Seismic Design of Earth Retaining Structures

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Introduction

The problem of retaining soil is one the oldest in the geotechnical engineering; some of the earliest and most fundamental principles of soil mechanics were developed to allow rational design of retaining walls. Many approaches to soil retention have been developed and used successfully. In the recent years, the development of metallic, polymer, and geotextile reinforcement has also led to the development of many innovative types of mechanically stabilized earth retention system.

Retaining walls are often classified in terms of their relative mass, flexibility, and anchorage condition. The common types of the retaining wall are:

1. Gravity Retaining wall
2. Cantilever Retaining wall
3. Counter fort Retaining wall
4. Reinforced Soil Retaining wall
5. Anchored bulkhead

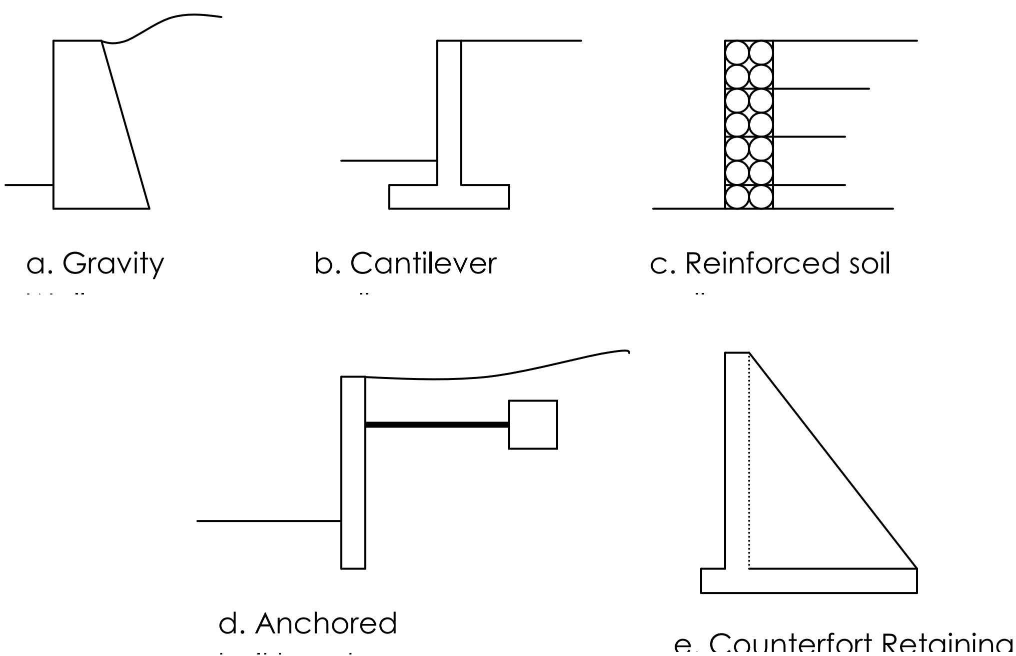


Fig. 1 Common type of Retaining Wall

Gravity Retaining walls (Fig 1 a) are the oldest and simplest type of retaining walls. The gravity wall retaining walls are thick and stiff enough that they do not bend; their movement occurs essentially by rigid body translation and or by rotation.

The cantilever retaining wall as shown in Fig.1b bends as well as translates and rotates. They rely on the flexural strength to resist lateral earth pressures. The actual distribution of lateral earth pressure on a cantilever wall is influenced by the relative stiffness and deformation both the wall and the soil.

In the present context considering the maximum applicability of free standing gravity retaining wall the presentation is focused mainly on the seismic design of gravity retaining wall. (For details of other type the book "Foundation Analysis and Design" by J.E. Bowles;McGraw-Hill International Edition, 1997 may be referred).

Type of Retaining Wall Failure

To design retaining walls, it is necessary to know how wall can fail. Under static condition the retaining walls are acted upon by the forces like;

1. body forces related to mass of the wall
2. by soil pressure
3. by external forces such as those forces transmitted by braces etc.

A properly designed retaining wall will achieve equilibrium of those forces including shear stresses that approach the shear strength of soil. During earth quake, however the inertial forces and changes in the soil strength may violate the equilibrium and cause permanent deformation of the wall. Failure whether by sliding, tilting, bending or some other mechanism, occurs when these permanent deformations becomes excessive. The types of failure of retaining wall are as shown below in Fig.2.

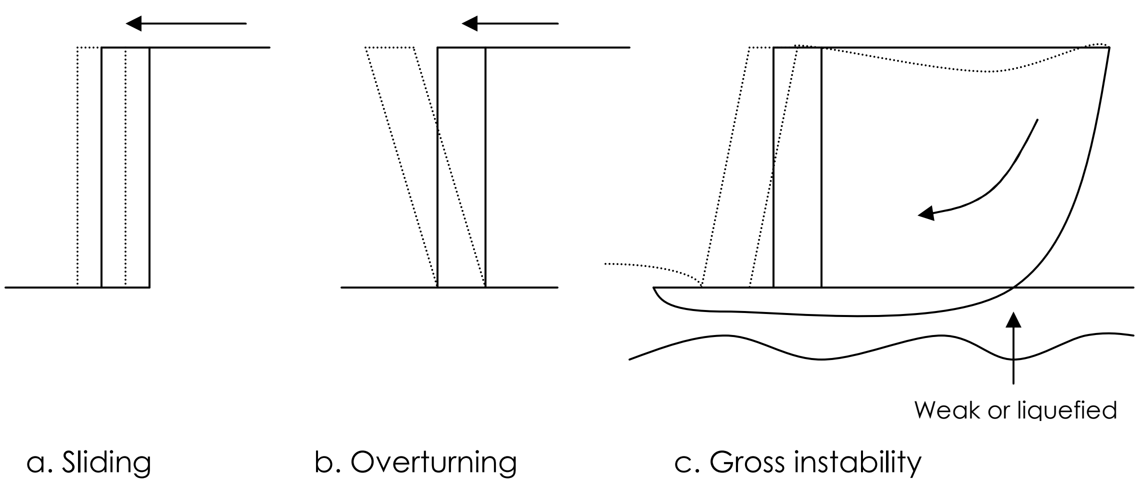


Fig.2. Typical failure mechanism of gravity wall

Gravity wall usually fail by rigid body mechanism such as sliding and/or overturning or by gross instability. Sliding occurs when horizontal force equilibrium is not maintained, that is when the lateral pressure on the back of the wall produces a thrust that exceeds the available sliding resistance of the base of wall. Overturning failure occurs, when moment equilibrium is not satisfied. In this situation bearing failure at the base are often involved.

