USE OF FRICTION DAMPERS ON ELEVATED WATER TANKS

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SUMMARY

This paper summarizes the seismic evaluation and retrofit design of the Renton Highlands and Rolling Hills elevated water storage tanks utilizing friction dampers. Due to the age, use, and recent earthquake damage of each tank, comprehensive seismic evaluations were performed to assess their earthquake vulnerability and options for performance-based seismic retrofit design enhancements. A site-specific geotechnical investigation was also performed to analyze near-surface conditions as well as physical and engineering characteristics of the soils.

Seismic design force levels from various design standards from the American Water Works Association (AWWA D100), International Conference of Building Officials (97 UBC) and Federal Emergency Management Association (FEMA 356) were compared and used for the seismic evaluations and subsequent concept retrofit designs. Three-dimensional nonlinear finite element models were used with the various standards to evaluate different combinations of hazard levels, performance objectives, and analytical methods.

The evaluation determined that both elevated water tanks had significant seismic deficiencies to critical elements, including the steel columns, diagonal braces, and horizontal struts. To achieve a higher level of performance, passive energy dissipation was evaluated using site-specific time history records for both 475- and 2475-year seismic events. Costs to retrofit the tanks using conventional strengthening and stiffening methods were determined to be approximately 50 to 80 percent of the cost to replace the tanks. Retrofitting the elevated tanks using energy dissipation (friction dampers) provides the highest level of performance and costs approximately 35 percent of the tank replacement cost. Retrofit design and construction using passive energy dissipation devices and FEMA 356 performance-based criteria was performed.

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INTRODUCTION

On February 28, 2001, a magnitude 6.8 earthquake occurred, centered approximately 37 miles south of Renton, between Olympia and Tacoma, Washington. The Rolling Hills and Highlands elevated water tanks in the City of Renton each sustained moderate amounts of damage. Due to the age, use, and observed earthquake damage, comprehensive seismic evaluations of the elevated water tanks were performed to assess their condition and to gain a better understanding of their earthquake vulnerability.

The structural evaluation involved analyzing the existing tanks and determining concept retrofit alternatives under several design standards, each with a differing combination of hazard level and performance objective. A geotechnical evaluation was also performed to confirm near-surface conditions and evaluate physical and engineering characteristics of the soils.

DESCRIPTION OF ELEVATED WATER TANKS

Highlands 750,000 Gallon Tank
The Highlands Reservoir was constructed in 1960 and has a capacity of 750,000 gallons with the top of water level at 135 feet above grade. The welded steel tank is spheroidal in shape, 66'-6" in diameter and 42'-6" in height at its center. The tank is supported by a 72-inch diameter riser column and eight 48-inch diameter pipe columns equally spaced around the perimeter at a 32'-3" radius. The column wall thickness varies from 0.375 inches at the top to 0.406 inches in the middle. The wall thickness for the lower column could not be determined from the drawings, but was assumed to be similar to the middle section.

The diagonal tension-only braces occur in three tier heights with each brace approximately 45 feet long. The braces consist of 6-inch by 2-inch steel flat bar and are welded to steel gusset plates at the pipe column connection. The horizontal struts between the bracing levels consist of built-up channel sections. Both the upper and lower struts consist of a vertical C9x13.4 and a horizontal C12x20.7, stitch-welded together. The columns are each supported by a 4-foot high, 5'-9" square pier that bears on a 2-foot deep, 13'-3" square footing. The 6-foot high pier and footing bears 5 feet below grade.

Figure 1. Highlands Reservoir.

Figure 2. Rolling Hills Reservoir.

Rolling Hills 300,000 Gallon Tank
The Rolling Hills Reservoir was constructed in 1970 and has a capacity of 300,000 gallons with the top of water level at 115 feet above grade. The tank is torospherical in shape, 45'-0" in diameter and 30'-0" in height at its center. The tank is supported by a 48-inch diameter center column and five 36-inch diameter steel pipe columns equally spaced around the perimeter at a radius of 28'-0". Each perimeter column has a constant wall thickness of 0.406 inches.

