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# Performance of Diaphragm Walls in Deep Foundation Excavations

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SYNOPSIS Two case studies of using diaphragm wall as earth retaining structure for deep excavation are reported. Case A concerns excavation down to a depth of 14.7 m in alternate layers of soft silty clay and silty sand. Diaphragm walls of 70 cm thick and 22 m deep were installed into a dense silty sand stratum. In Case B, diaphragm walls of 60 cm thick were constructed in very soft clayey soil without penetration into any firm stratum. In both cases, instrumentations including piezometers, inclinometers, earth pressure cells, reinforcing bar transducers, heave stakes, settlement points and strut strain gauges were installed and monitored throughout the construction. Comparison of predictions based on simplified theory and empirical relationships were made with the actual performance behavior of the diaphragm wall and the subsoils. It was found that elastic mode of soil-wall system can reasonably predict the behavior of diaphragm wall. In sandy soils, arching effect has significant effect on the magnitude and distribution of earth pressure on the wall. With the assistance of instrumentation monitoring during excavation work, factor of safety against base failure as low as 1.05 was used in the design.


## INTRODUCTION

Diaphragm wall or slurry trench has been used in civil engineering construction since the forties. In the early days, this type of construction was primarily used for cut off wall under dams. With the improvement of construction techniques and machineries, the quality and strength of concrete diaphragm walls have greatly improved. The application of this type of construction method in deep excavation work as both temperary and permanent retaining structure became gradually accepted. The first use of diaphragm wall in basement construction in Taiwan was the International Commercial Bank of China Building which was constructed by the Ret-Ser Engineering Agency in 1971. In recent years, due to the rapid expansion of economic development on the island, maximum utilization of the expensive urban land area is one of the most important consideration for both the public sector and the private investors. Increase of usable underground spaces becomes one of the obvious solution which has greatly contributed to the advancement of underground construction technology in Taiwan. In the last few years, construction of diaphragm walls for buildings has exceeded 100,000 sq.m. in wall area per year.

Design of diaphragm walls in the past has been relatively on the conservative side due to lack of understanding of the soil-structure interaction and soil behavior during and after construction of the wall. Development of instrumentation monitoring system for excavation control has greatly improved the understanding of soil behavior and enables engineers to adopt a less conservative approach in designing this type of construction system. This has resulted in significant savings in the construction cost.

This paper describes two case records on the design and construction of diaphragm walls in the soft sedimentary deposits of Taipei City in Taiwan, ROC. The behavior of the diaphragm walls was analyzed by using finite element method. Predictions are compared with actual measurement records as obtained from instrumentations installed in the wall.

## DESCRIPTIONS OF SITE CONDITIONS

### Case A

A highrise office complex was constructed by the Taiwan Power Company (TPC) in the southeastern part of Taipei City to accommodate its main office. The site is about 8,600 sq.m. in area and is bounded by a major 6 lane street (Roosevelt Road) on the north. The complex comprises of a 26-story tower block and a lowrise annex building. The two buildings are connected by a three level basement structure. The floor area of the tower block is about 1,990 sq.m. per floor and the basement is about 5,400 sq.m. per floor. The bottom of the basement level is located at 14.7 m below the existing ground surface. Moh and Associates, Inc. (MAA, 1978) was engaged by the TPC as the geotechnical consultant being responsible for site investigation, recommendation on foundation system and substructure construction method. During the construction stage, MAA served as consultant to the constructor BES Engineering Corporation for instrumentation installation, monitoring and interpretation for construction safety control.

The subsoils underlying the site are alluvial deposits consisting of alternative layers of silty clay and silty sand. On the basis of the subsurface exploration data, in situ and laboratory test results, the soil profile at the site can be divided into nine strata. The distribution of the soil over the entire site can be considered to be fairly uniform. Table I presents a simplified soil profile along with the soil parameters selected for geotechnical analysis and design.

TABLE I. Soil Profile and Design Parameters of the TPC Building Site

| Depth m | Description | N Value | $\gamma_t$, KN/m³ | W, % | $\bar{c}$, KN/m² | $\bar{\phi}$, degree | c, KN/m² | $\phi$, degree |
|---|---|---|---|---|---|---|---|---|
| 7 | Silty Clay | 3 - 8 | 18.9 | 29-35 | 59.8 | 9.6 | 69.6 | 6.5 |
| 10 | Silty Sand | 2 - 26 | 19.7 | 18-31 | 0 | 31 | – | – |
| 17 | | | | | 0 | 35 | | |
| 20.5 | Silty Clay | 4 - 14 | 18.8 | 28-35 | 30.4 | 26 | 48.1 | 14 |
| 30 | Silty Sand | 8 - 36 | 19.8 | 20-27 | 0 | 35 | – | – |
| 33 | Silty Clay | 10 - 20 | 19.4 | 23-29 | 0 | 34 | 62.8 | 21 |
| 38 | Silty Sand | 18 - 48 | 19.8 | 15-22 | 0 | 41.7 | – | – |
| 45 | Gravel | – | 21.8 | 12 | – | – | – | – |
| 47.5 | Clay | 20 - 27 | 19.7 | 22 - 28 | 33.3 | 29.2 | 19.6 | 6.7 |
| 50 | Gravel | – | – | – | – | – | – | – |

