7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Seismic Retrofit of Elevated Water Tanks at the University of California, Davis

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ABSTRACT

This paper describes the seismic evaluation and design procedure adopted for the retrofit of three steel cross-braced elevated water tanks at the University of California, Davis. Feasibility studies were made for retrofit with a conventional method and a technique of incorporating passive energy dissipators in the cross bracing. Three dimensional nonlinear time-history dynamic analysis was adopted for retrofit with passive energy dissipators. The comparative studies indicated superior technical performance of the alternative scheme using energy dissipators. The introduction of supplemental damping provided by the energy dissipators resulted in considerable reduction in forces, accelerations and overturning moments. Therefore, the expensive and time consuming procedure of strengthening columns and foundations was not necessary. Pall friction-dampers, which integrate well with the original tension-only x-bracing system, were chosen as they provided the most efficient and economical design solution. The use of this new technique resulted in an estimated 25% savings in retrofit construction cost.

INTRODUCTION

In recognition of the importance of its water supply after a major seismic event and reports of disturbing lateral movements of one of its tanks during the Loma Prieta earthquake of 1989, the University of California, Davis initiated a seismic retrofit study of its elevated water tanks.

The original tank manufacturers were contracted to provide seismic analysis of their towers at the University of California, Davis. The analysis criteria selected was the 1992 California Building Code - Title 24 (CBC) (ICBO 1992) for non-building structures. The equivalent static lateral force procedure for earthquake design was used by the manufacturers for evaluation. A supplemental equivalent static lateral force procedure was undertaken using Sec. 2338(d)3 of the CBC which allows the use of a national standard, but not less than 80% of the CBC base shear. The national standard used was the American Water Works Association Standard D100-84 (Section 13).

Using the above criteria, retrofit of all components of the support framework: braces, horizontal struts, columns, column anchorage including strengthening of foundations, was necessary to accommodate the higher earthquake overturning forces. An earthquake force reduction scheme using a permanent reduction in water level to 60 % capacity was considered. However, the estimated cost savings in the tank retrofits

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would not offset the additional pumping stations needed to assure a continuous water supply and adequate pressure.

The construction cost estimate for the conventional retrofit using steel pipe diagonal bracing and struts, column strengthening (including anchorage), and additional foundations for all three elevated tanks was approximately US$1.3 million.

As a limited amount of funds were available for seismic retrofit, alternative methods to the conventional retrofit were explored. The 1992 CBC allows force reduction systems of seismic isolation, energy dissipation and damping in accordance with Sec. 2333(j)2 when approved by the Building Official.

Base isolation was considered, but the difficulties and expense of isolating the riser, providing a rigid diaphragm level above the isolators, installing the isolators and developing reliable uplift capacity in the isolators deemed its use not feasible or economical for this tank application. Passive energy dissipation as a means of earthquake force reduction was selected as the most appropriate system for this retrofit.

Final construction documents for the retrofit of the three elevated water tanks using Pall friction-dampers were completed in 1994. Construction of the retrofit is expected to take place in the second half of 1995.

DESCRIPTION OF ELEVATED TANKS

The three elevated water tanks consist of two 100,000 gallon and one 200,000 gallon steel tanks all supported approximately 120 feet (37 m) above ground level. Column anchorage to the foundations consisted of two 1-3/4 inch (44 mm) diameter anchor bolts. The 200,000 gallon tank column anchorage also included shear lugs. The foundation system is drilled belled piers tied together at the top with grade beams. The tanks were originally designed and constructed between 1958 and 1967.

The 200,000 gallon tank (Figure 1a) is 36 feet (1097 cm) in diameter supported by six inclined columns arranged in a hexagonal pattern. The support system consists of 22 inch (559 mm) x 0.3125 inch (8 mm) thick circular steel tube columns, built-up channel struts and 1-3/8 inch (35 mm) diameter steel rod x-bracing.

The 100,000 gallon tanks (Figure 1b) are 30 feet (914 cm) in diameter supported by four inclined columns. The support system consists of 24 inch (610 mm) diameter x 0.28 inch (7 mm) thick circular steel tube columns, built-up channel struts and 1-1/2 inch (38 mm) diameter rod x-bracing.

