06 Apr 1995, 1:30 pm - 3:00 pm

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Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 2-7, 1995, Volume I, St. Louis, Missouri

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Paper No. 6.07

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SYNOPSIS Prior to 1970, the majority of earth and rockfill dams were constructed with little regard for earthquake resistant design — especially in the Pacific Northwest which, at that time, was considered an area of low to moderate seismic activity. Since the early 1970's, and in particular since the near-catastrophic failure of the Lower San Fernando Dam in 1971, the vulnerability of hydraulic fill dams to pore pressure build-up and loss of strength as a result of earthquake shaking is well documented. In contrast, very few case histories exist on the liquefaction susceptibility of saturated gravel filters in zoned embankments. This paper summarizes a detailed finite-element, seismic stability analysis for a dam which has a saturated gravel filter of unknown relative density under the entire downstream shell of the embankment.

INTRODUCTION

Bull Run Dam No. 2 is a 145-foot high, zoned earth and rockfill embankment located approximately 30 miles east of Portland, Oregon. The 34-year-old dam is part of a large facility which supplies water to the city of Portland. An aerial view of the dam, spillway structure and reservoir is shown on Figure 1. The main components of the dam consist of a central impervious core, a downstream sandy gravel filter, and outer rockfill shells (Figure 2).

The sandy gravel filter extends underneath the entire downstream shell of the embankment. A significant portion of this zone is saturated, lying below the tailwater elevation of 748 feet. In addition, this saturated section is sandwiched by relatively impervious random fill above, and predominantly clayey native foundation soils below. A review of construction records did not give any indication as to the relative density of the filter material.

Could a major earthquake cause liquefaction of the gravel material and subsequent failure of the embankment? In order to evaluate the liquefaction resistance of the saturated downstream gravel filter, a comprehensive seismic stability analysis was performed utilizing a procedure originally proposed by Seed (1966). This procedure has undergone numerous improvements and advancements over the past 25 years, which have been summarized in a recent paper by Seed and Harder (1990).

In general, the procedure consists of calculating the static stresses in the dam prior to the earthquake, and the dynamic stresses and accelerations within the dam during the earthquake using finite-element methodology. The Factor of Safety against liquefaction $(FS)_\ell$ is calculated by comparing the cyclic shear strength of the material (obtained using SPT-based correlations) with the cyclic shear stresses induced by the earthquake. Based on the calculated $(FS)_\ell$, residual excess pore pressure ratios are assigned to the gravel filter. Conventional slope stability analyses are performed on the embankment using the increased pore-water pressures within the filter zone. Finally, permanent embankment deformations are estimated based on maximum crest acceleration.

SITE INVESTIGATION

In order to evaluate the relative density of the downstream gravel filter, two pairs (4 total) of Becker hammer drillholes were advanced through the downstream shell of the embankment and the filter zone. Each pair of drillholes consisted of drilling the first hole with an open-bit to obtain samples and determine the depth to the filter zone; then an adjacent hole was drilled with a closed-end bit to obtain a continuous record of the blowcounts per foot of drilling. The Becker hammer blowcounts within filter material were then correlated to equivalent Standard Penetration Test $(N_1)_{60}$ blowcounts using a procedure developed by Harder and Seed (1986).

MATERIAL PROPERTIES

Material properties of the main components of the dam and the foundation soil were required for the static and dynamic finite element analyses. The dam is situated on approximately 90 feet of foundation soil which, in turn, overlies basalt bedrock.

For the static analysis, nine hyperbolic stress-strain and strength parameters were assigned to each material type based on existing information (e.g. boring logs, laboratory

Fig. 1 Aerial View of Bull Run Dam No. 2 (left-center of photo), Reservoir, Spillway Approach Canal, and Spillway.

tests, and construction records). Where there was insufficient information, a comparison of classification soil properties with typical values of hyperbolic parameters for similar material types was made (Duncan et al., 1980). The hyperbolic parameters for each material type are summarized on Table 1.