Since the site is located in a district with a number of multistory buildings in the vicinity, driving of sheet piles is undesirable. Diaphragm wall system was adopted as the retaining structure for the basement construction. This system has the additional advantage of serving as both temporary and permanent retaining structure which resulted in significant savings to the construction cost. In order to maintain stability for the deep excavation, it was found that the diaphragm wall should have a minimum penetration into the subsoil of 7.3 m below the bottom of excavation. In other words, the diaphragm wall should have a minimum depth of 22 m. The earth pressure distribution diagram recommended by PECK (1969) was used for design of the internal bracing system. Four levels of steel H struts were installed during construction. A plan of the building site is shown in Fig. 1

## Case B

In 1980 the Taiwan Power Company planned to construct an Electricity Distribution Center for the Taipei Northern Area (TNAEDC) at Section 5 of the Chung Shan North Road. The proposed building is a L-shaped R.C. structure with 3 stories above ground and 2 basement levels which occupies a site area of about 2,540 sq.m. The bottom of the foundation mat is located at a depth of 7.8 m below the existing ground surface. In the original design, steel sheet piles were used retaining structure for excavation. Due to the extreme soft condition of the subsoil, a major failure has occurred during excavation. The excavated area was subsequently refilled. It was decided to move the construction to the area immediately north of the previous site. MAA (1981) was invited by the constructor BES Engineering Corporation to carry out a thorough investigation of the subsoil conditions and to design the earth retaining system. During construction, instrumentations were installed to monitor the safety.

TABLE II. Soil Profile and Design Parameters of the TNAEDC Building Site

| Depth m | Description | N Value | $\gamma_t$, KN/m³ | W, % | $\bar{c}$, KN/m² | $\bar{\phi}$, degree | c, KN/m² | $\phi$, degree |
|---|---|---|---|---|---|---|---|---|
| 1-2.6 | Backfill | 2 - 6 | – | 28 | – | – | – | – |
| 5-11 | Silty Clay | 1 - 4 | 17.7 | 35-45 | 0 | 32.7 | 16.7 | 17.9 |
| 9-12.3 | Silty Sand | 5 - 6 | 18.1 | 30-34 | 0 | 31.5 | – | – |
| 23.8-40 | Silty Clay | 1 - 2 | 17.3 | 43-50 | 0 | 27.7 | 17.7 | 12.0 |
| Below 40 | Weathered Sandstone With Shale | 50 | – | – | – | – | – | – |

The subsoil profile at the site is comprised of 5 substrata. From the ground surface downward, they are surface fill, clayey silt, silty find sand, silty clay and weathered sandstone and shale. The fourth silty clay layer has a thickness varying from 15 m up to 30 m over the site. This soil has very high moisture content close to the liquid limit, is uniformly soft and possesses high compressibility. This stratum appeared to be the main soil layer which controls the geotechnical problem at the site. Table II presents a simplified soil profile along with their important properties.

Located approximately 4 m north of the present site is an existing transmission station. The ground surface of that station is about 1.20 m higher than the site under consideration. In order to reduce the effect of this overburden pressure on settlement of surrounding area and to prevent excessive lateral deformation of retaining structure of the new construction, a continuous row of 40 cm diameter cement-sand grout bore piles extending to a depth of 16 m was installed on the southern side of the transmission station. For construction of the proposed basement excavation, four different types of retaining system were evaluated. Figure 3 shows the factor of safety against base heave failure for different depths of excavation by assuming that the surcharge load was 1 ton per sq.m. (9.81 KN per sq.m.) and the depth of retaining structure was 16 m. In order to achieve a minimum factor of safety of 1.05 and to keep the construction cost as low as possible, the site was divided into two zones as shown in Fig. 2. For Zone A, YSP Type III sheet piles were driven, and the excavation work was started only after removal of about 70 cm thick layer of the top fill soil surrounding the excavation area. For the northern part of the site i.e. Zone B, diaphragm wall of 60 cm thick

