

DESIGN AND PERFORMANCE OF OPEN EXCAVATIONS IN TAIPEI

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SYNOPSIS

Taipei's subway system will involve the construction of numerous underground facilities in deep open excavations. Significant experience has been gained in the past in this work and excavation performance has largely been satisfactory. However, the combination of deeper subway excavations and rising groundwater pressures in the Taipei basin requires a more sophisticated approach to defining soil strength for excavation support design than has been the case in the past. Critical to this process is the assessment of the degree to which the material in the passive zone swells and the time required to establish steady state seepage conditions. Effective stress paths based on field monitoring data provide insight into the factors which affect swelling, seepage and other related processes.

INTRODUCTION

Deep open excavations are common in Taipei for construction of basements for buildings which are supported on floating or semi-floating raft foundations. Internally braced diaphragm wall systems are traditionally used for support of open excavations. Few structures are piled except bridge and overpass structures.

At the present time, there is a significant increase in excavation work due to the construction of the Taipei Rapid Transit System (TRTS). The priority network of the TRTS includes 72 km of track with 67 stations. About half of the stations are to be constructed below grade in open excavations which will typically be 200 to 300 m long and 15 to 28 m deep. An additional 12.5 km of cut and cover work is planned for running track, crossovers, pedestrian shopping malls and the like. The remaining 18 km of underground track will be constructed in paired 5.6 m dia. bored tunnels. The deeper excavations will be supported by internally-braced diaphragm walls; the total length of diaphragm wall construction will be about 45 km.

This paper focuses on the design of support systems for open excavations for the TRTS project and in particular the definition of soil strength. Emphasis is placed on total and effective stress paths determined from field monitoring data which provide a better understanding of actual soil behaviour during excavation. Information is also provided in relation to the performance of excavations in Taipei and requirements for future excavations.

GROUND CONDITIONS

Taipei is located in a flat-lying tectonic basin which is infilled by up to 200 m of Quaternary sedimentary deposits. The underlying Tertiary bedrock forms the hills to the south, west and east of the basin; the northern hills are of volcanic origin. The Quaternary deposits which infill the basin, comprise the upper recent deposits of

the Sungshan formation overlying an extensive Chingmei gravel deposit and the lower hard sandy clay deposits of the Hsin Chuang formation. Most of the TRTS underground construction work will take place in the Sungshan and Chingmei formations.

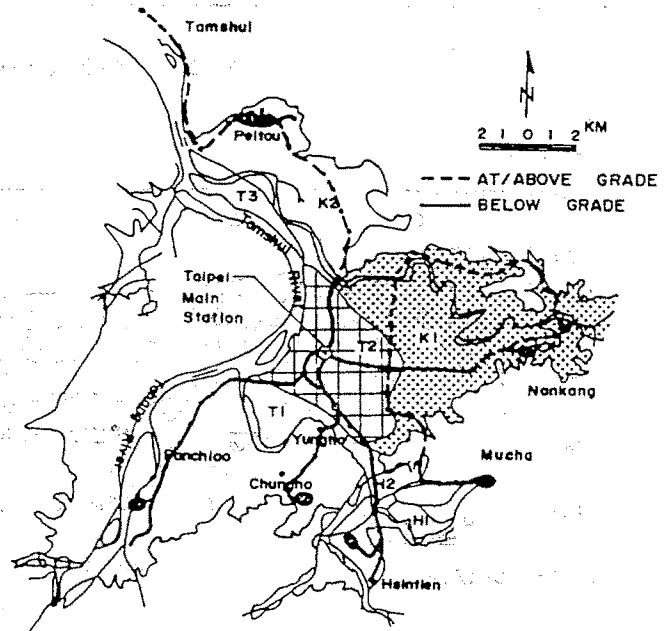


Fig. 1 Distribution of soils in the Taipei basin

Based on collation and synthesis of available data, Moh and Associates (1987) sub-divided the Sungshan deposits into areal zones based on depositional history and therefore material type. Most of the deeper excavations for the TRTS project will be located in the T2 and K1 zones as shown on Fig. 1. The stratigraphy in the T2 zone is relatively uniform, consisting of six alternating layers of clayey and sandy deposits. The uppermost layer VI together with layers IV and II consist of cohesive soils while layers V and III comprise silty sands. The lowermost layer I is variable and contains both clayey and sandy materials. In the K1 zone, sand layers are thin or absent and the profile is dominated by cohesive soils.

