Dynamic Response of Earth Dam-Foundation System

Lin Gao
Dalian University of Technology, Dalian, China

Guan Fei
Dalian University of Technology, Dalian, China

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 7.7

Dynamic Response of Earth Dam-Foundation System

Lin Gao
Professor, Dalian University of Technology, Dalian, China

Guan Fei
Dalian University of Technology, Dalian, China

Synopsis: A numerical procedure is proposed to analyze the seismic response of structure resting on layered medium. By this approach the effects of the dynamic soil-structure interaction (DSSI) upon the response of earth dam on multi-layered halfspace subjetced to seismic ground motions are studied. The dynamic responses of earth dams on rigid base are also evaluated to compare with those on a flexible homogenous halfspace. It is noted that great differences may arise between the results of the two cases. To investigate the influence of the truncated size of foundation in horizontal direction on the dynamic response of the earth dam on a single layered foundation, the conventional finite element method (FEM) are employed, in which the dam together with a finite size of foundation is analyzed. It is shown that, for most cases, DSSI decreases the response amplitudes of the earth dam significantly.

INTRODUCTION

Much progress has been made in recent years in the development of analytical procedure for evaluating the dynamic response of structures. It has become evident that the DSSI effect cannot be neglected for the dynamic response of massive structures. However, in the analysis of earth dam, much attention has been paid to the dynamic constitutive relations of soil. A few efforts have been made for the study of DSSI effects upon the response of earth dam. In the conventional analysis of earth dam, the foundations are assumed to be rigid or the dam-foundation systems are studied by analyzing the dams together with the truncated parts of infinite foundations. Errors may arise for the procedures described above due to the following aspects: 1) the rigid foundation assumption disregards the DSSI effects, so the dynamic behavior of the system can not be fully represented; and 2) if only limited part of the foundation is taken into consideration, it is difficult to determine the seismic motion and to cancel the reflected waves at the boundaries of the finite domain, which may spoil responses of dams. Three types of boundary conditions are normally prescribed at the boundaries of the domain simulating the existence of an infinite or semi-infinite medium outside the structure: 1. Elementary (non-transmmitting) boundary-the Dirichlet or Neumann boundary conditions are imposed at the boundary; 2. Local (imperfect) boundary or viscous boundary-viscous dashpots are placed at the boundary nodes to absorb the propagating wave from inside with different efficiency related to the wave propagating direction; 3. Consistent, nonlocal boundary-any type of wave can be transmmitted at this boundary and the structure-soil interface is taken to be the boundary to minimize the degrees of freedom.

For the first two alternatives, the straightforward finite element scheme cannot be implemented efficiently, since the elementary boundary are perfect reflectors in the sense that no energy is absorbed or transmmitted. And it is important to determine the position of edges for the finite domain modelling the semi-infinite medium due to the fact that the domain has to be large enough for the reflected waves to be damped out before reaching the region of interest. In the present paper, both the elementary and consistent boundary conditions are employed in the analyses.

The purpose of this paper is to study the effects of DSSI on the dynamic responses of earth dams and to investigate the errors that may occur during the seismic response analysis of the dam of truncated domain are used to simulate the infinite foundation.

FORMULATIONS

Foundation Part

The impedance of the foundation plays an important role in the analysis of DSSI problems. In the present approach, the foundation compliance is determined by the analytic solution of wave equation for the layered medium.

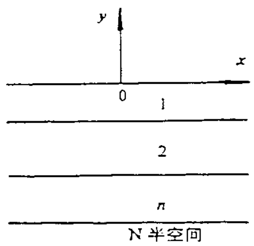

Fig. 1 Layered medium

For each isotropic and homogeneous visco-elastic layer, the Lamê potentials ϕ and ψ are introduced and the wave equations take the form

$$\nabla^2\phi + k_L^2\phi = 0$$

$$\nabla^2\psi + k_T^2\psi = 0 \tag{1}$$

$$k_L = \frac{\omega}{Cp}, \quad k_T = \frac{\omega}{Cs}$$

where k_L and k_T are P and SV-wave complex wavenumbers respectively, ω, Cp and Cs correspond to circular frequency, P- and SV-wave speeds respectively.

