SEISMIC RESPONSE OF THE VAN NORMAN RESERVOIR OUTLET TOWERS

by

S. A. Gadomski[I]

SYNOPSIS

The San Fernando earthquake of February 9, 1971 caused major damage to the Los Angeles Department of Water and Power's facilities at the Van Norman Reservoir Complex. Of particular interest are the three reinforced concrete outlet towers at the Upper and Lower Van Norman Reservoirs. The high frequency of this earthquake accompanied with the high accelerations, on the order of 0.5g, caused failure in two of the three towers. The response of these structures was determined using a lumped mass approach. Theoretical bending moments and shears were calculated, and the modes of failure predicted and compared with their actual behavior. From this information, recommendations are made for future design of similar structures in active seismic zones.

INTRODUCTION

On February 9, 1971 an earthquake struck the Los Angeles area with most of the major damage centered in the San Fernando Valley. At the northern end of the San Fernando Valley, the Los Angeles Department of Water and Power operates and maintains a facility known as the Van Norman Complex. Located approximately 25 miles (41 km) northwest of Los Angeles and about 7 miles (12 km) from the epicenter, this complex includes three reservoirs, two of which were heavily damaged. The remaining reservoir completed in 1970 experienced no structural damage. Included in this damage was complete failure in two of the three outlet towers from the Upper and Lower Van Norman Reservoirs, with no apparent damage to the third tower. The three reinforced concrete towers studied in this analysis are as follows:

1. East tower, Lower Reservoir
   Built in 1913
   Height - 139 feet (42.4 m), outside diameter - 20 feet (6.1 m)
   Condition: Failure 26 feet (7.9 m) above the base.

2. West tower, Lower Reservoir
   Built in 1914
   Height - 74 feet (22.6 m), outside diameter 20 feet (6.1 m)
   Condition: No damage.

3. Upper Reservoir Tower
   Built in 1919
   Height - 89 feet (27.1 m), outside diameter - 19 feet (5.8 m)
   Condition: Failure 27 feet (8.2 m) above the base.

[I] Assistant Engineer, Projects Design, Los Angeles Department of Water and Power, Los Angeles, California, U.S.A.

This earthquake registered a magnitude of 6.6 (Richter Scale), but the high frequency accompanied with the high accelerations, caused failure in two of the towers. This paper attempts to show why these failures occurred and presents recommendations on future design of similar structures.

ANALYSIS

Concrete core samples were taken from all three towers and tested to give the ultimate compressive strengths of the samples. From this information an average value for Young's Modulus and ultimate shear capacity was found using emperical formulae.[7] The towers were then idealized to analytical lumped mass models, using 32 masses to accommodate the various openings and changing geometry of the towers. A computer program was then utilized to assemble the mass and stiffness matrix, solve the standard eigenvalue problem, and calculate the shears and moments due to the chosen forcing function.[5] (the Lake Hughes No. 12 strong-motion seismograph station's record, located approximately 15 miles (25 Km) from the epicenter, scaled to 0.5g maximum acceleration was used as the forcing function for this analysis[2]). Modal damping was assumed to be 5.0% in all three towers. The effect of virtual mass[1,6] was also taken into consideration. The virtual mass was taken as 1.0 times the displaced volume of water.

The results of this analysis are presented in graphical and tabular form in Figure I. All three towers have very little reinforcing steel to withstand the large bending moments produced by this earthquake.[8] Although the construction drawings were somewhat sketchy on this point a conservative estimate for the ratio of steel to concrete was assumed to be 0.002. The east tower was the only tower to show extremely high stresses in flexure. The concrete stress at the outermost fiber showed stresses as high as 166% of the allowable, with the stresses in the steel reinforcement reaching 155% of the ultimate. The flexure stresses, both in the concrete and steel, in the other two towers were well within the allowables. The east tower also showed high stresses in shear reaching 115% of the allowable in the close vicinity of the failure. The Upper Van Norman Tower showed shear stresses well beyond the allowable by 155%, while the west tower showed shear stresses well within the allowables.

CONCLUSIONS

It is, therefore, concluded that shear was the governing mode of failure in the Upper Van Norman tower, while a combination of shear and flexure was the governing mode of failure in the east tower at Lower Van Norman. One would expect that these two towers would experience severe damage by the stresses produced by this earthquake. The upper tower was shifted 4 in. (10.2 cm) at the break. The east tower at Lower Van Norman toppled. A landslide on the upstream toe of the dam took place during the earthquake in the vicinity of the east tower. The stresses due to the earthquake loading were probably enough to damage the tower to a point where it had no strength to withstand any additional force, and the landslide was enough to topple the tower.

The reason for the wide variation in the behavior of the three towers may be explained by the fact that fundamental frequency of the west tower at Lower Van Norman was somewhat higher than the peak frequencies of the earthquake and, hence, did not reach a resonant mode of vibration as was experienced in the two towers that did fail.

RECOMMENDATIONS

This analysis showed good correlation with the actual behavior of these towers during the earthquake. The main recommendation that may be made is for an analysis of this type to be carried out on all similar structures in seismic active zones. It is impossible, at this time, to predict what the exact characteristics of a given design earthquake should be; however, an analysis of this type will aid the engineer in making decisions on the final design configuration.

Economics plays an important part in practical engineering design and there will probably be certain risk factors in the final design configuration[3]. These risk factors should be assigned according to the importance of the structure.

