**Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007**

NONLINEAR SEISMIC EVALUATION OF RETROFIT CONCEPTS FOR THE LAKE ALMANOR INTAKE TOWER

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ABSTRACT

Lake Almanor on the North Fork Feather River in Northern California was established in 1914 via construction of the earth-filled Canyon Dam for the purpose of water storage for use in hydroelectric power generation. The intake tower for releasing water from the reservoir is a lightly reinforced, 115' tall, cylindrical concrete structure with stepped wall thicknesses and 9 radially oriented buttresses of varying lengths around the circumference. The original structure, built in 1912-1914, was approximately 65' high. In the mid 1920's, the dam and intake tower were raised by 50' to increase the water storage capacity. A top slab and operating house were added on top for housing and operating the equipment for opening and closing the intake gates in the tower.

As part of the re-licensing conditions for continued operations of this 90 year-old structure, the intake tower must meet new seismic criteria. Engineering analyses, including response spectra analyses, linear time history analyses, and nonlinear time history cracking analyses, performed for the structure with the current seismic demands for this site indicate that structural damage of the intake structure is possible. Thus, seismic retrofitting may be required to bring the intake tower structure into compliance for re-licensing with the updated seismic criteria. This paper will describe the structural analyses used to evaluate the as-built structure and several retrofitting strategies to increase the seismic capacity of the structure. Some of the retrofitting strategies under investigation include adding post-tensioned rock anchor tendons, adding fiber reinforced wrapping, and providing additional confinement to the buttresses with a concrete and steel plate jacket. The structural evaluations consider 3-dimensional, nonlinear time history analyses using advanced concrete constitutive modeling.

Introduction

The Lake Almanor Dam intake tower is essential to regulate the level of the lake and to release water into the Feather River. During a seismic event uncontrolled releases of water from the reservoir could occur if the tower fails. The structural integrity must be assured during an earthquake scenario.

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At the upstream tow of the dam is the 115-foot tall reinforced concrete tower. The tower is used to regulate the flow into the reinforced concrete outlet conduit. The original as-built tower in 1914 was 65-feet tall, in 1927 an addition raised the height of the tower to its current 115-foot height.

Previously Pacific Gas & Electric Company (PG&E) reviewed the seismicity of the dam site and a stability analysis of the dam. This showed the Maximum Credible Earthquake (MCE) could be expected to produce peak ground accelerations up to 1.01 g for the 84th percentile earthquake. Subsequently the hazard potential for the dam has been reclassified and seismic records utilized have been scaled down to represent the 50th percentile ground motions for the MCE.

ANATECH Corp. was retained by PG&E to perform a seismic evaluation of the as-built tower and several retrofit concept alternatives. The seismic evaluation was performed with the general-purpose finite element program ABAQUS (Hibbit) coupled with the ANACAP-U (Anatech Corp.) nonlinear material constitutive models. The model developed is a full 3D finite element model, Fig. 1, of the intake tower using 8-node continuum elements. All reinforcement is included as truss-like sub-elements embedded within the concrete elements at the appropriate locations. The hydrodynamic effect of the water is included as nodal point masses active in the horizontal directions only. The added hydrodynamic masses were calculated following the methodology developed by Goyal and Chopra (Chopra). For these analyses the time history input was applied only in one horizontal direction perpendicular to the weak axis of the tower cross section. PG&E provided acceleration time histories that represent the 50th percentile of maximum ground motions for the MCE at the dam site.

