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SEISMIC UPGRADE OF THE FREEPORT WATER RESERVOIR, SACRAMENTO, CALIFORNIA

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SUMMARY

This paper describes the seismic analysis and design procedures used for the seismic upgrade of an existing 3,000,000 gallon (11,355,000 liters) steel elevated water tank supported by a steel framework in Sacramento, California. The initial seismic upgrade proposal consisted of a base isolation system, but budget overruns motivated the City of Sacramento to investigate alternative seismic upgrade schemes. The final seismic upgrade design incorporates a passive energy dissipation system in the braces of the framework to create a friction damped braced frame. A nonlinear time history dynamic analysis was undertaken for the design of the friction damped braced frame system. The seismic upgrade using the friction damped braced frame is compared with the originally proposed base isolation retrofit scheme

INTRODUCTION

In the past seven years, the City of Sacramento, Department of Utilities, has been performing seismic hazard assessments and seismic upgrades of their elevated water reservoirs. The City's elevated water reservoirs consist of reinforced concrete structures built in the late 1930's and a more modern steel elevated tank built in South Sacramento in the late 1950s. Each of these water reservoirs has storage capacities of 3,000,000 gallons (11,355,000 liters). For the elevated steel tank structure, force reduction systems such as base isolation and passive energy dissipation devices were considered for the seismic upgrade. The base isolation seismic upgrade scheme was first proposed for the elevated steel tank in 1994 [Cygna, 1994]. Concerns with the high construction cost for this scheme motivated the City of Sacramento to investigate an alternative seismc upgrade using a passive energy dissipation system. The Freeport Water Reservoir, after completion of seismic upgrade, is shown in Figure 1.

DESCRIPTION OF EXISTING ELEVATED STEEL TANK

The welded steel water tank has a capacity of 3,000,000 gallons (11,355,000 liters) with the top of water level at about 38 m (125 feet) above grade. The tank is spheroidal in shape, 38.4 m (126 feet) in diameter and 18.9 m (62 feet) in height at its center. The riser at the center of the tank consists of welded steel pipe 1829 mm (72 inches) in diameter with a 6 mm to 10 mm (1/4 to 3/8 inch) wall thickness.

The support columns consist of welded steel pipe. The outer circle of columns are 1219 mm (48 inches) in diameter, 18 in number and equally spaced at a radius of 18.9 m (62 feet). The column wall thickness varies from 10 mm to 11 mm (13/32 to 7/16 inch). These columns attach directly to the tank shell structure. The inner circle of columns are also 1219 mm (48 inches) in diameter, but with a 17 mm (11/16 inch) wall thickness, 9 in number and equally spaced at a radius of 9.5 m (31 feet). These columns attach to an interior ring girder which in turn attaches to the tank shell. All columns are vertical with no inclination and are spaced approximately 6.7 m (22 feet) apart.

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The diagonal braces occur in two tier heights on the outer circle of columns and are 17-18 m (55-58 feet) long. The braces are intended to be in tension only under lateral forces. The braces consist of 106 mm (4-1/16 inch) square solid steel bars at an angle of 26 degrees from vertical. The square bar diagonal braces attach to 25 mm (1 inch) thick gusset plates which enter into the interior of the pipe and are connected to horizontally orientated diaphragm plates welded to the pipe wall. The horizontal struts for the tiers of bracing consist of W12x53 steel sections bolted and welded to the gusset plates.

The columns are supported by isolated rectangular and trapezoidal spread footings bearing 2.7 m (9 feet) below grade. The column connection consists of shear lugs used in conjunction with two 44 mm (1-3/4 inch) diameter anchor bolts.

SEISMIC UPGRADE PROPOSAL WITH BASE ISOLATION

In 1997, an evaluation, seismic upgrade design and 90% construction documents were completed with the base isolation retrofit technique [Cygna, 1994 & ICF Kaiser, 1997]. The general criteria used for the base isolation seismic upgrade was the 1991 Uniform Building Code (UBC) [ICBO, 1991], Chapter 23, Appendix - Division III, Earthquake Regulations for Seismic Isolated Structures.

The performance level of life-safety was selected with two seismic hazard levels, the site-specific response spectrum and the 1991 Uniform Building Code (UBC) Zone 3, soil type S_1 response spectrum based on a 10% chance of exceedance in 50 years. However, the Maximum Credible Earthquake, based on a 10% chance of exceedance in 250 years, was used to establish maximum isolator displacements as required by the base isolation provisions of the UBC. The site-specific ground motions in the Sacramento area have been typically found to be less than that specified in the UBC, for the same return period, which was also the case for this project. As a result, 0.8 times the UBC response spectrum acceleration values (0.3g at zero period) were used for the analyses.