A typical east-west section along the TRTS Nankang Line (Fig. 2) indicates the gradual change in stratigraphy from the alternating

clayey/sandy layers in the T2 zone in the city centre area to the thick clay deposits in the K1 zone to the east. Fig. 3 shows a typical north-south section along the TRTS Hsintien Line. Typical T2 stratigraphy occurs in the northern section of this line; a promontory of sandstone and tuff abruptly delineates the T2 zone from the gravelly H2 area which occupies the southern section of the line.

The groundwater regime in the Taipei Basin has experienced complex changes during the past 30 years as a result of extensive pumping from the Chingmei formation to supply water for the city. Fig. 4 shows typical water pressure drawdown and associated settlement data for the downtown area. Pumping of groundwater from the Chingmei gravel began prior to 1960 and continued until the mid-1970's; by that time, a maximum drawdown of about 40 m and

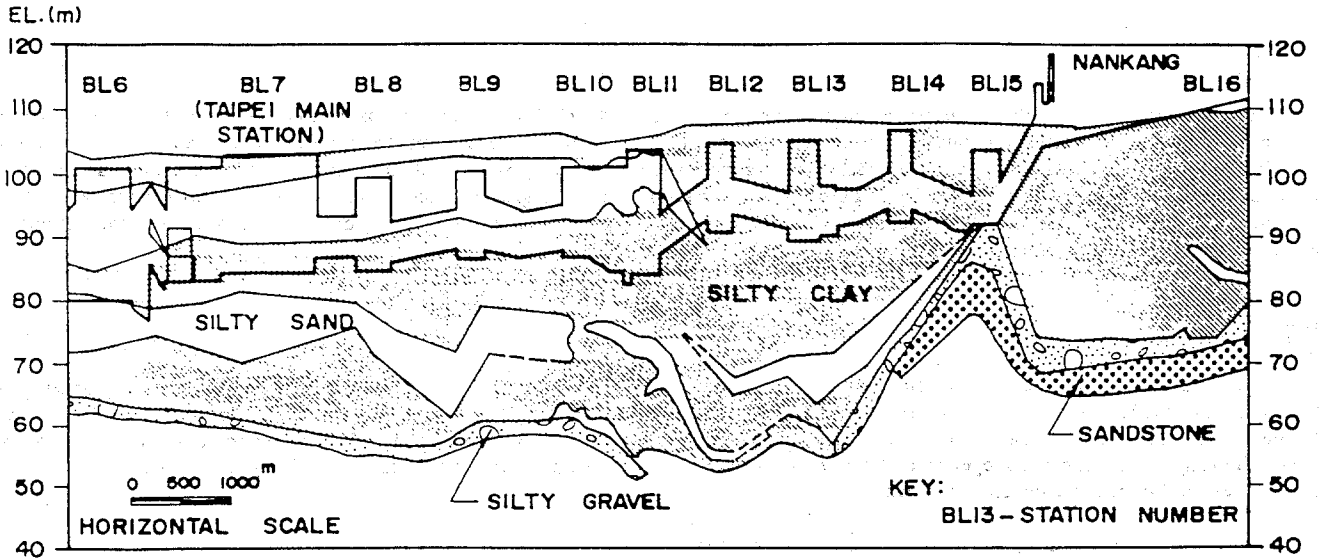


Fig. 2 Stratigraphy along the E-W Nankang Line

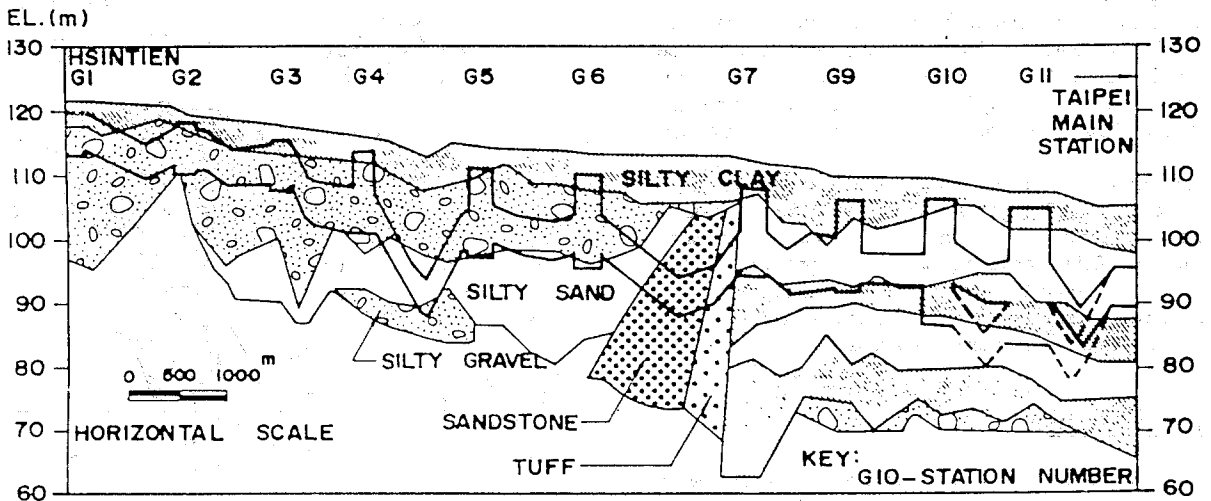


Fig. 3 Stratigraphy along the N-S Hsintien Line

associated settlement of 2 m had occurred. Drawdown and settlement were maximum in the city centre area and progressively decreased toward the edges of the basin. Since the mid-1970's, pumping has been restricted and there is a well-defined trend of recovery of water pressures in the gravel and the lower Sungshan deposits. As shown on Fig. 4, the ground surface settlement associated with pumping followed the drawdown curve closely. Since recovery of the water pressures began, the rate of settlement has slowed dramatically and has been close to zero throughout most of the basin in the recent years. In the central basin area, water pressures in the lower Sungshan deposits and Chingmei gravel are still sub-hydrostatic. Recovery of water pressures is continuing and is a major factor in the design and construction of underground works.

