NONLINEAR ANALYSIS OF A GRAVITY DAM FOR SEISMIC LOADS

by

D.P. Reddy[I] and H.S. Ts'ao[II]

SYNOPSIS

A nonlinear dynamic analysis of a gravity dam subjected to earthquake excitations was performed, taking into account the interaction of the reservoir water and the dam. The dam section was modeled by a two-dimensional finite element system. The results of the nonlinear analysis indicated that the dam was stable when subjected to the Safe Shutdown Earthquake (SSE) loads with the water level of the reservoir at 25-year flood condition.

INTRODUCTION

The concrete gravity dam is located in the southeastern United States. The dynamic analysis of the abutment section of the dam was performed previously for static and seismic loads (Ref. 1), using the EADHI computer program (Ref. 2). This program considers the effects of reservoir interaction with the dam and does not include any nonlinear effects. At the upstream and downstream faces of the dam section, tensile stresses of about 600 psi (4140 kN/m^2) were obtained, which far exceeded the estimated ultimate tensile strength of the concrete. Thus the present nonlinear analysis of the dam was performed to account for the development of cracks in the concrete.

This paper presents the results of a nonlinear analysis of the dam. The loading on the dam includes earthquake ground motions, the dead load of the dam, and the loads due to flood water in the reservoir. The vertical and S80E components of the Golden Gate Park 1957 earthquake records, scaled to provide response spectra that envelop the SSE criteria spectra in the frequency range of interest for the dam response, were used as input to the analysis. This corresponded to peak accelerations of 0.2g for both horizontal and vertical components. The flood water level in the reservoir corresponded to the 25-year flood.

ANALYSIS PROCEDURES

The nonlinear analysis of the dam was performed using the FEDRC code, a dynamic, inelastic, two-dimensional, continuum finite-element computer program (Ref. 3). The program accepts time-varying loads and time-varying motions, in addition to seismic motions, and solves the governing equations of motion of a finite element system with a step-by-step integration procedure. Although Rayleigh damping can be specified in the program, no damping value is assigned in the analysis. The nonlinear stress/strain relationship for concrete used in the analysis is based on the suggestion by Liu, Nilson, and Slate (Ref. 4). A cracking stress (maximum tensile stress) can also be specified to properly consider the effects of cracks in concrete and resulting redistribution of stresses.

To account for the interaction between the dam and the reservoir, the hydrodynamic forces on the upstream face of the dam were first computed using the EADHI computer program. These forces were then treated as time-varying input functions in FEDRC code. Since EADHI program is valid only for linear elastic systems, the hydrodynamic forces computed from the program are only approximate.

I Principal Engineer, Agbabian Associates, El Segundo, California, U.S.A.
II Member of Technical Staff, Agbabian Associates.

During the course of this study (Ref. 5), it was found that the true hydrodynamic forces were very close to the approximate values except at the top section of the dam. Since hydrodynamic forces are smaller at the upper part of the dam, the effects of using approximate values on the nonlinear response are not significant.

The finite element mesh of the dam is shown in Fig. 1. Part of the rock foundation is also included in the model in order to treat the compressibility of the foundation. The bottom and vertical boundaries of the foundation are assumed fixed but are sufficiently removed from the dam so that their effects on the dam response are negligible.

The rock foundation is treated as linearly elastic material, whereas the concrete in the dam is treated as nonlinear. The maximum tensile strength of concrete is assigned a value of 350 psi (2415 kN/m^2) for the interior elements and 100 psi (690 kN/m^2) for the exterior elements of the model. The smaller value used in the latter case is to account for surface cracks that occurred during construction from the heat of the cement hydration.