In cantilever retaining wall also, the similar type of failure occurs as that of in the gravity wall. In addition, the flexural failure mechanism also occurs in cantilever wall.

Static Pressure on Retaining Wall

The seismic behavior of retaining wall depends on the total lateral earth pressure that develops during the earth shaking. This total pressure includes both the static gravitational pressure that exist before earthquake occurs and the transient dynamic pressure induced by the earthquake. Therefore, the static pressure on the retaining wall is of significant in the seismic design of retaining wall and hence a brief review of static earth pressure is presented.

Calculation of Static Earth Pressure: Rankine Theory

Rankine (1857) developed the simplest procedure for computing the minimum active and maximum passive earth pressure. For minimum active condition, Rankine expressed the pressure at a point on the back of a retaining wall as

$$p_a = K_a \sigma_v^{/} - 2c\sqrt{K_a} \tag{1}$$

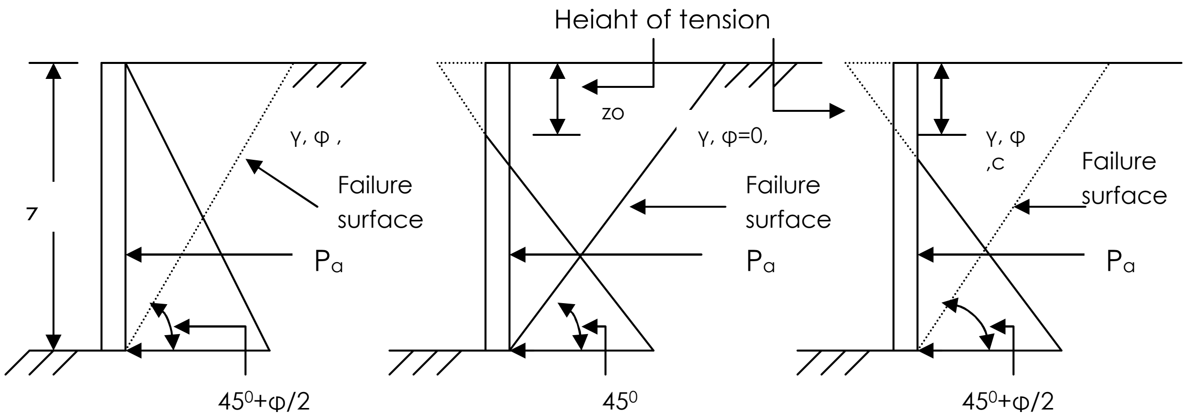
Where K_a is the coefficient of minimum active earth pressure, $\sigma_v^{/}$ is the vertical effective stress at the point of interest, and c is the cohesive strength of the soil. When the principal stress planes are vertical and horizontal (as in case of a smooth vertical wall retaining a horizontal backfill), the minimum active pressure coefficient is given by the equation:

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right) \tag{2}$$

For the case of the cohesionless backfill inclined at angle β with the horizontal infinite slope solution can be used to compute K_a as:

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \tag{3}$$

The pressure distributions on the back of the wall, as indicated by the equation (1), depend on the relative magnitude of the frictional and cohesive components of the backfill soil strength as given below in Fig 3.



$K_a = \tan^2(45-\phi/2)$ $z_o = 2c/\gamma$ $z_o = \left(\frac{2c}{\gamma}\right)\tan(45+\phi/2)$

$p_a = K_a\gamma z$ $p_p = \gamma z - 2c$ $p_a = \gamma z\tan^2(45-\phi/2) - 2c\tan(45-\phi/2)$

$P_a = K_a\gamma H^2/2$ $P_a = \gamma H^2/2 - 2cH + \frac{2c^2}{\gamma}$ $P_a = \left(\frac{\gamma H^2}{2}\right)\tan^2(45-\phi/2) - 2cH$

$\tan(45-\phi/2) + 2c^2/2$

(a). (b). (c).

Fig.4. Minimum Rankine active earth pressure distribution for back fill with various combination of friction and cohesive strength; (a) Frictional resistance, no cohesion; (b) Cohesive soil, no frictional resistance; (c) combined cohesion and friction (S.L Cramer)

Although the presence of cohesion indicates that tensile stresses will develop in between the upper portion of the wall backfill, tensile stresses do not actually develop in the field. The creep, stress relaxation and low permeability characteristics of the cohesive soil render them undesirable as backfill materials for the retaining structures. Hence their use as filling materials should be avoided.

