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Xhou Jian
Tongji University, Shanghai, China

Zeng Guoxi
Zhejiang University, Zhejiang, China

Wu Shiming
Zhejiang University, Zhejiang, China

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Zhou Jian
Associate Professor of Geotechnical Engineering, Tongji University, Shanghai, China

Zeng Guoxi
Professor of Civil Engineering, Zhejiang University, Zhejiang, China

Wu Shiming
Professor of Civil Engineering, Zhejiang University, Zhejiang, China

SYNOPSIS: Presented in this paper is a comparison between the results of 2-D and 3-D dynamic effective stress analyses of a hypothetical earth dam and two real tailing dams. The study is based on a method of rigorous nonlinear dynamic analysis by taking account the interaction of the fluid and soil phase of the material.

INTRODUCTION

In current engineering practice the dynamic response of large earth dams, whether in wide valleys or in narrow canyons, is usually determined by independently computing the dynamic response of the main section of the dams using 2-D analysis procedures. Since, for dams in narrow canyons, the response of structure is of a 3-D nature, considerable judgment is required to estimate the overall dynamic response from that computed for main section of the dam(Mejia et al., 1982). The most widely used methods for dynamic analysis are based on the equivalent linear model(Seed, 1979). In recent years, there has been a distinct shift towards the use of non-linear total or effective stress methods of analysis(Prevost et al.,1985, Finn et al.,1987). In effective stress analysis the pore water pressure can be computed, and its effects during the dynamic response calculation can be taken into account.

The purpose of this paper is to present a comparison between the results of 2-D and 3-D dynamic effective stress analyses of a hypothetical earth dam and two real tailing dams. The study is based on a method of rigorous nonlinear dynamic analysis by taking account the interaction of the fluid and soil phase of the material. The hyperbolic model by Hardin and Drnevich(1972) is taken as stress-strain skeleton curve, Masing rule is used to determine unloading and reloading, and Masing-type curve is revised with damping by Hardin's equation. The tendency to develop volumetric strains due to cyclic shear stress is taken into account by introducing a modified form of pore water pressure model presented by Seed into constitutional equation.

ANALYSIS METHOD

Governing Equation

In 1962, Biot established the basic equation of a saturated solid, porous medium under dynamic conditions. A version of his governing equations extended to nonlinear behavior is represented as following(the pore water acceleration has been neglected)

$$[L]^T[D][L]\{u\}-[L]^T\{m\}p = -[L]^T U_g - \rho\{g\} + \rho(\{\ddot{u}\}+\{\ddot{u}_g\}) \quad (1)$$

$$\{\nabla\}^T[k]\{\nabla\}p-\{m\}^T[L]\{u\}=\{\bar{f}\} \quad (2)$$

where
- $[L]$ = Appropriate differential operator defining strains in terms of displacements
- $[D]$ = Tangent modulus matrix
- $\{u\}$ = Displacement vector
- $\{m\}^T$ = $[1,1,1,0,0,0]$
- p = Pore water pressure
- U_g = Seismic pore water pressure
- ρ = Density of soil
- g = Gravity acceleration
- $\ddot{u}$ = Relative aceleration
- $\ddot{u}_g$ = Input earthquake acceleration
- $[\bar{k}]$ = $[k]/(\rho g)$
- $[k]$ = Permeability matrix
- $\{\bar{f}\}$ = Seepage discharge vector

Constitutive Model

A shear stress-strain backbone curve suggested by Harding(1972) is adopted, and the masing rule is used to determine unloading and reloading. Masing-type curve has been revised with damping by Hardin's equation(Hardin and Drnevich, 1972). After each load reversal,if stress is less then maximum stress occured before, the tangent shear modulus is determined by

$$G=G_{ms}+\kappa(\gamma_m)\{\frac{G_{max}}{[1+(\gamma-\gamma_a)/(2\gamma_r)]^2}-G_{ms}\} \quad (3)$$

where: G_{ms} = Secant modulus of the back-bone curve at the point of maximum shear stress occured before

