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STABILITY OF EARTH DAMS FOR EARTHQUAKE CONDITION

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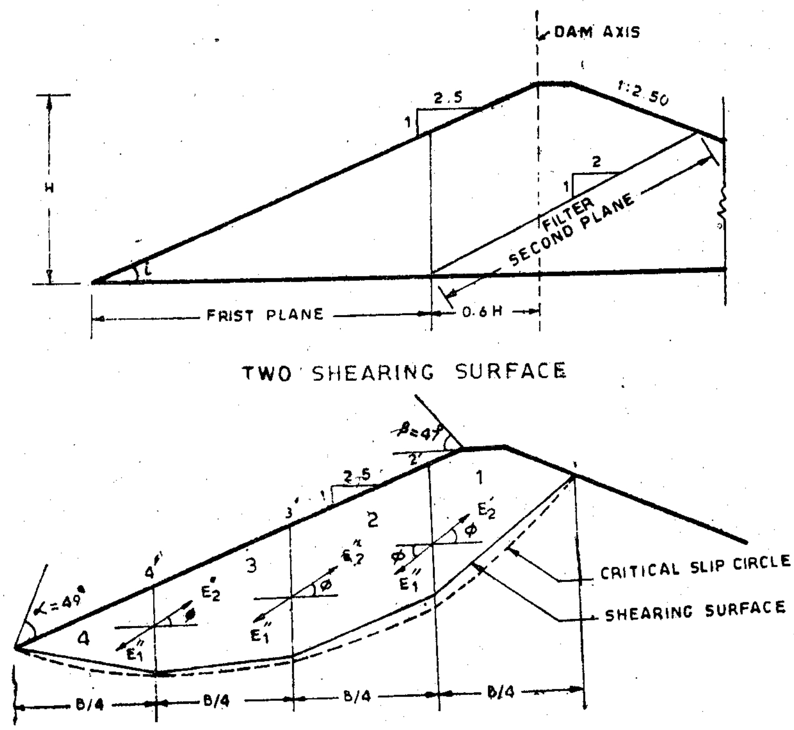
INTRODUCTION

The high earth dams are, in general, designed having thin clay core with filter at the down stream of this thin clay core. When this central Zone of Clay material is very thin, the usual method of stability analysis using cylindrical failure surface may not be the guiding criteria for the design of such failure surface has to pass through materials of diffe- rent physical properties. The failure surface in such a case could be surfaces bounded by one or more plane surfaces. The method having one plane surface given by Culmann (1866) will have the some drawback

The present work deals with failure surface bounded by (1) two plane surfaces and (2) four plane surfaces. In both cases the assumed surface is in the region suggested by Patel, Krishnayya and Arora (1964) (Fig. 1) which was suggested for stability analysis during rapid drawn-down conditions using cylindrical failure surface This enables the comparision of results for the same critical region.



FOUR SHEARING SURFACE

Fig. 1

EFFECT OF EARTHQUAKE ON THE NEUTRAL PRESSURE IN EARTH DAMS

This can be calculated using the method of superposition suggested by Patel and Bokil, (1962, 1963) and Patel and Arora (1965) using the following assumptions :

1. Instantaneous loading of saturated cohesive soil does not bring about sudden consolidation.

2. Increased water thrust due to earth is transmitted instantaneously throughout the body of the dam as the pressure is transmitted at the velocity of sound in water which becomes infinite. When water is considered incompressible the transmission of excess pressure would take place instantaneously, since the force transmitting it is the mass of earth

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acted upon by the earthquake acceleration. This is quite larger and the velocity of wave is much more than the velocity of seeping water.

3. Soil being highly impervious does not permit the flow lines to change their paths instantaneously. The potentials, however, are changed due to sudden (transmission of increased pressure. Thus, at any point on a flow line, the new potentials for full reservoir condition plus the increase in the pressnre at the starting point of this flow line on the upstream face.

The method of superposition can be valid only if it is verified that during earthquake, the velocity of wave travels without drainage of water indicating that there is no motion of the water relative to the solid skeleton, which amounts to saying that there is little loss of wave energy resulting from the friction between the solid and the water.

The analytical proof of the said hypothesis is given by Ishihara (1967). He took an elastic skeleton uniformly distributed in space and compressive fluid filling the pores of the skeleton. The equation derived by him for the phase velocity of propagation for the compressional wave produced similar to earthquake was given by

$$\frac{2}{V} = \frac{1+(w/\omega_c^2)}{\frac{1}{\overline{V}^2_l} + \frac{1}{\overline{V}_n^2}\;(\omega/\omega_c)} \qquad (1)$$

Where

V — phase velocity of propagation in saturated soil skeleton.