The dynamic analysis required an initial estimate of the low strain shear modulus (G_{max}) and damping. These initial values were iterated during the analysis to obtain strain compatible results. An initial estimate of G_{max} for cohesive materials was obtained from a relationship of G_{max}/S_u developed by Egan and Ebeling (1985). For cohesionless soils, the initial G_{max} was estimated from the equation $G_{max} = K_2(\sigma_m')^{1/2}$. Damping values were obtained from relationships developed by Seed et al. (1970 and 1984).

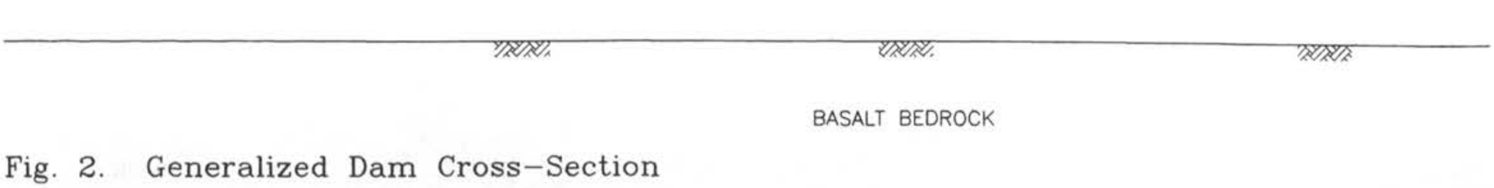

Fig. 2. Generalized Dam Cross-Section

Parameter	Foundation Material	Filter Material	Core Material	Shell Material
Friction Angle (Ø')	34	40	27.5	42
Change in Friction Angle (ΔØ')	0	8	0	8
Cohesion (c'), ksf	1	0	.46	0
Modulus Number (K)	340	450	235	400
Unloading Modulus No. (K_{ur})	510	675	350	600
Modulus Exponent (n)	.5	.4	.4	.5
Failure Ratio (R_f)	.70	.65	.88	.70
Bulk Modulus Number (K_b)	200	300	350	275
Bulk Modulus Exponent (m)	30	.28	.50	.30

Table 1. Hyperbolic Parameters for Static Finite Element Analysis

The foundation material underlying the embankment consists of a matrix of stiff clay interspersed with a heterogeneous mixture of sand, gravel, and boulders. This deposit is part of a large ancient landslide which originated in the slopes above the right abutment. Several triaxial shear tests on the clay fraction indicate an undrained shear strength ranging from 6 to 12 ksf. Atterberg limits on the clay fraction indicate an average PI = 35. The initial estimate of the low strain shear modulus was 5,000 ksf. Since the foundation material has a relatively high percentage of cohesionless soils which are not reflected in the Atterberg test results, the shear modulus degradation curve used was for a PI=0-10 material.

The impervious core consists of a stiff to very stiff, sandy, silty clay with abundant fine to coarse gravel. Relatively undisturbed, thin-walled tube samples were obtained during a 1986 FERC dam safety review. The results of laboratory tests on 5 samples indicate an average saturated unit weight of 123 pcf, with PI's ranging from 16-20. The undrained shear strength ranged from 2 to 4.5 ksf. Several values of G_{max} were input into the analyses to evaluate the sensitivity of the results to G_{max}. The shear modulus degradation curve for material with PI = 10-20 was input into the dynamic analysis.

The downstream gravel filter consists of a slightly silty, sandy, fine to coarse gravel. The gravel has a $D_{50} = 1/4''$, and 6 percent by weight passing the No. 200 sieve. The Becker blowcounts were correlated to an average SPT blowcount of $(N_1)_{60} = 27$ for the two closed-end drillholes. This is approximately equivalent to a relative density of 70-75 percent. Representative values of the shear modulus coefficient (K_2 = 60 to 100) versus the relative density were obtained from charts developed by Seed et al. (1984). The shear modulus degradation curve for sands was used in the dynamic analysis.

The embankment shell consists of blasted basalt rockfill with a maximum diameter of 3 feet. Seismic refraction data indicate a P-wave velocity of 1,600 to 2,500 fps within the shell material. The low strain shear modulus was estimated from the relationship $G_{max} = \gamma/g(V_s)^2$. Where V_s is the shear wave velocity, g is the acceleration due to gravity, and γ is the moist unit weight.

