GEOTECHNICAL EARTHQUAKE ENGINEERING

SEISMIC SLOPE SAFETY

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SEISMIC SLOPE SAFETY

In order to analyse the safety of an earth dam (or a slope) during an earthquake, the following information is needed.

a) The **inertia forces** that will be generated in the dam (or slope) by the earthquake;

b) The **resistance** of the dam (or slope) against these forces along with the pre-existing static forces;

c) The possible **consequences of failure**, when the resistance of the structure is not sufficient to withstand these forces temporarily, allowing the development of deformation.

Soils, being a non-linear inelastic material, the three stages above are interconnected. We need techniques, like Finite Element Analysis, to analyse such a problem rigorously. However, in a simplified analysis, the three stages are dealt with separately and gives reasonable answers.

In the simplified analysis, the inertia forces in the first stage are determined by assuming that soil is a visco-elastic material. The resistance in the second stage is determined by assuming that soil is a rigidly plastic material. The third stage is determined by using the results of the first two.

A) INERTIA FORCES

The determination of the response of a slope to an earthquake is quite complicated. There is no analytical solution to this problem. We have analytical solution for a dam with symmetrical slope and also we have solution for a horizontal layer with no slope. It is generally assumed that the accelerations are same everywhere. However, if more detailed information is required, then FE analysis with proper boundary condition is needed. Figure 1 shows how a vertically propagating SV wave will produce both horizontal and vertical motions in the slope.

Seismic Slope Safety

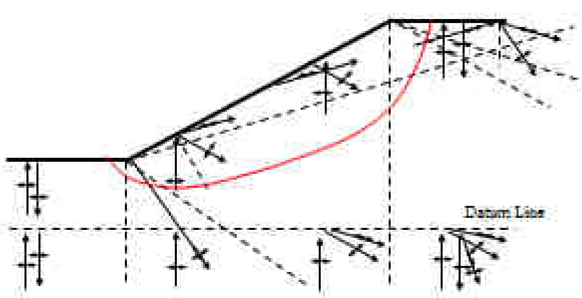


Figure 1: Wave characteristics in slopes

SV wave traveling from below. The long arrows show the wave direction, the short arrows show the particle motion. The particle motion at any point will be the combined effect of all waves. (Sarma&Irakleidis,2008)

Figure 1

However, in case of an earth dam, simplifying assumptions can be made and reasonable results can be obtained.

The inertia forces generated during an earthquake in a dam or a layer or a slope will depend on

i) The geometry of the dam
ii) The material properties
iii) The earthquake time history.

In order to determine these forces, we formulate a mathematical model with assumptions.

Assumptions for a dam: See figure 2

1) The length of the dam is great compared to the height. In this case, the presence of the abutments will not be felt except near the ends. (L>4H).

2) Slopes of the dam are fairly flat and the section is symmetrical about the y-axis. Amount of oscillations due to bending is small. Therefore, subjected to horizontal loading in shear, the response is assumed to be in shear only. (Slope <1:1.5) .

   With the above assumptions, only the y dimension and the shear stress is pertinent. Therefore, it is called **one-dimensional shear-beam** analysis.

3) The wedge is rigidly connected to the base. The rigidity of the foundation material is much greater than the dam. (The solution to the dam on a layer is available as well.)

4) The base is acted upon by an arbitrary disturbance in the horizontal direction only.

5) The material in the wedge is homogeneous and elastic.

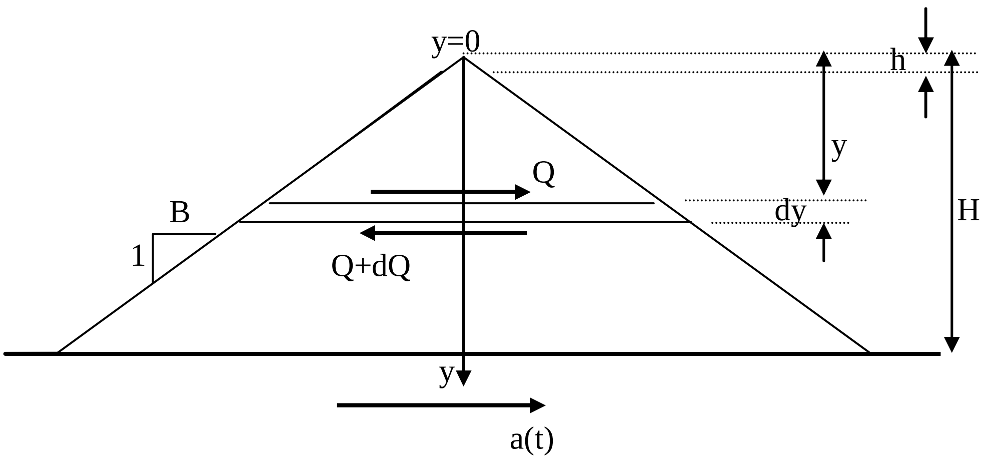


Figure 2

With the above assumptions, the equation of motion can be written and solution obtained. The solution is of the form of sum of response of many modes.

$$u(y,t) = \sum \Phi_n(y) I_n(t)$$

Φ_n gives the nth mode shape and I_n is the Du Hamel's Integral for the response of a single degree of freedom structure of nth mode frequency subjected to the ground acceleration a(t)

.
From the solution (not detailed here), we see that the pertinent parameters of the dam are the following:

a) Height of the dam, H
b) Average shear wave velocity, S
c) Energy loss capacity, λ

The first two parameters combine together to produce single parameter, which is the fundamental period of the dam,

$$T_1 = 2.61\ H/S \qquad (1)$$

All other mode periods are functions of the fundamental mode. The slope of the dam does not come into the picture. Therefore, the two real parameters are T_1 and λ.

[**Compare** the period of the dam with that of a soil layer which is $T_1 = 4\ H/S$]

Damping:

In the Voigt type material, the damping factor as a fraction of the critical becomes directly proportional to the mode frequency, producing higher damping in higher modes. Field tests do not support this finding. It is difficult to see any consistent trend in the variation of damping with modes. It is therefore generally accepted to have a constant value of λ in all modes. λ is found from cyclic tests on laboratory samples as an equivalent viscous damping factor which is given as:

$$\lambda = 1/(4\pi)\ \Delta W/W \qquad (2)$$

where ΔW and W are explained in figure 3.

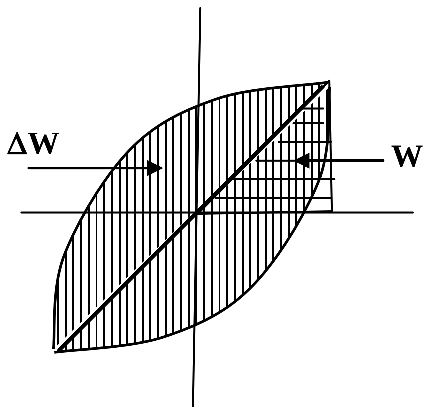


Figure 3

It is clear from lab tests that both the equivalent shear modulus and the damping are functions of the strain imposed. We therefore, choose these values from an average strain expected during an earthquake.