The Rankine theory predicts triangular active pressure distribution oriented parallel to the backfill surface for homogeneous cohesionless backfill. The resultant active earth pressure P_A acts at a point located at height H/3 above the base of the wall height with the magnitude:

$$P_A = \frac{1}{2}K_a\gamma H^2 \qquad (4)$$

Under maximum passive condition, Rankines theory predicts wall pressure given by the relation

$$p_p = K_p\sigma_v^1 + 2c\sqrt{K_p}$$

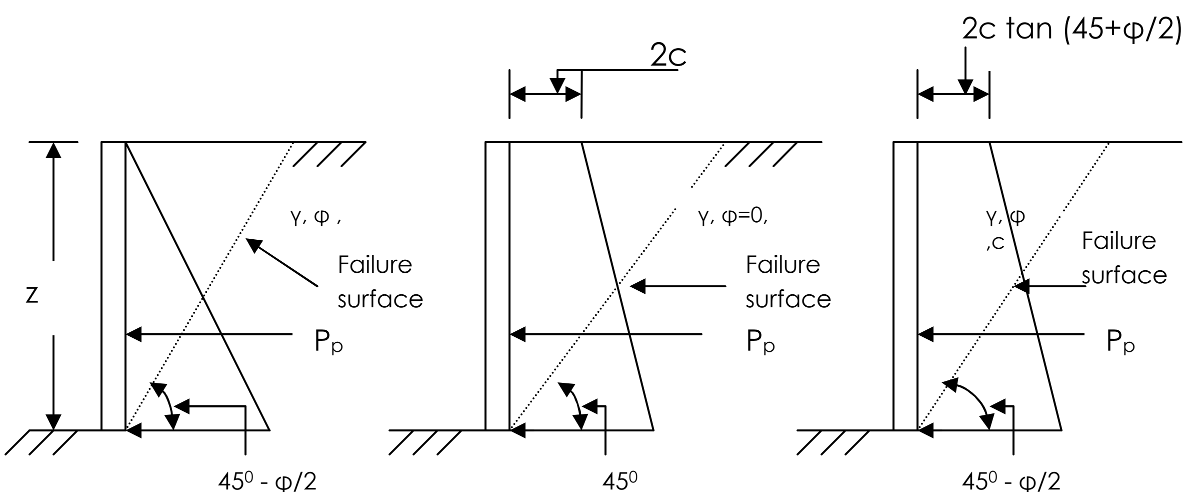
where K_p is the coefficient of maximum earth pressure. For smooth, vertical walls retaining horizontal backfills K_p is calculated from the relation;

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} = \tan^2\left(45+\frac{\phi}{2}\right) \quad (5)$$

and for backfill inclined at β to horizontal

$$K_p = \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \quad (6)$$

The passive pressure distribution for various backfill strength characteristics are shown in Fig 5.



$K_p = \tan^2(45+\phi/2)$

(a)	(b)	(c)
$p_p = K_p\gamma z$	$p_p = \gamma z + 2c$	$p_p = \gamma z\tan^2(45+\phi/2) - 2c\tan(45+\phi/2)$
$P = K_a\gamma H^2/2$	$P_a = \frac{1}{2}\gamma H^2 + 2cH$	$P_p = \left(\frac{\gamma H^2}{2}\right)\tan^2(45+\phi/2) + 2cH \tan(45+\phi/2)$

(a). (b). (c).

Fig.5. Maximum Rankine passive earth pressure distribution for back fill with various combination of friction and cohesive strength; (a) Frictional resistance, no cohesion; (b) Cohesive soil, no frictional resistance; (c) combined cohesion and friction (S.L Cramer)

For homogeneous dry backfill Rankine theory predicts a triangular distribution oriented parallel to the backfill surface. The backfill earth pressure resultant, or the passive thrust P_p, acts at a point located at H/3 above the base a wall of height H with the magnitude;

$$P_p = \frac{1}{2} K_p \gamma H^2 \tag{7}$$

Calculation of Static Earth Pressure: Coulomb Theory

By assuming that the forces acting on the back of the retaining wall resulted from the weight of the wedge of the soil above a planar failure plane surface coulomb used force equilibrium to determine the magnitude of the thrust acting on the wall for both minimum active and maximum passive conditions.

Under minimum active earth pressure conditions, the active thrust on a wall with the geometry shown in Fig 6 is obtained from the force equilibrium for critical failure surface, the active thrust on wall retaining a cohesionless soil can be expressed as;

$$P_a = \frac{1}{2} K_a \gamma H^2 \tag{8}$$

where,

$$K_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\delta + \theta)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi)\sin(\phi - \beta)}{\cos(\delta + \theta)\cos(\beta - \theta)}}\right]^2} \tag{9}$$

δ is the angle of wall friction between the wall and the soil, β is the angle of slope of filling and θ is the angle of inner face of wall with the vertical face.

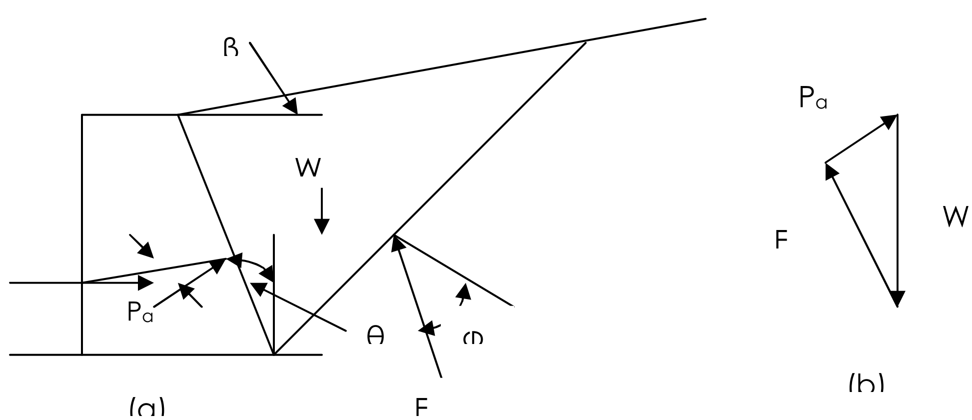


Fig.6. (a) Triangular active wedge bounded by planar backfill surface failure surface and wall, (b) force polygon for active Coulomb wedge (S.L Cramer)

Coulomb theory does not explicitly predict the distribution of active pressure, but it can be shown to be triangular for linear backfill surface with no loads. In such case, P_a acts at a point located at H/3 above the height of wall of height H.