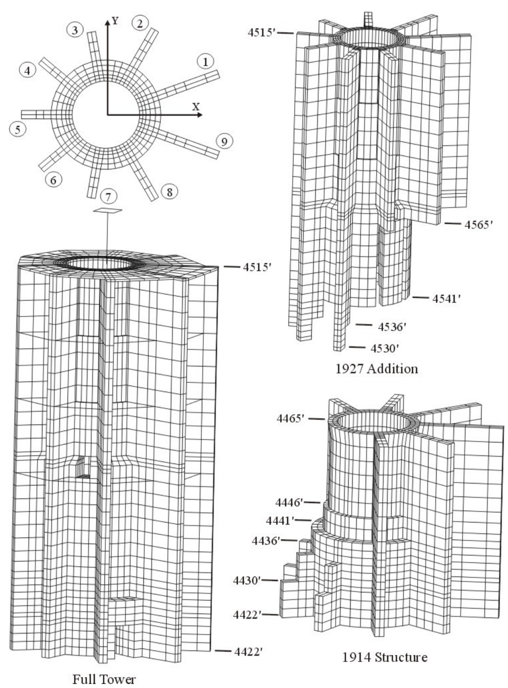

Figure 1. 3D finite element model of Lake Almanor intake tower

This provides a baseline seismic analysis using the input seismic time histories at the base of the model. Additionally, three alternative retrofit concepts were implemented and their efficacy evaluated with nonlinear seismic analysis. The three retrofit concepts are as follows:

1) Post-Tension Anchoring of the Tower:
Post tensioning of the tower to improve its strength, by placement of P/T anchors in permanent casing pipes socketed 100 feet into bedrock on both sides of the existing buttresses. (URS Corp.)

2) Carbon FRP Reinforcement of Tower

Application of carbon fiberglass reinforced plastic (FRP) to the inside face of the tower at several elevations in order to supply additional bending moment capacity needed to meet the demands of the design earthquake. (URS Corp.)

3) Buttress Confinement

Provide confinement around the buttresses with a one-foot thick layer of tremie concrete and an external jacket of steel plate. Two confinement variations were investigated:

A) Buttresses confinement from elevation 4422’ to 4483’ with one-foot thick tremie concrete and one-inch thick plate steel,

B) Buttress confinement from elevation 4422’ to 4436’ with one-foot thick tremie concrete and ½-inch thick plate steel.

The nonlinear material constitutive models have the ability to capture the effects of concrete cracking with subsequent opening and closing of cracks, load transfer to reinforcement with yielding and strain hardening (and rupture) of steel rebar as necessary, shear retention across concrete cracks due to aggregate interlock, shear stiffness and strength of the cracked concrete as a function of crack opening size, and compressive yielding with subsequent strain softening of concrete (crushing). Monotonic and cyclic uniaxial compressive stress strain behavior used in the modeling for the as-built concrete material is shown in Fig. 2. The performance and capacity of the structure will be based on material response and limit states under progressive damage rather than section shear and bending capacity limits imposed a priori and independently on beam element models. The evaluation of the structural performance was based on the tower displacement history including residual or permanent deformations, and concrete maximum principal stresses and strains for the extent and size crushing and cracking damage.