Analytical Procedures:

Three analyses were performed [Cygna, 1994], response spectrum (including frequency analysis), equivalent static analysis and a dynamic time-history analysis for the base isolators. Except for the nonlinearity of tension-only bracing in the equivalent static analysis, both the response spectrum and equivalent static analyses assumed linear elastic responding elements. For the response spectrum and dynamic time-history analysis, the impulsive and convective masses of the water were modeled. The fundamental period of the convective or sloshing mass was established using Housner's equations. The frequency characteristics of the base isolated structure which were reported include the sloshing mode period of 6.81 seconds and isolation mode of 3.0 seconds.

Base shears for the base isolation system were reported for the response spectrum and equivalent static analyses. The weight, including the water at high water level, the tank shell and support framing, was taken as approximately 12,836 kg (28,300 kips) total. 10% damping in the isolators was assumed for both analyses. Base shears were determined using 0.8 times the UBC Zone 3 response spectrum and further divided by 1.25 for 10% damping (resulting in 0.192 g at zero period). The resulting base shears reported include the equivalent static, R_{wi} = 1.5, equal to 6810 kN (1531 kips) and response spectrum, unreduced, equal to 9563 kN (2150 kips).

Displacements were also provided for the response spectrum and equivalent static analyses. The lateral displacements at the top of isolator were reported for the equivalent static analysis, *D*, equal to 286 mm (11.25 inches) and response spectrum analysis equal to 279 mm (11.0 inches).

Seismic Upgrade Design:

The proposed base isolation seismic upgrade design [ICF Kaiser, 1997] which was based on the above analyses, consisted of:

- New 127 mm (5 inch) square solid bar braces to replace the existing braces.
- Extension of existing gusset plates and new internal stiffening plates.
- Addition of a steel tube ring beam around the perimeter of the tank at the intersection of the column to tank shell.

- 27 high damping rubber isolators and one teflon slider for the riser located on top of the existing foundation pedestals.

- A horizontal steel tube braced diaphragm at the top of the isolators.

- New piping with expansion joints at the riser intended to accommodate the total displacement of the isolators.

In addition, the existing valve house located within the plan of the tank had to be moved to accommodate the horizontal steel tube braced diaphragm and the movement of the isolators. The final construction cost estimate for this system was approximately US$ 1.98 million. Concerns with construction estimate budget overruns for this proposed seismic upgrade motivated the City of Sacramento, Department of Utilities, to investigate alternative seismic upgrade schemes.

SEISMIC UPGRADE USING PASSIVE ENERGY DISSIPATION SYSTEM

The investigation of alternative schemes for seismic upgrade led the City of Sacramento to the University of California, Davis elevated water tanks which had recently completed a seismic upgrade using friction dampers [Hale et. al., 1995]. In hopes of reducing construction costs for the seismic upgrade of the Freeport Water Reservoir with a passive energy dissipation system, a specific study was undertaken using the Pall friction damper system for this tank structure. Since the passive energy dissipation system allowed the base of the tank structure to move with the ground, the expensive and time consuming work on modification of column bases and piping was not necessary. The study indicated the feasibility was valid which allowed the seismic upgrade design and construction documents to be completed for this system.

Pall Friction Dampers:

Pall friction dampers suitable for cross bracing, were selected to be the primary component to dissipate earthquake energy. A typical Pall friction-damper for the tension-only cross-bracing system for the tank is shown in Figure 2. When tension in one of the braces forces the damper to slip, it activates the four outer links to shorten simultaneously the other brace thus keeping it taut. In the next half cycle, the other brace is immediately ready to activate the damper in the other direction.

The Pall friction damper consists of a series of steel plates with slotted holes which are specially treated to develop reliable friction surfaces. These plates are clamped together with high strength bolts and are allowed to slip at a predetermined load. The friction-dampers possess rectangular loops with negligible fade over several cycles of reversals that can be encountered in successive earthquakes. The Pall friction-dampers have successfully gone through proof-testing on shake tables in Canada and the United States (Filiatrault 1986, Aiken 1988) and have found many applications in new construction and seismic upgrade of existing buildings also (Pall, 1993).

Design Criteria:

The final design criteria adopted the *NEHRP Guidelines for the Seismic Rehabilitaiton of Buildings*, FEMA 273 [BSSC, 1997] as the primary source document for the seismic upgrade. The Performance Objective agreed upon for the Freeport Water Reservoir is outlined in Table 1.