The Fourier transformed solution of Eq.(1) is obtained as

$$\phi = A_1 e^{k_1 x_2} + A_2 e^{-k_1 x_2}$$

$$\psi = B_1 e^{k_2 x_2} + B_2 e^{-k_2 x_2} \tag{2}$$

$$k_1 = (\xi^2 - k_L^2)^{1/2}, \quad k_2 = (\xi^2 - k_T^2)^{1/2}, \qquad \mathrm{Re}(k_1, k_2) > 0$$

The displacements and stresses are expressed in terms of ϕ and ψ as

$$\begin{Bmatrix} u_1 \\ u_2 \\ t_{12} \\ t_{22} \end{Bmatrix} = \begin{bmatrix} -i\xi & k_2 & -i\xi & -k_2 \\ k_1 & i\xi & -k_1 & i\xi \\ 2\mu^* i\xi k_1 & \mu^*(2\xi^2 - k_T^2) & 2\mu^* i\xi k_1 & \mu^*(2\xi^2 - k_T^2) \\ \mu^*(2\xi^2 - k_T^2) & 2\mu^* i\xi k_2 & \mu^*(2\xi^2 - k_T^2) & -2\mu^* i\xi k_2 \end{bmatrix} \begin{bmatrix} \exp(k_1 x_2) & & & \\ & \exp(k_2 x_2) & & \\ & & \exp(-k_1 x_2) & \\ & & & \exp(-k_2 x_2) \end{bmatrix} \begin{Bmatrix} A_1 \\ B_1 \\ A_2 \\ B_2 \end{Bmatrix} \tag{3}$$

or $$\{\mathbb{S}\} = [\mathbb{T}][\mathbb{E}]\{\mathbb{A}\}$$

where $\{\mathbb{A}\}$ is the coefficient vector and the above equations hold for each layer with its own constitutive properties.

By applying the interface continuity conditions between the nth layer and the (n+1)th layer, the coefficient vector of the nth layer is related to that of the (n+1)th layer

$$[\mathbb{T}]_n [\mathbb{E}]_n \{\mathbb{A}\}_n = [\mathbb{T}]_{n+1} [\mathbb{E}]_{n+1} \{\mathbb{A}\}_{n+1}$$

or $$\{\mathbb{A}\}_n = [\mathbb{E}]_n^{-1} [\mathbb{T}]_n^{-1} [\mathbb{T}]_{n+1} [\mathbb{E}]_{n+1} \{\mathbb{A}\}_{n+1} \tag{4}$$

Sucessively applying the continuity conditions, the following is readily available

$$\{\mathbb{A}\}_1 = [\mathbb{D}]\{\mathbb{A}\}_N \tag{5}$$

With the aid of Eq.(5) and Eq.(3) the displacements can be expressed in terms of the tractions on the surface

$$\{\mathbb{U}\} = [\mathbb{F}]\{t\} \tag{6}$$

By the invers Fourier transform

$$f(x_1, \omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} f(\xi, \omega) e^{-i\xi x_1} d\xi \tag{7}$$

the solution to Eq.(6) in space domain is available. The numerical integration of Eq.(7) may be referred to [3].

The impedance of the foundation is obtained as

$$[S_{bb}^g] = [M][F]^{-1} \tag{8}$$

where [M] is a transform matrix.

Structure Part

For convenience the earth dam is assumed to be visco-elastic. The soil dynamic shear modulus can be expressed as[2]

$$G_{max}=22.1K_{2max}(\sigma_m)^{0.5} \qquad (kg/cm^2)$$

$$\sigma_m=(1+2K_0)\gamma z/3 \tag{9}$$

$$G^*_{max}=(1+2i\eta)G_{max}$$

where K_{2max} is the shear modulus coefficient, ranging from 80 to 180 for relatively dense well-graded gravels. In this paper K_{2max} equals to 110. σ_m is the effective mean principal stress; K_0 is the confining pressure coefficient; and η is the hysteretic damping coeffient of the soil. While the dam is subjected to moderate earthquake loadings, the shear strain amplitude induced in the soils is about $10^{-3}\%$-$10^{-2}\%$. Therefore the modulus G_{max} is to be multiplied by a reduction factor of 0.6. After the dam is discretized, by applying the continuity condition at the interface of dam and foundation, the formulation is readily available for the dam foundation system

