Module 2 : Theory of Earth Pressure and Bearing Capacity

Lecture 9 : Development of Uplift Capacity Theory [Section 9.1: Intoduction]

Objectives

In this section you will learn the following

- Development of Uplift Capacity Theory
- Introduction
- Different failure surfaces assumed
- Seismic vertical uplift capacity of strip anchors
- Collapse mechanism
- Application of the upper bound theorem of limit analysis
- Uplift equations

9. DEVELOPMENT OF UPLIFT CAPACITY THEORY

9.1 Introduction

An anchor, whether it is placed in soil or rock is essentially a sub-structural member which transmits a tensile force from main structure to the surrounding ground. The shear strength of the surrounding ground is used to resist this tensile force and in general attempts are made to fasten the anchor to the firm ground well away from structure. The most common anchor consist of a high strength steel tendon installed at the required inclination and to the required depth to resist the applied load in an sufficient manner so that the tendon material is stressed to the economic levels and the ground in which it is embedded is also realistically stressed. The tensile force in anchor is that force which is necessary for equilibrium between the anchor, the structure to which it is attached and the ground in which the anchor is embedded so that the movements of structure and surrounding ground are kept to acceptable levels.

Types of ground anchors:

The anchors can be classified in different ways.

- **Depending upon the use they can be classified as,**

I **Temporary anchors** which are provided to structures which serve a limited purpose during the construction stage only and may not be required in final service stage. The temporary ground anchors are used for the tying back of the retaining walls in deeper excavations to avoid congestion of propped excavations.

II **Permanent anchors** which are provided to structures that are called upon to withstand the stresses for their entire service phase which may last decades. The permanent ground anchors are used to support sea defense and tidal protection walls. These are also used for anchoring down the structures against buoyancy in dry docks, underground train stations, reservoirs and semi-underground storage structures, underground passes, submerged tunnels and strengthening of dams. The stability of the tall structures e.g. radio and TV masts, elevated towers and similarly proportioned structures against overturning has been achieved with ground anchors and many structures built on hills and mountainsides have similarly been secured by anchor cables. Permanent ground anchors have been used for certain types of light cable roof structures built to give extremely large but inexpensive clear span structures ideal for supporting areas where simple roof covering is required. The stability of the hillsides to enable road networks to be built or for open cast mining to take place has made a very considerable difference to the economy of these projects by allowing steeper sided excavations and saving enormous quantities of soil removal.

The use of rock anchors to improve the mechanical properties of rock in the construction of tunnels or underground caverns, such as in the features of underground power stations, is another widespread application of this technique together with that of rock bolting. Also, there is small but significant use of the ground anchors for very specialized applications, such as pile testing, the preloading and preconsolidation of foundations.

- **Depending upon the shape they can be classified as,**

I **Plate anchors** :- Refer fig. 2.29. These can be of any shape such as strip, rectangular, circular etc. These are mostly used in tall steel structures. These types of anchors are the most common anchors used in the field.

II **Helical anchors** :- Refer fig. 2.30. These foundations have the advantages of rapid installation and immediate loading capabilities that offer cost-saving alternatives to reinforced concrete, grouted anchors and driven piles. Modern helical anchors are earth anchors constructed of helical shaped circular steel plates welded to a steel shaft. The plates are constructed as a helix with a carefully controlled pitch. The anchors can have more than one helix located at appropriate spacing on the shaft. The central shaft is used to transmit torque during installation and to transfer axial loads to the helical plates. The central shaft also provides a major component of the resistance to lateral loading.

III **Grout anchors** :- In this type, the anchors are installed and the surrounding area is filled with the grout. In some cases, the enlarged bulb of the grout(fig. 2.31a) is formed which acts as a anchorage. The anchors with underreams (fig. 2.31b) are mostly used in clay soils.

- **Depending upon the capacity anchors can be classified as,**

I **Horizontal anchors :-** Refer fig. 2.29. In these anchors the plate is horizontally aligned. These are used in towers, pipelines etc.

II **Vertical anchors :-** Refer fig. 2.29. In these anchors the plate is vertically aligned. These are used for anchoring sheet pile walls.