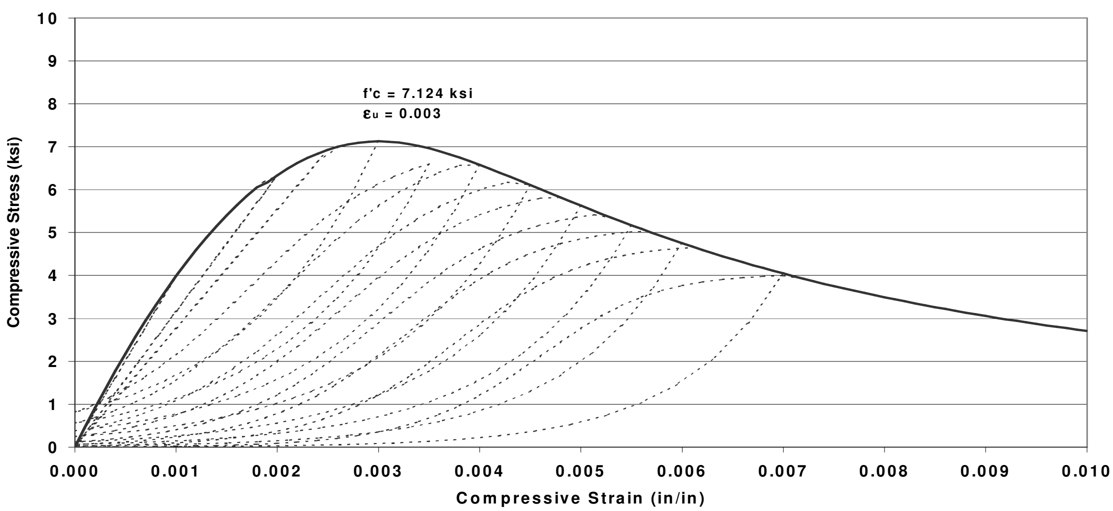

Figure 2. Concrete material model compressive stress strain curve

As-Built Baseline

The baseline analysis was performed with the model representing the as-built intake tower constructed in 1914 and the 1927. A nonlinear time history analysis was performed in displacement control. Displacement boundary conditions were applied in the global Y direction only at the base of the tower. The global X and vertical degrees of freedom of the base were held fixed.

The tower displacement response is shown in Fig. 3. This plot shows the top absolute displacement at elevation 4515’, the top relative displacement between elevations 4515’ and 4422’, and the base acceleration record. The analysis terminates due to lack of convergence and excessive element deformations at 9.5 seconds into the seismic record. Prior to the analysis terminating, the tower top relative displacement reaches –4.5 inches. From this plot, the relative tower top displacement can be estimated as oscillating at a frequency slightly more than 5 Hz prior to the peak acceleration pulse at approximately 8.2 seconds.

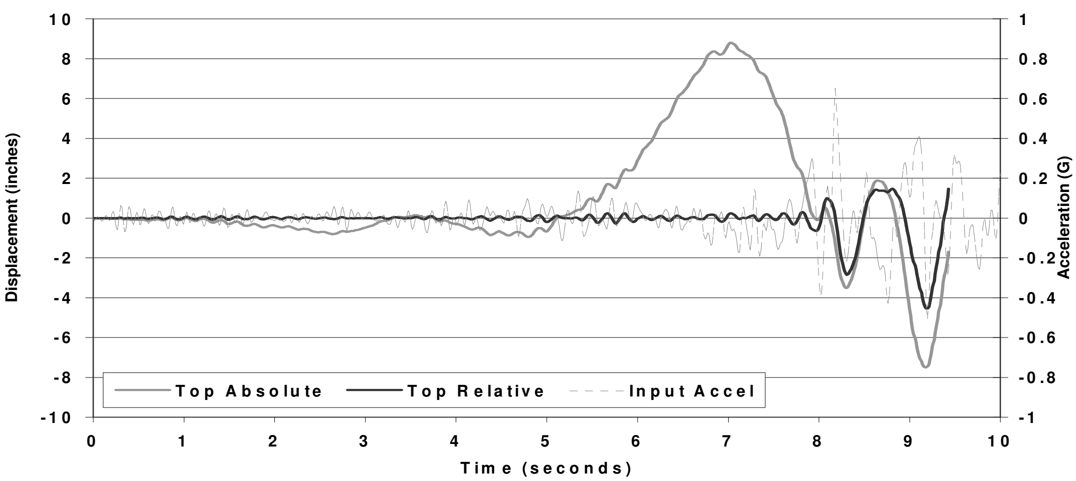

Figure 3. Displacement history response for as-built baseline structure.

The localized damage to the structure occurs due to loss of section capacity at the base of the tower between elevations 4422' and 4430'. The section loss is due to crushing failure of the concrete at the ends of the buttresses that progress radially in towards the main tower wall.

Post-Tension Anchoring of Tower

This retrofit concept involves post tensioning of the tower to improve its strength. Rock anchoring tendons would be placed in permanent casing pipes and socketed 100 feet into bedrock on both sides of the existing buttresses. These tendons would be attached to the operating house slab via installation of new anchor blocks above the slab and corbels beneath the slab on either side of the buttresses.