For the maximum passive conditions in cohesionless backfill (Fig. 7) Coulomb theory predicts a passive thrust as

$$P_p = \frac{1}{2} K_p \gamma H^2 \tag{10}$$

where, $$K_p = \frac{\cos^2(\phi+\theta)}{\cos^2\theta\cos(\delta-\theta)\left[1+\sqrt{\dfrac{\sin(\delta+\phi)\sin(\phi+\beta)}{\cos(\delta-\theta)\cos(\beta+\theta)}}\right]^2} \tag{11}$$

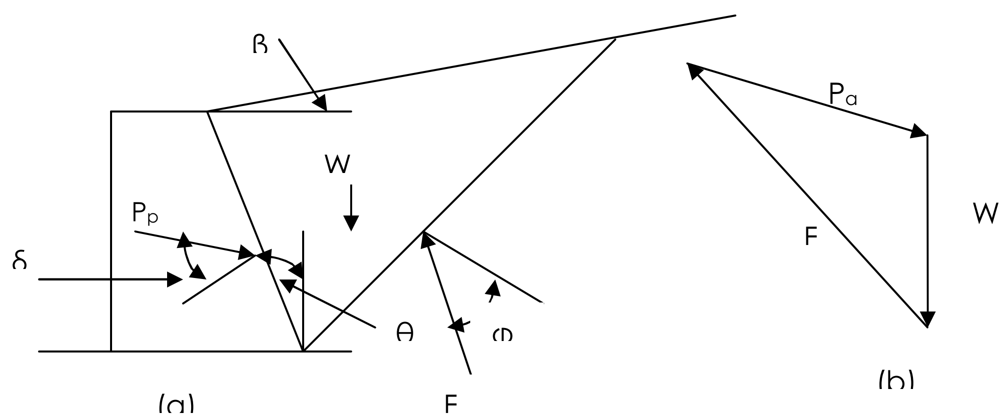


Fig.7. (a) Triangular passive wedge bounded by planar backfill surface failure surface and wall, (b) force polygon for active Coulomb wedge (S.L Cramer)

The calculation of passive earth pressure coefficient K_p by Rankine method should be avoided when the slope of top surface of the backfilling that β>0. In such situation the Coulombs method should be followed. Further neither the Coulomb method nor Rankine method explicitly incorporated the cohesion as an equation parameter in lateral earth pressure computation. Therefore when cohesive soil are used as filling materials Bell equation should be used in computation of lateral earth pressure. Readers interested on the topic may refer to the book on Foundation Analysis and Design by J.E. Bowles (McGraw-Hill International Edition, 1997).

For practical field application of the Coulomb theory coefficient in the annexure attached may directly be referred which taken from the book by J.E. Bowles mentioned above.

Dynamic Response of Retaining Walls

The dynamic response of even simplest type of retaining wall is quite complex. Wall movement and pressure depends on the response of the soil underlying the wall, the response of the backfill, the inertial and flexural response of the wall itself, and the nature of the input motions. Most of the current understanding of the dynamic response of retaining wall has come from the model test and numerical analyses. These tests and analyses, the majority of which involved gravity wall indicate that;

1. Wall can move by translation and or by rotation. The relative amounts of translation and rotation depend on the design of the wall; one or the other may predominate for some wall, and both may occur for others.

2. Magnitude and distribution of dynamic wall pressure are influenced by the mode of wall movement (e.g. translation, rotation about the base, or rotation about the top).

3. Maximum soil thrust acting on the wall generally occurs when the wall has translated or rotated towards the backfill (when the inertial force on the wall is directed towards the backfill). The minimum soil thrust occurs when the wall has translated or rotated away from the backfill.

4. The shape of the earth pressure distribution on the back of the wall changes as the wall moves. The point of application of the soil thrust therefore, moves up and down along the back of the wall. The position of the soil thrust in highest when the wall moves towards the soil and lowest when the wall moves outwards.

5. Dynamic wall pressures are influenced by the dynamic response of the wall and backfill and can increase significantly near the natural frequency of the wall-backfill system. Permanent wall displacement also increases at frequency of the wall-backfill system. Dynamic response effect can also cause deflections of different parts of the wall to be out of phase. This effect can be particularly significant for wall that penetrates into the foundation soil when the backfill soil moves out of phase with the foundation soils.

6. Increased residual pressures may remain on the wall after an episode of strong shaking has ended.

[Geotechnical Earthquake Engineering By S.L. Cramer, Pearson Education]

In summarizing, it may be seen that the damage of retaining wall under seismic forces has been due to the increase in the pressure resulting from the movement of the structure during earthquake. Therefore, separate evaluation of dynamic earth pressure and stresses on the retaining structures should be done for retaining wall constructed in the seismic area. The one of the commonly used method adopted in the evaluation of dynamic seismic coefficient for lateral earth pressure is discussed in the following pages.

Mononobe-Okabe Seismic Coefficient Analysis

The most commonly adopted method for determining the dynamic lateral pressure on retaining structures was develop by Mononobe (1929) and Okabe (1926). The method was developed for dry cohesionless materials and was based on the assumption that:

(1) the wall yields sufficiently to produce minimum active pressure

(2) when the minimum active pressure in attained, a soil wedge behind the is at the point of incipient failure and the maximum shear strength is mobilized along the potential sliding surface.

(3) the soil behind the wall behaves as a rigid body so that acceleration are uniform throughout the mass; thus the effect of the earthquake motion can be represented by the inertia forces $k_h \times W$ and $k_v \times W$ where W is the weight of the sliding wedge $k_h g$ and $k_v g$ are the horizontal and vertical components of the earthquake acceleration at the base of the wall.