The response time histories of motion at four locations in the dam are shown in Figs. 2 through 5. On the lower two-thirds of the dam, the maximum positive and negative displacements generally occur at 1.94 sec and 2.05 sec, respectively (Figs. 4, 5). These values coincided with the corresponding values in the elastic analysis (Ref. 1). The effect of material nonlinearities on the dam response is seen clearly in the upper one-third portion of the dam section. In Figs. 2 and 3, the displacement oscillations attenuate at about 4 sec to values that differ from the static responses because the cracking of the concrete elements has caused permanent displacements. The permanent displacements at Nodes 1 and 17 are about 0.1 ft (3.05 cm) toward the reservoir. The maximum horizontal displacement at Node 17 (Fig. 3) from the nonlinear analysis is 0.18 ft (5.5 cm), about three times that from the elastic analysis.

The compressive stresses developed in the dam during the earthquake excitations appear to be small. Fig. 6 shows the vertical stress distribution in the dam section at 2.05 sec. The maximum compressive stresses are far below the estimated allowable limit of 6500 psi (44,850 kN/m^2) for the concrete. The tensile stresses, on the other hand, frequently exceeded the tensile strength. As a result, cracks developed one time or another in the concrete elements shown shaded in Fig. 1.

The cracks first began to appear in the upper portion of the downstream face of the dam. These cracks opened and closed several times before another crack was found at the heel of the dam. After the cracks propagated lower on the downstream face, they began to form at the bottom of the upstream face. At 1.5 sec, all the elements on the upstream face lost their tensile resistance. Shortly after, the elements on the downstream face also experienced cracks. New cracks ceased to appear only after 2.1 sec, when ground motion began to subside.

The formation of cracks weakens the strength of the dam against overturning. For this reason, three critical sections denoted in Fig. 1 as A-A, B-B, and C-C were investigated. Fig. 7 shows the vertical stress distributions along Section A-A at critical times. The moment of these stresses with respect to either end of the dam section is the overturning capacity at that particular instant. Since the center of gravity of the stresses is relatively far from either end, Section A-A is in a very stable position. Fig. 8 shows the distributions of stresses along Section B-B at three instants of time.

The overturning moment at t = 1.99 sec is very small due to the presence of tension in the middle part of the section. The maximum compressive stress is very small at other instants of time, and therefore failure due to overturning is not indicated along this section. The stress distribution in the elements along Section C-C, listed in Table 2 for nine time instants, indicates that all six elements along Section C-C developed cracks but not simultaneously. The stresses shown are relatively small in comparison to the allowable stresses in the concrete, and failure along this section due to overturning is not indicated. However, if the dam is subjected to significantly stronger seismic input motions, failure is more likely to occur along Section C-C than at the base of the dam.

When cracks are opened along the upstream face of the dam, water can penetrate the cracks. Whether the water will significantly increase the pore water pressure in the dam depends on the width of the crack and the duration in which the crack remains open. Table 3 gives the longest time interval during which cracks remain open in each element on the upstream face of the dam. As shown, cracks open for a duration of less than 0.15 sec in all elements. Considering the maximum water level of 213 ft (65m), the water can penetrate only 1 to 2 ft (0.3 to 0.6m) into the dam before the crack is again closed. Therefore, pore water-pressure buildup is not a potential problem.

CONCLUSIONS

The nonlinear analysis indicates that the abutment section of the dam is stable under the assumed SSE loads. If the dam is subjected to significantly stronger seismic input excitation, the upper part of the dam along a section about 65 ft (19.8m) from the top (Sec. C-C, Fig. 1) may fail by toppling into the reservoir, but the major part of the dam should remain stable.

BIBLIOGRAPHY

1. *Analysis of a Gravity Dam for Static and Seismic Loads*, R-7523-1-3703. El Segundo, CA: Agbabian Associates, 29 Dec. 1974.
2. Chakrabarti. P. and Chopra, A.K. *Earthquake Response of Gravity Dams Including Reservoir Interaction Effects*, EERC-73-6. Berkeley: Univ. of Calif., Dec. 1972.
3. *User's Guide for FEDRC Code*, U-7523-2-3868. El Segundo, CA: Agbabian Associates, May 1975.
4. Liu, T.C.Y.; Nilson, A.H.; and Slate, F.O. "Biaxial Stress/Strain Relations for Concrete," *J. Struct. Div.*, ASCE, 98:ST5 (May), 1972.
5. *Nonlinear Analysis of a Gravity Dam for Seismic Loads*, R-7523-2-3894. El Segundo, CA: Agbabian Associates, 24 Jul. 1975.