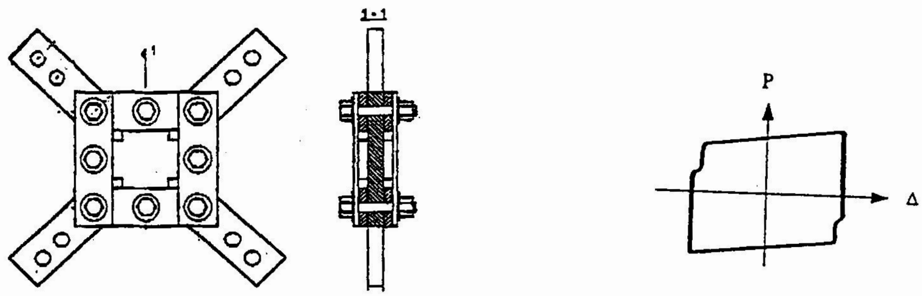
RETROFIT USING PASSIVE ENERGY DISSIPATION SYSTEM

Selection of Damping Device

A damper device which integrates into a tension-only x-bracing system (similiar to the original bracing system) is a natural evolution of this structural system. Diagonal brace lengths of 40 to 60 feet (1220 cm to 1830 cm) occuring in these towers are not conducive to dampers requiring compression braces to function. Tension-only bracing without a mechanism to insure a continual taut brace under shortening results in undesirable behavior and inefficiency in the dissipation system. The Pall friction-damper was selected for this retrofit since it best satisfies all criteria for this

(a) 200,000 Gallon (b) 100,000 Gallon

FIGURE 1. EXISTING ELEVATED WATER TANKS



(a) Friction-damper (b) Hysteretic Behavior

FIGURE 2. PALL FRICTION-DAMPER

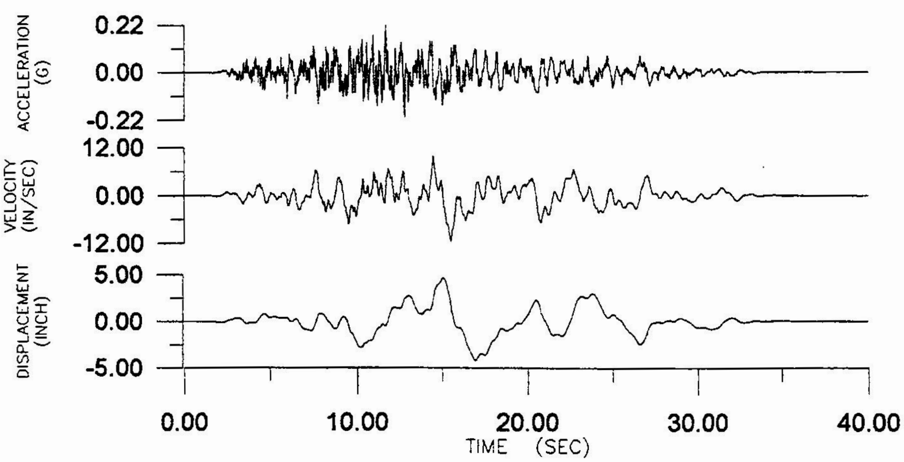


FIGURE 3. SYNTHETIC TIME-HISTORY, LOMA PRIETA, AGNEWS STATE HOSP. STATION.

particular structural system.

Pall Friction-Dampers

These friction-dampers are simple and fool proof in construction (Pall 1982). Their performance is reliable and repeatable. They basically consist of a series of steel plates with slotted holes which are specially treated to develop reliable friction. These plates are clamped together with high strength bolts and allowed to slip at a predetermined load. The friction-dampers possess large rectangular loops with negligible fade over several cycles of reversals that can be encountered in successive earthquakes. A much greater quantity of energy can be disposed of in friction than in any other method involving the yielding of steel plates or viscoelastic materials. The performance of friction-dampers is not affected by temperature, velocity or stiffness degradation due to aging. Friction-dampers do not need replacement after an earthquake. They need no maintenance and are always ready to be activated when required, irrespective of how many times they have performed. The friction-dampers are not designed to slip during wind loading. During a major earthquake, they slip at a predetermined load before yielding of the structural members. After the earthquake, the strain energy of the framed structure brings the dampers back to their near original alignment.

These friction-dampers have successfully gone through rigorous proof-testing on shake tables in Canada and the United States (Filiatrault 1986, Aiken 1988). Patented Pall friction-dampers are available for single diagonal, Chevron-Brace and tension-only cross bracing systems. These friction-dampers have found several applications in new construction and retrofit of existing buildings (Pall 1987, Pall 1991, Vezina 1992, Pall 1993, Pasquin 1993, Wagner 1995, Savard 1995, Godin 1995) and several others under construction.

A typical friction-damper for tension-only x-bracing system for the tanks is shown in Figure 2a. 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 dissipate energy. The hysteretic behavior of the damper is shown in Figure 2b.