The diagonal tension-only braces occur in three tier heights and each brace averages 48'-6" in length. The braces consist of 3 1/2-inch wide by 1 1/4-inch thick steel flat bar and are welded to steel gusset plates at the column connections. Both the upper and lower struts consist of a vertical C10x15.3 and a horizontal C12x20.7, stitch-welded together. The columns are each supported by a 3'-5" high, 6'-0" square pier that bears on a 2-foot deep, 13'-0" square footing.

NISQUALLY EARTHQUAKE DAMAGE ASSESSMENT

Following the Nisqually Earthquake, the two tanks were inspected for damage. Overall damage consisted of spalled base plate grout at various locations, sagging and kinked radial rods, and cracks in paint film along the attachment welds. Additionally, each tank had the following earthquake damage:

Highlands 750,000 Gallon Tank
42 of the 48 "strap plate" braces were yielded and partially buckled and showed signs of structural overload. The buckling was not as severe as the braces on the Rolling Hills tank, but it was recommended that all braces be replaced. Of the yielded braces, 6 braces were bowed between 6 and 12 inches and 36 braces were bowed less than 6 inches.

Rolling Hills 300,000 Gallon Tank
21 of the 30 "strap plate" braces were yielded, resulting in 8 braces to be bowed between 6 and 8 inches and 13 (8 observed plus 5 presumed) braces to have moderate bowing of approximately 2 to 3 inches; the remaining 9 braces (3 observed and 6 presumed) did not have a significant bow.

Due to the excessive number of buckled diagonal braces, a seismic repair and rehabilitation scheme was implemented to restore lateral capacity to the essential structures.

SEISMIC DESIGN STANDARDS AND PERFORMANCE CRITERIA

Seismic rehabilitation design of existing structures can be performed with numerous standards and criteria, customized to suit the specific project requirements. The following documents were used for the evaluation and concept retrofit design of the two elevated water tanks:

- American Water Works Association (AWWA) D100-96
- 1997 Uniform Building Code (UBC)
- FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings

A Demand/Capacity Ratio (DCR) approach was used as the acceptance criteria. The demand is determined by distributing the seismic forces to the various elements based on their weight, stiffness, and ductility. Stiffness is the force-displacement characteristic of an element and ductility is a measure of its ability to deform past its elastic limit. The capacity is the strength of the element. Elements with a DCR greater than 1.0 indicate that the element may not meet the specified performance objective.

SEISMIC EVALUATION AND ANALYSIS METHODS

Seismic evaluations of the Highlands and Rolling Hills water tanks were performed to determine the most appropriate design approach and associated costs. Three dimensional finite element models were developed in SAP 2000^{TM} (Computers and Structures, Inc.) using the existing member sizes and assumed material properties. Five analyses were performed for each tank using different design standards:

- Linear static procedure using AWWA D100-96, Zone 3
- Linear static procedure using 1997 UBC, Zone 3
- Linear static procedure using 1997 UBC, Zone 4
- Linear dynamic procedure (site-specific spectra) using 1997 UBC
- Linear dynamic procedure (site-specific spectra) using FEMA 356

Under the linear static procedure, the structure is modeled with linearly-elastic stiffness and equivalent viscous damping. Design earthquake demands are represented by static lateral forces whose sum is equal to the seismic base shear. The basis, modeling approaches, and acceptance criteria of the linear dynamic procedure are similar to that for the static procedure. The main exception is that the response calculations are carried out using either modal spectral analysis or time-history analysis. Corresponding base shears using each of the previously noted standards were determined and the following structural components of the tank were evaluated:

- *Columns*, the critical vertical-load carrying components, were evaluated at all levels under combined flexural and axial loading.
- *Braces*, the main components of the lateral-force resisting system that provide stiffness and strength to the structure, were evaluated at all tiers under tension force demands.
- *Struts*, the horizontal elements that provide lateral stability for the columns, were evaluated at the upper and lower levels for compression and lateral torsional buckling.
- *Reactions* (maximum compression, uplift, and shear) at the base of the columns were also evaluated.