III **Inclined anchors :-** Refer fig. 2.29. In this type, the anchor plates are inclined at an angle with the horizontal. These can be used in slopes, towers etc. This is the generalized form of the above two anchors.

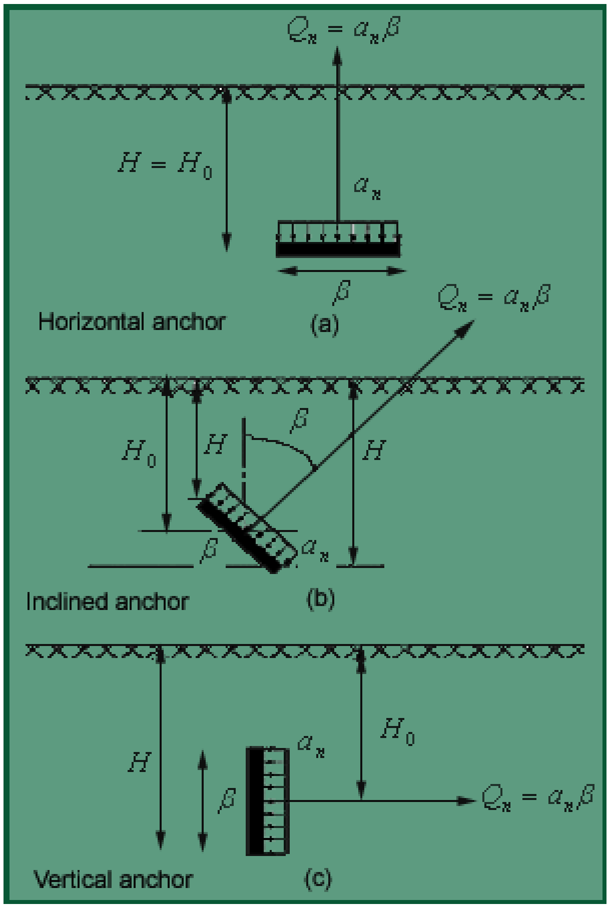

Fig. 2.29 Different types of anchors

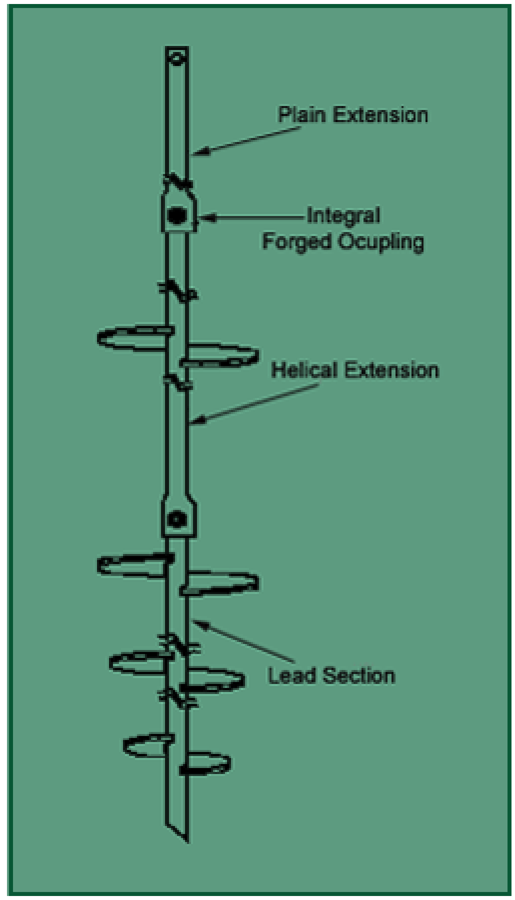

Fig. 2.30 Helical anchors

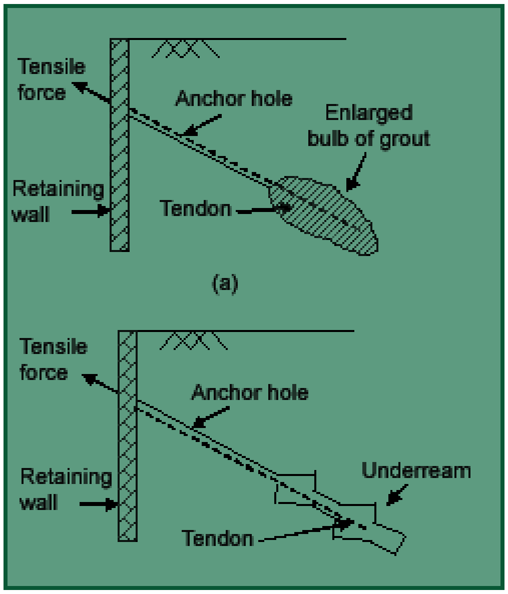

Fig. 2.31 Different types of grout anchors

Common applications of ground anchor in India (Fig.2.32)

- Raising or strengthening of dams to stabilize and to protect against sliding,
- Anchors for diaphragm wall and other retaining structures,
- To resist wind and earthquake forces,
- Prestressed piles,
- Tie backs for cable ways,
- Stabilization of hill slopes,