For the seismic evaluation, the baseline model was modified to include the post-tensioned rock anchors modeled as truss elements. The tendon/truss elements were placed 12 inches from the end of the buttresses and were fixed at the base and attached to the top slab. The tendons were modeled as Grade 270 25 strand tendons with a cross sectional area of 15 in^2. At each buttress, the tendons were stressed to the following force levels:

Buttress 1 and 9, 600 kips/tendon,
Buttress 2 and 8, 760 kips/tendon,
Buttress 3, 4, 6 and 7, 780 kips/tendon.

The tower displacement response is shown in Fig. 4. The post-tensioned model terminates at approximately 15.12 seconds into the seismic record. The peak relative displacement cycles occur from approximately 8.5 to just past 9 seconds with successive displacement cycles of 2.25, -2.22, and 2.5 inches.

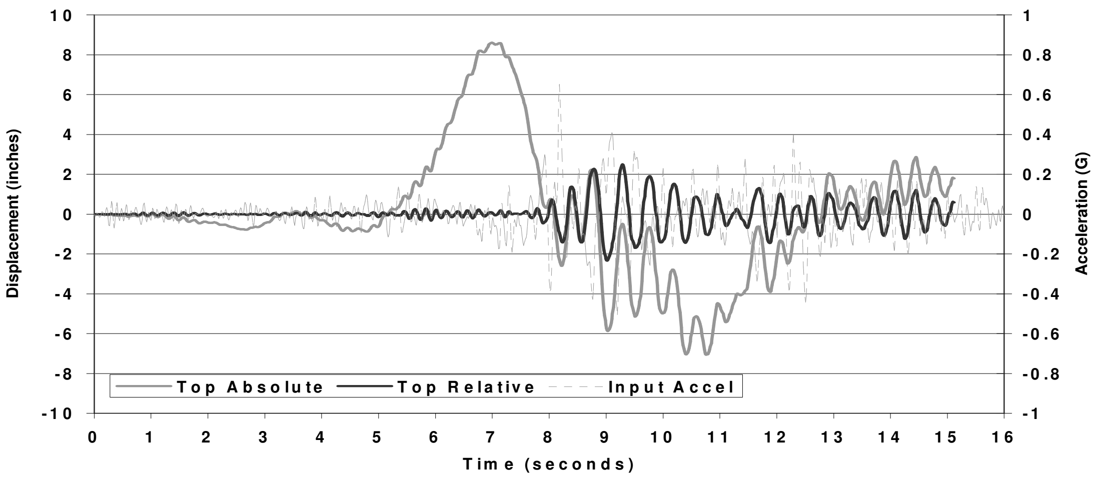

Figure 4. Displacement history response for tower with post-tensioned anchors.

The retrofit for post-tension anchoring of the tower was able to survive longer than the baseline analysis, but the analysis terminates before the end of the seismic event. This retrofit reduced the overall displacement of the tower, but it did not significantly control the seismic induced rocking and large compressive stresses experienced at the ends of the buttresses. Similar to the as-built tower, damage at the base of the tower will occur due to loss of section capacity from crushing failure of the concrete at the ends of the buttresses. At the peak acceleration pulses, uniform diagonal shear stress bands develop through the main tower wall indicating the possibility that a significant shear failure of the main tower could occur.

Carbon FRP Reinforcement of Tower

This retrofit concept consists of applying Carbon FRP reinforcement around the inner surface of the intake tower. The arrangement and construction as presented in (Saadatmanesh), is as follows. From Elev. 4422 to Elev. 4430, ¼" wide by 2 inch deep, vertical slots will be cut in the concrete 2-in. o.c. Three layers of Carbon Laminates will be placed in each slot and HJ3 epoxy resin will be injected into the slot to assure full composite action of Carbon and Concrete. An additional three layers of Carbon Laminate will be bonded on the inner surface of the cylinder from elevations 4422 to 4468.