SUBSURFACE CONDITIONS AND SEISMICITY

The subsurface conditions at the Highlands and Rolling Hills sites were evaluated by drilling borings to depths of 75 feet. Subsurface conditions at each site were similar and consisted of 3 to 5 feet of medium dense granular fill overlying glacially consolidated soils. The glacial soils consist of about 25 feet of very dense silty sand with gravel (glacial till) overlying very dense sand and gravel (advance outwash).

Seismicity in the Puget Sound area is primarily driven by the Cascadia Subduction Zone (CSZ) - the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. Three potential seismic source zones are generally acknowledged for the Puget Sound area:

1. Shallow Crustal Earthquakes. The closest presumable active fault is the Seattle Fault, situated roughly 4 miles north of the Highlands site and approximately 7 miles north of the Rolling Hills site.

2. Intraplate Earthquakes. CSZ intraplate earthquakes occur within the subducting Juan de Fuca Plate at depths of 25 to 37 miles within the Puget Sound area. The Olympia 1949 (M=7.1), Seattle 1965 (M=6.5), and Nisqually 2001 (M=6.8) earthquakes are considered to be intraplate earthquakes.

3. Interplate Earthquakes. CSZ interplate earthquakes occur along the interface between the Juan de Fuca and North American Plates. Although there are currently no recorded seismic events of large interplate earthquakes for the CSZ, this source is characterized as being capable of generating earthquakes of large magnitudes (M=8 to M=9).

SEISMIC EVALUATION RESULTS RELATIVE TO DIFFERENT DESIGN CRITERIA

Highlands 750,000 Gallon Tank
The weight of the stored water and support tank structure is approximately 6700 kips. Using the existing framing members, the fundamental period of the structure was determined to be 1.7 seconds.

AWWA D100-96, Zone 3: A seismic base shear of 0.16W, or 1085 kips, was used for analysis. The upper columns were found to have a DCR of 1.28 (i.e., 28 percent overstressed) due to combined bending and axial compression stresses. Due to their thicker wall sections, the middle and lower columns were determined to have adequate strength. The diagonal braces in all tiers were overstressed in tension. The top, middle, and lower tier braces have DCRs of 1.30, 1.56 and 1.63, respectively. The upper and lower struts have DCRs of 1.41 and 1.64, respectively. The overturning moment at the base of the tank resulted in a maximum downward vertical reaction of 1265 kips and a maximum shear of 271 kips. There is no uplift at the base of the columns.

1997 UBC, Zone 3: The nonbuilding base shear equation controlled for the 97 UBC Zone 3 analysis. The 0.23W limit state force was converted to a working stress level (by dividing by 1.4), resulting in a base shear of 0.17W, or 1140 kips. The upper columns were found to have a DCR of 1.31 while the middle and lower columns were determined to have adequate strength. The top, middle, and lower tier braces have DCRs of 1.34, 1.63, and 1.69, respectively. The upper and lower struts have DCRs of 1.48 and 1.72, respectively. The base overturning moment resulted in a maximum downward vertical reaction of 1290 kips and a maximum shear of 284 kips. There is no uplift at the base of the columns.

1997 UBC, Zone 4: The nonbuilding base shear equation also controlled for the Zone 4 analysis. Converting the 0.41W limit state force to a working stress level resulted in a base shear of 0.29W, or 1943 kips. The upper columns were found to have a DCR of 1.74, the middle columns were adequate, and the lower columns have a DCR of 1.15. The top, middle, and lower tier braces have DCRs of 2.20, 2.60, and 2.70, respectively. The upper and lower struts have DCRs of 2.49 and 2.88, respectively. The base overturning moment resulted in a maximum vertical reaction of 1667 kips and a maximum shear of 478 kips. There is no uplift at the base of the columns.