$\kappa(\gamma_m)$ = Revision coefficient of damping ratio corresponds to the maximum shear stress occured before

γ_a = Shear strain at the point of reversal

γ_r = So-called reference strain

G_{max} = Initial tangent shear modulus

Formula of Pore Water Pressure under Cyclic Loading

A modified form of pore water pressure model presented by Seed (Seed, at al., 1976) is used for analysis as follows:

$$\frac{U_g}{\sigma_m}=\frac{1}{2}+\frac{1}{\pi}\arcsin[\beta(\frac{N}{N_m})^{1/\alpha}-1] \quad (4)$$

where: σ_m = Total mean stress

N = Number of loading cycles in t time

N_m = Number of loading cycles to cause maximum pore water pressure

α = Parameter related with dynamic shear stress ratio

β = Parameter related with static shear stress ratio

Dynamic Analytical Procedures

The equation (1), (2) can be solved numerically under giving boundary and initial condition by FEM. The weighted residual method and 3-D isoparametic element with eight nodes is used to formulate the following set of finite equations:

$$[K]\{\delta\}+[Q]\{p\}+[M]\{\ddot{\delta}\}=\{F\} \quad (5)$$

$$[Q]^T\{\delta\}+[H]\{p\}=\{\bar{F}\} \quad (6)$$

where [K] = Stiffness matrix

[Q] = Couple matrix

[M] = Mass matrix

[H] = Permeability matrix

{F} = Nodal earthquake load vector

$\{\bar{F}\}$ = Nodal seepage discharge vectc

$\{\delta\}$ = Nodal displacement vector

$\{\ddot{\delta}\}$ = Acceleration vector

Let $\Delta\delta$ and Δp be the increments of node variable δ and p during time increment Δt the following set of finite diferenc equation is obtained from Eqs (5), (6):

$$[K]\{\Delta\delta\}+[Q]\{\Delta p\}+[M]\{\Delta\ddot{\delta}\}=\{\Delta F\} \quad (7$$

$$[Q]^T\{\Delta\delta\}+[H]\Delta t\{\Delta p\}=\{\Delta\bar{F}\} \quad (8$$

The equation (7),(8) is solved by the fron solution methed. The main steps are a follows

1. Pre-front, form the identificatio vector for assembly and elimination o dynammic equation.

2. Compute the element matrices of dynami equation and the loading vector.

3. Front solution, calculate noda displacements, velocities, acceleration and pore water pressure at each time step

4. Compute strain and stress field fro nodal displacements, determine the ne value of modulus G according to revise masing rule.

5. Calculate seismic pore water pressur increment, and converted it into equivalen nodal force and add it into the loadin terms of Eqs. (7) .

6. Repeat step 2-5 until the end o earthquake motion.

7. Continue post-earthquake static analysi until no further dissipation of pore wate pressure is taking place.

MATERIAL PROPERTIES AND INPUT MOTION

A hypothetical earth dam in triangula shaped canyon, Tonglin minor tailings da and Nanfen tailings dam are presented her for analysis.

The max. height of the hypothetical earth dam is 60m with a max. length of 360m. The dam material is assumed to be tailings fine sand ,as same as that of Tonglin tailings dam(see table 1).

Table 1. Index Properties of Materials

Material	Fine Sand	Slime (Tonglin)	Caly	Fine Sand (Nanfen)
Saturated Unit Weight γ (kN/m^3)	21.9	21.2	19.0	19.4
Specific Gravity G_s	3.27	3.25	2.70	3.27
Uniformity Coefficient U	5.80	9.10	--	6.50
Median Grain Size d_{50} (mm)	0.04	0.02	--	0.05
Effective Cohesion C' (kPa)	0	0	9.8	0
Angle of Internal Friction ϕ'(°)	30	28	24	33
Possion Ratio μ	0.35	0.37	0.40	0.32
Coefficient of permeability $k(10^{-5} cm/s)$	52.0	4.8	0.0087	120.0
Maximum Shear Modulus Coefficient k_{2max}	47	43	30	30
Maximum Damping Ratio D_{max}	0.30	0.29	0.28	0.30

The max. height of Tonglin minor tailings dam is 51m with a max. length of 150m. Upstream and downstream slopes are 1:65 and 1:4 respectively. The dam material consists of tailings fine sand, slime and clay as listed in table 1. The bedlock is covered with 5m thick soil deposits.