$\overline{V}_l$, $\overline{V}_n$ — are two other kind of phase velocities.

ω — frequency of vibration.

ω_c — "characteristic frequency" of soil skeleton.

From equation (1), it is clear that the phase velocity of wave becomes equal to $\overline{V}^l$ at the limit $\omega/\omega_c \rightarrow 0$. He has further proved that $\overline{V}_l$ equals the modulus of the soil that it realised in the rapid process or in the drained condition divided by the gross unit weight of the material From these conditions, it can be concluded that the velocity of the compressional wave at small value of ω/ω_c is identified to be the one that travels without drainage of the water and that this velocitv can be computed by using the modulus of the soil obtained in the undrained test, and the gross unit weight of the soil. Wave being able to travel without drainage of the water indicates that there is no motion of the water relative to the solid skeleton. This amounts to saying that, there is a little loss of wave energy resulting from the friction between the solid and water.

In case of earthquake the value ω/ω_c will be extremely small. In the motion of an earthquake, for example, the major peak of the recorded motion seems to appear at the most 10 times in one second. Whereas the characteristic frequency of dense sand of normally consolidated clay is 5630ft/sec which shows that the value of ω/ω_c is extremely smıll and hence $\overline{V} = \overline{V}_l$. The values of $\overline{V}$ for typical dense sand and normally consolidated clay given by Ishihara (1967)comes out to

$\overline{V}$ = 1800 m/sec for sand having $k = 10^{-2}$ cm/sec.

$\overline{V}$ = 1520 m/sec for clay with $k = 1 \times 10^{-6}$ cm/sec.

Thus we take an earth dam of 100 m height with upstream slope of say 1 vertical to 3 horizontal, the time required for the wave to reach the horizontal distance of 300 m. will be less than 1/5th of a second and hence it can be said that the wave travels instantaneously and during that time as already stated no drainage takes place, i. e. the flow pattern remains the same with the result that the excess pressure due to earthquake is transmitted instantaneously along a given flow line.

Thus increase in pressure along the upstream slope for different earthquake intensities (α) of 0.05, 0.1,, and 0.15 was calculated using the method suggested by Zanger (1962) and new neutral pressure at different points along flowline of full reservoir condition was calculated using the method of superposition (Fig. 3).

STABILITY ANALYSIS

The present work deals with the stability analysis of an earth dam by assuming plane shearing surface and interaction force at the vertical boundaries to make an angle equal to the angle of internal friction (ϕ) with the horizontal (Fig. 2). Chugaev (1964) presented this method for the stability of earth dams for steady condition. His work is taken as the basis and is extended here to be utilised for earthquake condition.

The following forces will act in the state of limit equilibrium on a given slice (Fig. 2).

1. The weight W of the slice.
2. The cohesive force C acting along the base of the slice.
3. The vertical component of water pressure (P_v).
4. The horizontal component of water pressure (P_h).
5. Horizontal thrust αW because of the acceleration due to earthquake (α).
6. Total vertical seepage force.
9. The resultant W_N of the normal pressure acting on the base of a sliee.
10. Friction along the base of the slice W_T where

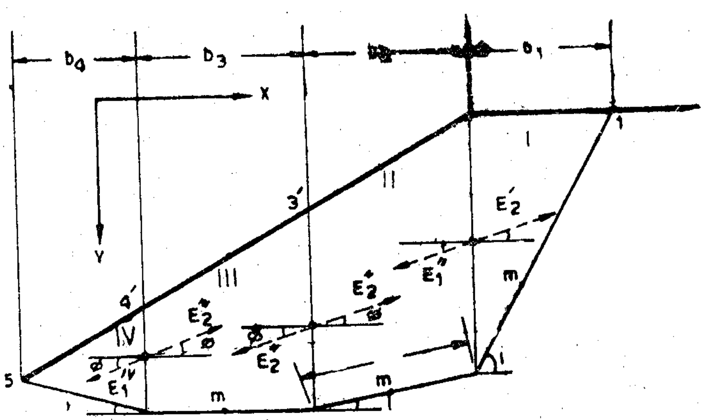


Fig 2 (a)