1997 UBC, Site Specific Spectra: A seismic base shear (working stress level) of 0.15W, or 1005 kips, was determined using the site-specific spectral analysis for a 10 percent/50-year event, assuming 5 percent damping. The spectra was increased by 1.25 (importance factor for essential facilities) and decreased by a factor of 2.2 (ductility factor). The calculated base shear is 88 percent of the Zone 3 static base shear and 52 percent of the Zone 4 static base shear. The upper columns were found to have a 1.23 DCR while the middle and lower columns were determined to have adequate strength. The top, middle, and lower tier braces have DCRs of 1.20, 1.48, and 1.47, respectively. The upper and lower struts have DCRs of 1.33 and 1.57, respectively. The base overturning moment resulted in a maximum vertical reaction of 1275 kips and a maximum shear of 258 kips. There is no uplift at the base of the columns.

FEMA 356: Site specific response spectra was also used for the FEMA 356 performance-based analysis. The 5-percent damped spectrum corresponding with a 10 percent/50-year event was not scaled by a response modification factor, nor was it scaled to a working stress level. Two modes of vibration were required to capture at least 90 percent of the participating mass of the reservoir in each of the principal horizontal directions. The peak member forces, displacements, and base reactions for each mode of response were combined by the SRSS (square root sum of squares) method.

The columns, braces, and struts were analyzed as either force- or deformation-controlled elements. While the middle and lower portions of the columns met the Life-Safety (LS) performance objective, the upper portion of the columns did not meet Collapse Prevention (CP) requirements under combined compression and flexural stresses. The diagonal braces on the upper tier met LS criteria for tensile capacity; however, the middle and lower tier braces only met CP criteria. The upper and lower struts did not meet CP performance criteria.

The overturning moment at the base of the tank resulted in a maximum downward vertical reaction of 1650 kips and a maximum shear of 453 kips. There is no uplift at the base of the columns. The column reactions were not factored down to account for ductility, as recommended for Immediate Occupancy (IO) performance.

Table 1 is a summary of the seismic base shears calculated for the Highlands 750,000 Gallon water tank based on the different standard documents:

Table 1. Seismic Base Shear, Highlands 750,000 Gallon Tank

Standard	AWWA Zone 3	97 UBC Zone 3	97 UBC Zone 4	UBC Site Spectra
Base	0.16W	0.17W	0.29W	0.15W
Shear	1085 k	1140 k	1943 k	1005 k

Table 2 summarizes the DCRs for the different elements of the Highlands tank under the various analytical methods and standards. Also included is the expected performance level of the existing tank using FEMA 356 methodologies. The maximum column reactions under each standard are also shown.

Table 2. DCRs & Performance, Highlands 750,000 Gallon Tank

	Demand Capacity Ratio (DCR)				Expected Performance*
Element	AWWA Zone 3	97 UBC Zone 3	97 UBC Zone 4	UBC Site Spectra	FEMA 356
Columns:					
Upper	1.28	1.31	1.74	1.23	< CP
Middle	0.72	0.73	0.90	0.71	LS
Lower	0.87	0.89	1.15	0.86	LS
Braces:					
Upper	1.30	1.34	2.20	1.20	LS
Middle	1.56	1.63	2.60	1.48	CP
Lower	1.63	1.69	2.70	1.47	CP
Struts:					
Upper	1.41	1.48	2.49	1.33	< CP
Lower	1.64	1.72	2.88	1.57	< CP
Reactions:					
Downward	1265 k	1290 k	1667 k	1275 k	1650 k
Uplift	0	0	0	0	0
Shear	271 k	284 k	478 k	258 k	453 k

*The following abbreviations were used: < CP = does not meet Collapse Prevention Level; CP = meets Collapse Prevention Level; LS = meets Life-Safety Level.

Rolling Hills 300,000 Gallon Tank
A seismic evaluation of the Rolling Hills tank was performed in a similar approach to that of the Highlands tank. The weight of the stored water and existing support structure is approximately 2730 kips. The period of the structure was determined to be 2.1 seconds. Table 3 is a summary of the seismic base shears calculated for the Rolling Hills tank based on the different standard documents:

Table 3. Seismic Base Shear, Rolling Hills 300,000 Gallon Tank

Standard	AWWA Zone 3	97 UBC Zone 3	97 UBC Zone 4	UBC Site Spectra
Base	0.09W	0.17W	0.26W	0.17W
Shear	246 k	464 k	710 k	453 k

Table 4 summarizes the DCRs for the different elements of the Rolling Hills tank under the various analytical methods and standards. Also included is the expected performance level of the existing tank using FEMA 356 methodologies. The maximum column reactions under each standard are also shown.