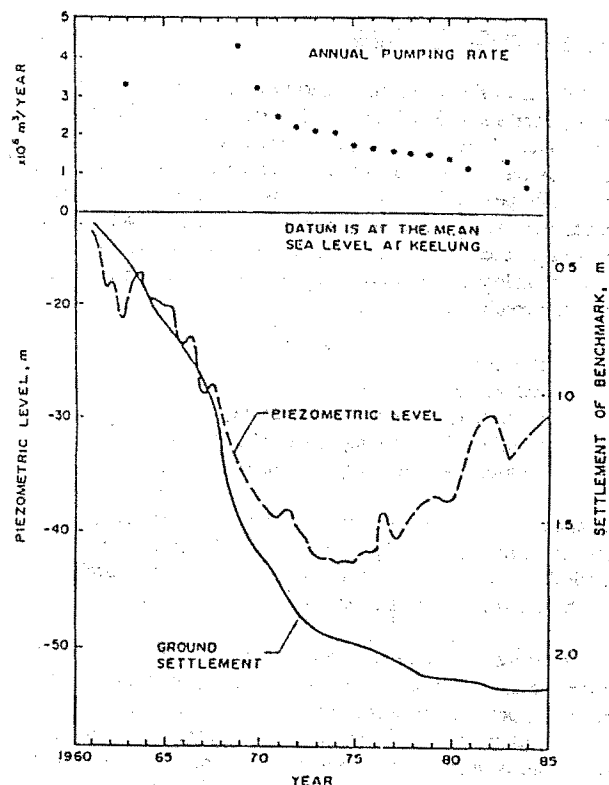


Fig. 4 Drawdown and settlement due to past pumping

SOIL PROPERTIES

The bases of excavations for many of the deeper TRTS structures will be underlain by layers III and IV of the Sungshan formation. Layer

III is sandy in nature and the strength properties of this material are relatively well understood. The major concern in the prediction of excavation performance is the behaviour of the layer IV materials. These are variable in terms of silt-size content, plasticity and water content. Further, layer IV acted as an aquitard and prevented water pressure reduction in the overlying Sungshan deposits while the water pressures were depressed in the underlying materials. Thus the degree of over-consolidation within layer IV varies both spatially across the basin and with depth.

The strength behaviour of the layer IV materials in the T2 zone and the cohesive soils in the K1 zone depends to a large extent on material gradation. This is illustrated by the results of CIU tests on samples with varying index properties obtained from different locations across the basin (Fig. 5). There is a consistent trend of decreasing strength ratio (S_u/σ'_c) with increasing water content and decreasing average grain size (i.e. as the material becomes finer). Materials with an initial water content >40% behave in the same manner as normal insensitive clays. As the material becomes coarser, the behaviour is more representative of a granular soil with dilation increasing progressively; as a result, the strength ratio also increases.