Response

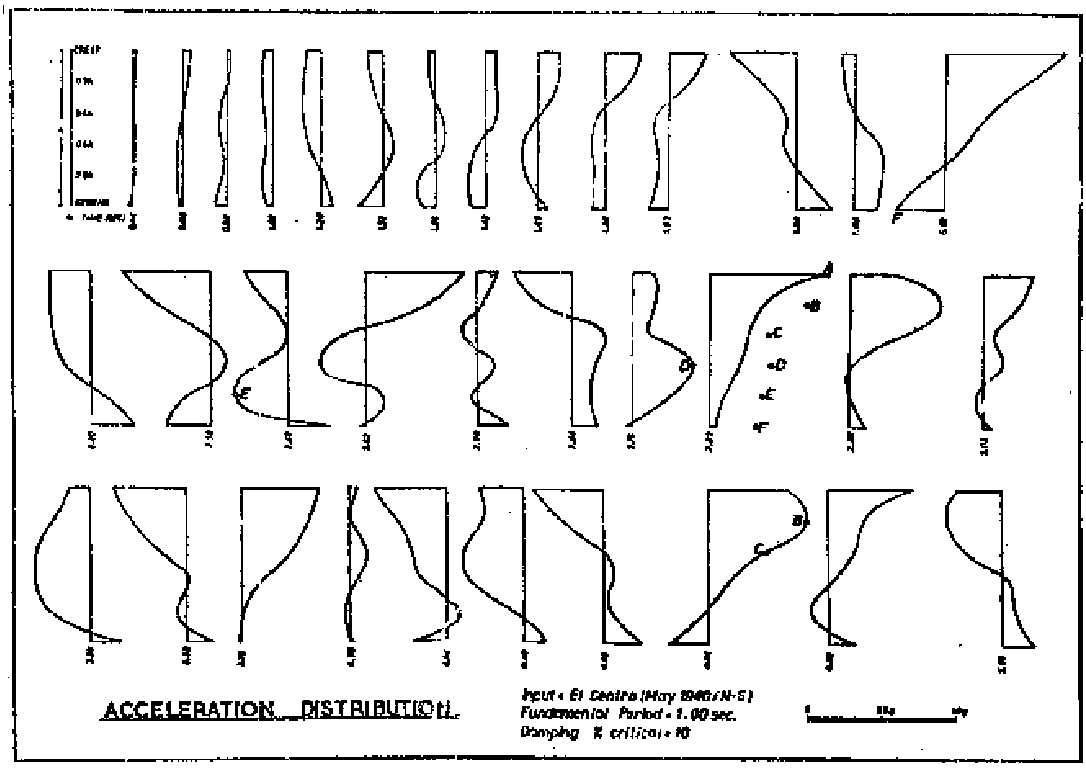
The response of the dam depends upon

a) T_1 b) λ c) a(t) = type of earthquake record

An earthquake record contains many frequencies. The dam also has many natural frequencies. It will therefore select those frequencies with which it can resonate and respond vigorously in those frequencies. In a record, which represent the response of the structure, say, a record obtained at the crest of the dam, the predominant frequencies will be those of the structure. However, the predominant frequency of the earthquake record will also be maintained at the same time.

The response shows that the acceleration changes with height and in time. The peak accelerations at different heights occurs at different times and in different directions. The accelerations vary rapidly with time, the peak values lasting for only a fraction of a second. These values may occur only once or twice during the whole earthquake. See Ambraseys and Sarma (1967).

FIG. 1



AVERAGE SEISMIC COEFFICIENT $\overline{k}_\alpha$

Because the peak accelerations in the dam may occur at different times and may be in different directions, the use of the peak accelerations in design would produce conservative result. We have therefore developed the concept of the average acceleration. The averaging process requires a pre-defined possible slip surface.

Average seismic acceleration as a function of time $A_\alpha(t)$ is defined as the total inertia force on the mass contained within the slip surface and the free surface divided by the total mass.

$$A_\alpha(t) = \text{Total Inertia force}(t)/\text{Total mass}$$

The average seismic coefficient $\overline{k}_\alpha$ is defined as

$$\bar{k}_{\alpha} = \frac{|A_{\alpha}|_{\max}}{|a|_{\max}} = \text{maximum average acceleration/ maximum ground acceleration} \qquad (3)$$

The average seismic coefficient may be expressed as a fraction of the maximum ground acceleration as shown here or as a fraction of gravity.

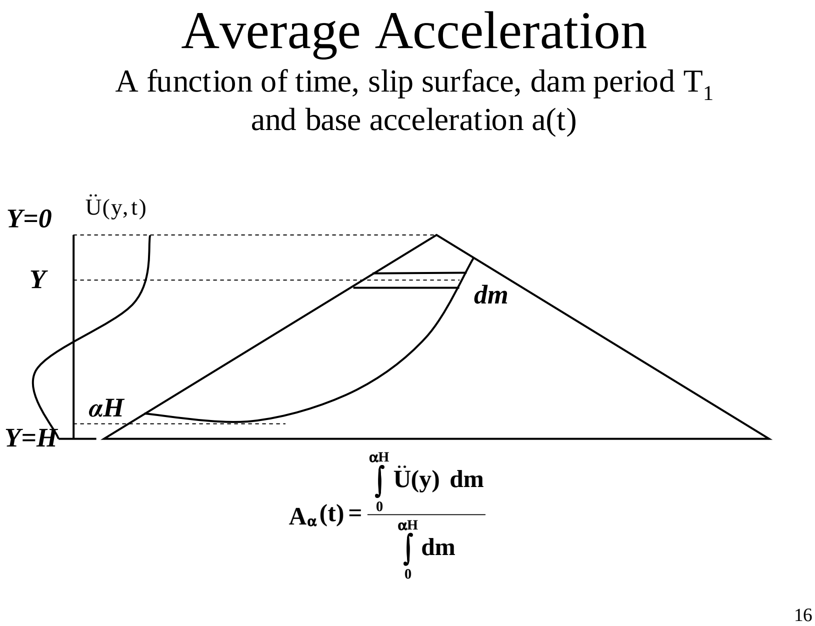


Figure 4

It is therefore necessary to pre-define a possible slip surface. This does not mean that the actual sliding surface will be the one defined here. This surface will be representative of many similar ones.

a) One parameter sliding wedge (see figure 5)Ambraseys and Sarma (1967)