TABLE 1. VERTICAL STRESSES ALONG SECTION C-C (see Fig. 1)

Time, sec	Stresses in Elements, psi (1 psi = 6.9 kN/m^2) (+ Tension, - Compression)					
	19	20	21	22	23	24
1.71	0	-105	72	6	-52	0
1.72	0	-274	40	2	1	-156
1.73	0	-340	13	-1	11	- 7
1.74	0	-233	5	-5	1	0
1.75	0	- 53	11	-4	- 5	- 21
⋮						
1.94	-134	0	-17	4	21	0
1.95	-176	-101	-21	5	17	0
1.96	-151	-251	23	18	- 6	0
1.97	-134	- 38	106	22	-11	-372

TABLE 2. LONGEST TIME INTERVALS DURING WHICH CRACKS REMAIN OPEN IN ELEMENTS ON UPSTREAM FACE OF DAM

Element No.	Time, sec	Element No.	Time, sec
10	0.145	70	0.040
14	0.140	82	0.050
19	0.120	95	0.040
25	0.020	109	0.030
32	0.080	140	0.120
40	0.070	159	0.130
49	0.020	180	0.100
59	0.030		

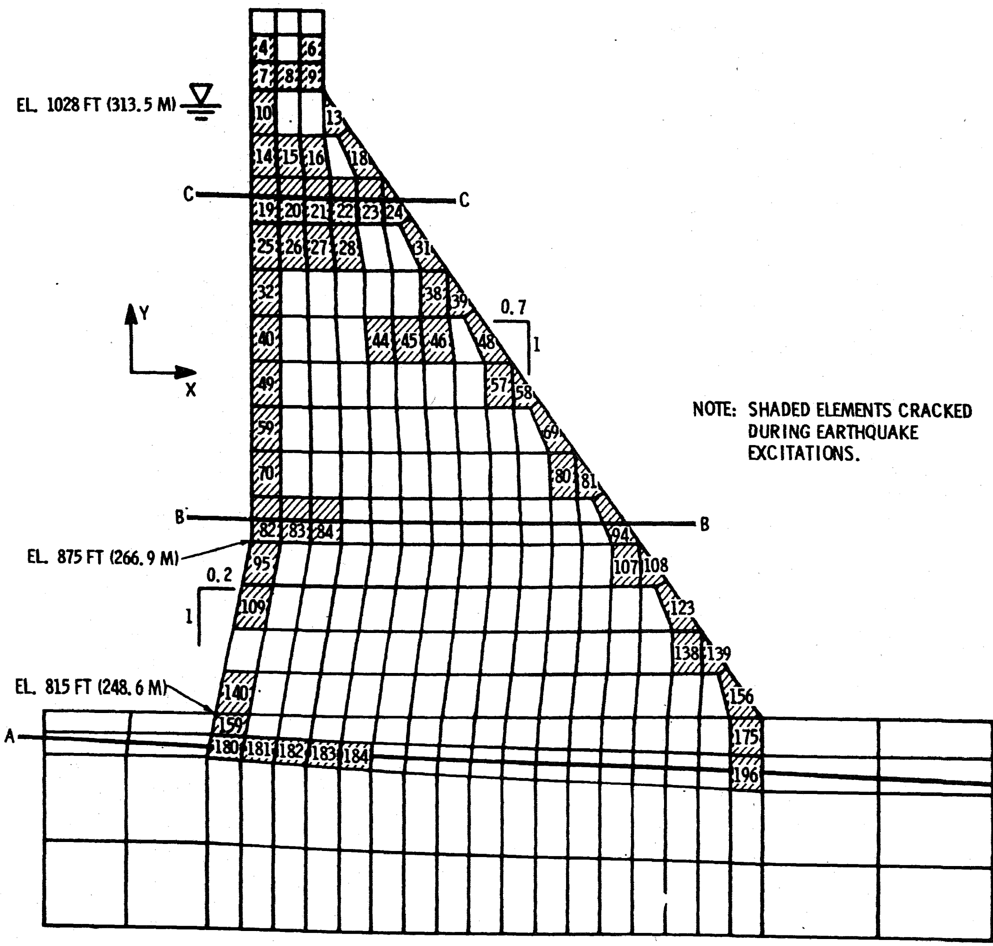