FINITE ELEMENT STATIC STRESS ANALYSIS

A detailed finite element model consisting of 971 nodes and 934 elements was developed for the static stress analysis (Figure 3). The static horizontal shear (τ_{hv}) and effective vertical stress (σ_v') in the two-dimensional, plane strain model were calculated using the program FEADAM84 (Duncan et al., 1984). The program calculates the stresses and strains by incrementally loading each finite element layer from the base of the dam to the crest. The stresses in the lower layers are recalculated for each successive load increment. The materials under the water table were modelled using buoyant unit weights. Following the incremental loading of the dam, hydrostatic forces assuming full reservoir conditions were applied to the upstream face of the impervious core.

A contour plot of the ratio of the static, horizontal shear stress to the vertical effective stress, alpha ($\alpha = \tau_{hv}/\sigma_v'$), is shown on Figure 4. Alpha values are used to modify the

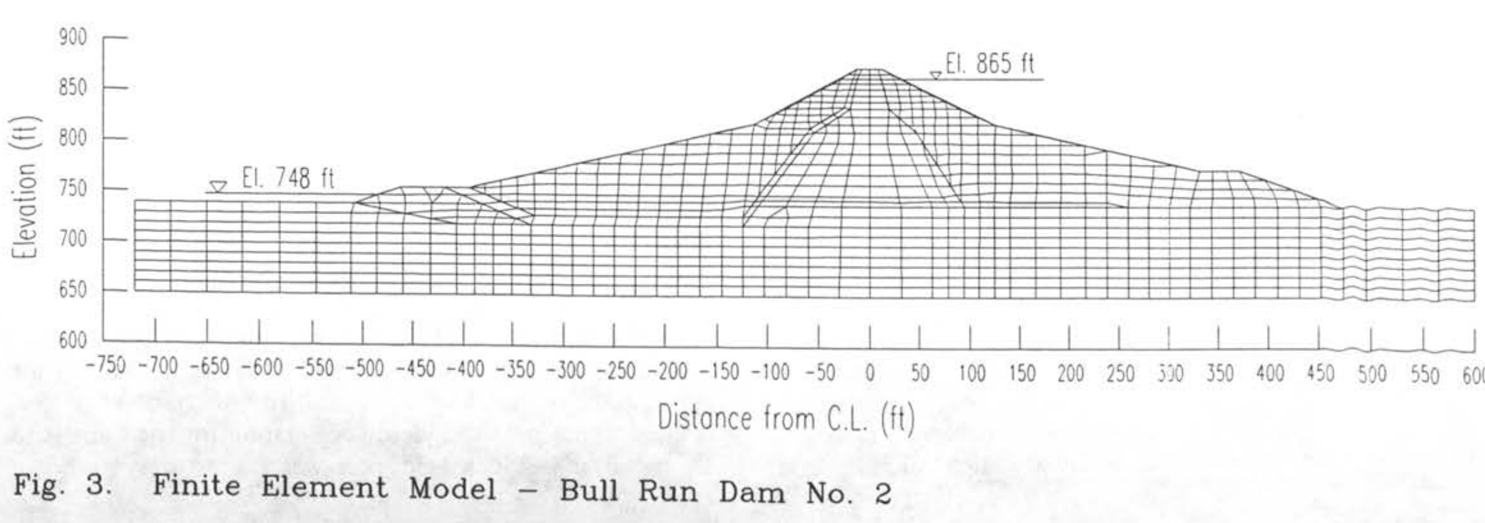

Fig. 3. Finite Element Model – Bull Run Dam No. 2

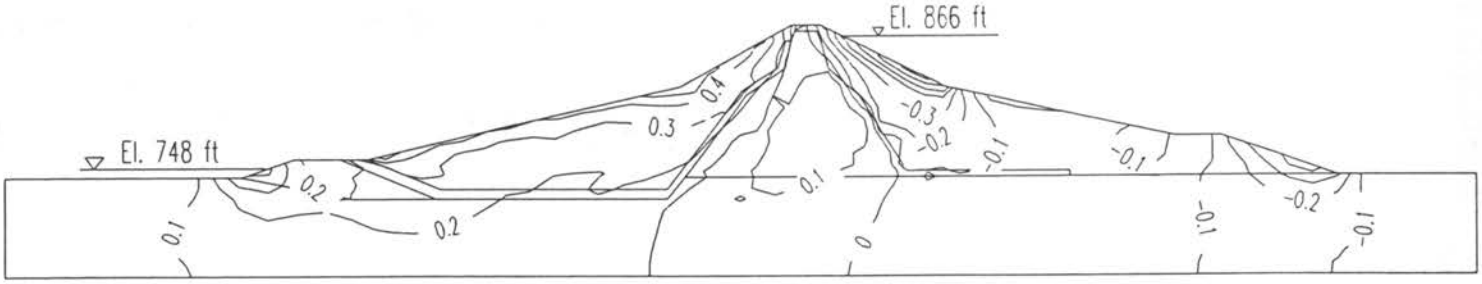

Fig. 4. Contours of Alpha Values

cyclic shear strength of a material to account for the effects that in-situ shear stresses have on the potential for a material to develop excess pore-water pressures. Alpha values for the saturated portion of the filter range from 0.12 to 0.30.

GROUND MOTION

A seismic hazard evaluation of the dam site was performed prior to the stability evaluation (Cornforth Consultants, 1992). This study evaluated maximum credible earthquakes for three seismogenic sources: (i) various crustal faults occurring in the North American plate, (ii) an intraplate earthquake occurring in the subducting Juan de Fuca plate, and (iii) an interface earthquake occurring in the subduction zone between the Juan de Fuca and the North American plate.

This paper will only discuss the results of the critical crustal earthquake since the highest accelerations were produced by this event. The controlling crustal event would have a magnitude of 6.5, be located 15 km from the site and would be capable of producing an 84th percentile peak acceleration of 0.36g. An acceleration time history was developed by modifying a naturally recorded time history to match a smooth acceleration response spectra for rock.