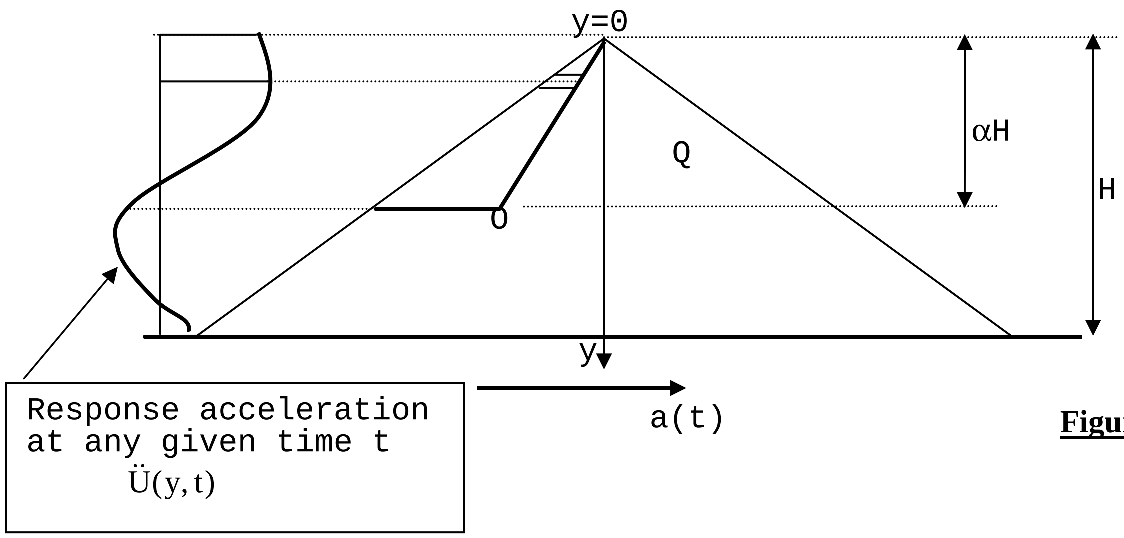


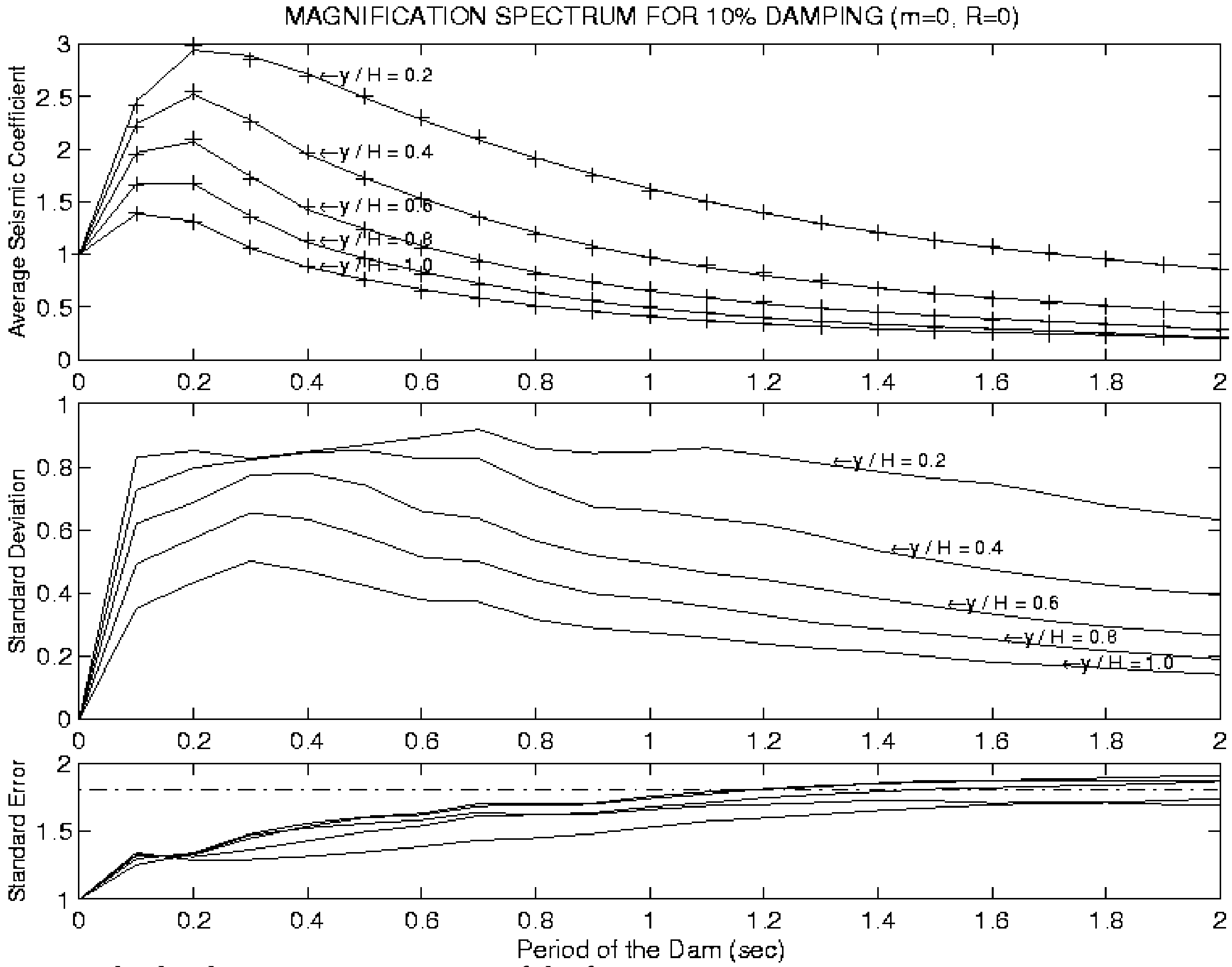
Figure 5

In this kind of wedge, α becomes the parameter defining the slip surface. The position of the point O becomes immaterial.

$$A_\alpha(t) = \frac{\int_0^{\alpha H} \ddot{U}(y)\, dm}{\int_0^{\alpha H} dm} \qquad (4)$$

(Note that the one parameter wedge may represent many different slip surfaces approximately)

<u>AVERAGE SEISMIC COEFFICIENT SPECTRA</u>



Dichotomous third order regression equation of the form

$$\mathbf{k=1 + a_1T + a_2T^2 + a_3T^3} \text{ for } \mathbf{T < T_C} \qquad \mathbf{(5)}$$

$$\mathbf{k=1 + a_1T_C + a_2T_C^2 + a_3T_C^3 + a_4(T-T_C) + a_5(T-T_C)^2 + a_6(T-T_C)^3} \text{ for } \mathbf{T \geq T_C} \qquad \mathbf{(6)}$$

has been fitted to the seismic coefficient spectra, where :

•**T** is the fundamental period of the dam

•$\mathbf{T_C}$ is a 'critical period' chosen so as to minimize the residual of the regression. For almost all cases $\mathbf{T_C}$**= 0.4** sec gives the optimum solution and this value is therefore adopted.