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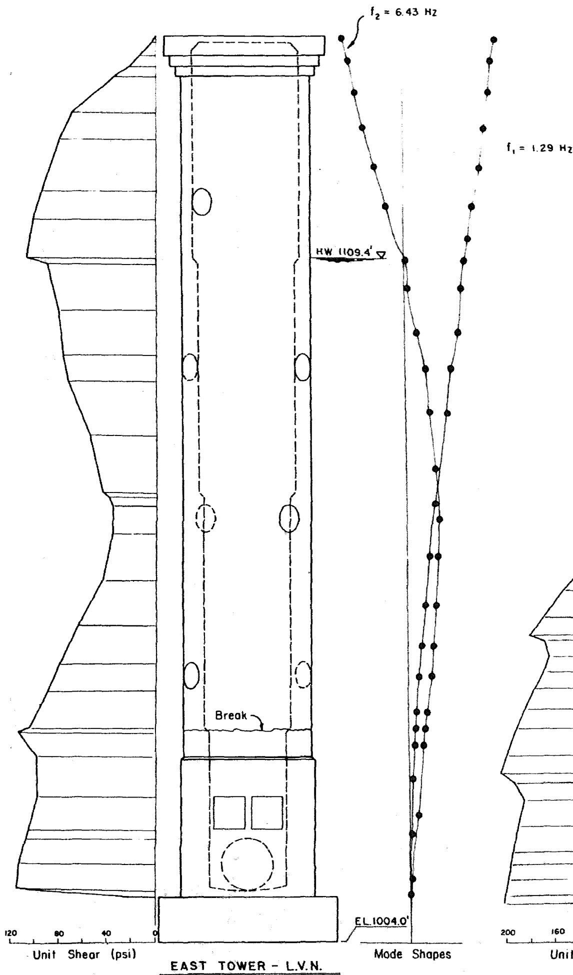

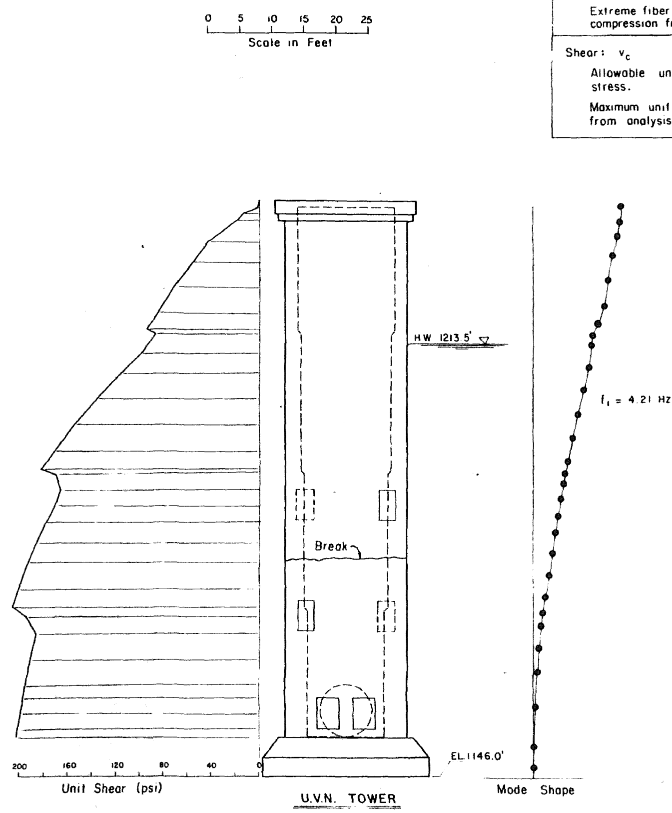

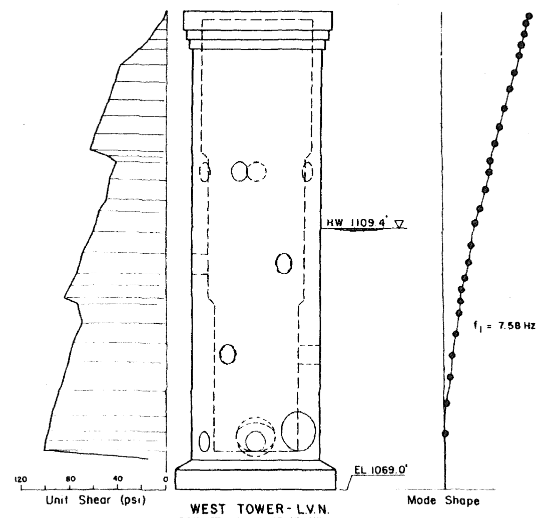

Description		East Tower - L.V.N. (psi)	U.V.N Tower (psi)	West Tower - L.V.N (psi)
Strength of Concrete: f_c'				
Strength of concrete as determined from testing.	f_c'	2475	3615	3790
Average compression strength from analysis.	f_c'	1395	891	930
Flexure: f_c				
Extreme fiber stress in compression (allowable).	f_c	1113	1626	1705
Extreme fiber stress in compression from analysis.	f_c	1850	1225	1260
Shear: v_c				
Allowable unit shear stress.	v_c	100	120	123
Maximum unit shear stress from analysis.	v_c	115	206	101

Note: 1 psi = .07037 kg. per sq. cm.
1 ft = .3048 m

FIGURE I RESULTS OF ANALYSIS