$$\begin{bmatrix} S_{ss} & S_{sb} \\ S_{bs} & S^s_{bb}+S^g_{bb} \end{bmatrix} \begin{Bmatrix} U^t_s \\ U^t_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ S^g_{bb}U^g_b \end{Bmatrix} \tag{10}$$

where subscript 'b' denotes the point on the interface of the dam and foundation; subscript 's' denotes all the remained points of the dam. U^t_s and U^t_b are the corresponding displacements. S_{ij} is the dynamic stiffness matrix

$$S_{ij}=(1+2\eta i)\quad [K_{ij}] \quad -\omega^2[M_{ij}] \tag{11}$$

where $[K_{ij}]$ and $[M_{ij}]$ are the stiffness matrix and mass matrix; η is the hysteretic damping ratio; and ω is the exciting frequency. U^g_b is the free field ground motion at the interface of dam and foundation.

NUMERICAL EXAMPLES

Firstly, the dynamic responses of the dam shown in Fig.2 are evaluated under seismic ground motions. Cases of rigid and flexible homogeneous foundations are considered. The dynamic soil properties of the dam and foundation are as follows: Poisson ratio $\upsilon=0.3$; hysteretic damping coefficient $\eta=0.75$; unit weight $\gamma=2.0t/m^3$ and the SV wave speed in the foundation is 300m/s. The peak acceleration amplitudes of the input earthquakes are adjusted to unit. Time histories of three typical earthquake ground motions are selected as input seismic motions: 1. El-centro earthquake record, May 18,1940, USA; 2. Tangshan subearthquake record at Tianjin Hospital, Nov.11, 1976, PRC, characterized for the softer foundation and 3. artificial earthquake characterized for rock foundation. The spectral repsponse curves of them are dipicted in Figs.3-4. Peak acceleration amplitudes at dam crest are shown in Table I for the two types of foundations under different input seismic ground motions. It is noted that the DSSI decreases peak response amplitudes significantly.

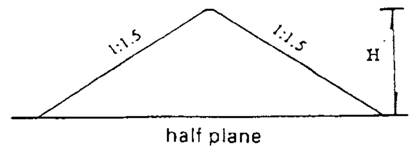

Fig.2 Dam on a halfspace

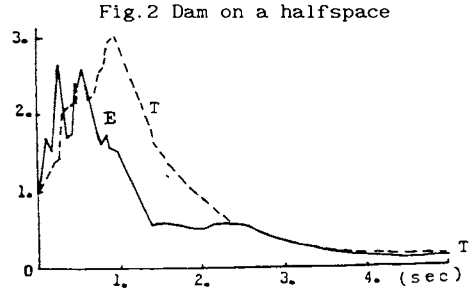

Fig. 3 Spectral response

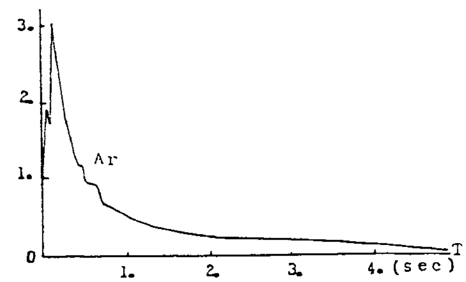

Fig.4 Spectral response

Table I H=100m

seismic motion / Foundation	1	2	3
fexible	2.21	2.39	1.15
rigid	3.67	4.24	2.11

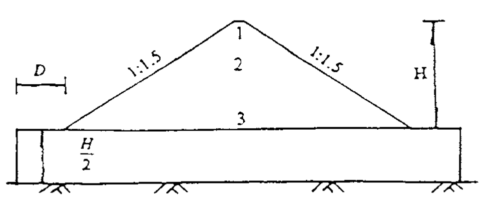

Fig.5 Dam foundation system

The dam-foundation system shown in Fig.5 analyzed by the FEM with a truncated part of foundation is demonstrated as the second numerical example. In Table II,peak acceleration amplifying factors at the dam crest are given for three input motions and for dams with different ratios of D/H, where H denotes the height of the dam and D denotes the length of the truncated foundation outside the dam heels. The case of D/H=∞ corresponds to the system analyzed by Eq.(10).In table III peak acceleration response amplitudes at different points on the central section of dam body are given for foundation with different ratios of D/H as the system subjected to artificial seismic ground motion. The input seismic motions are assumed to be incident from the infinite, which can be obtained by the deconvolution from free field ground motions.