Table 1

y/h	a1	a2	a3	a4	a5	a6	St. Dev. (x10e-4)
0.2	20.786	-69.728	71.234	-2.402	1.09	-0.196	0.642
0.4	18.649	-69.875	72.992	-2.394	1.48	-0.359	0.404
0.6	15.465	-65.174	72.931	-1.967	1.341	-0.35	0.377
0.8	11.228	-51.408	60.093	-1.591	1.108	-0.291	0.159
1	6.979	-35.71	43.768	-1.213	0.849	-0.223	0.092

Figure (6) shows the average seismic coefficients to be used in design from one parameter sliding wedge. These are obtained as the mean of several strong motion records.

The figure shows that seismic coefficients for slip surfaces towards the top of the dam are generally higher. Therefore, if a cross section is designed for a factor of safety greater than one for a toe slip, the section may not have sufficient factor of safety towards the top of the dam because of the increased acceleration.

B) RESISTANCE

The resistance of the slope against the inertia forces along with the static forces can be defined by the critical acceleration or the factor of safety of the slope.

CRITICAL ACCELERATION $k_c g$:

Critical acceleration is defined as that acceleration which when applied on the sliding mass produces a condition of limiting equilibrium. The acceleration in this case is horizontal. In this context, the factor of safety is defined as that factor by which the soil strength parameters are to be reduced to produce a condition of limiting equilibrium. Limiting equilibrium implies a factor of safety of one.

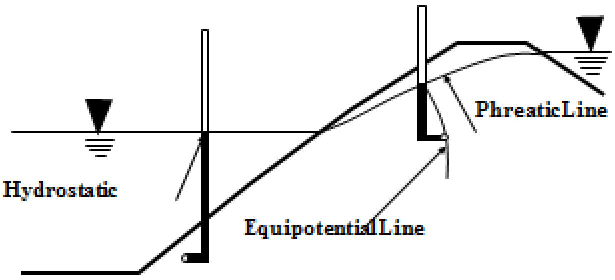
We may adopt any stability analysis methods for this purpose. However, the aim is to find the critical acceleration and not the factor of safety. In this context, Sarma(1973) method is most appropriate since it determines the critical accelerations directly. A better method is Sarma(1979) method with inclined slices.Sarma (1999) gives a set of relationships for the critical acceleration factor k_c for simple homogeneous slopes of different inclinations and different strengths. An enhanced limit equilibrium technique is recently developed which uses the acceptability criterion as a starting point and determines both the critical surface and the critical acceleration at the same time. See Sarma& Tan (2006) and Tan &Sarma (2008).

For seismic stability analysis, the strength parameters to be used are those that refer to the dynamic (cyclic) undrained condition. We may use total strength parameters (strength determined from appropriate laboratory tests) or effective stress parameters includingpre-seismic pore pressures and dynamic pore water pressure parameters (again determined from appropriate laboratory tests).

Seismic Slope Safety Assessment

Pore water Pressure

Hydrostatic and Pressure due to flow



Pore water pressures under static (non-seismic) conditions.

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The excess pore pressures due to seismic loading will depend on many factors. It depends on the level of loading and the number of cycles, dissipates slowly after the earthquake and the dissipation depends on the permeability of the soil. In this case, slopes may fail some time after the earthquake.

The critical acceleration for an **infinite slope** can be determined easily as shown in equation 16 later.

Results for critical acceleration $k_c g$,Sarma(1999) (Determined from log-spiral slip surfaces)

$k_c = k_{co} + [c'/(\gamma H)] .f_c$ (7)

$k_{co} = (1-r_u) \tan(\phi'-\beta) - r_u \tan \beta$ (8)

$f_c = a. \tan \phi' + b$ 10)

$a = p_a \tan^2\beta + q_a \tan \beta + r_a$ (11)

$b = q_b \tan \beta + r_b$ (12)

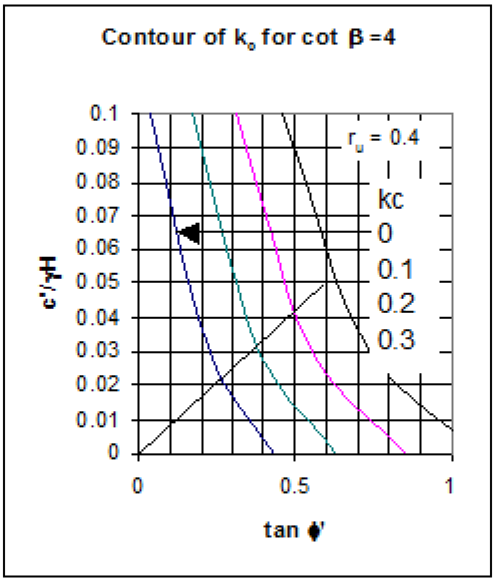
$r_u = u/\gamma h$ (13)

u= pore pressure and

γh= over burden pressureat a point

Table 2

r_u	$c'/\gamma H$	p_a	q_a	r_a	q_b	r_b
0						
	0.025	-7.2719	3.4739	2.2959	9.7347	0.8386
	0.05	-6.8671	3.1763	1.679	8.1369	0.6444
	0.1	-4.7764	1.6411	1.3844	6.6296	0.5272
0.2						
	0.025	-6.5929	3.1774	2.1229	9.3458	0.7807
	0.05	-5.8664	2.5665	1.6102	7.9287	0.5898
	0.1	-4.1681	1.3018	1.3069	6.6086	0.4742
0.4						
	0.025	-6.8318	3.4838	1.8372	8.9259	0.6832
	0.05	-4.9198	1.9128	1.5311	7.6775	0.526
	0.1	-4.0763	1.3315	1.1502	6.5747	0.4098



Homogeneous slope (1/4 Gradient)
PWP parameter $r_u = u/\gamma H$

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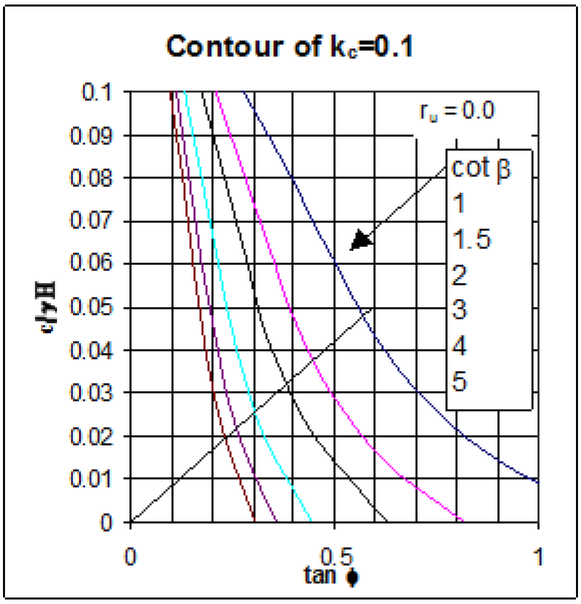

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Critical accelerations are determined for slip surfaces at different heights and the critical surfaces found (defined as the surface with minimum critical acceleration). The critical accelerations are then compared with the average seismic coefficients. If the critical accelerations are bigger than the average seismic coefficients, the slip surface has a factor of safety greater than one. Factor of safety less than one is implied when the average seismic coefficients are bigger than the critical. In this case, it is possible that the mass will slide on the slip surface.

