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Technical Note

Response of Flyash Ponds under Dynamic Loads

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Introduction

Appreciating the overall concern for environment and the need for safe disposal and gainful utilization of abandoned fly ash ponds for human settlement, it is essential to explore the seismic response of fly ash pond under dynamic loading. Out of over forty thousand acre land covered under abandoned fly ash ponds in India, one thousand acres of land is left unutilized in Panipat and Badarpur, which are seismically active regions, and obviously liquefaction under very severe shaking is a possibility.

In order to evaluate the response of any natural loose deposit under dynamic loads, it is necessary to determine the dynamic properties of the loose deposit. There are several types of field tests in vogue to measure shear modulus and damping. In the present study, block resonance test as recommended in IS 5249-1977 (Method of Test for Determination of Dynamic Properties of Soil) has been conducted at the site and dynamic properties of fly ash have been determined.

Methodology

A standard block 1.5 × 0.75 × 0.70 m high was cast using plain cement concrete of M15 grade, after uniformly excavating top 30 cm of fly ash. The block was excited only in the vertical mode. The mechanical oscillator, which works on the principle of eccentric masses mounted on two shafts rotating in opposite directions was mounted on the block so that it generates purely vertical sinusoidal vibrations. Oscillator was mounted such that the line of action of vibrating force passes through the center of gravity of the block, For vertical vibration mode, two acceleration pickups were fixed on the top

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of the block to sense the vertical motion of the block. These vertical motions were picked up by FFT analyzer. The oscillator frequency was increased in steps 0.25 Hz between the range of 1 to 18 Hz while the step of 0.1 Hz was applied between frequency range of 18-25 Hz and signals were recorded. The same procedure was repeated for the various dynamic force values by changing the eccentricities namely 120°, 90° and 70° between the masses. The maximum values of acceleration at given frequency was recorded. The amplitude of vibration (A_z) at a given frequency (f) is given by

$$A_z = \frac{a_z}{4\pi^2 f^2}$$

where a_z = vertical acceleration of vibration in mm/sec^2 of the block at frequency f, and

f = frequency in Hz.

The amplitude versus frequency curve has been plotted in Fig. 1 for each eccentricity to obtain the natural frequency of the flyash deposit and the

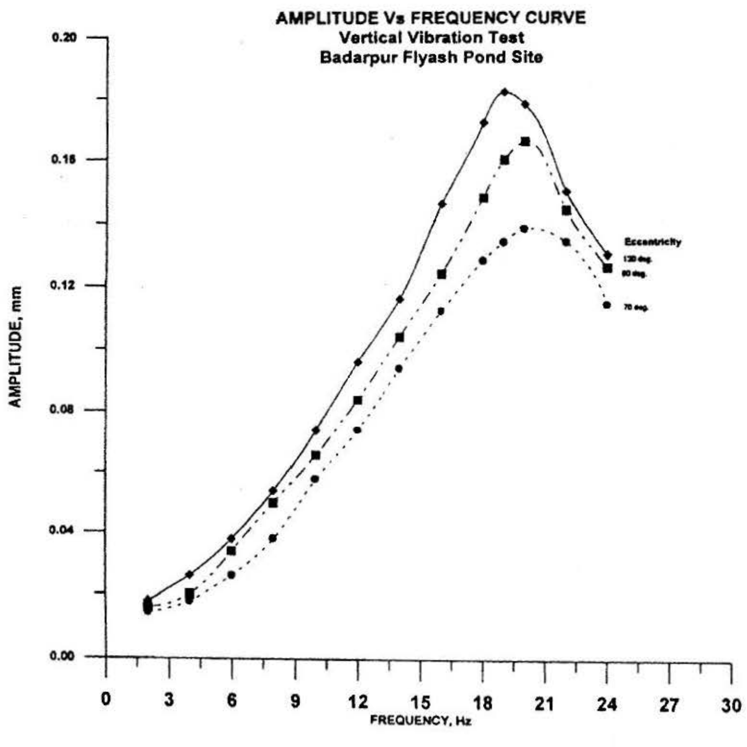

FIGURE 1 : Amplitude Vs. Frequency Curve for badarpur Flyash Pond Site

foundation block tested. The natural frequency (f_{nz}) was seen to be 18.9 Hz.

The coefficient of elastic uniform compression (C_u) for fly ash was computed from the expression

$$C_u = \frac{4\pi^2 f_{nz}^2 M}{A}$$

where A = contact area of the block with soil (1.5 × 0.75 m)

M = mass of the block, oscillator and motor (W/g = 189 kg/cm/sec^2)

Substituting these values in the above equation, C_u comes out to be 2.39 kg/cm^3. The computed coefficient of elastic uniform compression and modulus of elasticity is related by the expression

$$C_u = \frac{1.13E}{(1-\nu^2)\sqrt{A}}$$

Similarly, modulus of elasticity and Poisson's ratio are related to shear modulus by the expression

$$G = E/2(1+\nu)$$

Poisson's ratio (ν) was taken as 0.35 for the site under consideration. The above computation results E =160.73 kg/cm^2 and G = 59.6 kg/cm^2. Further, shear modulus is related with shear wave velocity by

$$V_s = \sqrt{\frac{G}{\rho}} = 76.001 \text{ m/s}$$

where mass density of flyash is taken as 1.1 gm/cc.