Table II H=100m

D/H / seismic motion	2.0	3.0	4.0	5.0	∞
El-centro	3.27	3.00	3.34	3.35	3.27
Tangshan	3.64	3.39	3.67	3.67	5.76
Artificial	1.29	1.16	1.35	1.33	2.05

Table III H=50m Artificial earthquake

D/H / Point No.	2.0	3.0	4.0	5.0	∞
1	2.15	1.55	2.18	2.19	2.94
2	1.36	1.12	1.36	1.36	1.78
3	0.78	0.61	0.77	0.77	1.29

It is indicated in Table II and Table III that the ratio D/H affects the dam responses not substantially.The peak response amplitudes approach to a definite value for D/H≥4 and fluctuate for D/H<4 more or less depending on the characteristics of seismic motions and dams itself. However, according to the numerical studies carried out in this paper, the response amplitudes for different values of D/H do not exceeds 30%. The numerical results by FEM are different compared with those of the case for D/H ⇒∞. The single layer medium exhibits a strong spectral characteristics. For the vertical propagating SV wave, the free vibration frequencies of the layer are $(2n-1)H/2C_s$, where n is an integer. When the foundation is modelled by a finite domain with limited number of elements, it is impossible to represent the realistic problem perfectly. which may cause error in the numerical analyses. Much work has to be done to study the wave propagating in the finite element grids. Figs.6-9 are the horizontal acceleration distributions along the height of dams.

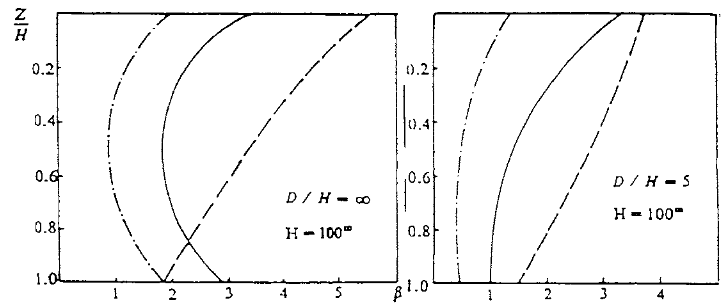

Fig.6— Fig.7 Acceleration distribution along dam height

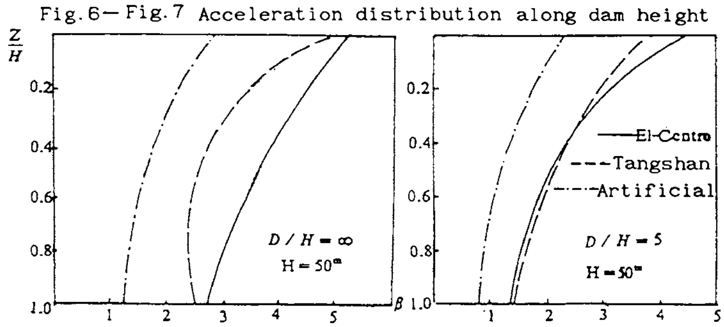

Fig.8—Fig.9 Acceleration distribution along dam height

CONCLUDING REMARKS

The dynamic response of earth dam-foundation system is studied in this paper. It can be conluded on the basis of the numerical examples that the DSSI has a significant influence upon the response of dam. By analyzing the system by FEM, it is shown that the response of dam is not affected by the truncated size in horizontal direction greatly, and the analysis by FEM is only an approximation since the finite domain cannot model the infinite foundation completely. For the ratio D/H in the range of above 3-4, the response amplitude approaches to a definite value. Otherwise the it will fluctuate with varying degree depending on the characteristics of dam and the seismic motion. For some cases, resonable results cannot be obtained by the conventional FEM. And it seems that the errors may increase as the dominant frequencies of the seismic wave increase.

REFERENCES

1. J.P.Wolf,Dynamic Soil-Structure Interaction, Prentice-Hall, 1985.
2. G.R.Seed et al, Moduli and Damping Factors for Dynaimc Analyses of Cohesionless Soils, ASCE, GT, Vol.122,1986
3. Guan Fei and Lin Gao, The Dynamic Compliance of Strip Rigid, Massless Foundation Lying on Multi-layered Visco-elastic Halfspace, Asia Vibration Conference'89, Shenzhen, China, Nov. 1989