Table 4. DCRs and Performance, Rolling Hills 300,000 Gallon Tank

	Demand Capacity Ratio (DCR)				Expected Performance*
Element	AWWA Zone 3	97 UBC Zone 3	97 UBC Zone 4	UBC Site Spectra	FEMA 356
Columns:					
Upper	0.71	0.89	1.10	0.98	LS
Middle	0.79	1.04	1.33	1.04	LS
Lower	0.84	1.15	1.50	1.12	CP
Braces:					
Upper	1.47	2.44	3.53	2.20	< CP
Middle	1.40	2.21	3.13	2.10	< CP
Lower	1.40	2.10	2.90	2.00	< CP
Struts:					
Upper	0.54	1.04	1.60	1.05	CP
Lower	0.63	1.43	2.17	1.40	< CP
Reactions:					
Downward	974 k	1312 k	1690 k	1399 k	2012 k
Uplift	0	271 k	668 k	273 k	884 k
Shear	40 k	134 k	202 k	255 k	341 k

*The following abbreviations were used: < CP = does not meet Collapse Prevention Level; CP = meets Collapse Prevention Level; LS = meets Life-Safety Level.

CONCEPT SEISMIC RETROFIT DESIGN ALTERNATIVES

Several conceptual retrofit designs were prepared for both the Highlands and Rolling Hills elevated water tanks utilizing different methodologies. Conventional stiffening and strengthening schemes were developed using Zone 3 and 4 criteria from the 1997 UBC. Linear static analyses were performed on the finite element models with assumptions identical to those used in the evaluation phase. The columns, braces, and struts found to be overstressed were typically replaced with larger sections.
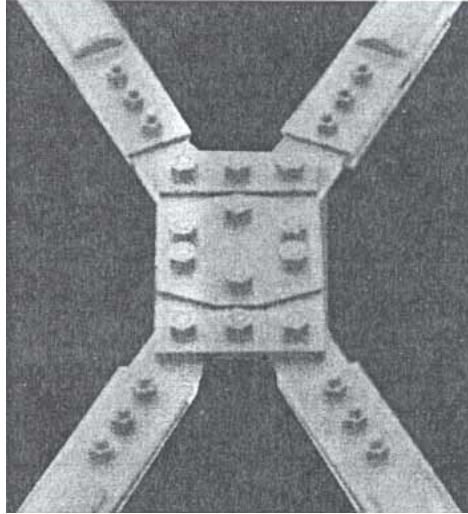
Figure 3. Friction Damper (courtesy of Pall Dynamics, Inc.).

Energy dissipation was evaluated as an alternative approach to the conventional strengthening schemes. Passive energy dissipation is an emerging technology that enhances the seismic performance of the structure by adding damping. The primary use of energy dissipation devices is to reduce earthquake displacement and the corresponding internal forces of the structure. Friction dampers suitable for cross bracing were evaluated to be the primary component to dissipate earthquake energy. The friction damper consists of a series of steel plates that are treated to develop reliable friction surfaces. The plates are mated together with high strength bolts in slotted holes and are allowed to slip at a predetermined load. Refer to Figure 3.

When tension in one of the braces forces the damper to slip, the four outer links are activated, shortening the other brace and keeping it taut. In the next half cycle, the other brace resists forces in tension and activates the damper in the other direction. The friction dampers are displacement-dependent devices that possess rectangular (rigid-plastic) hysteretic loops with negligible fade over several cycles of reversals. The response of friction dampers have an added benefit of being independent of velocity and/or frequency of excitation.