- For anchoring cantilever spans of bridge and also for importing stability for foundations of cantilever roofs,
- Strengthening of foundations for heavy machinery subjected to dynamic loads,
- To strengthen abutments of bridges subjected to heavy loads,
- Pile load tests for large capacity piles

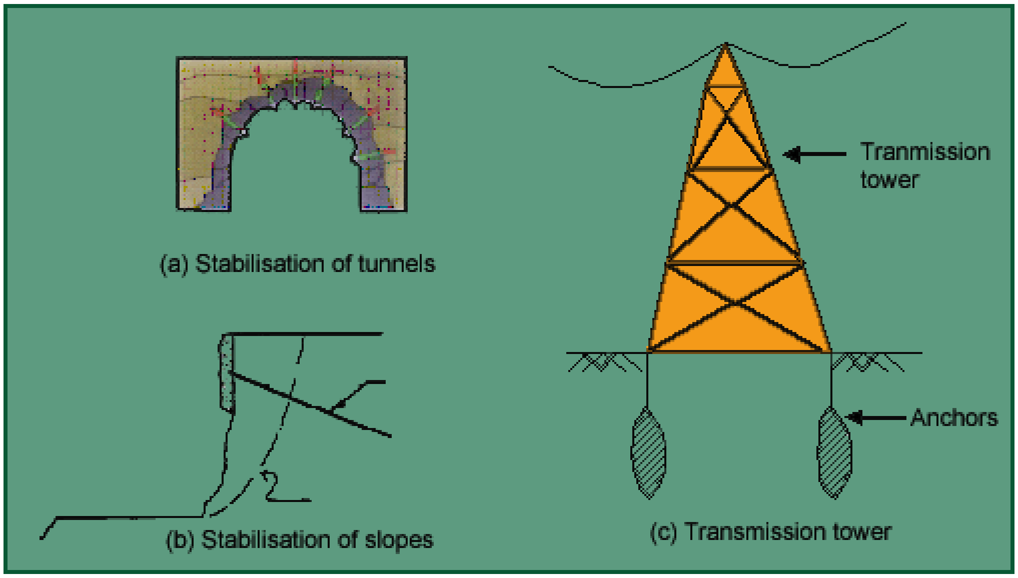

Fig. 2.32 Applications of anchors

Different failure surfaces assumed

First the linear failure surface is considered by different researchers as shown in fig. 2.33. In this cd is considered as the imaginary retaining wall. As the wall (cd) moves towards backfill(B), the portion A moves upwards relative to portion B, so this case is of negative friction angle. Meyerhof and Adams (1968) considered the failure surface as logspiral, as shown in fig. 2.34.

