Lateral Earth Pressure “At Rest”

10.1 Introduction

Under conditions where there is no lateral strain within the ground mass, the value of the lateral soil pressure is commonly called the *lateral earth pressure at rest* ($K_0$). Sometimes it is also defined as the *neutral lateral earth pressure* or the *lateral earth pressure at consolidated equilibrium*. The ratio of lateral to vertical earth pressure in this “no lateral strain” condition is termed the *coefficient of earth pressure at rest*, $K_0 = \frac{\sigma_3}{\sigma_1}$. Here $\sigma_3$ is the principal horizontal stress and $\sigma_1$ is the principal vertical stress, both in a soil mass having a cylindrical form of test apparatus.

According to the theory of elasticity, the coefficient of earth pressure at rest depends solely on the value of Poisson’s ratio $\nu$, i.e., $K_0 = \frac{\nu}{1 - \nu}$ (Tschebotarioff 1973). In some practical situations, e.g., the design of rigid unyielding walls such as gravity-type walls constructed on stiff foundation soils or basement walls backfilled with sand, $K_0$ will typically be equal to 0.4–0.5. In clays of normal sensitivity, $K_0$ is usually 0.5–0.6.

In granular material the magnitude of $K_0$ depends on the amount of frictional resistance mobilized at contact points between particles (Lambe and Whitman 1969). When a granular soil is loaded for the first time, the frictional forces at the contacts act in such a direction that $\sigma_3$ is less than $\sigma_1$, i.e., $K_0 < 1$. Jaky (1944), on the basis of his experiments, suggests that in granular soils the coefficient of lateral earth pressure at rest is best represented by $K_0 = 1 - \sin\phi$, where $\phi$ is the angle of internal friction.

By running a series of odometer tests in which not only the vertical effective stress but also the lateral effective stress was measured, Brooker and Ireland (1965) developed relationships for $K_0$ as an average for all normally consolidated clays: $K_0 = 0.95 - \sin\bar{\phi}$, where $\bar{\phi}$ is the effective angle of internal friction.
Monitoring of Soil-Structure Interaction

normally consolidated clays, Alpan (1967) recommends the empirical formula

\[ K_0 = 0.19 + 0.233 \log P_L, \]

where \( P_L \) is the plasticity index of clay, in percent. Since the early 1920, numerous experiments have been conducted by many workers in order to determine a credible value of \( K_0 \). Terzaghi (1920) found that \( K_0 \) does not depend on whether the soil is loose or dense. For sands he found \( K_0 = 0.42 \) and for clays \( K_0 = 0.7 \) to 0.75. Fedorov and Malyshew (1954) found that \( K_0 \) in loose soil has a greater value than in dense soils. Rowe (1954) found that the value of \( K_0 \) decreases with increases in vertical pressure \( \sigma_1 \). It should be noted that the investigators who tested soils in triaxial cells usually obtained larger values of \( K_0 \) than those who tested loose soils. For more information on \( K_0 \) values, the interested reader is referred to Clayton et al. (1993).

This author, with co-workers, undertook a large-scale model investigation to determine the value of \( K_0 \) in granular soils using the calibration chambers, soil pressure measuring systems described in the preceding chapters and some special proprietary devices.

10.2 Experimental Determination of \( K_0 \) in a Granular Soil Mass

Clean, dry quartz sands of various granulometric compositions were used in these experiments: two types of coarse granular sands, two kinds of fine granular sands, and a medium-sized granular sand. The principal stresses in the soil mass were determined using stiff flat pressure cells. Under test conditions it is possible to assume that \( \sigma_1 = \sigma_2 \) and \( \sigma_2 = \sigma_3 = \sigma_5 \). The first series of tests was carried out in order to test the basic criteria of the experiment methodology.

At this early stage the medium sand was tested in the smaller calibration device, the SCC (for details consult Chapter 3), by means of nine small-diameter \( (d = 26 \text{ mm}) \), stiff cells. The membrane deflection of these cells did not exceed \( 1/2000 \ d \) at maximum pressure. In these tests the value of \( K_0 \) was determined from two experiments as the \( \sigma_3/\sigma_1 \) ratio as the average of 18 measurements for dense \( (D_r > 67\%) \) and loose \( (D_r < 30\%) \) sands. In order to obtain a value of \( \sigma_1 \) in a mass of soil, the cells were arranged horizontally and loaded to pressures of 0.5 and 0.1 MPa. Then, in order to measure a \( \sigma_3 \) value, the same cells were rearranged vertically and loaded with similar loads. In all tests the soil’s density was carefully controlled to ensure comparable soil conditions in all relevant tests.

In order to account for effects of possible displacement of pressure cells in the mass of soil during load application, the cells were installed in three different configurations: “cross,” “circle” with pressure-bearing membranes facing the wall of the chamber, and “circle” with pressure-bearing membranes facing the chamber’s center. From these tests the value of \( K_0 \) was found to vary from 0.53 to 0.66 with larger values related to loose sand (Lazebnik 1967). It must be noted that in this preliminary series of tests the results were quite scattered. This apparently was attributed to effects of the small diameter and high stiffness of