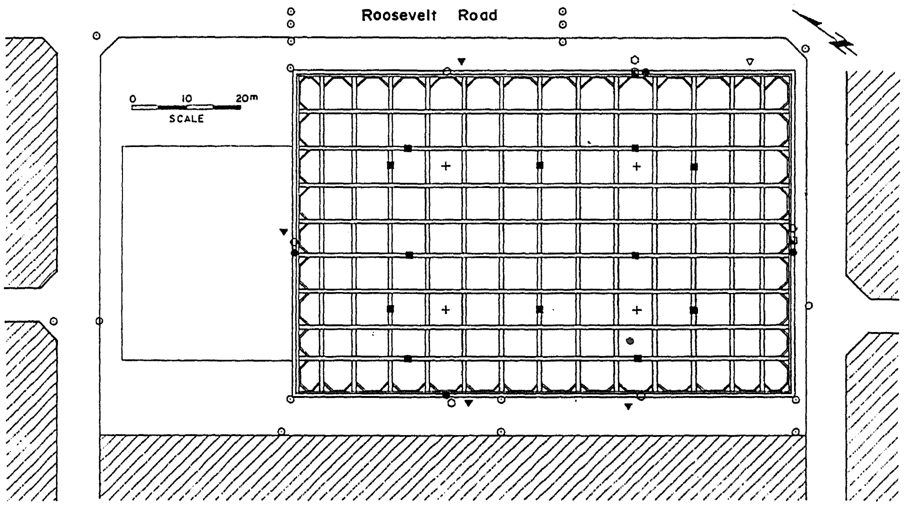



Fig. 1. Site Plan and Instrumentation Layout of Case A

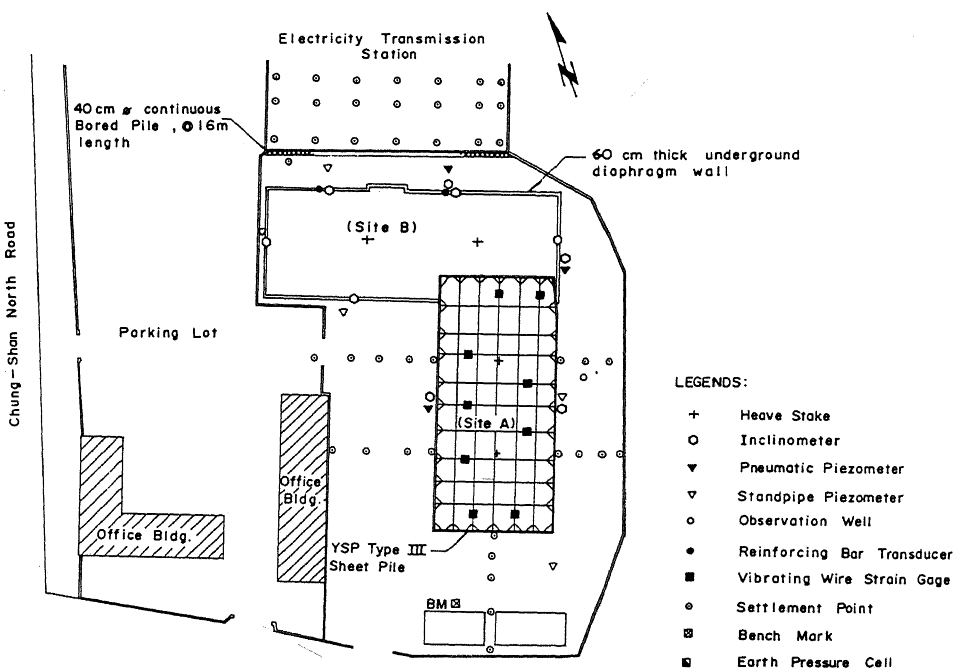



Fig. 2. Site Plan and Instrumentation Layout of Case B

was installed as the retaining structure. It was further specified that the surcharge load along the northern boundary within a 4 m wide strip was maintained at less than 0.5 tons per sq.m. (4.9 KN per sq.m.) in order to ensure construction safety. Internal bracing system consists of steel H struts was used during the excavation work.

INSTRUMENTATIONS

In view of the large area and extremely soft soil formation involved in the strutted excavations for the two projects reported above, seven types of monitoring instruments were installed. These include piezometers, inclinometers, earth pressure cells, and reinforcing bar transducers in the diaphragm wall, piezometers and settlement points within and around the excavated area, heave stakes in the excavation area, and vibrating wire strain gauges on the steel struts. The locations of the various instruments except the strain gauges for Case B are shown in Figs. 1 and 2 for the TPC and TNAEDC projects, respectively.