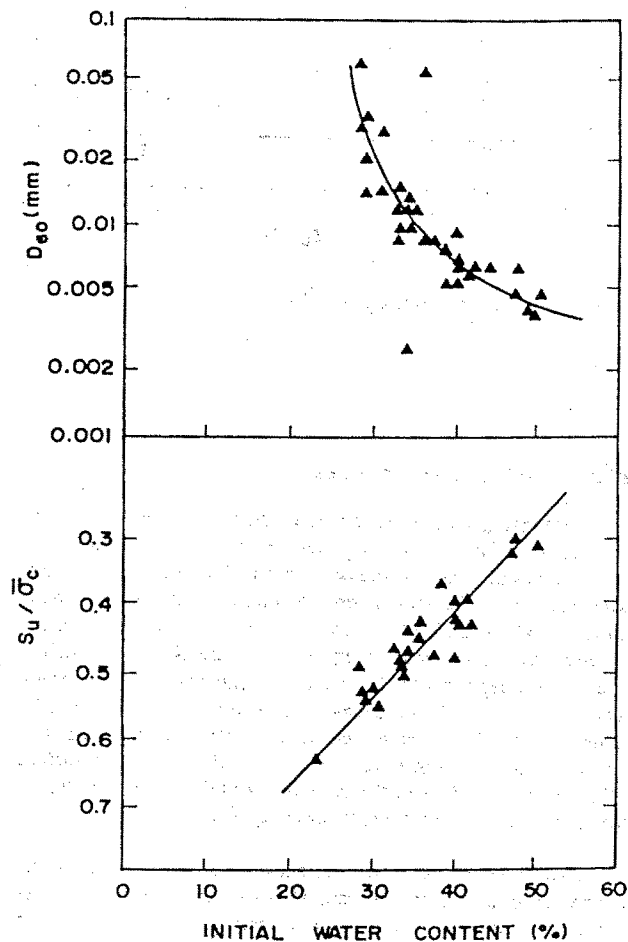


Fig. 5 Dependence of strength on material type

Definition of the undrained strength of the more clayey soils is based on CAU triaxial compression and extension tests. The results of these tests are shown on Fig. 6 and indicate significant strength anisotropy which is often associated with low plasticity soils. It is noted that the degree of undrained strength anisotropy increases as the plasticity decreases. This is the result of increased normalized strength in compression similar to that observed in the previously discussed CIU tests (Fig. 5) and reflects increasing dilation with decreasing plasticity. Based on preliminary studies using the SHANSEP approach proposed by Ladd and Foott (1974), the undrained strength of the clayey layer IV materials can be described as follows:

$$S_u = S \cdot \sigma'_{vo} \cdot OCR^m$$

where $S = 0.32$ for compression and 0.18 for extension. Based on published data for other soils, m is taken as 0.8 (Chin et al., 1989).

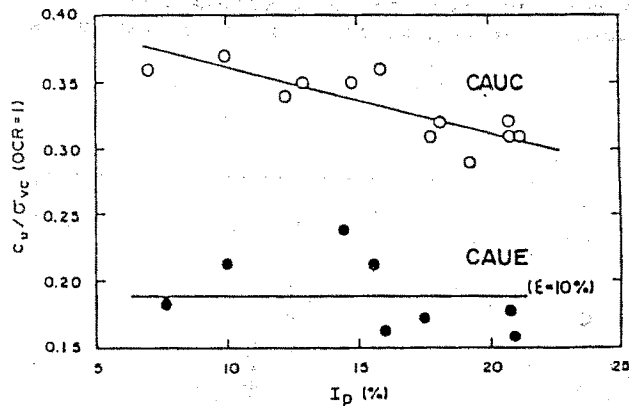


Fig. 6 Strength anisotropy of clays

EXCAVATION PERFORMANCE

Field monitoring data from previous diaphragm wall supported excavations in Taipei forms a valuable basis for evaluating wall performance and predicting the behaviour of TRTS excavations. For excavations in the T2 area, where conditions are more favourable, maximum wall deflections are typically equal to about 0.5% of the maximum excavation depth (Fig. 7a). In the K1 area which is underlain by deep deposits of soft clay, maximum wall deflections are greater at 0.5 to 0.75% of the maximum excavation depth. In Fig. 7b, maximum ground surface settlement is related to maximum wall deflection. In the more sandy T2 area, maximum ground surface settlement is about 50% of the maximum wall deflection. In the K1 area, this ratio is generally higher at about 75%.