The free-field ground motion time history at the surface of the foundation soil was calculated using the program SHAKE. The foundation soil was subdivided into 10 layers for computational purposes. The modulus reduction curve for low plasticity soils was input into the program. The results of the SHAKE analysis indicates that the peak acceleration at the free-field ground surface would be 0.41g.

FINITE ELEMENT DYNAMIC RESPONSE ANALYSIS

The free-field time history of the foundation soil was input into the finite element program FLUSH. The same finite element model of the embankment that was developed for the static analysis was used for the dynamic analysis. The results of the dynamic analysis indicate that the maximum crest acceleration would be 0.40g (Fig. 5), which is essentially the same acceleration as the free-field motion. The cyclic shear stresses on a horizontal plane were combined with the static vertical effective stress to estimate the cyclic stress ratio generated by the earthquake, $(CSR)_{eq} = \tau_{eq}/\sigma_v'$, for the elements within the saturated portion of the gravel filter.

LIQUEFACTION EVALUATION

An estimate of the cyclic stress ratio required to cause liquefaction (CSR) in the saturated filter was obtained from the empirical chart developed by Seed et al. (1984) based on SPT $(N_1)_{60}$ blowcount values. The cyclic stress ratio obtained from this chart was modified for sloping ground conditions, vertical effective stresses other than 1 tsf, and for an earthquake magnitude other than M7.5. These steps have been well documented in numerous papers and will not be discussed in this paper.

The cyclic stress ratio induced by the earthquake $(CSR)_{eq}$ was compared with the cyclic shear strength of the gravel to obtain a Factor of Safety against liquefaction $(FS)_{\ell}$. The results of the analysis indicated that the various elements within the gravel filter had Factor of Safety against liquefaction ranging from 1.6 to 2.6.

SEISMIC STABILITY ANALYSIS

Based on the calculated factor of safety against liquefaction, residual excess pore pressure ratios were assigned to each element based on a chart developed by Marcuson and Hynes (1990). These increased pore-water pressures were converted into an equivalent head in feet of water above the existing water table. Static slope stability analyses were then performed to evaluate the post-earthquake stability of the embankment. The results of the analysis indicated that the embankment would have a minimum factor of safety against shear failure of 1.7.