Table 1: Performance Objective

Performance Level	Level of Ground Motion
Operational	10% chance of exceedance in 50 years (475 year return period) BSE-1
Near Collapse	2% chance of exceedance in 50 years (2475 year return period) BSE-2

The operational performance level was defined as that where the reservoir continues in operation (excluding disruption from incidental and ground supported piping and valves not properly braced) with minor damage and minor disruption. The Near Collapse performance level was defined where structural collapse is prevented, but life-safety is at risk, damage is severe and a substantial loss of water is expected.

Ground Motion Input:

The site specific ground motion characterization option in FEMA 273 was utilized for this project. A detailed Site Specific Seismic Hazard Report [Kleinfelder, 1998] was completed providing response spectra and pairs of synthetic time-history records for both levels of ground motion. The ground motion records selected were scaled to the site response spectra. The synthetic time-histories were derived from the following events and stations.

- 1992 Landers earthquake, Yermo Fire Station - 360 and 270 degree components
- 1989 Loma Prieta earthquake, Agnews State Hospital - 0 and 90 degree components
- 1979 Imperial Valley earthquake, El Centro Array No. 8 - 140 and 230 degree components

The site specific acceleration at zero period was established as 0.20g for the 10% chance of exceedance in 50 years event (BSE-1) and 0.28g for the 2% chance of exceedance in 50 years event (BSE-2).
Of the three different time-histories, the one that produced the maximum response of interest was used for member verification and displacement evaluation.

Preliminary Analysis and Seismic Upgrade Schemes:

Two preliminary seismic upgrade schemes were analyzed using similar time histories to those provided by Kleinfelder, Inc. The computer program used for these nonlinear 3-D analyses was PC-ANSR [Maison, 1992]. Friction dampers, were modeled using the inelastic truss element of PC-ANSR with tension only capacity [Aiken, 1990]. The analytical hysteretic loop for the friction damper was modeled as elastic-perfectly plastic with the slip force taken as a fictitious yield force for the inelastic truss element [Aiken, 1990]. The liquid sloshing mass was explicitly modeled using methods developed by Housner [ASCE, 1984]. Of the water mass, 61% was conservatively considered impulsive and 39% convective. The sloshing period was calculated as 6.15 seconds.

The first scheme incorporated a total of 36 friction dampers in x-bracing at both levels of the outer circle of existing columns. The existing solid square braces were replaced with 6 in. diameter steel pipe bracing and a link beam added to provide moment resistance at mid-height of the column. No changes were made to the interior columns. A preliminary nonlinear analysis indicated satisfactory performance from this scheme and was therefore selected as the seismic upgrade model and scheme for further study.

The second scheme incorporated 18 friction dampers in new x-bracing in the interior circle of columns. The existing x-bracing in the outer circle of columns remained as is. This scheme has the advantage that temporary lateral bracing of the elevated tank is not necessary. However, the interior columns are heavily loaded and would require extensive strengthening at the x-bracing joints. Also, the existing valve house piping would not allow installation of x-bracing for the dampers at the bottom level in one of the nine bays. Nonlinear analyses indicated yielding of the existing braces at a lateral drift (at tank level) of approximately 84 mm (3.3 inches). The existing braces on the outermost circle of columns were found to resist most of the lateral load resulting in a possible brace or connection failure. Strengthening of the columns, braces, gusset plates, and foundations would be necessary at the outermost ring of columns with this scheme. The existing bracing also hinders the energy dissipation capability of the friction dampers. As a result, this second scheme was not selected.

Elastic Characteristics of the Final Model:

A frequency analysis of the model was carried out to understand the elastic response of the structure. The results are shown below in Table 2. The effective mass was taken as 120100 kN (27,000 kips).

Table 2: Frequency Characteristics

Mode	Period (sec.)
Sloshing	6.38
Support Structure - 1st mode	1.86

Final Nonlinear Time History Analysis:

The final analysis was carried out using orthogonal pairs of time histories, applied 100% each simultaneously, to obtain the maximum member responses and drift. The analyses included both the BSE-1 and BSE-2 time histories. The computer program PC-ANSR was also used for these 3-D nonlinear analyses. The analytical model for the final time history analysis is shown in Figure 3.

The damper slip forces were established primarily to minimize the seismic overturning forces on the columns and still keep drifts to within tolerable limits for the column to tank connection integrity. Finally, the slip forces were adjusted to maximize energy dissipated over the two levels of bracing. In consideration of all the above, the damper slip forces of 667 kN (150 kips) for the bottom x-braces and 533 kN (120 kips) for the top x-braces were established.