C) CONSEQUENCES OF FAILURE

Since the instantaneous acceleration during an earthquake may be large enough to reduce the factor of safety to below one, surfaces of discontinuity (slip surface) may be produced and displacements may occur along such slip surfaces. The displacements of the sliding mass may be roughly estimated by using the sliding block model. This idea was promulgated by Newmark (1965)

SLIDING BLOCK ANALYSIS:

From the stability analysis, we can determine the normal and shear forces on the slip surface.

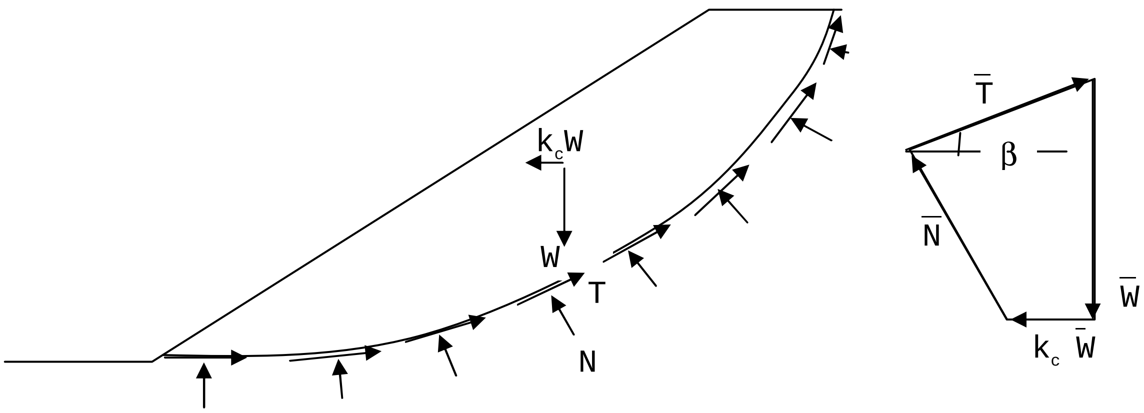

Figure 7

$\sum \overline{N}$ = Vector sum of all N forces
$\sum \overline{T}$ = Vector sum of all T forces
β= Equivalent Inclination of the Plane

When the factor of safety is less than one, we use the sliding block analysis. In this case, we assume that the mass rests on an equivalent inclined plane surface. The equivalent inclination is found from the limiting equilibrium condition, figure 7.

There are more complex sliding mechanism in the literature,e.g. Sarma (1981), Ling &Leshchinsky (1995), Ambraseys andSrbulov (1995), Stamatopoulos (1996), Sarma&Chlimintzas (2001). Comparison of displacements computed with complex sliding mechanism shows that for relatively small displacements, the single block sliding gives reasonable good approximations. For large displacements, the single block sliding gives conservative results. See Chlimintzas (2003).

SLIDING BLOCK MECHANISM [Newmark (1965) model]

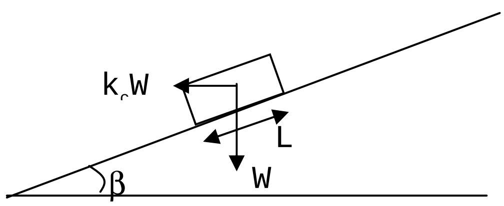

Figure 8

Figure 8 shows an equivalent sliding block. The shear strength parametrs c’ and ϕ’ are such that the critical acceleration $k_c g$ is the same for the sliding block as in the analysed slip surface as shown in figure 7.

D = Driving force down the slope
R = Resisting Shear force up the slope
U = Force due to pore water pressure on the block
L = Length of the block.

$$D = W (\sin \beta + k \cos \beta) \quad (14)$$

$$R = [W(\cos \beta - k \sin \beta) - U] \tan\phi' + c'L \quad (15)$$

At the critical stage, when factor of safety equals one, D = R, which gives $k=k_c$= critical acceleration factor.

$$k_c = \tan(\phi' - \beta) - \frac{\sin\phi'}{\cos(\phi' - \beta)}\frac{U}{W} + \frac{\cos\phi'}{\cos(\phi' - \beta)}\frac{c'L}{W} \quad (16)$$

If $k < k_c$ then the factor of safety $F_d > 1$ and

$$F_d = \frac{[\cos\beta - k \sin\beta] \tan\phi'}{\sin\beta + k \cos\beta} - \frac{U \tan\phi'}{D} + \frac{c'L}{D} \quad (17)$$

If we write $\tan i = k$, then

$$F_d = \frac{\tan\phi'}{\tan(\beta + i)} - \frac{U}{W}\frac{\tan\phi' \cos i}{\sin(\beta + i)} + \frac{c'L}{W}\frac{\cos i}{\sin(\beta + i)} \quad (18)$$

This shows that the application of a pseudo-static horizontal load (kW) is equivalent to tilting the base of the slope by an angle i where

$$i = \tan^{-1}(k) \quad (19)$$

If $k > k_c$, then the factor of safety $F_d < 1$ and there is a net driving force acting on the mass down the slope in which case, the mass must accelerate in the direction of the net force (Newton's Second Law) which gives:

$$m \ddot{x} = D - R = \frac{W \cos(\phi' - \beta)}{\cos\phi'} [k - k_c] \quad (20)$$

where x = Relative displacement of the block with respect to the base.

This formulation assumes that k_c remains unchanged during the movement. In the displacement analysis for a non-planar failure surface, use the critical acceleration as determined from the stability analysis and do not recompute from the sliding block model. It may be necessary to derive an equivalent ϕ' (weighted average value over the slip surface).