The finite element model was modified to simulate the Carbon FRP wrap on the interior surface of the tower from elevation 4422' to 4468'. The wrap was simulated with 4-node shell elements attached to the surface of the continuum concrete elements. The slotted laminate was simulated with truss elements aligned vertically along the inside surface of the tower. The Carbon FRP material was modeled as an elastic material with zero compressive capacity and the following characteristics: E_f = 17,500,000 psi. Failure strength f_{fu} = 300,000 psi. Sheet thickness t_f = 0.0625 in. The shell elements were defined to have a thickness of 0.1875 in, equivalent to three layers of carbon laminate. To simplify the model, isotropic behavior of the Carbon FRP was assumed.

It was assumed that for the carbon FRP to develop adequate bond strength at elevation 4422' the Carbon Laminate would need to extend deeper into the tower. To simulate this the bottom row of nodes connected to the 4 node shell elements were fixed vertically and allowed to transmit tensile loads across the base interface. All other base conditions are the same as in the baseline and post-tensioned tendon models.

The tower displacement response is shown in Fig. 5. The carbon FRP wrap model fails at approximately 9.5 seconds into the seismic record. The peak relative displacement is 4.75 inches at 9.48 seconds.

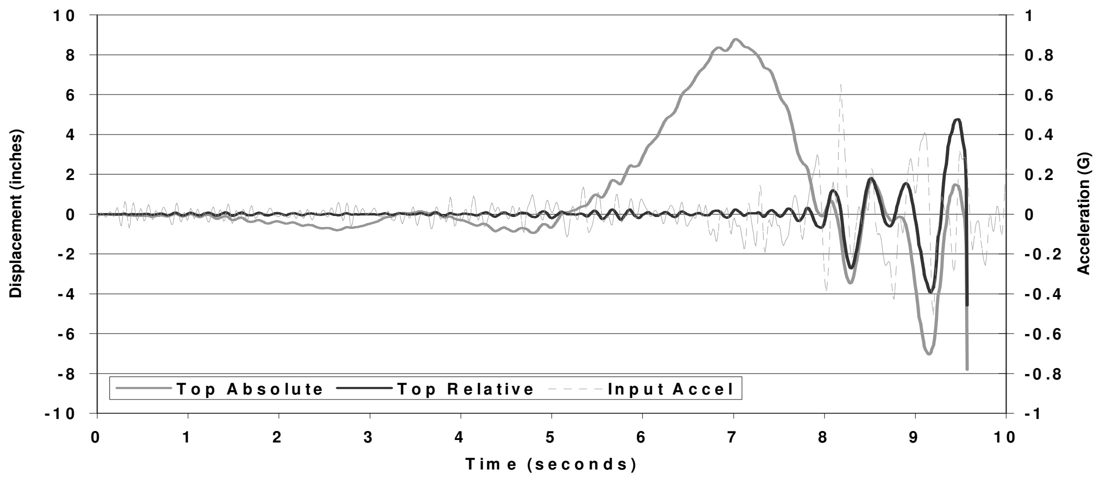

Figure 5. Displacement history response for tower with carbon FRP reinforcement

The Carbon FRP Reinforcement retrofit of the tower showed very little improvement in performance over the as-built baseline analysis. Tower displacements and rocking were similar. The addition of the carbon laminate to the interior surface of the wall, while expected to increase the flexural capacity also attracted more shear to the inside surface. Significant shear stresses developed in the narrow wall sections between the inlet gates leading to shear failure of the wall in this region. Additionally, as was seen in the previous analyses, the small bearing area of the buttresses developed large compressive stresses at the ends of the buttresses that led to concrete crushing at these locations.

Buttress Confinement, Elevation 4483'

This retrofit concept entails providing confinement and additional bearing area for the buttresses. A one-foot thick layer of tremie concrete is to be placed around each buttress inside a one-inch thick steel plate jacket. The retrofit is to extend from elevation 4422' to 4483'.The baseline model was modified to include additional 8-node continuum elements around the perimeter of the existing buttresses to simulate the one-foot thick tremie concrete. The steel plate was modeled with 4-node shell elements attached to the outer surface of the new continuum elements.