The peak crest acceleration was used to estimate the permanent displacement which would occur as a result of the earthquake. A series of slope stability analyses were performed to evaluate the yield acceleration for each analyzed shear surface. Increased pore-water pressures were not

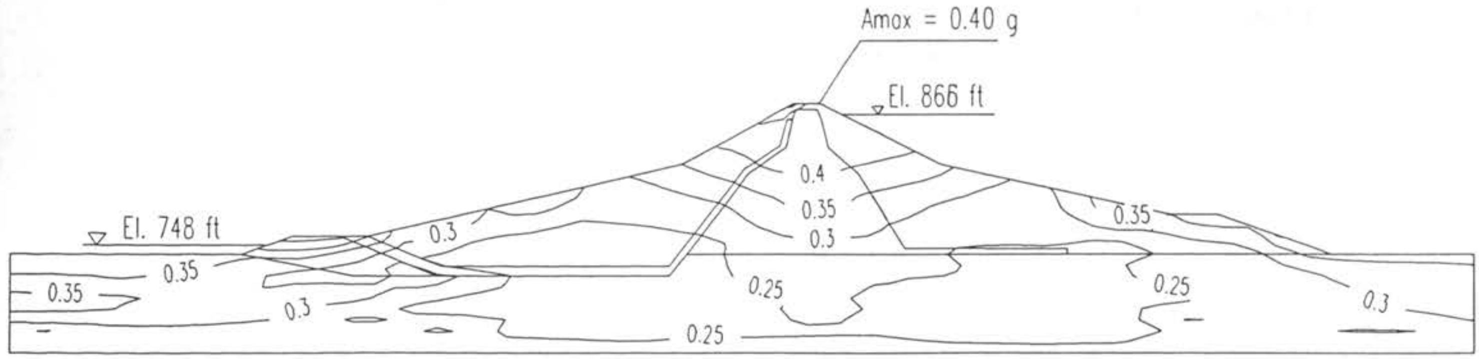

Fig. 5. Peak Horizontal Acceleration

modelled in the saturated filter zone as it was reasoned that the cyclic motions would have ceased prior to the pore pressure generation. The results of the Makdisi-Seed procedure indicate relatively small deformations on the order of 4 to 8 inches.

CONCLUSIONS

A case history has been presented for a finite element, seismic stability analysis of a zoned embankment dam with a saturated gravel filter zone of unknown relative density. Becker hammer drillholes were used to obtain blowcounts of the filter zone. The Becker blowcounts were correlated to equivalent SPT $(N_1)_{60}$ blowcounts to obtain an estimate of the cyclic shear strength of the gravel. The cyclic strength was modified for initial static shear stress using alpha values from FEADAM84; for a M6.5 earthquake; and for effective vertical stress. The cyclic shear stresses and accelerations induced by the earthquake were calculated using FLUSH. The cyclic strength was compared with the cyclic stress induced by the earthquake to obtain an estimate of the factor of safety against liquefaction. Based on the factors of safety, increased pore-water pressures were modelled in the filter zone and the stability of the embankment was analyzed using conventional, limit equilibrium slope stability methods.

The results of the liquefaction analysis indicate that the saturated gravel filter zone in Bull Run Dam No. 2 will maintain adequate shear strengths following a major earthquake. Although the results are favorable for this dam, $(FS)_\ell = 1.6$ to 2.6, the issue of potentially liquefiable filter zones in other zoned embankment dams is one which deserves the attention of geotechnical engineers, whether designing a new embankment or performing a safety evaluation on an existing dam.

ACKNOWLEDGEMENTS

The work described in this paper was performed for the City of Portland, Bureau of Water Works. The writers wish to thank the following people for their experience and assistance during the project analyses and with the preparation of this paper:

Derek H. Cornforth, President
Cornforth Consultants, Inc.

Ignacio Arango, Manager of Geotechnical Engineering
Bechtel Corporation

Farhang Ostadan, Chief Soils Engineer
Bechtel Corporation

Kenji Yamasaki, Geotechnical Engineer
Cornforth Consultants, Inc.

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