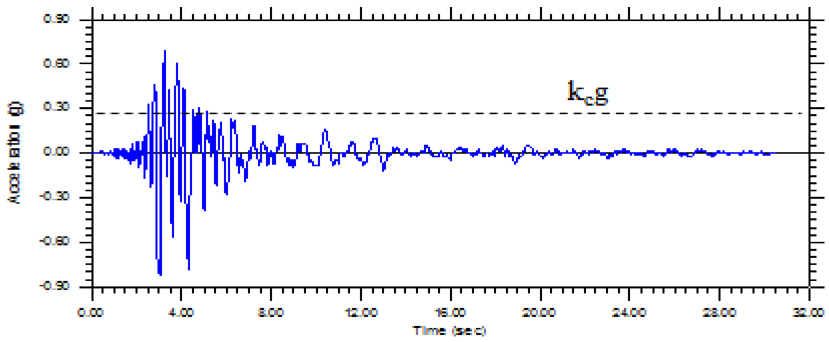

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Figure 9

Since k is a function of time [= $A_{\square}(t)$ for the average seismic acceleration], see figure 9, the equation can be integrated to obtain the maximum displacement x_m. We can safely assume that the block does not move upward.

Figure 10gives the displacement of the block as a function of k_c/k_m. The figure gives the solutions for k(t) as:

a) Rectangular pulse (See the graphical and analytical solution later in figure 11)
b) Half-sine pulse (See Sarma 1975)
c) Triangular pulse (See Sarma 1975)
d) Many earthquake records. For these records, T represents the predominant period of the record. For a record, representing the response of the dam, the predominant period is likely to be the fundamental period of the dam. Since the record also contains the information about the predominant period of the original record, it is difficult to guess which of these two will predominate. It is therefore conservative to use the longer one of the two predominant periods.

From the study of the many different records, the following equation holds good for practical purposes.

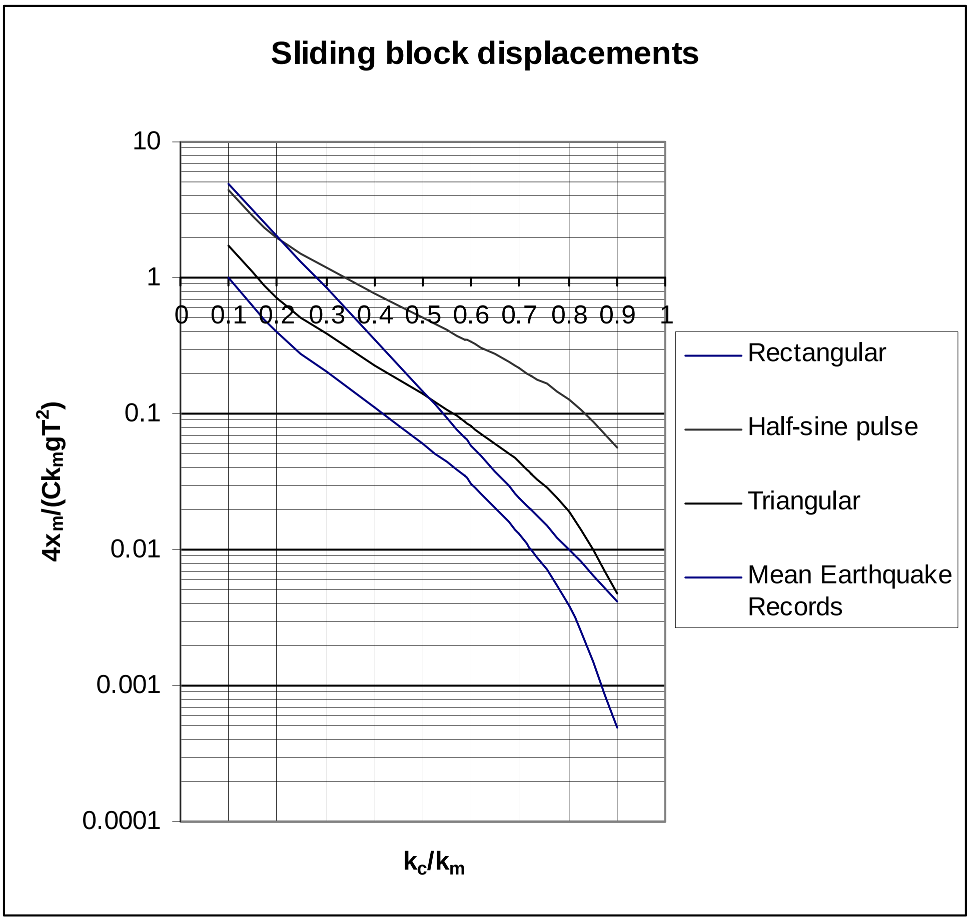
a) $\log 4x_m/(Ck_m gT^2) = 1.07 - 3.83\, kc/km$ Sarma (1988) (21)

where $C = \dfrac{\cos(\phi' - \beta)}{\cos\phi'}$

There are other formulae as well

b) $\log\{x_m(cm)\} = 2.3 - 3.3\, kc/km$ Ambraseys(1972) (22)

c) $\log\{x_m(cm)\} = 0.90 + \log\ [(1-k_c/k_m)^{2.53}\,(k_c/k_m)^{-1.09}$ Ambraseys& Menu(1988) (23)



(After Sarma 1975) **Figure 10**

Sarma&Kourkoulis (2004) studied many strong motion records to find the dominant parameters of these records which control the resulting displacements. The variability of the computed displacements at any level of the k_c/k_m ratio is high. See figure 10.

Factors affecting Displacements are:

- k_c/k_m ratio
- k_m or v_{max}
- Duration of acceleration pulses
- Number of pulses

And also,

- Change of strength parameters due to displacement
- Change of the geometry of the slide