FIGURE 1. FINITE ELEMENT MESH OF GRAVITY DAM MODEL

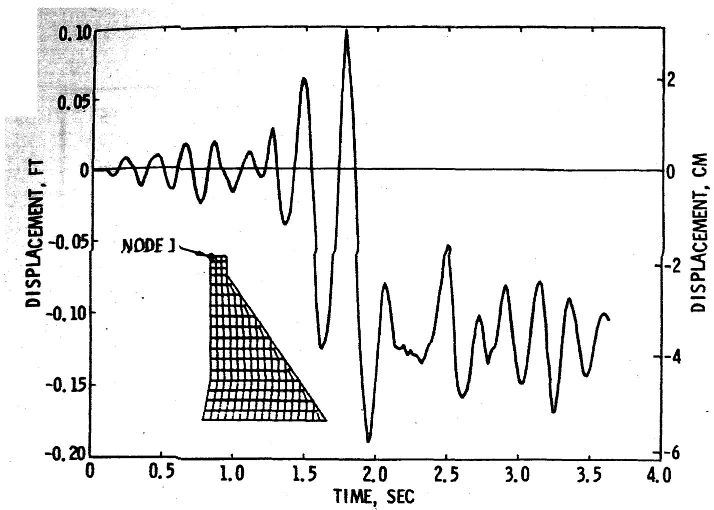

FIGURE 2. HORIZONTAL DISPLACEMENT TIME HISTORY OF DAM AT NODE 1

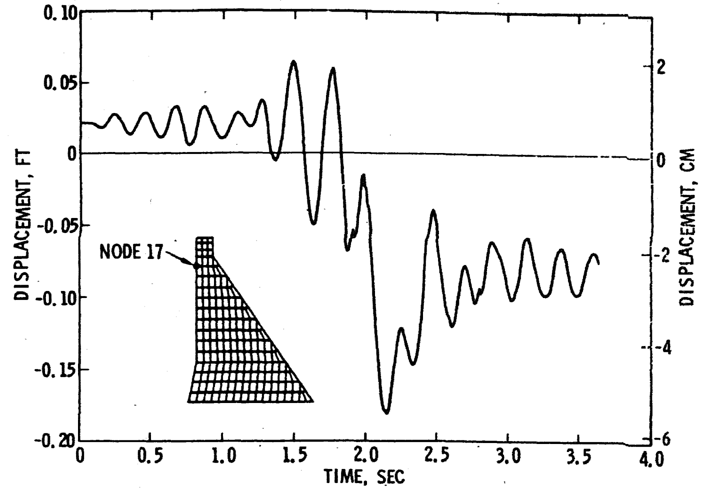

FIGURE 3. HORIZONTAL DISPLACEMENT TIME HISTORY OF DAM AT NODE 17

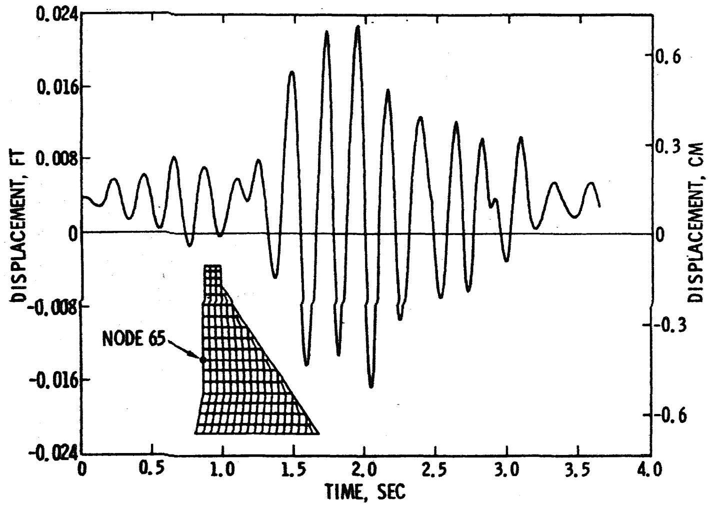

FIGURE 4. HORIZONTAL DISPLACEMENT TIME HISTORY OF DAM AT NODE 65

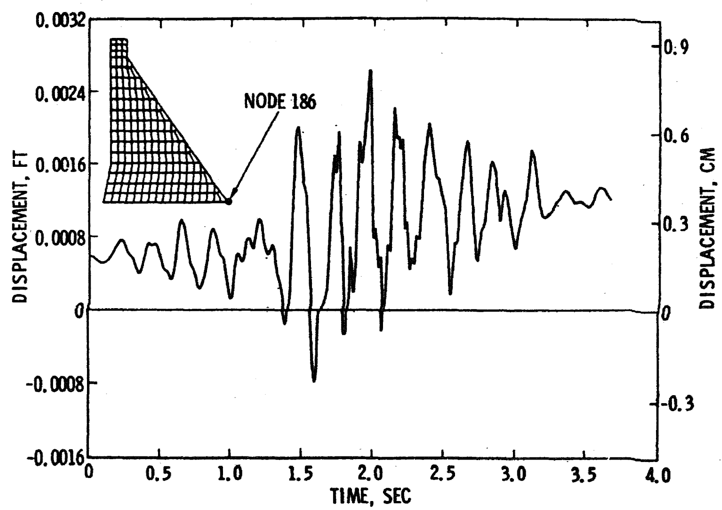

FIGURE 5. HORIZONTAL DISPLACEMENT TIME HISTORY OF DAM AT NODE 186

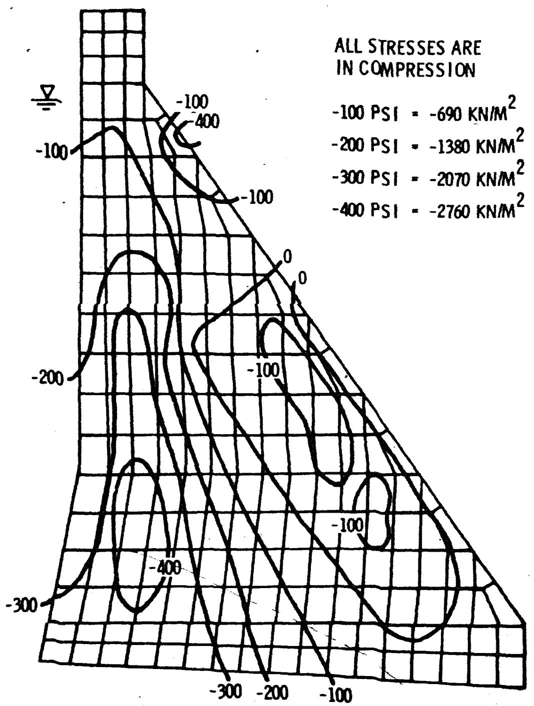

FIGURE 6. VERTICAL STRESS CONTOURS IN PSI AT t = 2.05 SEC

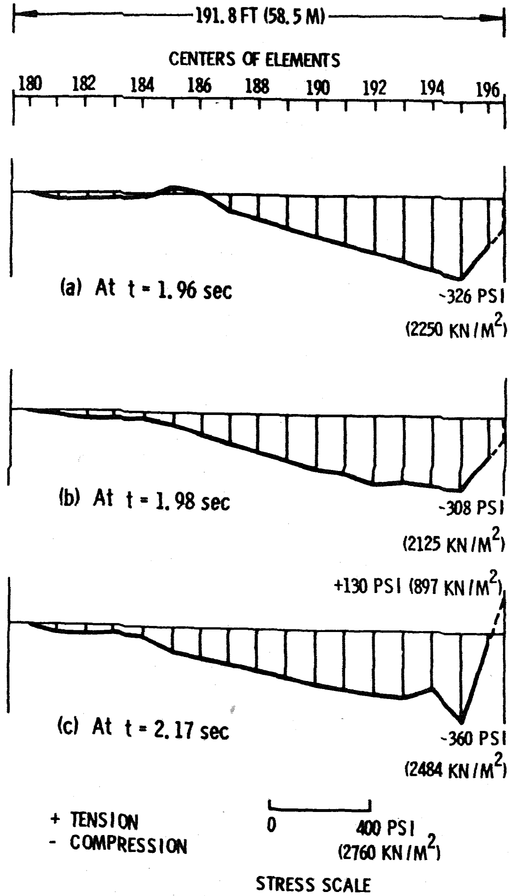

FIGURE 7. VERTICAL STRESS DISTRIBUTION ALONG SECTION A-A (see Fig. 1)

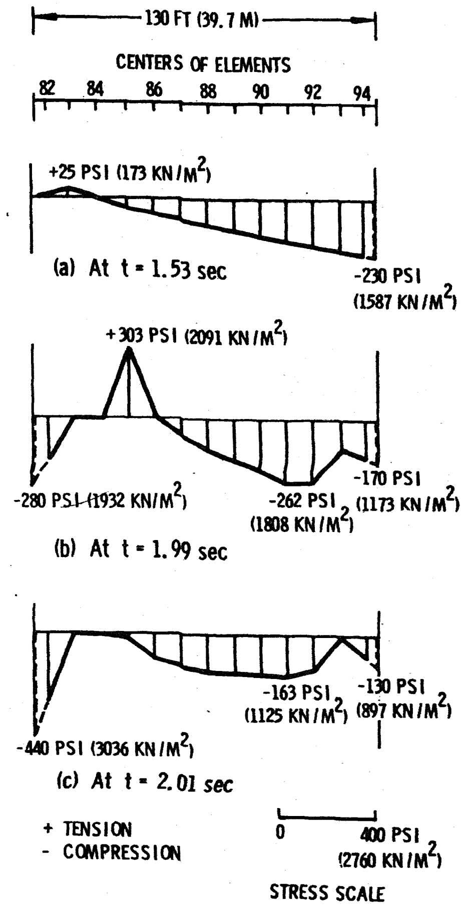

FIGURE 8. VERTICAL STRESS DISTRIBUTION ALONG SECTION B-B (see Fig. 1)

DISCUSSION

K. Madhavan (India)

The paper raises some very interesting questions. It is not clear from the paper as to how the pore pressure have been included in the analysis and whether the stresses are those obtained after including pore pressures.

Regarding the occurrence of tensile stresses and their significance, it is seen that overturning may not be critical. As far as sliding is considered, it will be interesting know the magnitude of crack opening width which occurred. The reduction in shear strength will depend upon this (1).

If we consider the opening as a conduit with a closed end, the build up of excess pore pressure occuring at the closed end may rise upto twice the difference between initial pressure at the end and reservoir pressure at the face. Probably an iterative analysis may be required to trace the progress of the cracks.

It appears that the displacement is altered by nonlinear characteristics of the concrete. Would the authors comment whether as a result of analysis a triangular distribution of equivalent static acceleration with maximum at the top would represent acceptable for design when choosing the section of the dam. At the design stage it is important to have a realistic distribution of equivalent static load so that the section will be optimal.

1. "Mechanism of Shear Resistance of Concrete Beams", Journal Structural Division ASCE October 1968.

Author's Closure

Not received.