Three-dimensional nonlinear dynamic analyses were performed using site-specific time history earthquake records. The 1974 and 1975 Alaska and the 2001 Nisqually Earthquake records, scaled to 10 percent and 2 percent probabilities of exceedance in 50 years, were used for analysis. Multidirectional excitation effects were accounted for by analyzing the model using simultaneously imposed pairs of earthquake ground motion records along each of the primary horizontal axes of the structure.

Seismic isolation systems were also considered, but not pursued as an option. Typical isolation systems reduce forces transmitted to the superstructure by decoupling the superstructure from the ground, lengthening the period of the structure, and adding some amount of damping. Base isolation is ideal for short, squat buildings, with a low fundamental period of vibration. Since the natural fundamental periods of the two elevated tanks were moderately long, and since potential uplift demands on isolators can reduce their reliability, base isolation was not determined to be a viable alternative.

Highlands 750,000 Gallon Tank
Under UBC criteria, the period of the stiffer retrofitted structure was reduced from 1.7 to 1.2 seconds. Assumptions regarding tank capacity, sloshing, orthogonal effects, vertical load distribution, and member properties and end conditions remained the same as the evaluation phase.

To meet UBC Zone 3 criteria, a 3/8-inch thick steel jacket is required around the upper portion of all columns. Since the middle and lower portions of the columns have adequate capacity with their thicker wall sections, only the upper columns need to be strengthened. To meet UBC Zone 4 seismic criteria, concrete would need to be pumped into the steel pipe columns. The concrete is required for the full height of the column to provide adequate compression capacity. Thru-bolts, headed studs, embedded plates, or other connection devices would be required to provide necessary shear transfer between the steel column and the concrete infill.

All existing braces and struts need to be removed and replaced with larger sections under both Zone 3 and 4 criteria. Preliminary design suggests that 10-inch square tube steel sections are required for the braces and struts for Zone 3; 14-inch sections are required for Zone 4. Although tube steel was used for the new members, solid plate, steel pipe, or two pairs of back-to-back angles in a cruciform configuration could also be used. Based on the geotechnical assessment, the foundations need to be retrofitted for additional shear resistance capacity under UBC Zone 4 criteria. Additional shear capacity can be achieved either by increasing the area of the footing base or providing supplemental piles. Table 5 summarizes the retrofit requirements for the Highlands water tank under Zone 3 and Zone 4 UBC criteria.

Table 5. UBC Retrofit, Highlands 750,000 Gallon Tank

Element	Existing Members	Proposed Retrofit	
		97 UBC Zone 3	97 UBC Zone 4
Columns:			
Upper	T = 0.375"	t = 0.75" (add 3/8" sleeve)	Fill with concrete
Middle	T = 0.406"	Unchanged	Fill with concrete
Lower	T = 0.406"	Unchanged	Fill with concrete
Braces:			
Upper	2"x6" (As = 12 sq. in)	TS 10x10x3/8 (As = 14.1 sq. in)	TS 14x14x1/2 (As = 26.4 sq. in)
Middle	2"x6" (As = 12 sq. in)	TS 10x10x1/2 (As = 18.4 sq. in)	TS 14x14x5/8 (As = 32.4 sq. in)
Lower	2"x6" (As = 12 sq. in)	TS 10x10x1/2 (As = 18.4 sq. in)	TS 14x14x5/8 (As = 32.4 sq. in)
Struts:			
Upper	C9x13.4 w/ C12x20.7	TS 10x10x3/8	TS 14x14x3/8
Lower	C9x13.4 w/ C12x20.7	TS 10x10x3/8	TS 14x14x3/8

For the FEMA 356 alternative utilizing passive energy dissipation, 70-kip dampers are required at the intersection of all braces in all bays for the full height of the tank. The 24 friction dampers limit the drift in the structure to 5.6 inches (0.39 percent) for a 10 percent/50-year event and 8.6 inches (0.61 percent) for a 2 percent/50-year event. The 0.39 percent drift meets the IO criteria for the 10 percent/50-year event, and the 0.61 percent drift meets LS criteria for the 2 percent/50-year event. Based on the preliminary analysis, the existing columns and struts meet acceptable criteria for IO and LS performance for the 10

percent and 2 percent/50-year events, respectively, and need not be replaced. The existing undamaged 2-inch by 6-inch braces may be reused. Due to the amount of energy dissipation, the existing foundations have adequate bearing capacity and frictional resistance to sliding and are not subjected to any uplift forces.

