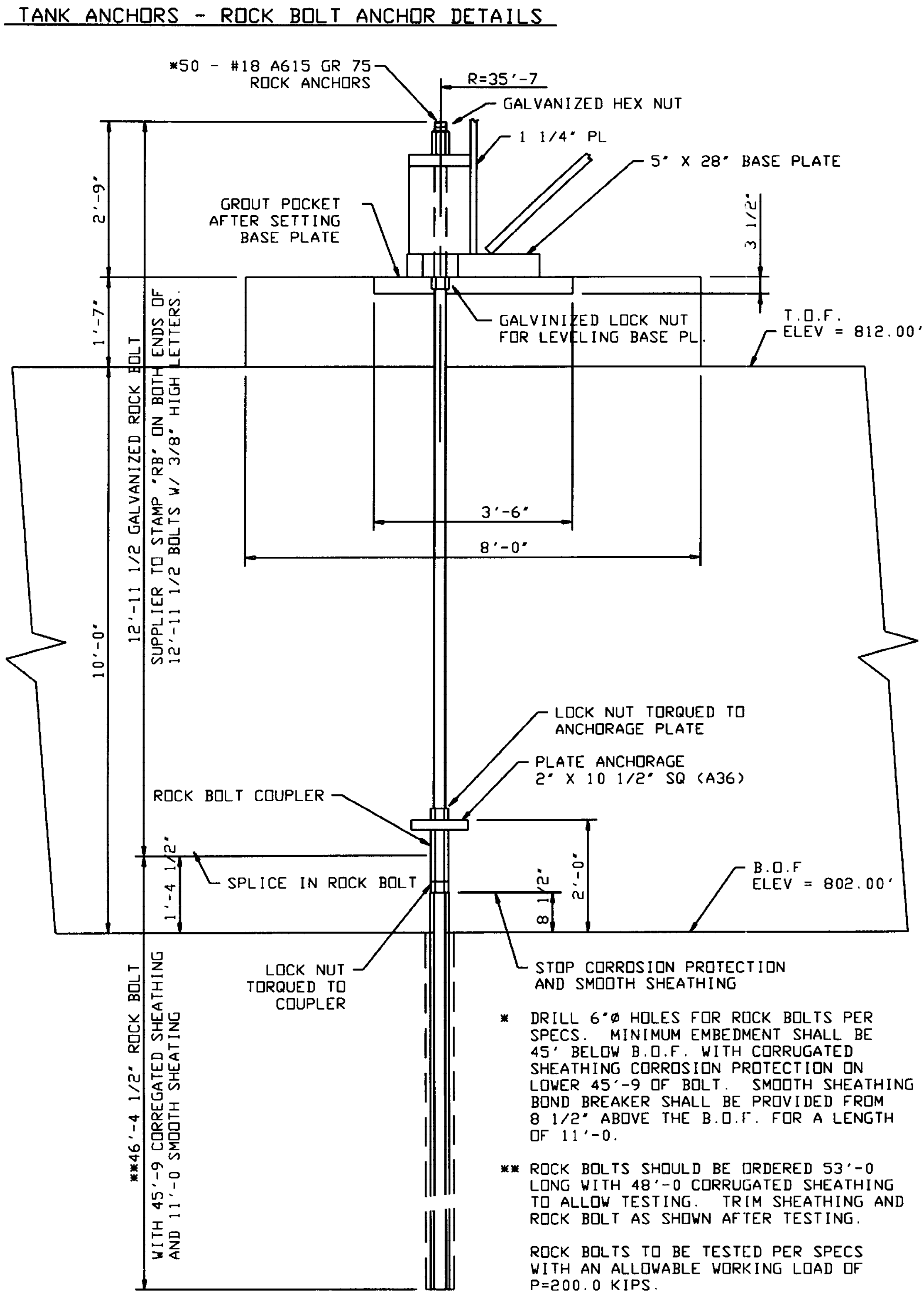

Figure 2 - Anchorage Details

Figure 3 - Existing Tanks

Figure 4 - Concrete Mat Foundation Under Construction