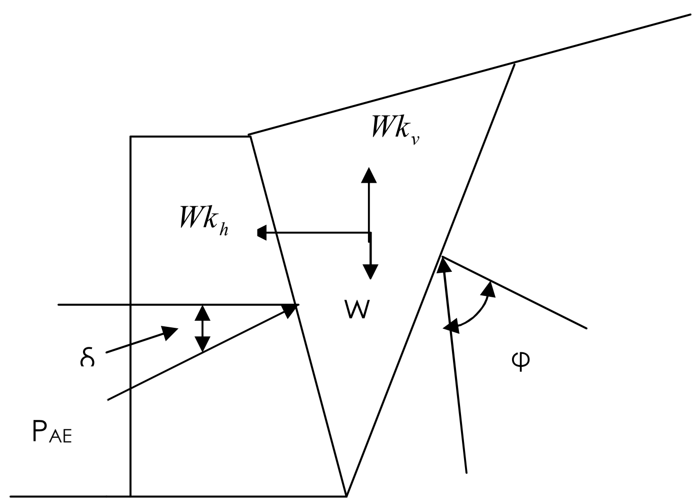


Fig.8. Forces considered in Mononobe-Okabe Analysis

In effect, the active pressure during the earthquake P_{AE} is computed by the Coulomb theory except that the additional forces $k_h \times W$ and $k_v \times W$ as shown above in Fig.8 are included in the computation. Determining the critical sliding surface is the usual way and the active pressure corresponding to this surface lead to the following expression:

$$P_{AE} = \frac{1}{2}\gamma H^2 (1 - k_v) \times K_{AE} \qquad (12)$$

where $K_{AE} = \dfrac{\cos^2(\phi - \theta - \beta)}{\cos\theta \cos^2\beta \times \cos(\delta + \beta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2}$ (13)

$\theta = \tan^{-1}\dfrac{k_h}{1 - k_v}$

$\gamma =$ unit weight of soil

$H =$ height of wall

$\phi =$ angle of friction of soil

$\delta =$ angle of wall friction

$i =$ slope of ground surface behind the wall

$\beta =$ slope of back wall to the vertical

$k_h =$ horizontal ground acceleration /g

$k_v =$ vertical ground acceleration /g

The horizontal component of the force P_{AE} may be expressed as P_{AEh} where

$$P_{AEh} = P_{AE}\cos(\delta + \beta) \qquad (14a)$$

$$P_{AEh} = \frac{1}{2}\gamma H^2(1 - k_v) \times K_{AE}\cos(\delta + \beta) \qquad (14b)$$

For wall with vertical inside face that is $\beta = 0$

$$P_{AEh} = \frac{1}{2}\gamma H^2(1 - k_v) \times K_{AE}\cos(\delta) \qquad (14c)$$

Mononobe and Okabe considered that the total pressure computed by their analytical approach would act on the wall at the same position as the initial static pressure; that is at the height of H/3 above the base. With the analysis on effect of the vertical components on the dynamic pressure with varied data; it was also found that in most of earthquakes the horizontal acceleration components are considerably greater that the vertical components and it seems reasonable to conclude that in such cases the influence of vertical components k_v can be neglected for the practical purpose.

Finally it may be noted that the values of the K_{AE} represent the total maximum earth pressure developed on the wall. For many purpose it convenient to separate this pressure into two components – the initial static pressure on the wall and the dynamic pressure increment due to the base motion. For practical purpose we may write

$$K_{AE} = K_a + \Delta K_{AE} \tag{15}$$

and the dynamic lateral pressure components becomes

$$\Delta P_{AE} = \frac{1}{2}\gamma H^2 \times \Delta K_{AE} \tag{16}$$

Methods of determining both active and passive lateral pressure by the Mononobe-Okabe method, but utilizing the graphical constructions, such as coulomb or Melbye construction procedure has been described by Kapila (1962), who also showed that using the same general approach, the passive pressure resistance under seismic conditions may be expressed by the equations;

$$P_p = \frac{1}{2}\gamma H^2 (1 - k_v) K_{PE} \tag{17}$$

where $$K_{PE} = \frac{\cos^2(\phi + \theta - \beta)}{\cos\theta \cos^2\beta \times \cos(\delta - \beta + \theta)\left[1 - \sqrt{\dfrac{\sin(\phi - \delta)\sin(\phi + i - \theta)}{\cos(\delta - \beta + \theta)\cos(i - \beta)}}\right]^2} \tag{17a}$$

In addition to the qualitative indications of the lateral earth pressure developed during earthquake, model tests, in which small scale structures are subjected to base motion by means of shaking tables, have been used done by numbers of investigators for quantitative evaluations of the magnitude of the dynamic pressures. The general conclusions of the experimental studies by various investigators are as summarized below:

(1). All the investigators have concluded that the lateral earth pressure coefficients for the cohesionless backfill computed from the Mononobe-Okabe analysis are in reasonably good with values developed in small scale (model) structures.

(2). In case of unanchored retaining structures, most of the investigation agree that the increase in the lateral pressure due to the base excitation are greater at the top of the wall and the resultant increment acts at the height varying from 0.5H to 0.67H above the base of the wall.

(3). The increase in the lateral pressure due to dynamic effect may be accompanied by an outward movement of the wall, the amount of movement increasing with the magnitude of the base acceleration.

(4). After a retaining structures with a granular backfill materials has been subjected to a base excitation, there is a residual pressure which is substantially greater than the initial pressure before the base excitation; this residual is also a substantial portion of the maximum pressure developed during the excitation.

Provision of IS 1893:1984 for Calculation of Dynamic Lateral Pressure

The provision of calculation of lateral dynamic earth pressure in IS: 1893:1984 [which is in process of revision]; is in the line of the Mononobe-Okabe method as described in the preceding pages.