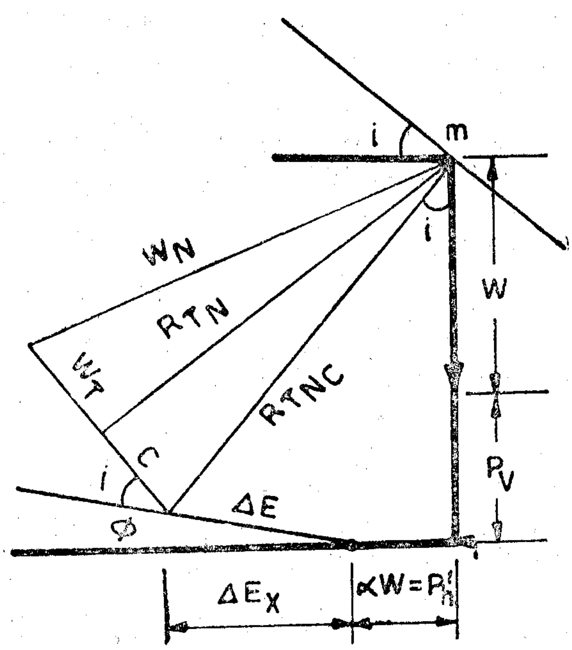
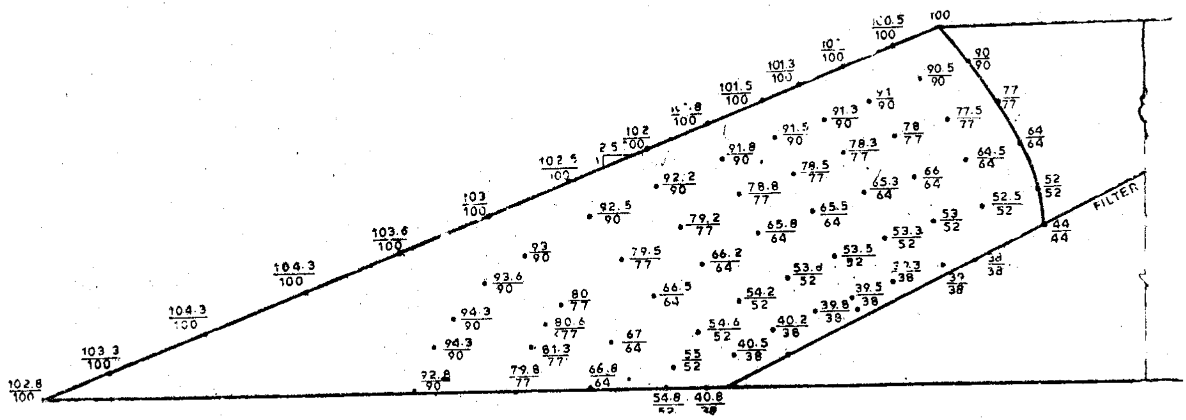


Fig 2 (b)



Potentials for U. S. Slopes of 1 : 2.5 $=\alpha$ 0.15

Note : At every point upper figure is for Earthquake and lower figure is for Full Reservoir.

Fig. 3

$$R_{TN} = \sqrt{W_N^{\,2} + W_T^{\,2}}$$

Similarly

$$R_{TNC} = \sqrt{W_N^{\,2} + (W_T + C)^2}$$

In the case of slopes similar to one shown in Fig. 2, the ratio of h/b is not usually large. It is then possible not to impose any conditions about the height at which the line of action of the forces E is located (Chugaev, 1967). Similarly we can argue the same for P_v and P_h. Thus for the sake of higher determinancy we shall arbitrarily assume, that the intersection of the axis of action of the forces W and W_N for each slice lies on the shearing surface.

Let us consider homogenous isotropic earth dam as shown in the figure 2 (a) having number of slices 22, 33, 44... The force polygon using notations given in App. 1 for any slice is shown in Fig. 2 (b).

From this it can be shown that

$$W_N = (W + P_v) \cos i + (\alpha W - P_h) \sin i + \frac{|\Delta E_x|}{\cos \phi} \sin (i \mp \phi) \tag{2}$$

Where $\quad |\Delta E| = |E_2| - |E_1| = [\Delta E_x / \cos \phi]$ (3)

on the other hand

$$|\Delta E| = R_{TN} \sin (i \mp \phi) - C \cos i = W_N \sin \alpha \, [1 \mp \text{ctg}\, i \,\text{tg}\, \phi] - C \cos i \tag{4}$$