Friction dampers are typically not designed to be active and slip under wind loads and consideration of this is also necessary as a check on the design slip force. In this case, using the 1994 Uniform Building Code (ICBO, 1994) wind loading with a 1609 meters per hour (75 mph) wind speed, the factor of safety against wind activating the dampers was determined to be approximately 2.4.

The tank level horizontal displacement time-histories for selected directions are shown in Figures 5 and 6 for the controlling BSE-1 and BSE-2 time histories, respectively. Base shear for the tank was investigated for the synthetic time-history derived from the Loma Prieta 0 degree record. The base shear demand from the analysis was found to be 3883 kN (873 kips).

Member responses from the existing tank support framework were found to be elastic in the BSE-1 and minor to no yielding in the BSE-2 (using a material strength reduction factor, $\phi = 1.0$ for the BSE-2). Column strength evaluation at each time step was based on a P-M yield interaction surface similar to that given by the AISC LRFD [AISC, 1993] Specifications. The member capacities for the BSE-1 were calculated considering the normal material strength reduction factors for new LRFD member design. The W18x86 remains elastic under the BSE-1 and BSE-2 loading also. The Landers earthquake (Yermo Fire Station) acceleration time-histories produced the maximum member responses.

The effect of soil-structure interaction was considered by implementing soil springs using foundation stiffnesses provided in the Site Specific Seismic Hazard Report. Since the foundations bear on a hardpan soil layer at a shallow depth, the effect of the soil flexibility on the response of the structure was found to be negligible and therefore was not used for the final design parameters.

Soil bearing pressures below the spread footings, including the effects of overturning on the individual footing, were found to range from 177 kPa to 320 kPa (3.7 ksf to 6.7 ksf) for the BSE-1. The allowable ultimate soil bearing pressure was taken as 575 kPa (12 ksf) for the BSE-1 based upon the ultimate bearing pressure capacities given by the Geotechnical Report [Kleinfelder, 1994].

Final Seismic Upgrade Scheme:

The final recommended seismic upgrade scheme incorporates friction dampers in both x-bracing levels of the outermost circle of existing columns [CYS, 1998]. New components consist of the following:

- 36 Pall friction dampers in the cross bracing.

- 152 mm (6 inch) diameter extra-strong steel pipe bracing replacing the existing solid square braces.

- A supplemental moment resisting steel frame is created by using W18x86 beams at column mid-height with moment resisting connections at each end.

The existing gusset plates could be utilized without modification due to the low slip force demand on the braces. The W18x86 occurs at mid-height between the bracing levels and will also function as a horizontal strut. The W18x86 moment resisting frame beam is expected to remain elastic under the BSE-1 and BSE-2 and essentially acts as a spring to help return the tank to its near original position and to reduce column moments. Since the foundations bear on the hardpan soil and the friction dampers limit the lateral force transferred to the top of footing, seismic upgrade of the foundations was not required.

Non-structural items, such as stairs and the valve house cantilever roof portion, were modified so they did not inhibit the lateral deformation of the elevated tank and columns above the ground surface.

Construction with this scheme began in November 1998 and was complete in June 1999. The construction cost for the seismic upgrade of elevated tank with Pall friction dampers was US$ 0.737 million, compared to US$ 1.98 million for the original scheme with base isolators - a savings of more than 60 %.

Friction Damper Design Criteria:

Table 3 provides slip force and damper displacement (parallel to the pipe brace) requirements for the friction damper design. Maximum damper displacement demands are based on 1.3 times the MCE/BSE-2 level earthquake displacements in accordance with FEMA 273, Chapter 9. The design criteria was based on release of any initial compression in the braces due to filling of the tank with water. The hysteretic behavior of the prototype test damper for this project is shown in Figure 4.

Table 3: Friction Damper Design Criteria

Damper Location	**Slip Force**	**Displacement ±**	**Total Displacement**
Upper Bracing	533 kN (120 kips)	27 mm (1.06 in)	54 mm (2.12 in)
Lower Bracing	667 kN (150 kips)	43 mm (1.69 in.)	86 mm (3.38 in.)

CONCLUSIONS

The use of Pall friction dampers in very long tension-only cross bracing has shown to provide a very practical and economical solution for the seismic retrofit of elevated water tanks. The seismic response of the elevated cross-braced water tank, with friction dampers as the passive energy dissipation mechanism, was found to be equal or less than the base isolated seismic upgrade scheme.