Design Criteria

The Tentative Seismic Design Requirements for Passive Energy Dissipation Systems (SEAONC 1993) was used as the primary design criteria. The design provisions of this document relate to the allowable stress design methodology of the CBC. A LRFD steel member check in conjunction with the 1994 NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings (BSSC 1994) under non-building structures was adopted as more applicable to the limit state analysis using the nonlinear time-history dynamic analysis.

Ground Motion Input

Detailed site specific ground motion criteria was established in accordance with the Tentative Seismic Design Requirements for Passive Energy Dissipation Systems (Woodward-Clyde Consultants 1994). Response spectra and pairs of synthetic earthquake time-history records were generated for an event with a 10% exceedance in 50 years (Design Basis Earthquake), five percent damping and a base acceleration of 0.22g. The synthetic time-histories were scaled to match the Design Basis Earthquake response spectrum. They were derived from the following events and stations: Loma Prieta, Agnews State Hospital Sta., orientation 0/90; Imperial Valley, 1979, El Centro No. 1 Sta., orientation 140/230; Loma Prieta, Gilroy No.2 Sta., orientation 0/90. One synthetic time-history record is given in Figure 3.

Three-Dimensional Nonlinear Time-History Dynamic Analysis

The SADSAP computer program (Wilson 1994) was mainly used for the analysis. This program performs three-dimensional linear elastic analysis with the capability of performing nonlinear dynamic analysis using a limited number of predefined nonlinear elements. Locations of yielding or other nonlinearity must be known by the user in order for the structure to be analyzed. Ritz direction vectors are applied by the user to drive the modal analysis and recover the effect of the nonlinear element forces on the adjacent linear elastic members. SADSAP allows rapid solution of structures with nonlinear elements that contain a large number of degrees of freedom.

The linear tower framework was modeled with beam elements as shown in Figures 4a and 4b. The SADSAP plasticity element with a very low compression force capacity and elastic-perfectly-plastic axial deformations was used as x-bracing to model the friction damper system. A force-deformation hysteresis loop of the plasticity element modeled in SADSAP is shown in Figure 6. Tension yield of the plasticity element was taken as the slip force in the damper. Continuity between new wide flange struts and the existing columns was provided to form a supplemental moment resisting frame (MRF). This dual lateral force resisting system or friction braced moment resisting frame is recommended by the Tentative Seismic Design Requirements for Passive Energy Dissipation Systems for rate-independent dampers. The riser containing water was explicitly modeled due to its significant mass. A detailed study indicated that sloshing must be considered for tanks of this size. Impulsive and convective masses and the sloshing period were determined using Housner's equations (ASCE 1984). Spring stiffness from the convective mass to the structure was determined and entered into the model. Superposition of ground motion components was taken into account by scaling non-primary directions to 30% (Rosenblueth 1980) and simultaneously exciting the base in all three directions.

A three dimensional inelastic time-history dynamic analysis computer program DRAIN-TABS (Powell 1977), using a similiar model to that used in SADSAP, was employed to check results from SADSAP.

Results of Analysis

1. Sloshing effects on the tanks, when compared with water mass assumed as rigid, significantly increased the overturning moments on the support framework.

2. The minimization of overturning forces on the columns, which were critical for the stability of the tanks, governed the optimization of the friction-damper slip load. For all three tanks, the optimum slip forces from top to bottom tier were 35 kips - 30 kips - 25 kips (156 kN - 133 kN - 111 kN) for each damper. This graduated slip force level used to achieve optimum damper/structure behavior has also been recognized by other studies (Aiken 1990). However, final slip force recommendations revised the bottom tier from 25 kips (111 kN) to 30 kips (133 kN) due to considerations of infrequent high wind loading. In all, 24 dampers for the 200,000 gallon tank and 12 dampers for each 100,000 gallon tank were used to achieve the response reductions.

3. The maximum base shear was significantly reduced for tanks incorporating friction-dampers. For friction damped tanks, the base shears were 113 kips (500 kN)