The max. height of Nanfen tailings dam is 120m with a max. length of 520m. The dam is assumed to be homogeneous. Main material properties and computation parameters are listed in table 1.

An accelerogram recorded at Qian-an, Tangshan during a major after-shock(magnitude 6.3) on August 31, 1976, is used as input motion with the scaled peak acceleration 0.2g and predominant period 0.28 second. The motion is assumed to be along the upstream-downstream direction and the shock duration is assumed as long as 10 second(Fig. 1).

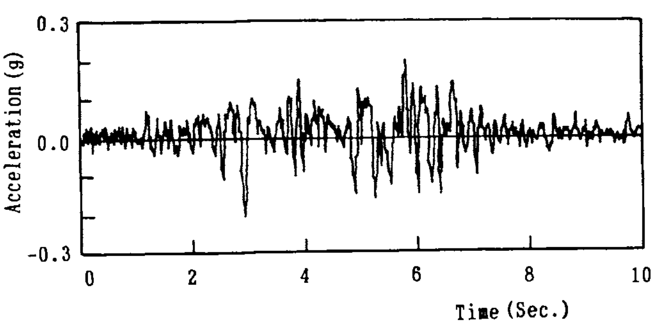

Fig. 1 Input Motion

COMPARISON OF RESULTS

Fig. 2 shows distribution of residual pore water pressure ratio(p/σ_m) for the hypothetical dam at the end of earthquake. It can be seen that there exists a high pore water zone together with a restricated zone of liquefaction near the crest in section D-D'which is near the abutment. The pore water pressure in the maximum section A-A' is lower than that in section D-D', which indicates that the maximum section may not be the most critical from a stability point of view.

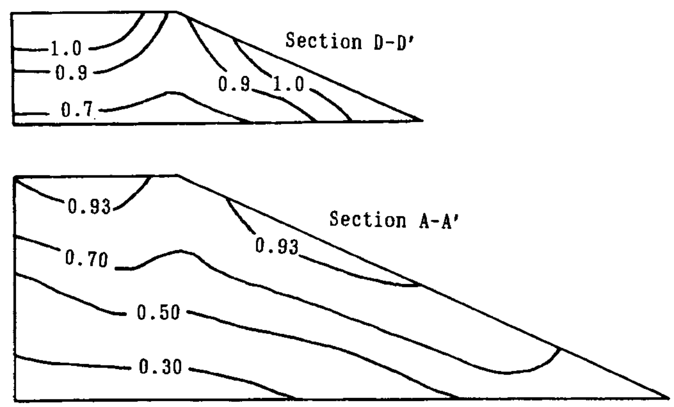

Fig.2 Distribution of Pore Water Pressure Ratio,p/σ_m, Computed for Maximum and Quarter Section of Hypothetical Dam

Fig. 3 shows the distribution of residual pore water pressure ratio(p/σ_m) for Tonglin minor tailings dam. It is again observed that the pore water pressure in the maximum section A-A' is lower than that in section E-E'.