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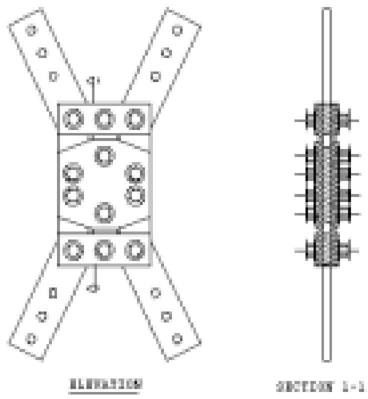
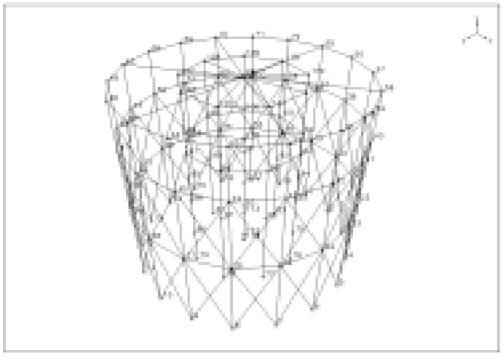


Figure 1: Elevation of retrofitted tank

Figure 2: Pall friction damper



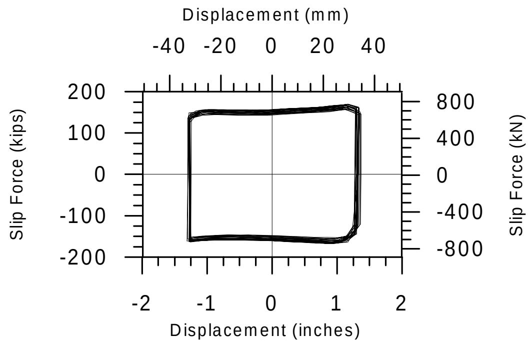


Figure 3: Analytical model of tank

Figure 4: Hysteretic loop of prototype friction damper

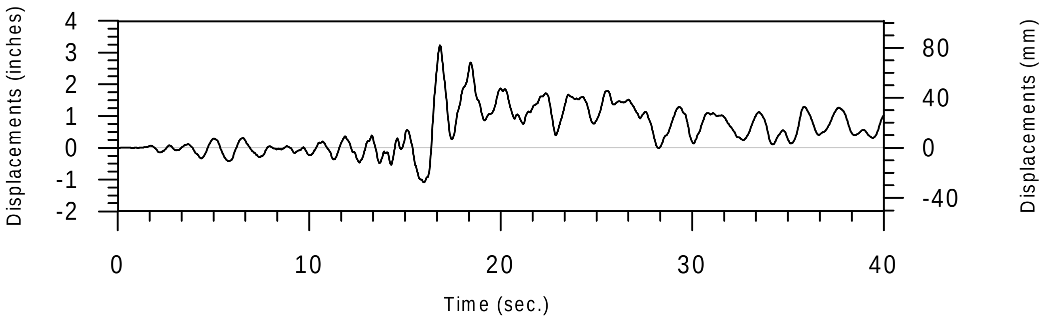


Figure 5: Horizontal displacement at tank level - Z direction - BSE-1 Landers earthquake

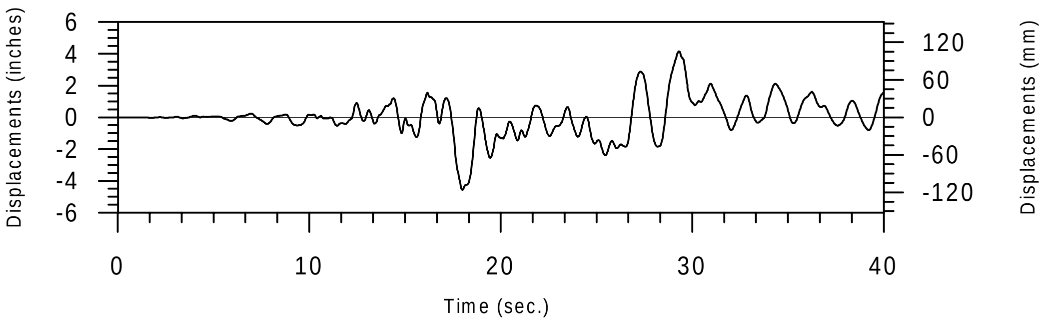


Figure 6: Horizontal displacement at tank level - X direction - BSE-2 Landers earthquake