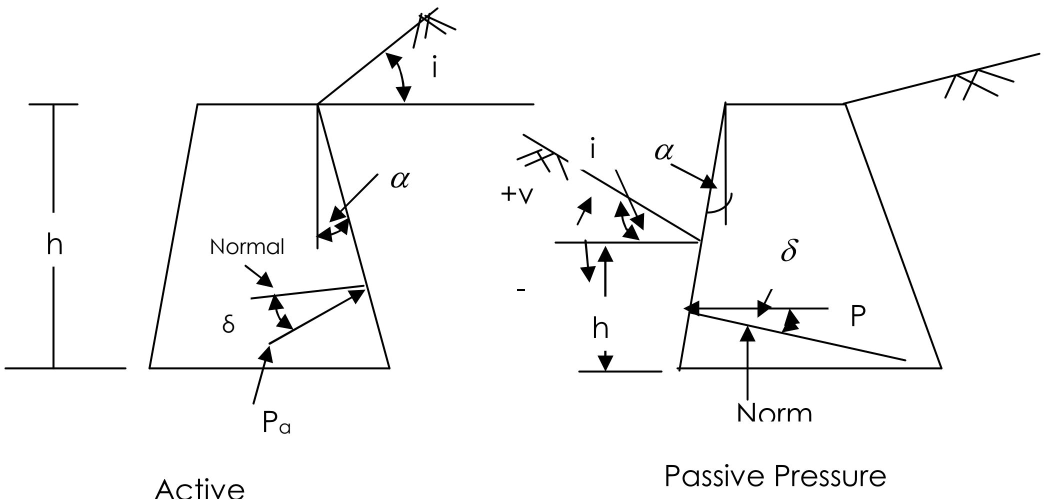


Fig. 9 Earth Pressure Due to Earthquake on Retaining Wall

As per the provision of IS: 1893:1984 the general conditions encountered for the design of retaining wall is illustrated in the Fig. 9 above. The active earth pressure exerted against the wall is given by

$$P_a = \frac{1}{2} w h^2 C_a \quad (18)$$

where,

P_a = active earth pressure

w = unit weight of soil

h = height of wall

$$C_a = \frac{(1 \pm \alpha_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{ \frac{\sin(\phi + \delta)\sin(\phi - \iota - \lambda}{\cos(\alpha - \iota)\cos(\delta + \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2 \quad (18a)$$

Two values shall be calculated from above equation, one for $1+\alpha_v$ and the other for $1-\alpha_v$ and maximum of the two shall be the design values. The values of the notations shall be taken as below:

α_v = vertical seismic coefficient – its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{2}{3}\alpha_h$

ϕ = angle of internal friction of soil

$\lambda = \tan^{-1}\frac{\alpha_h}{1 \pm \alpha_v}$

α = angle which earth face of the wall makes with the vertical as shown in Fig.9

i = slope of earth fill as shown in Fig.9

δ = angle of friction between the wall and earth fill

α_h = horizontal seismic coefficient

From the total pressure computed from the relation given above subtract the static earth pressure calculated by putting $\alpha_h = \alpha_v = \lambda = 0$ in the expression given above or from the equations available for calculation of static earth pressure using Coulomb theory. The remainder is the dynamic increment. The dynamic increment shall be considered separately in addition to the static pressure and this will be considered to act at the mid-height of the wall. The point of application of the dynamic increment pressure shall be at mid height of the wall as per the provision of the code.

Similarly the general conditions encountered in the design of retaining wall for passive pressure is also illustrate in Fig.9. The passive pressure against the wall shall be given by

$$P_p = \frac{1}{2} w h^2 C_p \quad (19)$$

where, $$C_p = \frac{(1 \pm \alpha_v)\cos^2(\phi + \alpha - \lambda)}{\cos\lambda \cos^2\alpha \cos(\delta - \alpha + \lambda)} \times \left[\frac{1}{1 - \left\{ \frac{\sin(\phi + \delta)\sin(\phi + \iota - \lambda}{\cos(\alpha - \iota)\cos(\delta - \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2 \quad (19a)$$

The equation (19) gives the total passive pressure on the face of the wall at the time of the base acceleration. The static passive pressure calculated based on the Coulomb theory shall be deducted from the total passive pressure and the remainder shall be the dynamic passive pressure decrement. The point of application of the dynamic decrement is assumed to act an elevation of 0.66h above the base of the wall.

Effect of Surcharge on Dynamic Pressure

Further the code also provides that the active pressure against the wall due to uniform surcharge in intensity q per unit area of the inclined earth fill, during the acceleration of the base shall be:

$$(P_a)_q = \frac{qh\cos\alpha}{\cos(\alpha - i)} C_a \qquad (20)$$

The point of application of the dynamic increment in active pressure due to uniform surcharge shall be at an elevation of $0.66h$ above the base of the wall, while the static component shall be applied at mid-height of the wall.

The passive pressure against the wall due to uniform surcharge on intensity q per unit area of the inclined earthfill during the acceleration of the base shall be:

$$(P_p)_q = \frac{qh\cos\alpha}{\cos(\alpha - i)} C_p \qquad (21)$$

The point of application of decrement in the passive pressure due to uniform surcharge shall be at an elevation of $0.66h$ above the base of the wall; while the static component shall be applied at the mid-height of the wall.