It should be noted that the data shown on Fig. 7 were mainly obtained from past basement excavations which were typically less than 15 m deep. Care is required in extrapolating these data to predict the behaviour of TRTS excavations which will be deeper and will remain open for significantly longer periods. Further, the performance of excavations reflected by the data shown on Fig. 7 was largely achieved when the groundwater conditions in the basin were

favourable (i.e. in most cases the water pressure in the passive zone was below the base of the excavation). As a result of continuing water pressure recovery and the greater depth of the TRTS excavations, water pressures were well above the base of the TRTS excavations in all cases. Thus, unless effective groundwater control measures are implemented, much greater wall deflections and associated ground surface settlements can be expected. In the extreme, basal failure due to blow-in and/or piping could occur.

Finally, it is noted that much of the previous experience as summarized on Fig. 7 has been gained from excavations in the T2 area. A considerable portion of the TRTS works will be constructed in the clayey K1 zone where there is little experience in the construction of deep excavations and some failures have been reported (Kao et al., 1987).

DESIGN CONSIDERATIONS

One of the most significant factors to be considered in the design of diaphragm walls is the definition of soil strength, particularly in the passive zone. Since soil strength is controlled by effective stress, accurate assessment of porewater pressures throughout the entire construction period is required. This is an extremely difficult problem since the porewater pressure regime that develops in the vicinity of an excavation during construction will be the result of a number of independent processes. A full analysis would have to include the combined effects of:

- swelling,
- unloading,
- shear induced porewater pressure,
- transient flow leading to steady state flow,
- measures to control groundwater pressures.

Such analyses are not warranted given the uncertainties involved in terms of selection of soil parameters and stratigraphic model, numerical modelling, etc. Instead, a simplified approach which basically involves only an assessment of swelling, is adopted.

Given the more permeable nature of the layer IV soils in the T2 area and the presence of thick sandy layers, it is expected that complete swelling will occur in excavations in this area. As a result, effective stress strength parameters are used in wall design. Total stresses are based on the assumption that principal stress rotation does not occur and that fully active and passive states develop. Porewater pressures are predicted assuming that steady state conditions will develop with boundary conditions which reflect the imposed effect of groundwater control measures on the in situ water pressure regime. The effect of shear induced porewater pressures is ignored.

In the K1 area where excavations will be in deep deposits of soft clays, analyses indicate that little or no swelling should occur during the time that the excavations remain open. Design is based on undrained strength which implicitly includes shear induced porewater pressures. In these cases, the effects of groundwater control measures are not relevant for wall design; however, such measures are still required in some cases for control of blow-in.

As noted previously, the approach described above is highly simplified. Given the silty nature of the materials together with likely differences between assumed and actual stratigraphy, predicted porewater pressures will likely not be realized in many cases. Thus

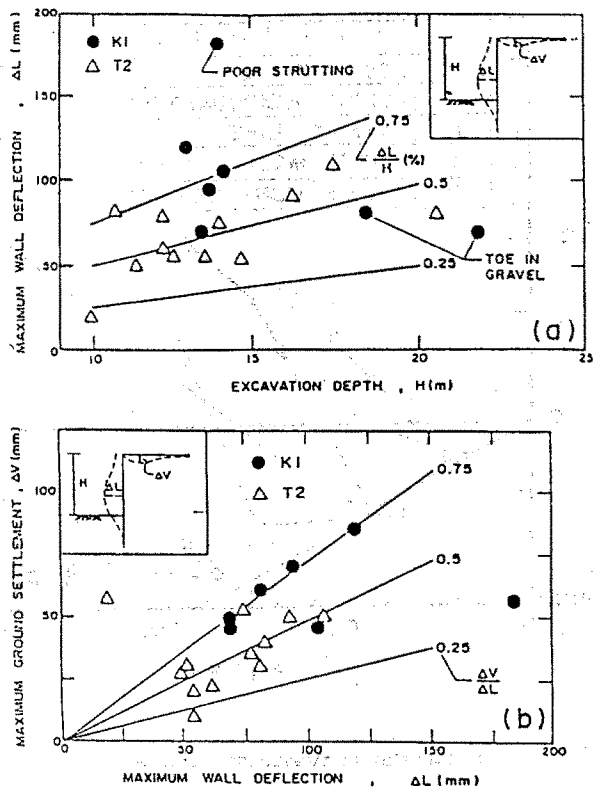


Fig. 7 Deformations associated with open excavations

monitoring of water pressures throughout the construction period will be essential. Because water pressures were depressed in the past, there has been little monitoring of water pressures within and adjacent to excavations in Taipei. However, some information is available from earth pressure/piezometer cells installed on the active and passive faces of diaphragm walls. Total and effective stress paths for soil elements adjacent to walls can be developed based on this data and provide insight into actual soil behaviour.