Viscous Damping

In case of vertical vibration test, the value of damping coefficient ξ of soil is given by following equation

$$\xi = \frac{f_2 - f_1}{2 f_{nz}}$$

Dynamic Properties of BTPS Flyash Pond

C_u (kg/cm^3)	2.390
E (kg/cm^2)	160.730
G (kg/cm^2)	59.600
Vs (m/s)	76.001
ξ	0.255

where f_2, f_1 = two frequencies at which the amplitude is equal to $x_m/\sqrt{2}$

x_m = Maximum amplitude at f_{nz}

The damping coefficient (ξ) is found to be 0.255.

Analysis and Results

Seismic Response

With the above properties, a one-dimensional model for the pond as shown in Fig. 2 was analyzed using SHAKE91, a software for conducting equivalent linear seismic response analyses of horizontally layered soil deposits.

It was assumed that the rock is at 60 m depth below the ground level and flyash is underlain by silty sand. At the rock level, El-Centro (1940, N-S

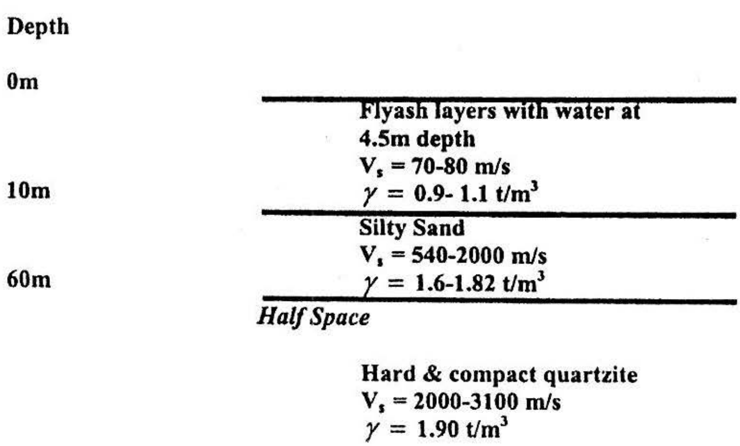

FIGURE 2 : One Dimensional Soil System at Badarpur Flyash Pond Site

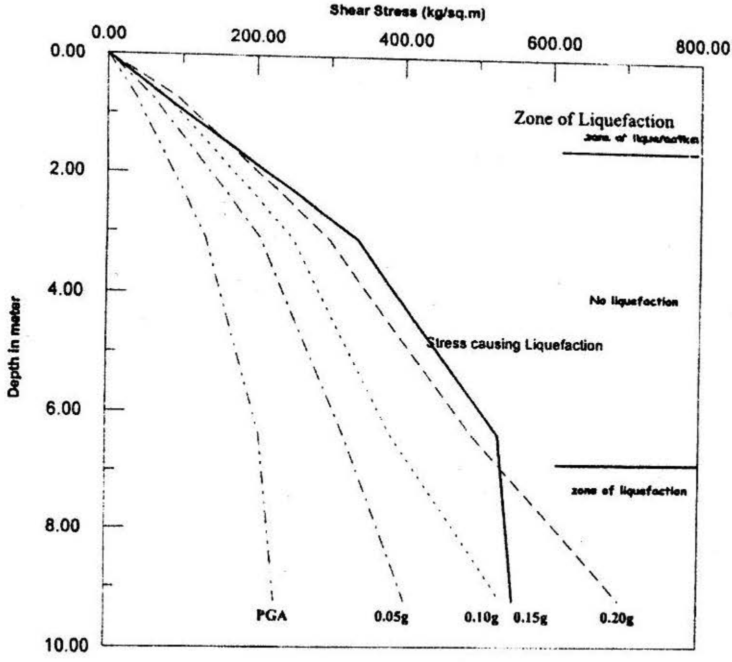

FIGURE 3 : Maximum Shear Stress Distribution due to Different Earthquakes

Component) earthquake accelerogram has been applied. The peak acceleration has been scaled to 0.05 g, 0.10 g, 0.15 g and 0.20 g and the response due to these four earthquakes has been found. From the response, it was found that the maximum acceleration is attenuated maximum by the flyash pond.

One of the major causes of destruction during an earthquake is failure of foundation due to loss in bearing. This loss in strength may take place in sandy soils due to increase in pore pressure. The increase in pore pressure causes reduction in shear strength, which may even be lost completely. Soil that has lost all its shear strength behaves like a viscous fluid and this phenomenon is termed liquefaction. Flyash is essentially a silty soil and an attempt has been made to evaluate its liquefaction potential. The shear stress distribution along the depth during an earthquake has been plotted in Fig. 3 for all the four cases i.e. for peak accelerations equal to 0.05 g, 0.10 g, 0.15 g and 0.20 g. Using the simplified procedure as given by Seed and Idriss (1971), the stress causing liquefaction has been calculated and plotted in Fig. 3. It may be seen from the figure that maximum shear stress is larger than the stress causing liquefaction between 0 to 1 m and 7 to 10 m only for the severest case where peak rock acceleration is 0.2 g. In all the cases the pond does not liquefy. Flyash ponds are distributed all over the country. Not all the ponds are located in seismically active regions. The ponds at Panipat and Badarpur are in an earthquake prone region and hence liquefaction under very severe shaking is a possibility. However, considering the fact that the reported 50 year return period peak ground acceleration (Khattri, 1992) for regions near Delhi is 0.2 g, the risk due to liquefaction is considered manageable.

Conclusion

The above results are valid for unstabilised ponds. Any stabilization will improve the shear strength and hence liquefaction may not occur always. Further experimental and analytical studies on different flyash ponds under varying earthquake excitations are necessary to draw firm conclusions.

Acknowledgement

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