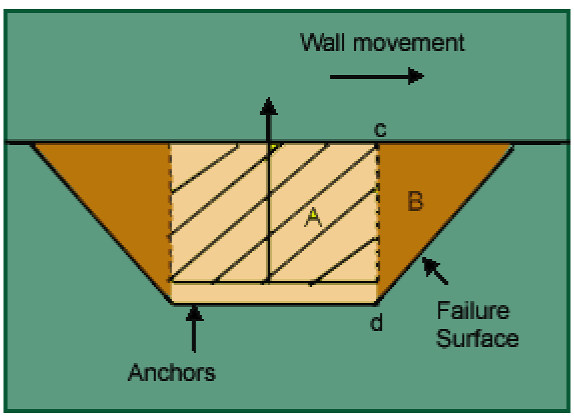

Fig. 2.33 Linear failure surface

Fig. 2.34 Logspiral failure surface

Seismic vertical uplift capacity of strip anchors

Introduction

In this, the influence of horizontal earthquake acceleration on the vertical uplift capacity of shallow strip anchors buried in cohesionless material has been theoretically examined by using the upper bound theorem of limit analysis and with the assumption of planar rupture surfaces. Seismic factors e_q and e_γ due to surcharge and unit weight respectively have been obtained with respect to variation in earthquake acceleration coefficient, soil friction angle, and embedment ratio of anchors.

The earthquake motion is assumed to be a uniform horizontal acceleration applied at the same time to the whole domain under consideration. This is in common with the conventional pseudo-static analyses. The mass of the anchor is negligible, so that if an earthquake occurs the anchor itself will not be subjected to any horizontal force. Also, the soil lying below the surface of anchor has been assumed not to offer any resistance to uplift. Soil is a rigid, perfectly plastic medium satisfying the Mohr-Coulomb failure criterion and associated flow rule, and the occurrence of an earthquake does not affect the values of soil parameters c, ϕ and γ.

Collapse mechanism

Consider an anchor, having width b and embedded at a depth d from the ground surface as shown in Fig.2.35(a), to be pulled out in an inclined direction with an inclination a to the vertical. The collapse mechanism is assumed to be a combination of three different triangular rigid blocks. Soil block OFG moves with velocity V_0, the same as that of the anchor plate. Soil blocks OEF and OGH move with velocities V_1 and V_2 making an angle F with the velocity discontinuity lines EF and GH. V_{01} is the relative velocity of block OEF with respect to that of block OFG, and V_{02} is the relative velocity of block OGH with respect to that of block OFG. The directions of velocities V_{01} and V_{02} are inclined at an angle ϕ to the velocity discontinuity lines OF and OG. Referring to the velocity hydrographs shown in Figs 2.35(b) and 2.35(c), it can be shown that

$$V_1 = V_0 \frac{\cos(\beta_1 - \phi + \alpha)}{\cos(\phi + \theta_1 - \beta_1)}$$

$$V_{01} = V_0 \frac{\sin(\theta_1 + \alpha)}{\cos(\phi + \theta_1 - \beta_1)}$$

$$V_2 = V_0 \frac{\cos(\beta_2 - \phi + \alpha)}{\cos(\phi + \theta_2 - \beta_2)}$$

$$V_{02} = V_0 \frac{\sin(\theta_2 + \alpha)}{\cos(\phi + \theta_2 - \beta_2)}$$

Here β_1 and β_2 are the horizontal inclinations of velocity discontinuity lines OF and OG, and ϕ_1 and ϕ_2 are the vertical inclinations of the directions of velocities V_1 and V_2. The positive directions of these angles are marked in Fig. 2.35(a).

Application of the upper bound theorem of limit analysis

An upper bound estimate of the ultimate vertical pullout resistance, P_u, per unit length of strip can be determined by equating the rate of total work done by the external and body forces to the rate of dissipation of total internal energy. Soil blocks OFG, OEF and OGH will be subjected to vertical and horizontal body forces of magnitudes equal to W_0 and m W_0, W_1 and m W_1, W_2 respectively. Here W_0, W_1 and W_2 are the weights of the soil blocks OFG, OEF and OGH, and m is the magnitude of the horizontal earthquake acceleration coefficient. If q is the surcharge pressure due to the soil overburden, there will be vertical and horizontal forces Q_1 and mQ_1 on the top of block OEF and Q_2 and mQ_2 on the top of OGH, where $Q_1 = qL_{EO}, Q_2 = qL_{OH},$, and L_{EO} and L_{OH} are the lengths of EO and OH. The rate of dissipation of total internal energy for a cohesionless material will become equal to zero: that is, δ (Energy) = 0. With the application of the upper bound theorem of limit analysis, it can be shown that