Idealized stress paths for typical soil elements in the active and passive zones have been presented by Lamb (1967) and are reproduced on Fig. 8. For element A located on the active side of the wall, the horizontal stress decreases while the vertical stress remains constant during excavation. At the end of excavation, assuming that the soil does not reach the limiting active state, the water pressure is smaller than the initial pressure (U_s) but larger than the pressure at steady state seepage (U_{ss}). Subsequently, the water pressure will decrease to U_{ss} and the effective state of stress at element A will move from A_1 to A_{ss} . On the passive side, soil element B experiences a decrease in vertical stress. On completion of the excavation, assuming that a limiting passive state has not been reached, the water pressure is negative. The pressure increases as swelling occurs and the effective stress point B_1 moves to B_{ss} as water pressures associated with steady state seepage conditions are established.

It should be appreciated that the stress paths shown on Fig. 8 are simplified and that actual stress path behaviour will vary. Typical stress paths for both clayey and sandy areas based on actual earth pressure/piezometer cell data are discussed below.

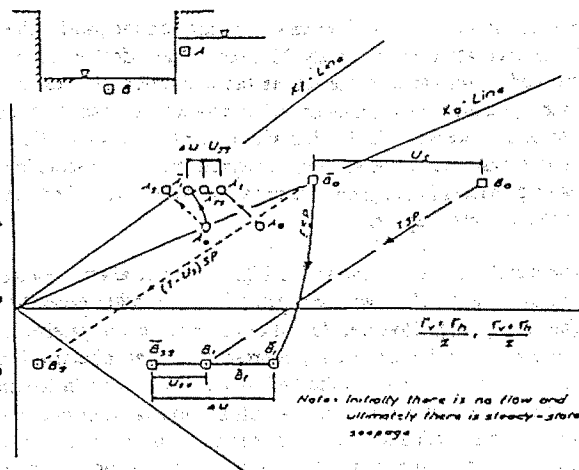


Fig. 8 Idealized stress paths for excavations

Stress paths have been developed for a 12 m deep excavation supported by internally-braced diaphragm walls (Moh and Chin, 1991). The site is located in the K1 area and is underlain by deep silty clay deposits. Stress paths at a depth of 18 m below ground surface on both active and passive sides are shown on Fig. 9. For the active side, information from the initial condition until 6 months following completion of the excavation is included. However, for the passive side, only data up to the end of the excavation is available. On the active side, the stress paths are as would be expected with decreasing horizontal stress and pore pressure during excavation. The effective stress path reaches an apparent active state at the end of the final excavation stage but a further decrease in pore pressure occurs as indicated by the horizontal section of the effective stress path. On the passive side, the stress paths indicate a decrease in both horizontal and vertical stresses during excavation. Further, swelling appears to occur as the excavation proceeds since the effective stress path does not exhibit a significant negative water pressure.