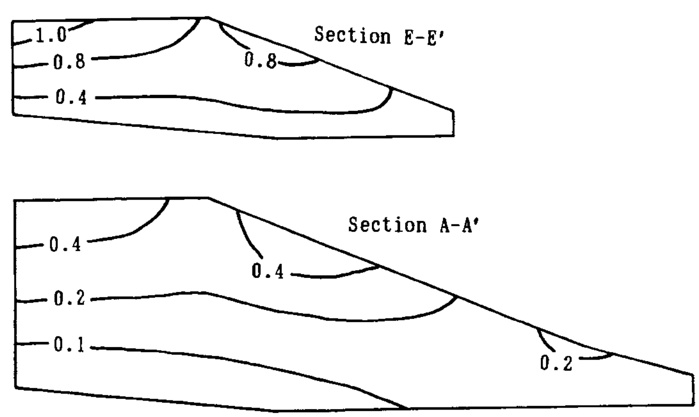

Fig.3 Distribution of Pore Water Pressure Ratio,p/σ_m, Computed for Maximum and Quarter Section of Tonglin Minor Dam

Fig. 4 - Fig. 6 show the distribution of ratio between the pore water pressure using 2-D and 3-D models, p_{2D}/p_{3D} , for three dams. It can be seen that in the zone near the crest and slop of the dam, the pore water pressures computed by 2-D analysis are 10% to 40% higher than those calculated from 3-D analysis. Since these areas are more sensitive to liquefy, 2-D dynamic analysis will give more critical results than 3-D analysis do.

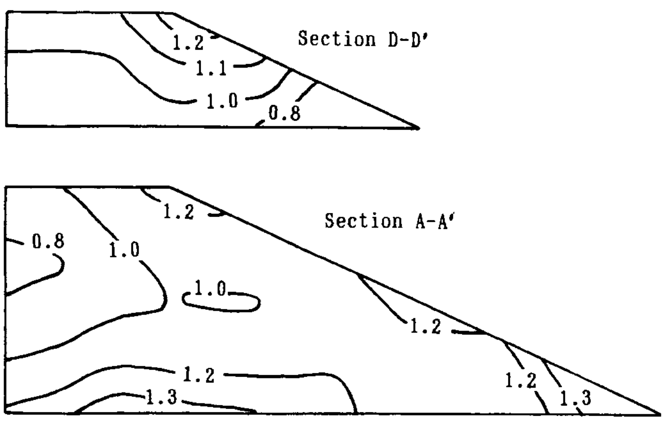

Fig.4 Distribution of Ratio, p_{2D}/p_{3D} , Computed for Maximum and Quarter Section of Hypothetical Dam

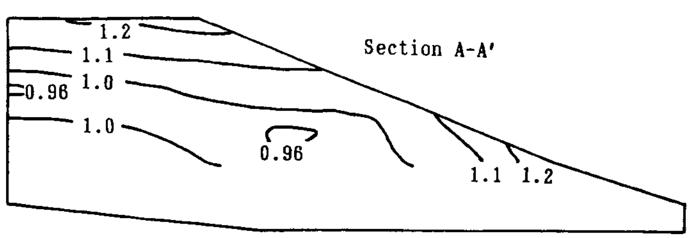

Fig.5 Distribution of Ratio, p_{2D}/p_{3D} , Computed for Maximum Section of Tonglin Minor Dam

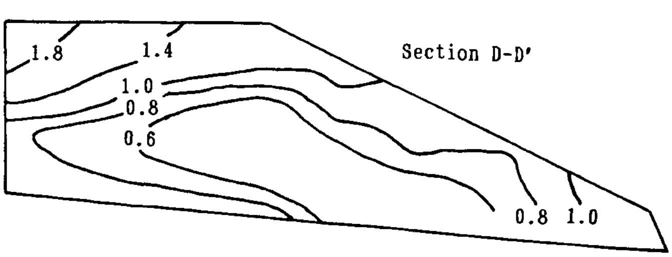

Fig.6 Distribution of Ratio, p_{2D}/p_{3D} , Computed for Maximum Section of Nanfen Tailings Dam

CONCLUSIONS

On the basis of the results of comparision it can be concluded:

a. The maximum pore water pressure rati can occur at sections other than th maximum section, which indicates that th maximum section may not be the mos critical from a stability point of view.

b. In the zone near the crest and slop o the dam, the pore water pressures compute by 2-D analysis are higher than thos calculated from 3-D analysis. Since thes areas are more sensitive to liquefy, 2- dynamic analysis will give more critica results than 3-D analysis do.

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