Rolling Hills 300,000 Gallon Tank
Retrofit requirements were also determined for the Rolling Hills water tank using Zone 3 and Zone 4 criteria from the 1997 UBC. The requirements are summarized in Table 6.

Table 6. UBC Retrofit, Rolling Hills 300,000 Gallon Tank

Element	Existing Members	Proposed Retrofit	
		97 UBC Zone 3	97 UBC Zone 4
Columns:			
Upper	t = 0.406"	Unchanged	Unchanged
Middle	t = 0.406"	Unchanged	Unchanged
Lower	t = 0.406"	Unchanged	t = 0.906" (add 1/2" sleeve)
Braces:			
Upper	1.25"x3.5" (As = 4.4 sq. in)	TS 7x7x1/2 (As = 12.4 sq. in)	TS 9x9x1/2 (As = 16.4 sq. in)
Middle	1.25"x3.5" (As = 4.4 sq. in)	TS 7x7x1/2 (As = 12.4 sq. in)	TS 9x9x1/2 (As = 16.4 sq. in)
Lower	1.25"x3.5" (As = 4.4 sq. in)	TS 7x7x1/2 (As = 12.4 sq. in)	TS 9x9x1/2 (As = 16.4 sq. in)
Struts:			
Upper	C10x15.3 w/ C12x20.7	TS 9x9x3/8	TS 10x10x1/2
Lower	C10x15.3 w/ C12x20.7	TS 10x10x5/16	TS 10x10x1/2

For the FEMA 356 alternative, 60-kip dampers are required at the intersection of all braces in all bays for the full height of the tank. The 15 friction dampers limit the drift in the structure to 5.7 inches (0.40 percent) for a 10 percent/50-year event and 8.1 inches (0.57 percent) for a 2 percent/50-year event. The 0.40 percent drift meets the IO criteria for the 10 percent/50-year event and the 0.57 percent drift meets the LS criteria for the 2 percent/50-year event.

Based on the preliminary analysis, the existing columns and struts meet acceptable criteria for IO and LS performance for the 10 percent and 2 percent/50-year events, respectively, and need not be replaced. The existing undamaged 1 1/4 inch by 3 1/2 inch braces may be reused; however, if the majority of braces are being repaired or replaced due to damage, smaller brace sizes may be used. Due to the amount of energy dissipation, the existing foundations have adequate bearing capacity and frictional resistance to sliding and are not subjected to any uplift forces.

ESTIMATED COSTS OF SEISMIC RETROFIT DESIGN ALTERNATIVES

Concept level cost estimates were prepared for each retrofit alternative for both the Highlands and Rolling Hills tanks. Summaries of the retrofit costs are presented in Table 7 and Table 8. Also tabulated are budget level replacement costs for tanks with identical capacities and meeting UBC Zone 3 and 4 criteria. The replacement costs are presented in a range depending on the type of tank (i.e., leg tank, waterspheriod, or hydropillar).

Table 7. Retrofit and Replacement Cost, Highlands 750,000 Gallon Tank

	Retrofit Costs			Replacement Cost
Criteria	Construction	Design & Construction Services Fees	Total	Tank Replacement (Excluding Foundations)
UBC Zone 3	$397,000	$60,000	$457,000	$825,000 - $950,000
UBC Zone 4	$749,000	$113,000	$862,000	$975,000 - $1,125,000
FEMA 356	$274,000	$55,000	$329,000	

Table 8. Retrofit and Replacement Costs, Rolling Hills 300,000 Gallon Tank

	Retrofit Costs			Replacement Cost
Criteria	Construction	Design & Construction Services Fees	Total	Tank Replacement (Excluding Foundations)
UBC Zone 3	$177,000	$27,000	$204,000	$450,000 - $550,000
UBC Zone 4	$331,000	$50,000	$381,000	$525,000 - $650,000
FEMA 356	$178,000	$36,000	$214,000	