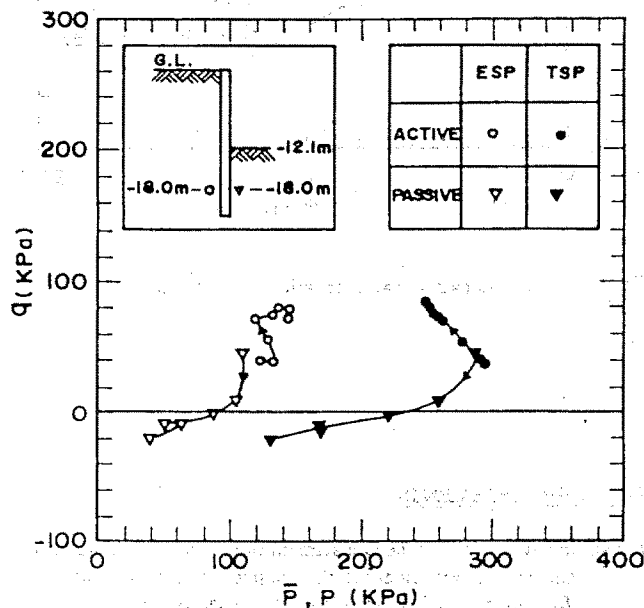


Fig. 9 Stress paths for an excavation in clay

Fig. 10 shows total and effective stress paths on the passive side of a 12 m excavation in the sandy T2 area. The effective stress path proceeds downwards to the left as the excavation progresses. The water pressure on completion of excavation is consistent with a hydrostatic pressure distribution with respect to the excavation base. In this case, complete swelling occurred during construction and water pressures in the passive zone can be readily described in terms of steady state seepage.

The simple behaviour shown on Fig. 10 is not always realized as indicated by the effective stress paths for some other excavations in Taipei (Fig. 11). However, for all of these cases, the passive failure line appears to be consistently represented by an effective friction angle of 32 degs. which is the typical value measured in laboratory tests on the sands and silts, and a wall friction value of one third of the effective friction angle. A further point of interest is that the initial effective stress state at the start of excavation for the cases shown on Fig. 11, can be represented by a horizontal to vertical effective stress ratio of about 0.5. It should be appreciated that this is not the geostatic stress state (i.e., K_0 ratio) but instead represents the stress state adjacent to the wall after straining due to trench installation. The actual K_0 value can be expected to be higher.

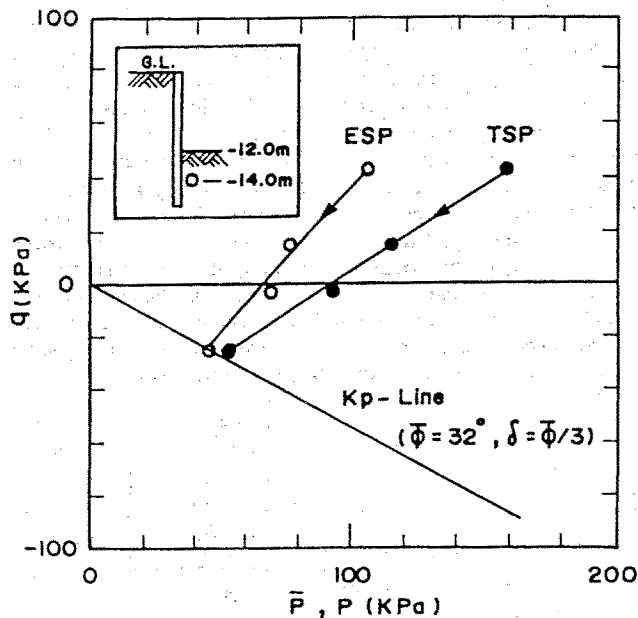


Fig. 10 Stress path for an excavation in silty sand

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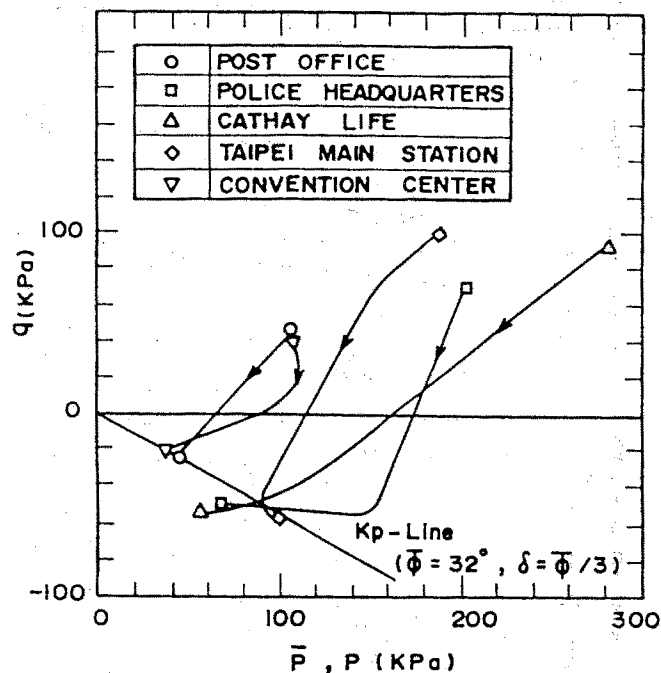


Fig. 11 Stress paths for excavations in silty sands

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