$$P_u = \frac{W_0 V_0 \left(\cos\alpha - m\sin\alpha\right) + V_1 (W_1 + Q_1)(\cos\theta_1 + m\sin\theta_1) + V_2 (W_2 + Q_2)(\cos\theta_2 - m\sin\theta_2)}{V_0 \cos\alpha}$$

The magnitude of P_u can be minimised by varying independently the kinematically admissible values of the four variables $\alpha, 01, 02$ and $\beta 1 (or\, \beta 2)$

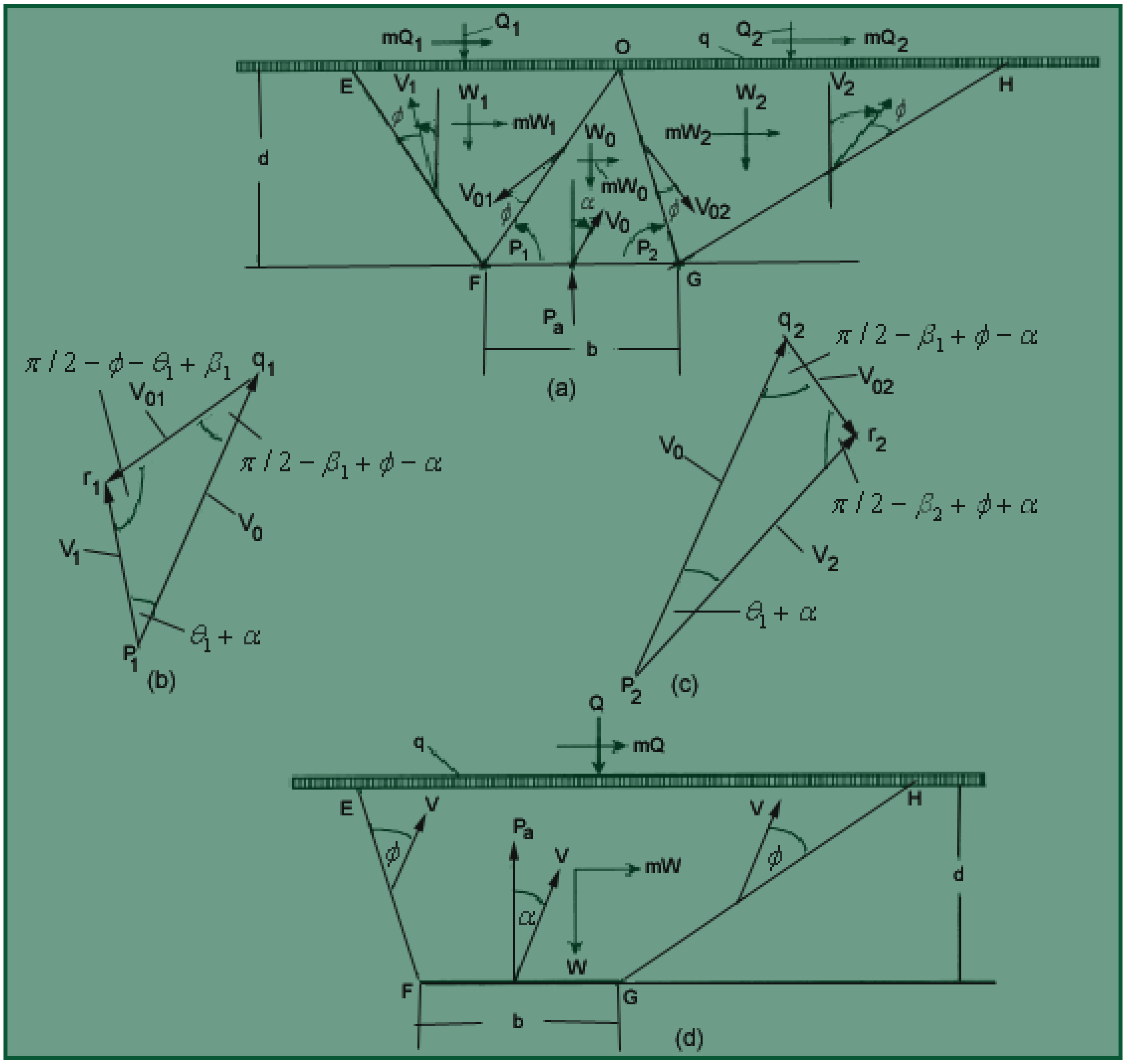

Fig. 2.35 Collapse mechanism

Results

It was found that, for any value of α, the magnitude of P_u becomes a minimum when the magnitudes of parameters 0_1 and 0_2 simultaneously reach their minimum admissible values $(i.e. 0_1 = -\alpha, 0_2 = \alpha)$. As a result, the magnitudes of V_1 and V_2. both become equal to zero, and the velocities V_1 and V_2. become simply equal to V_0. In other words, for the magnitude of P_u to be a minimum, no relative movement occurs among the blocks OEF, OFG and OGH: that is, the entire soil mass EFGH translates as a single rigid block with the same velocity as that of the anchor itself. The variable $\beta_1 (or\ \beta_2)$ does not affect the results since there is no relative movement among the three blocks. These findings justify that the magnitude of P_u can be simply calculated by considering a single rigid block collapse mechanism as shown in Fig. 1(d). This mechanism requires the minimisation of P_u with respect to only a single variable, namely α. Using this mechanism, the magnitude of P_u can be expressed in the following form:

$$P_u = \frac{V_0 (Q+W)(\cos\alpha - m \sin\alpha)}{V_0 \cos\alpha}$$

in which Q is total surcharge load on EH, and W is the weight of soil mass EFGH. Substituting the values of Q and W in equation (3), and simplifying:

$$P_u = \{(q+\gamma d)b + d(q+0.5\gamma d)[\tan(\phi-\alpha)+\tan(\phi+\alpha)]\} x (1 - m\tan\alpha)$$