Effect of Saturation on Dynamic Lateral Earth Pressure

Further the researcher has also analyzed that the presence of water in the backfill further increase the dynamic pressure during seismic excitation [H.Bolton Seed and Robert V. Whiteman]. For saturated earthfill the saturated unit weight of the soil shall be adopted while calculating the dynamic active earth pressure increment or passive earth pressure decrement using the equation (18), (19), (20) and (21) as discussed in preceding pages. For submerged earthfill also, the dynamic increment or decrement in active and passive earth pressure during earthquake shall be found from the expression given in the equation (18), (19), (20) and (21) with the following modification:

(1). The value of δ shall be taken as $\frac{1}{2}$ the value of δ in dry condition

(2). The value of λ shall be taken as follows:

$$\lambda = \tan^{-1}\frac{w_s}{w_s^{-1}} \times \frac{\alpha_h}{1 \pm \alpha_v} \qquad (22)$$

where w_s = saturated unit weight of soil in gm/cc

α_h = horizontal seismic coefficient

α_v = vertical seismic coefficient which is $\frac{2}{3}\alpha_h$

(3) Buoyant unit weight shall be adopted

Simplified Method of Determining Dynamic Lateral Earth Pressure Coefficient

The complex calculation of dynamic earth pressure coefficient some time becomes cumbersome when the immediate solution at site is required. In such situation, for simple cases of vertical wall and horizontal dry backfills, the methods proposed by Seed [Bolton Seed & V. Whiteman] can be adopted in determining the Mononbe-Okabe earth pressure effects as below:

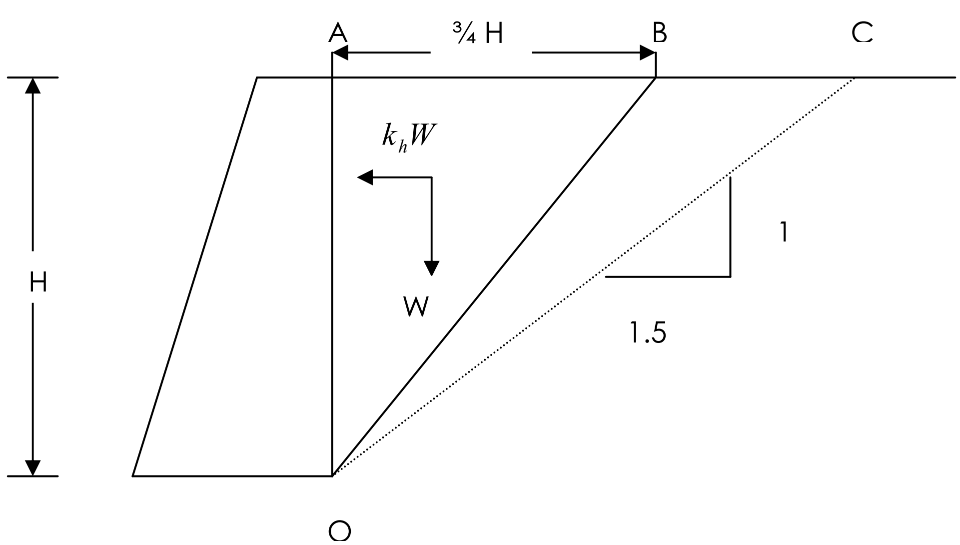


Fig. 10 Empirical Rules for Determining Dynamic Lateral Earth Pressure

The following simple rules are adopted in determining the coefficient of the dynamic lateral Mononbe-Okabe earth pressure coefficient:

1. Maximum dynamic active pressure P_{AE} is equal to the sum of the initial static pressure and the dynamic increment ΔP_{AE}

   Thus $P_{AE} = \text{Static pressure} + \Delta P_{AE}$

$$= \frac{1}{2} K_a \gamma H^2 + \Delta P_{AE}$$

2. For a backfill with angle of friction equal to about 35^0, the dynamic pressure increment is approximately equal to the inertia force on a soil wedge extending a distance of $\frac{3}{4}H$ behind the crest of the wall

   Thus $\Delta P_{AE} = W_{OAB} \times k_h$

$$= \frac{1}{2} \times H \times \frac{3}{4} H \times \gamma \times k_h$$

$$= \frac{1}{2} \times \gamma H^2 \times \frac{3}{4} k_h$$

$$P_{AE} = \frac{1}{2} K_a \gamma H^2 + \frac{1}{2} \times \gamma H^2 \times \frac{3}{4} k_h$$

$$\frac{1}{2} \times \gamma H^2 \left(K_a + \frac{3}{4} k_h \right)$$

   Therefore, the dynamic increment of pressure in $\frac{3}{4}$ times the horizontal seismic acceleration coefficient.

3. The dynamic pressure increment, ΔP_{AE}, acts on the wall at a height of 0.6H above the base.

Design Example

Carry out the stability analysis of the retaining wall of concrete M10 as given in Fig. 11 and calculate the base pressure.

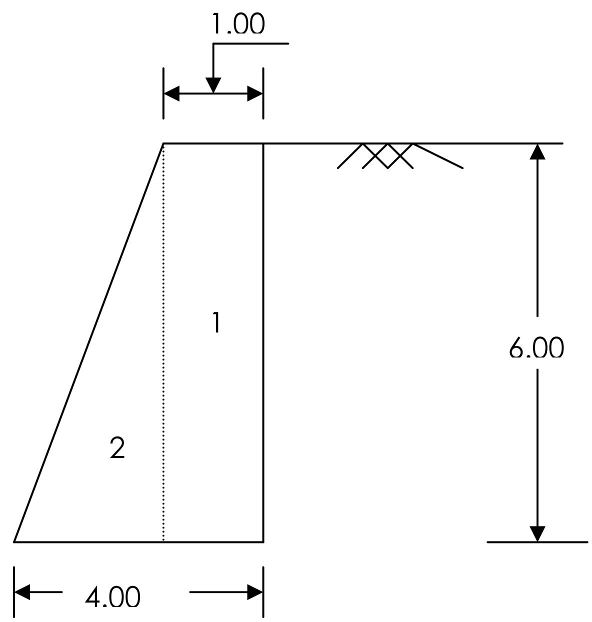


Fig. 11 Section of Retaining wall

The retaining wall is located in seismic Zone V. The properties of the backfill materials are as detailed below:

Unit weight of backfill soil $(\gamma) = 18kN/m^3$

Angle of internal soil friction $(\phi) = 30^0$

Angle of Wall friction $(\delta) = 20^0$

Angle of back face of wall with vertical $(\alpha) = 0^0$

Adopting Coulomb theory for calculation of static earth pressure the coefficient of active earth pressure $K_a = 0.297$ (values can also be directly read from the table appended)

Calculation of Horizontal and Vertical Seismic Coefficient

Since the relevant code dealing with the provision of seismic design of retaining wall is still under revision the data provided in the IS: 1893:2002 Part I is referred for relevant seismic data.