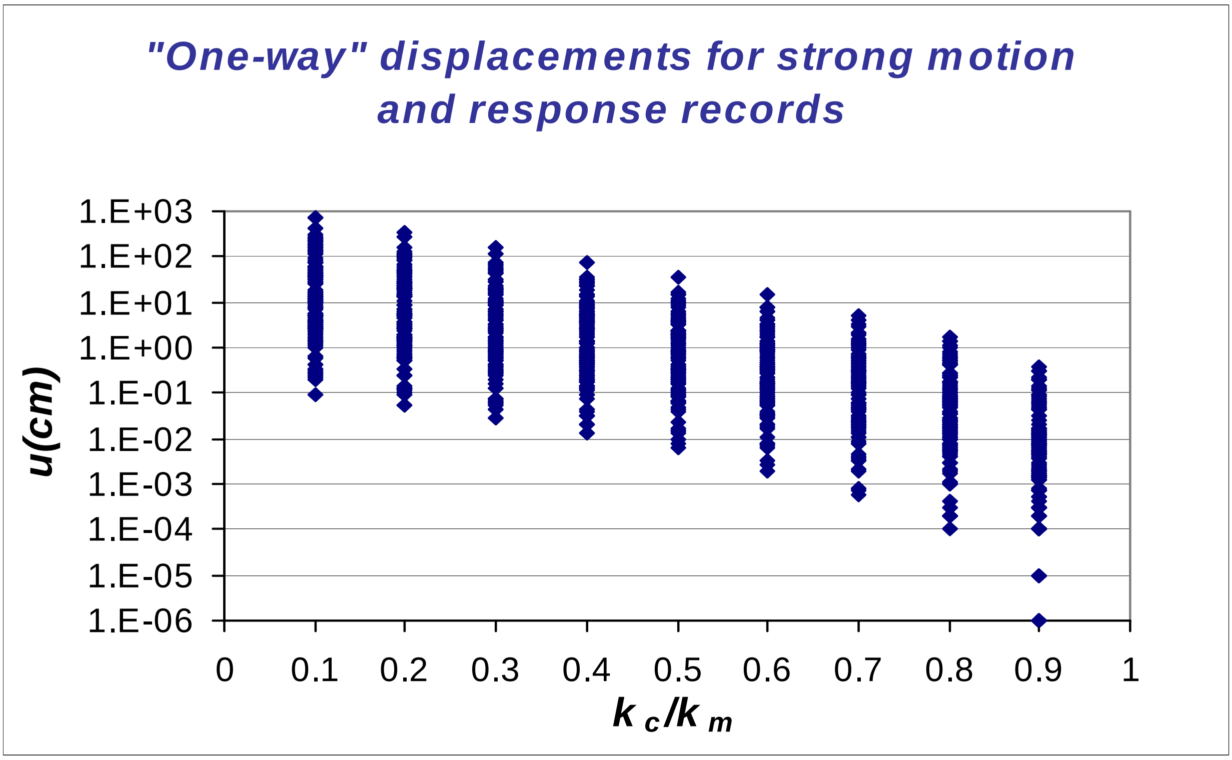


Figure 11, After Sarma&Kourkoulis (2004)

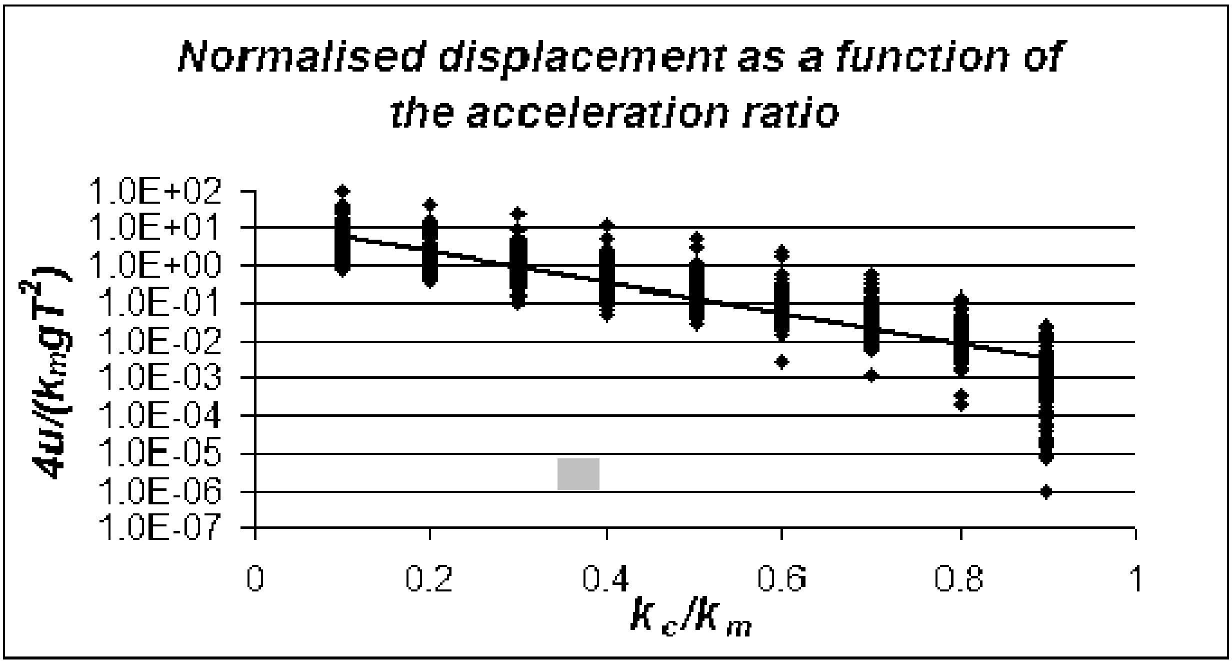


Figure 12 ,After Sarma&Kourkoulis(2004)

Sarma&Kourkoulis (2004) shows how other parameters affect the displacements.

Sarma&Chlimintzas (2001) produced a multiblock sliding model, figure 13, which takes into account the change of geometry of the sliding mass during movements and how it affects the displacements. Their results show that for small displacements, simple sliding block model is sufficient but for very large displacements, the simple model may be unconservative.