The construction costs tabulated above include 10 percent for mobilization, 10 percent for permits and sales tax, and a 25 percent contingency for unforeseen conditions. Standard unit costs for items such as steel erection account for construction in 2003 and the inefficiencies in requiring the water tanks to remain operational during construction. Fees for design, preparation of plans, specifications and cost estimates, and services during construction are estimated to be 15 percent of the construction cost for UBC Zone 3 and 4 retrofit criteria. A 20 percent fee was assumed for the FEMA 356 criteria due to the increased analytical work and testing required for the friction dampers.

Costs to retrofit each tank to meet either the UBC Zone 3 or Zone 4 criteria are approximately 50 to 80 percent of the cost to replace the tank, assuming no foundation work is required. Retrofitting the tank with friction dampers to meet FEMA 356 performance-based criteria is the most economical solution and provides the highest level of performance. The estimated cost to retrofit both the Highlands and Rolling Hills elevated water tanks with friction dampers is approximately $500,000.

CONSTRUCTION OF THE WATER TANK RETROFIT

As part of the quality control process, prototype testing was performed on two full-size friction damper devices prior to construction. Each device was loaded with alternating tension and compression forces applied to opposite damper legs for 20 cycles, with each cycle lasting approximately 15 seconds. The cycle was reversed at a point when the distance between the two outermost points of the damper reached the MCE (Maximum Credible Earthquake) stroke. Since friction dampers are not velocity-dependent, the speed at which the dampers were tested is irrelevant.

Pressure readings were taken and recorded by a computer at displacement increments of approximately 0.02 inches, and force-displacement curves were plotted for each test. The capacity of each prototype damper was maintained over the 20 continuous cycles and the variation in the slip load was within the specified tolerance. The temperature of the metal after 20 cycles was approximately 130°F, roughly 65°F above room temperature. Refer to Figure 4 for a photograph of the prototype testing setup.
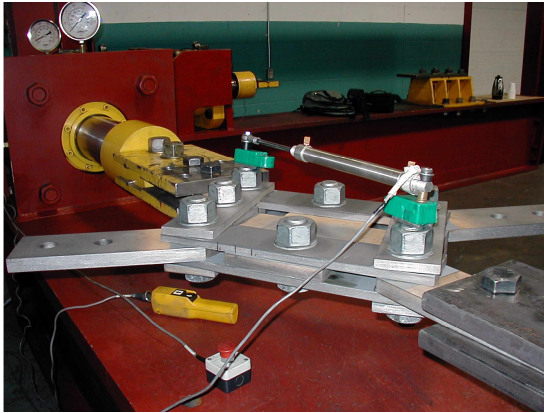
| Figure 4. Prototype Testing. | Figure 5. Completed Installation. |

A photograph of the completed friction damper installation is shown in Figure 5. The friction dampers were installed on each tank structures using the following sequence:

1. Water storage in each tank was lowered to 60 percent of maximum capacity prior to removing any structural members.

2. The existing braces were cut at the brace intersection and 1-inch steel splice plates were welded on to each side of the severed member ends.

3. Erection brackets with 1-inch diameter A36 threaded rods were installed at the severed brace locations to provide temporary shoring. The pairs of rods were then tensioned simultaneously – tensioning the existing braces, removing the slack, and avoiding subsequent lateral vibrations.

4. The friction damper devices were installed with each leg sandwiched and bolted between the four pairs of splice plates. The process was performed in one bay at a time.

5. The tank was refilled and the slack in the braces was removed using the erection rods and brackets.

The contractor installed two to three friction dampers per day using one man-lift, which resulted in a total construction time of less than two weeks for each tank.

REFERENCES

1. American Water Works Association, 1996, *AWWA D-100-96: Welded Steel Tanks for Water Storage*.
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3. International Conference of Building Officials, 1997, *1997 Uniform Building Code*.