The nonlinear time history analysis for the buttress confinement retrofit survives the entire seismic input record. Fig. 6 shows the displacement time history response of the tower. The peak relative displacement is approximately 1.56" at 12.6 seconds of the seismic record.

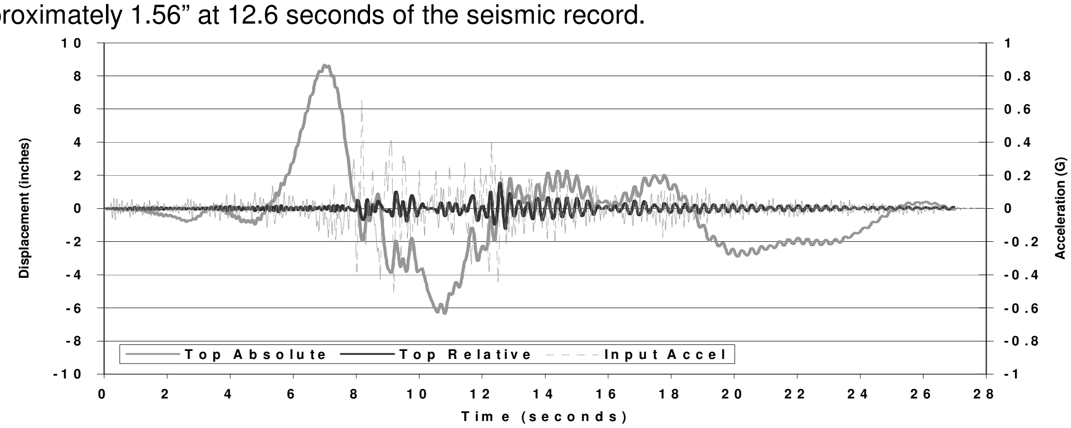

Figure 6. Displacement history response for tower with buttress confinement to elevation 4483'

The analysis shows that the buttress confinement retrofit to elevation 4483’ survives the full input seismic record with little damage and only moderate compressive stresses and strains in the buttresses and walls. During this analysis at the peak acceleration pulse at 8.2 seconds, the calculated base shear, 13600 kips, exceeded the shear capacity, 11209 kips, based on ACI-318 for a single time step of 0.0117 seconds. At the peak acceleration pulse, a significant band of shear stress exceeding 3.5√fc’ develops through the main wall above the inlet gate between buttresses 1 and 9, but principal tensile strains in the walls at the high shear stress bands are below the cracking threshold and significant shear failure is not expected. Elevated tensile strains do develop at the base of the tower and around the inlet gate openings that indicate concrete cracking will occur in these locations but it is not significant enough to indicate severe damage develops. The deformation of the inlet gates was investigated by calculating the displacement along the diagonal across the inlet opening at the inside and outside of the wall. The peak diagonal displacement is 0.155” and at the end of the analysis the maximum change in diagonal length is 0.07” indicating that there is no significant damage and permanent deformation at the gate inlets. With this retrofit no significant structural failure or collapse should be expected to occur.

Buttress Confinement, Elevation 4436’

This retrofit concept is a modification of the buttress confinement retrofit described in the previous section. The confinement and added concrete bearing area around the buttresses are provided with a one-foot thick layer of tremie concrete and a 1/2-inch thick jacket of steel plate. The extent of the confinement region is reduced from elevation 4422’ to 4436’.

The nonlinear time history analysis for this buttress confinement retrofit also survives the entire seismic input record. Fig. 7 shows the displacement time history response of the tower. The peak relative displacement is approximately 1.36” at 12.6 seconds of the seismic record.