Uplift equations

No seismic forces (m = 0). For m = 0, it was found that the minimum value of vertical uplift resistance occurs when the value of a is set equal to zero. The magnitude of average ultimate uplift pressure, P_u = P_u=b, will be given by the following equation:

$$p_u = qf_q + \gamma bf_\gamma \qquad \text{---------(a)}$$

$f_q = (1 + 2\lambda \tan \phi)$ where,

$$f_\gamma = \lambda(1 + \lambda \tan \phi)$$

$\lambda = b/d$ is a embedment ratio.

Presence of seismic forces (m $\neq$ 0)

Defining seismic factors e_q and e_g separately for the effects of surcharge and unit weight as the ratio of the pullout resistance for a component with a given value of m to the magnitude of the corresponding component of the pullout resistance with m = 0, equation (a) can be written in the following generalised fashion:

$$p_u = qf_q e_q + \gamma bf_\gamma e_\gamma$$

From the above equations,

$$e_q = \frac{\{1 + \lambda[\tan(\phi - \alpha_q) + \tan(\phi + \alpha_q)]\}(1 - m \tan \alpha_q)}{(1 + 2\lambda \tan \phi)}$$

$$e_\gamma = \frac{\{1 + 0.5\lambda[\tan(\phi - \alpha_\gamma) + \tan(\phi + \alpha_\gamma)]\}(1 - m \tan \alpha_\gamma)}{(1 + \lambda \tan \phi)}$$

In the above expressions α_q and α_g are the values of a for which the pullout resistance components separately on account of surcharge and unit weight become minimum.

Recap

In this section you have learnt the following

- Development of Uplift Capacity Theory
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- Application of the upper bound theorem of limit analysis
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Congratulations, you have finished Lecture 9. To view the next lecture select it from the left hand side menu of the page