Substituting W_N from equation 2 in equation 4, we get

$$|\Delta E_x| = \pm [(W + P_v) + (\alpha W - P_h) \,\text{tg}\, \alpha] \frac{\sin (i \mp \phi)}{\cos (i \mp 2\phi)} \cos\phi - C \frac{\cos^2 \phi}{\cos (i \mp 2\phi)} \tag{5}$$

in case of mass failure $\Sigma (\Delta E_x) = 0$ (6)

Putting $\Sigma\,(\triangle E_x)=0$ in equation 5 and solving for C, we get

$$C = \frac{\sum\left[\pm\frac{\sin(i\mp\phi)\cos\phi}{\cos(i\mp 2\phi)}\left\{(W+P_v)+(\alpha W-P_h)\,tg\,\alpha\right\}\right]}{\sum\left[\frac{\cos^2\phi}{\cos(i\mp 2\phi)}\,L\right]} \tag{7}$$

Where $C = cL$

Equation 7, which is for dry slope, while incorporating seepage forces, it can be written as :

$$C = \frac{F_n\,\gamma_w + \sum\left[\pm\frac{\sin(i\mp\phi)}{\cos(i\mp 2\phi)}\left\{(W-F_v\,\gamma_w+P_v)+(\alpha W-P_h)\,tg\,\alpha\right\}\right]}{\sum\left[\frac{L}{\cos(i\mp 2\phi)}\right]}\;\frac{1}{\cos\phi} \tag{8}$$

To make the L. H. S. non-dismensional in form of the stability number, dividing both sides by $H_{cr}\,\gamma_{sat}$, we get

$$\frac{C}{H_{cr}\,\gamma_{sat}} = \frac{\sum\left[\pm\frac{\sin(i\mp\phi)}{\cos(i\mp 2\phi)}\left\{\left(b_1-a_2\frac{\gamma_w}{\gamma_{sat}}+d_1\frac{\gamma_w}{\gamma_{sat}}\right)+\left(\alpha\,b_4-d_2\frac{\gamma_w}{\gamma_{sat}}\right)tg\,\alpha\right\}\right]}{\sum\left[\frac{e_1}{\cos(i\mp 2\phi)}\right]}\;\frac{1}{\cos\phi}$$

$$+\frac{1}{\cos\phi}\;\frac{a_1\,\gamma_w}{\gamma_{sat}}\Big/\sum\left[\frac{e_1}{\cos(i\mp 2\phi)}\right] \tag{9}$$

Where

$W = b_1\,H_{cr}^2\,\alpha_{sat}$

$F_h = a_1\,H^2_{cr}$

$F_v = a_2\,H^2_{cr}$

$P_v = d_1\,H^2_{cr}\,\gamma_w$

$P_h = d_2\,H_{cr}^2\,\gamma_w$

$L = e_1\,H_{cr}$

H_{cr} = critical height i.e. for factor of safety one

RESULTS AND DISCUSSION

The equation 9 is used to calculate the stability number for upstream slope of vertical to 2.5 horizontal for earthquake condition for different earthquake intensities (α) taking two shearing planes and four shearing planes. The results of the same are given in Table 1. For comparision of the stability number results of : (1) Ref. 6 draw down condition taking slip circle shown in Fig (2), and (2) Ref. 9 dealing with the stability number for earthquake condition taking the same slip circle of Fig. (2) are used.

From the study of the Table 1, it is clear that the slip circle method gives the highest stability number or the least factor of safety the validity of which is questionable in case of cores, whereas plane shearing surfaces give very low stability number or high factor of safety. Time will show which method is valid for earth dams with thin clay core.

Table 1

Critical Ttability Number $\left(\frac{C}{\gamma_{sat}\ H_{cr}}\right)$ for different angle of internal friction (ϕ)

Condition	$\phi\rightarrow$	0°	5°	10°	15°	20°	25°	30°
Rapid draw down slip circle method		0.128	0.096	0 068	0.038	0.01	—	—
Earthquake slip circle	0.05	0.091	0 016	0 062	0.048	0.033	0.02	0.006
	0.10	0.083	0.098	0.032	0.067	0 052	0 036	0.022
	0 15	0.14	0.123	0.107	0 091	0.074	0.059	0.03
	Condition, Plane shearing surfaces							
Two Planes	0 05	0.0349	0 010	-0.034				
	0.10	0.0360	0.0071	-0.0270				
	0.15	0.041	0.013	-0.016				
Four Planes	0.05	0.057	0.015	-0.022				
	0.10	0.061	0.022	-0.017				
	0.15	0.066	0.029	-0.008				

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