(a) 200,000 Gallon Tank (b) 100,000 Gallon Tank

FIGURE 4. COMPUTER MODELS OF ELEVATED TANK SUPPORTS

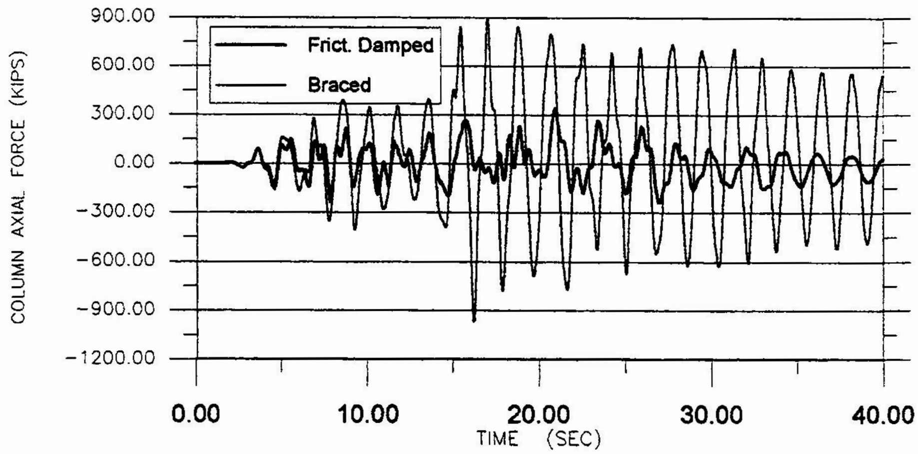


FIGURE 5. COLUMN AXIAL LOAD OF 200,000 GALLON TANK SUPPORT.

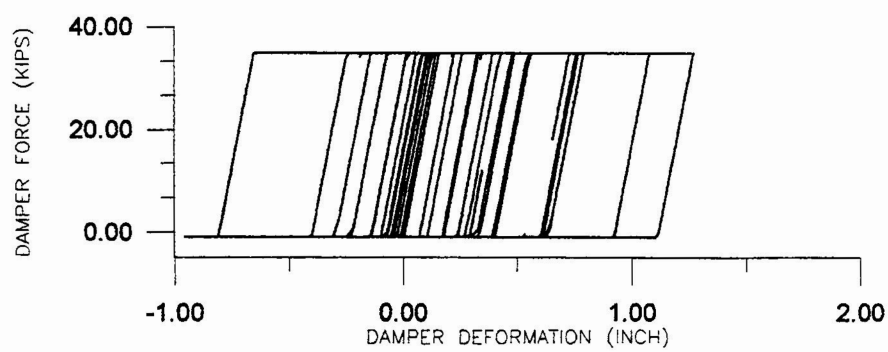


FIGURE 6. ANALYTICAL FORCE-DISPLACEMENT LOOP OF DAMPER.

and 60 kips (267 kN) for the 200,000 and 100,000 gallon tanks respectively. For the equivalent static lateral force procedure, the base shears were 300 kips (1335 kN) and 134 kips (600 kN) respectively for the conventional retrofit.

4. A time-history of column axial loading is given in Figure 5 for the 200,000 gallon tank. A comparison is made in that figure using a conventional retrofit tension-only bracing model. This illustrates that a substantial reduction in column loads has occurred in the friction-damped tanks. This reduction varied from 30% to 300% depending on the tower and the time-history record.

5. While the maximum force in a brace with the friction-damper was limited to the slip load of the damper, the forces in an ordinary brace of the conventional retrofit were much higher and varied with the tank and time-history record. Maximum diagonal bracing forces for the conventional retrofit were 255 kips (1134 kN) and 135 kips (600 kN) for the 200,000 and 100,000 gallon tanks respectively under the Loma Prieta, Agnews State Hospital Sta., orientation 0 record.

6. The DRAIN-TABS analysis results were reasonably close to the SADSAP analysis results and confirmed that the use of linearly elastic beams and columns was valid. Since the plasticity element of SADSAP does not allow compression yielding without buckling, the energy dissipated from this element does not reflect the true energy dissipation capability of the Pall friction-damper. Therefore, column forces from the SADSAP analysis are considered to be the upper bound.

7. Using the earthquake record which produces the maximum response, column demand/capacity ratios varied from 1.1 to 1.4. The beam demand/capacity ratios were less than one. The 1994 NEHRP provisions for non-building structures allow an R = 2 or demand/capacity of 2. Since the DRAIN-TABS model indicated essentially no inelastic activity, these demand/capacity ratios were accepted.

CONCLUSIONS

The seismic response of the elevated cross-braced water tanks, when using the Pall friction-device, was found to be significantly less than when using the conventional retrofit. This reduced response did not require the strengthening of columns and foundations as did the conventional retrofit. Estimated construction cost of the seismic retrofit using friction dampers was approximately US$0.98 million which resulted in significant saving over the conventional retrofit estimate of US$1.3 million.

ACKNOWLEDGMENT

Chris V. Tokas and Tom H. Hale wish to thank Cole/Yee/Schubert & Associates, Sacramento, California for the opportunity to work on the seismic retrofit design of the elevated tanks.

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