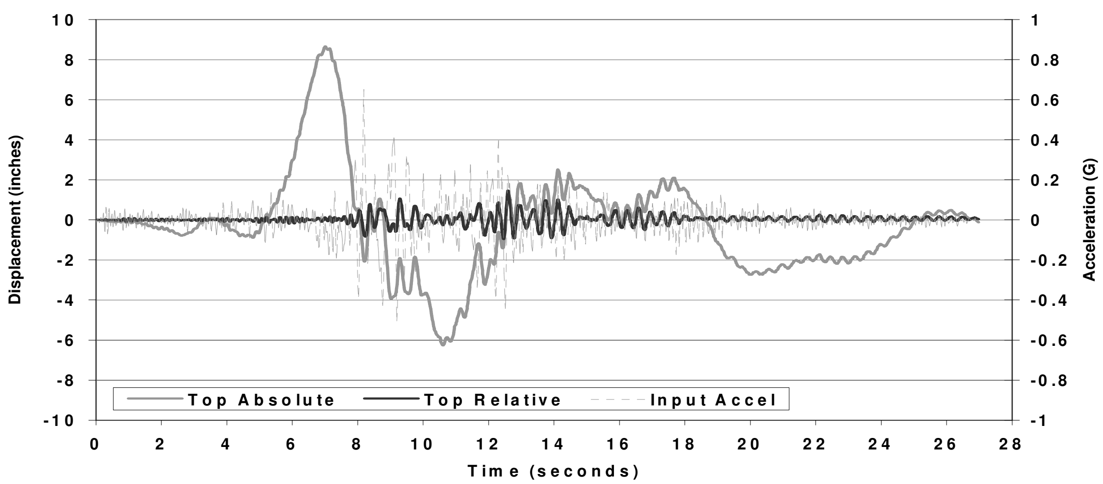

Figure 7. Displacement history response of tower with buttress confinement to elevation 4436'

The analysis shows that this retrofit concept performs well. The retrofitted structure was able to survive the full seismic input record. Some damage would be expected to occur at the top interface of the confinement retrofit. Primarily, cracking and rebar yielding at the ends of the buttresses directly above the top of the retrofit is expected to occur. The maximum crack opening width at these locations are predicted to be approximately 0.14” and tensile rebar strains to be 0.7%, and cracking would be expected to penetrate into the main wall. Rebar fracture or concrete crushing would not be expected to occur at the interface location. The tower top displacements are slightly increased compared to the buttress confinement retrofit to elevation 4483’. This model did not see the large shear stresses develop in the main tower wall that was observed in the buttress confinement to elevation 4483’ evaluation. The peak diagonal displacement is 0.081" and at the end of the analysis the maximum change in diagonal length is 0.02" indicating that there is no significant damage and permanent deformation at the gate inlets. With this retrofit no significant structural failure or collapse should be expected to occur.

Conclusions

Nonlinear time history finite element analyses were performed for the Lake Almanor Intake Tower to evaluate the seismic response of the as-built condition and the efficacy of several retrofit concepts. Eigenvalue analyses were performed on all of the models in their virgin state to calculate the first mode frequency. These frequencies ranged from a low of 5.53 Hz for the Baseline and Carbon FRP models to a high of 8.20 Hz for the Buttress Confinement to Elevation 4483' model. After the first and largest acceleration pulse at 8.2 seconds, all of the models experienced a drop in their modal frequency, estimated to range from slightly less than 2 Hz to 3 Hz. The reduced structural frequency was evaluated based on the cyclic tower top relative displacement history.

The seismic analyses performed for the as-built tower, the post-tension tower anchoring, and Carbon FRP retrofit concepts show that the tower will experience significant local damage at the base of the buttresses and the tower wall during the 50th percentile MCE. These three analyses indicate that the 2-foot wide buttresses do not have enough bearing area to resist the compression forces developed at the base of the tower as it starts rocking about the weak section axis. This leads to significant crushing of concrete starting at the end of the buttresses and progressing radially in towards the main tower wall.