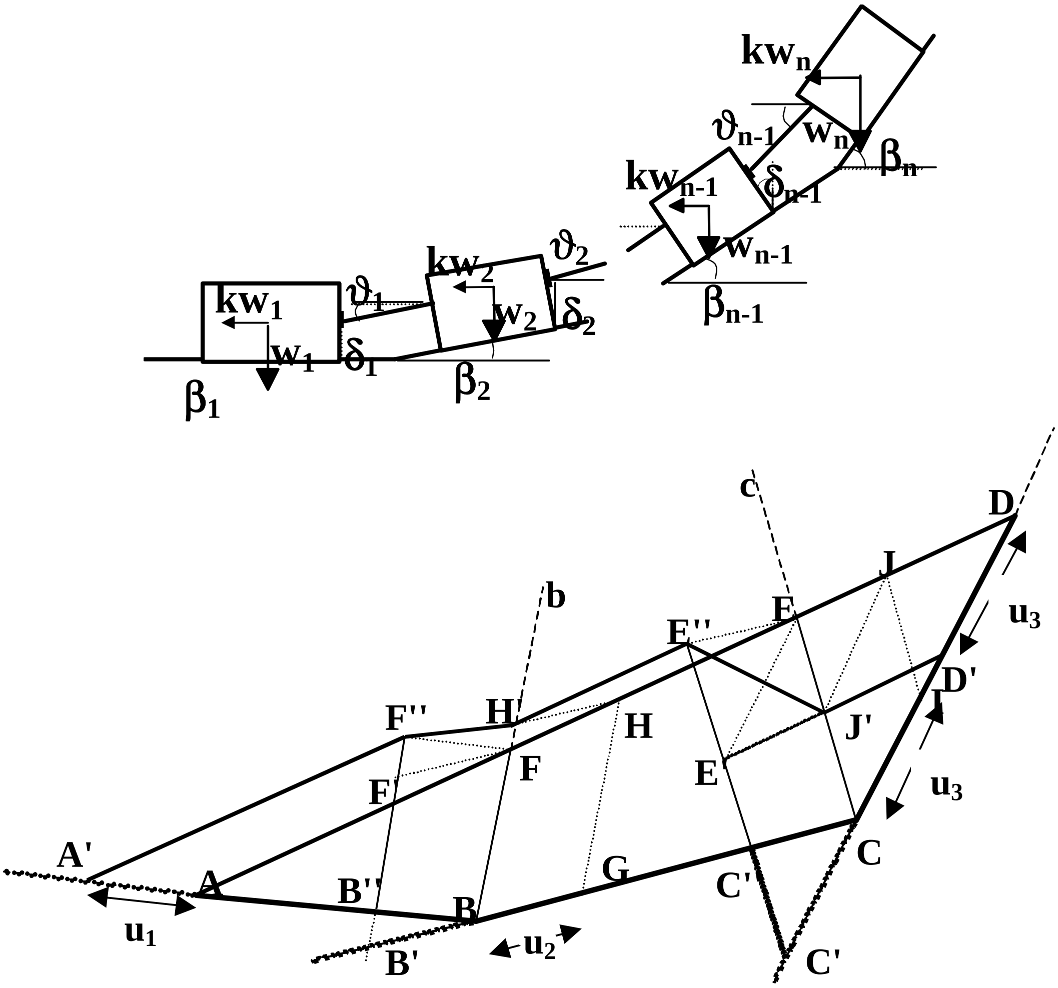


Figure 13, After Sarma&Chlimintzas (2001)

<u>Effect of vertical Acceleration:</u>

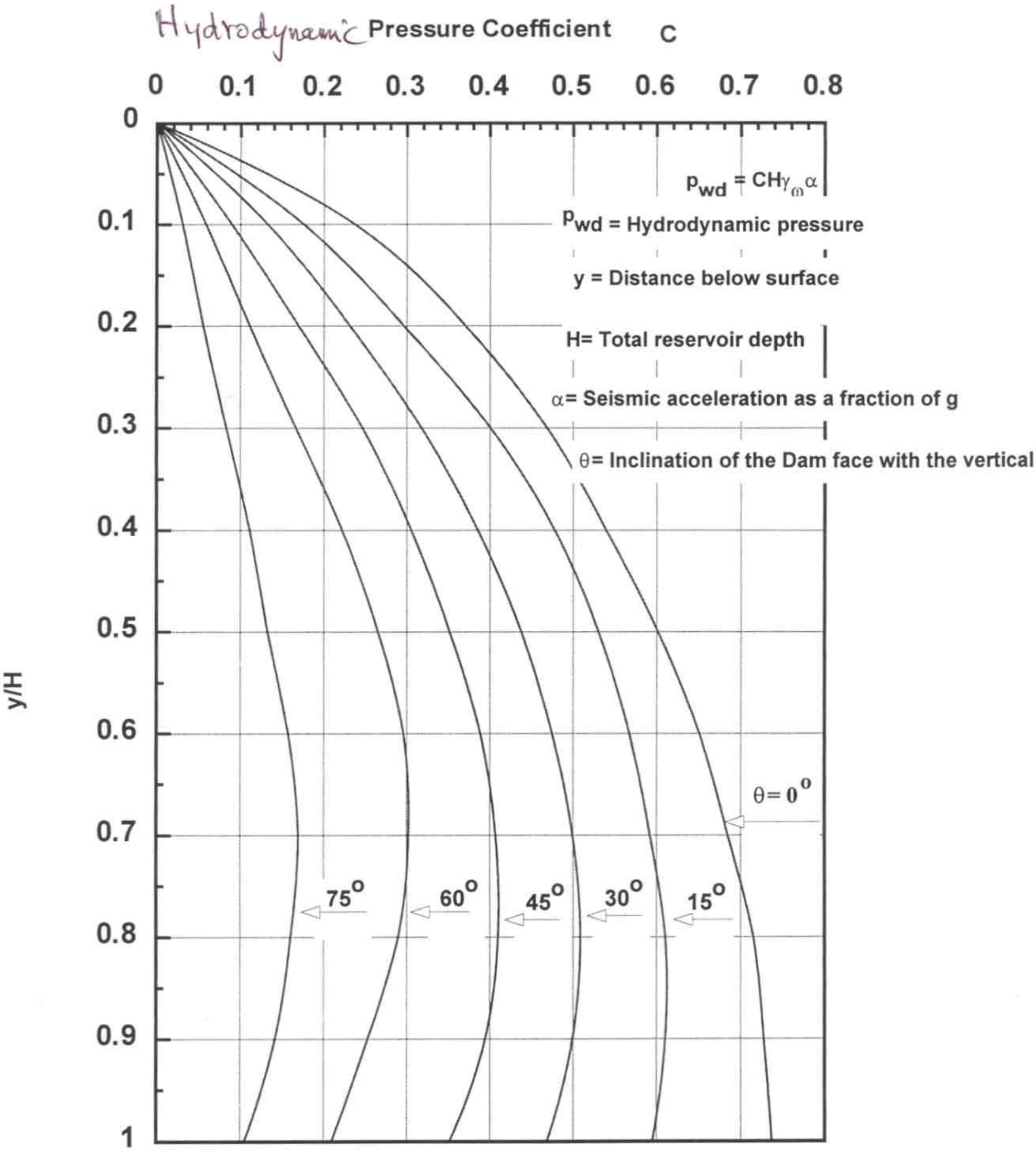
In the limit equilibrium analysis, vertical accelerationk_vg can be taken care of simply by increasing the unit weight of materials including that of water by the factor (1+k_v), where vertical acceleration is considered positive upward (inertia force is positive downward). In this case, the horizontal acceleration is reduced by the same factor to determine the factor of safety. Alternatively, the critical horizontal acceleration determined with the modified weight is increased by the same factor to determine the true horizontal critical acceleration.

For cohesive soils, the net effect is small. For cohesionless soil, negative vertical inertia load will always reduce the critical horizontal acceleration.

Even though the factor of safety is affected by the vertical acceleration slightly, the net effect on seismic displacements is very small and therefore, for practical purposes, vertical acceleration can be ignored. See Sarma& Scorer (2009).

<u>Hydrodynamic pressures</u>:

When there is external free water in a slope as in a reservoir, there will be hydrodynamic pressures. This acts in a direction normal to the slope and always towards the direction of the inertia force. This is therefore always detrimental for the slope. This pressure is highest when the slope face is vertical and reduces for flatter slopes.



Note: The pressure acts normal to the face of the dam

Figure : Values of pressure Coefficients

[After Zangar & Haefeli (1952)]

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(This is a reading list and not all papers are referred in the text.)

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