The calculation of the horizontal seismic coefficient is calculated as

$$\alpha_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$$

where, α_h = Horizontal seismic coefficient

Z = Seismic Zone factor which is taken as 0.36 for seismic Zone V

I = Importance factor which is taken as 1.00 for the retaining wall

R = Response reduction factor taken as 1.50 for unreinforced concrete wall

$\frac{S_a}{g}$ = Spectral acceleration coefficient or flexibility factor

The solid retaining wall is almost rigid and no differential displacement shall take place in the wall during seismic acceleration. Hence the wall is taken as zero period structure and the spectral acceleration coefficient of the wall is taken as 1.00. Putting the values we have

$$\alpha_h = \frac{0.36}{2} \times \frac{1.00}{1.50} \times 1.00 = 0.12$$

The vertical acceleration coefficient $\alpha_v = \frac{2}{3} \times 0.12 = 0.08$

Calculation of active pressure coefficient under seismic condition:

Adopting the method prescribed in IS: 1893: 1984 the dynamic active pressure coefficient is given by the equation:

$$C_a = \frac{(1 \pm \alpha_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{ \frac{\sin(\phi + \delta)\sin(\phi - \iota - \lambda}{\cos(\alpha - \iota)\cos(\delta + \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2$$

Putting the values of soil properties and seismic coefficients we calculate

$$\lambda = \tan^{-1} \frac{\alpha_h}{1 \pm \alpha_v}$$

λ = (Corresponding to $+\alpha_v$) = 6.34

$$Cos(\phi - \lambda - \alpha) = Cos(30 - 6.34 - 0) = 0.91$$

$$Cos(\delta + \alpha + \lambda) = Cos(20 + 0 + 6.34) = 0.896$$

$$Cos\lambda = Cos6.34 = 0.993$$

$$Cos\alpha = Cos(0) = 1.00$$

$$Sin(\phi + \delta) = Sin(30 + 20) = 0.766$$

$$Sin(\phi - i - \lambda) = Sin(30 - 0 - 6.34) = 0.401$$

$$Cos(\alpha - i) = Cos(0 - 0) = 1.00$$

$$C_a = \frac{(1 + 0.08) \times (0.91)^2}{0.993 \times (1.00)^2 \times 0.896} \times \left[\frac{1}{1 + \left[\frac{0.766 \times 0.401}{1.00 \times 0.896} \right]^{1/2}} \right]^2$$

$$C_a = 1.00 \times \left[\frac{1}{1 + 0.58} \right]^2 = 0.40$$

By Empirical method:

$$C_a = K_a + \frac{3}{4} k_h = 0.297 + \frac{3}{4} \times 0.12 = 0.387 \text{ [The values are almost comparable]}$$

Calculation static earth pressure

Active pressure $= 0.297 \times 18 \times 6.00 = 32.07 kN/m$

Height of action from the base of the wall $= \frac{H}{3} = 2.00m$

Dynamic pressure $= 0.40 \times 18 \times 6.00 = 43.20 kN/m$

Dynamic increment $\Delta E = 43.20 - 32.07 = 11.13 kN/m$

Height of action from the base of the wall $0.5H = 3.00m$

Self weight of wall

Section 1 $= 1 \times 6.00 \times 20.00 = 120.00 kN/m$

Section 2 $= \frac{1}{2} \times 6.00 \times 3.00 \times 20.00 = 180.00 kN/m$

Stability analysis

Calculation of overturnig moments

SL NO	Description	Vertical Weight (kN)	Horizontal Force (kN)	Lever arm from the base of Wall (m)	Overturnig moment (kNm)
	Static condtion				
1	Active pressure		32.07	2.00	64.14
	Seismic condition				
2	Sectio 1	120.00	14.40	3.00	43.20
3	Section 2	180.00	21.60	2.00	43.20
4	Dynamic pressure		11.13	3.00	33.39
	Total	**300.00**	**79.20**		**183.93**

Calculation of Restoring moments about the toe of wall

SL NO	Description	Vertical Weight (kN)	Lever arm from the toe of wall	Restoring moment (kNm)
	Static condtion			
1	Sectio 1	120.00	2.50	300.00
2	Section 2	180.00	2.00	360.00
	Total	300.00		660.00

Factor of safety against overturning $\frac{660}{183.93} = 3.50 > 1.2$ [The section can further be reduced]

Coefficient of frictional of the soil and wall at base $\tan\phi = \tan 30 = 0.57 \approx 0.50$

Factor of safety against sliding $= \frac{0.50 \times 300}{79.20} = 1.89 > 1.10$ OK

Net moment $= 660 - 183.93 = 476.07 kNm$

Vertical load $= 300 kN$

Eccentricity $e = \frac{B}{2} - \frac{M}{P} = \frac{4}{2} - \frac{476.07}{300} = 0.41 m$

Base Pressure $f = \frac{P}{B}\left(1 \pm \frac{6e}{B}\right) = \frac{300}{4}\left(1 \pm \frac{6 \times 0.41}{4}\right)$

$f_{\max} = 121.12 kN/m^2$ [Should not be more than allowable bearing capacity of soil]

$f_{\min} = 28.87 kN/m^2$ [No tension exist hence Ok]

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   Part I: General Provisions and Buildings