Post-tension anchoring of the tower increases the stiffness of the tower by putting the tower in a uniform state of compression. This will increase the section shear capacity based on ACI-318 by approximately 8%. During the seismic evaluation, due to the increased stiffness of the tower, larger shear forces develop at the base of the tower that exceed the calculated shear capacity several times between 8 and 10 seconds of the seismic record. Shear stress contour plots show distinct diagonal shear planes that extend through the main tower wall indicating a potential shear failure of the structure in addition to the crushing of the buttresses.

Typically, wrap type retrofits are applied to provide added confinement for lightly reinforced members. Carbon FRP Reinforcement has been shown to be very effective in retrofitting columns and beams subjected to high flexural demands. The stout character of the intake tower tends to be subjected to higher shear demands and rocking behavior, and flexural demands are less of a concern. This particular retrofit only locally strengthens the inside surface of the tower wall. The time history analysis indicates that the added stiffness introduced on the inside wall surface attracted large shear forces to this region of the tower causing damage to the structure particularly near the gate inlet openings.

Confining the buttresses with one-foot thick tremie concrete and a one-inch thick steel jacket to an elevation of 4483' is shown to provide adequate strengthening for the tower to survive the 50th percentile MCE without structural collapse. There was no evidence of concrete crushing at the ends of the buttresses. The increased bearing area was adequate to resist the elevated compressive forces during rocking of the tower. The increased mass due to the tremie concrete increased both the base shear developed at the base of the tower and the calculated shear capacity. The shear capacity is increased by approximately 36% over the as-built tower condition. During the seismic event, the tower did experience a significant shear demand that exceeded the design capacity at the peak acceleration pulse at 8.2 seconds, although stress and strain contour plots do no show any significant damage occurring.

Significantly reducing the buttress confinement by extending only to elevation 4436' and reducing the thickness of the steel jacket is shown to perform as well as the confinement to elevation 4483'. The tower top relative displacements are slightly increased but remain much lower than the displacement demands observed in the baseline, post-tensioned, and carbon wrapped configurations. The base shear and uplift demands are reduced from the more extensive confinement concept. Primarily, this is due to much less added mass from the buttress confinement resulting in lower base shears occurring during the seismic event. The analysis does show that strain concentrations develop in the ends of the buttresses at the discontinuous retrofit interface at elevation 4436'. The corresponding strains calculated from the relative vertical displacement over this region indicate that cracking and rebar yielding would be expected to occur, with a maximum rebar strain nearing 0.7%. While this cracking damage would be significant, the tower does not suffer structural failure and collapse.

Table 1. Summary of analyses and peak demands

Analysis	First Mode Frequency	Peak Relative Displacement	Peak Base Shear	Peak Uplift	Analysis Duration	Outcome
	(Hz)	(in)	(kips)	(in)	(seconds)	
As-Built (Baseline)	5.53	-4.53	-7485	0.21	9.43	Concrete Crushing
Post-Tensioned Anchoring	5.61	2.50	11234	0.55	15.12	Concrete Crushing
Carbon FRP Reinforcement	5.53	4.78	17106	0.46	9.57	Concrete Crushing
Confinement El. 4483'	8.20	1.56	13600	0.46	27.0	Minimal Damage
Confinement El. 4436'	6.33	1.33	10269	0.26	27.0	Moderate Damage

Based on the seismic evaluations of the Lake Almanor intake tower as-built condition and four retrofit alternatives, the best approach for strengthening the tower for the new seismic demands would be to provide increased bearing area and confinement for the buttresses. Neither the post-tension anchors nor the carbon FRP wrap provide any increased bearing capacity at the ends of the buttresses where the local concrete crushing failure of the as-built structure develops.

It should be noted that the earthquake record was only applied in one horizontal direction for all the analyses performed. In the field, the odds of seismically induced ground motions to hit the tower perfectly in the same direction as they were applied to the finite element models are very low. A more robust seismic evaluation of the intake tower would apply biaxial seismic input in the two horizontal directions and in the vertical direction, if the site conditions warrant.

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