PREDICTION AND COMPARISON WITH MEASUREMENTS

Method of Analysis

For analyzing stress and deformation of diaphragm walls under the action of earth pressures, the method proposed by JAMES and JACK (1975) on the basis of elastic theory has been widely adopted for its relative simplicity. The main assumptions of this method may in fact deviate considerably from the actual behavior of the retaining structure. In applying this method, selection of the magnitudes of the active force and the coefficient of subgrade reaction $k_s$ often becomes a state-of-the-art for the designer. For the two cases discussed in this paper, the above mentioned elastic model was used for prediction of the wall behavior. In the analysis, the soil-wall system was idealized as illustrated in Fig. 4. The diaphragm wall was divided into a number of beam element, the earth pressure acting on the wall was assumed to be in a trapezoidal distribution, and the soil reaction was repretented by a series of springs.

Before excavation, the earth pressure is in an at-rest condition. This pressure tends to decrease as the excavation proceeds due to inward movement of the retaining structure. After completion of the substructure, the earth pressure increases again. For the analysis it was therefore assumed that the earth pressure acting on the diaphragm wall was equal to the average value of the at-rest pressure and active pressure.

The modulus of subgrade reaction $k_s$ for soils below the excavation surface can be estimated from the following equation:

$$k_s = C\ F\ q_a$$

where 'F', is the factor of safety, and '$q_a$',

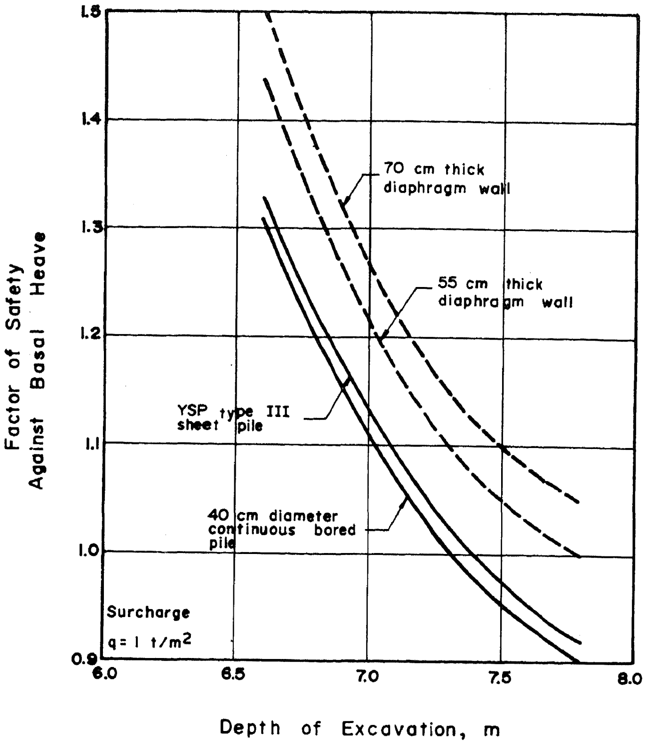



Fig. 3. Factor of Safety Against Base Failure of Different Types of Retaining Systems for the TNAEDC Strutted Excavation

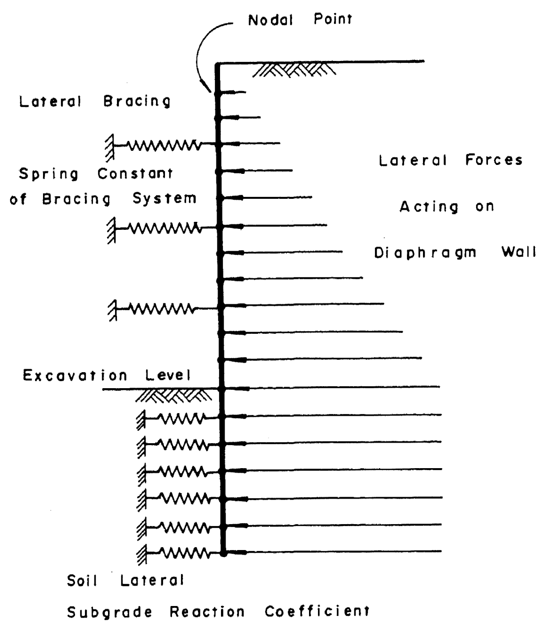



Fig. 4. Idealization of Soil-Wall System

is the allowable bearing capacity. BOWLES (1982) suggested a value of 40 for the factor C. However, he pointed out that these values will give good values of bending and other items of interest but deflections may be substantially in error. Based on past experiences in Taipei, C values of 6 to 10 were considered for the two cases reported in this paper.

## Deformation of Diaphragm Wall and Ground Settlement

It is a well known fact that earth retaining structure will undergo both lateral deformation and angular distortion during excavation. From the deformation characteristics it is possible to determine the degree of safety and to evaluate the effect of excavation on the environment and structures situated in the surrounding area.

For the TPC project, 70 cm thick reinforced concrete diaphragm wall was designed as the earth retaining structure. Figure 5 presents the deflection curves of the diaphragm wall

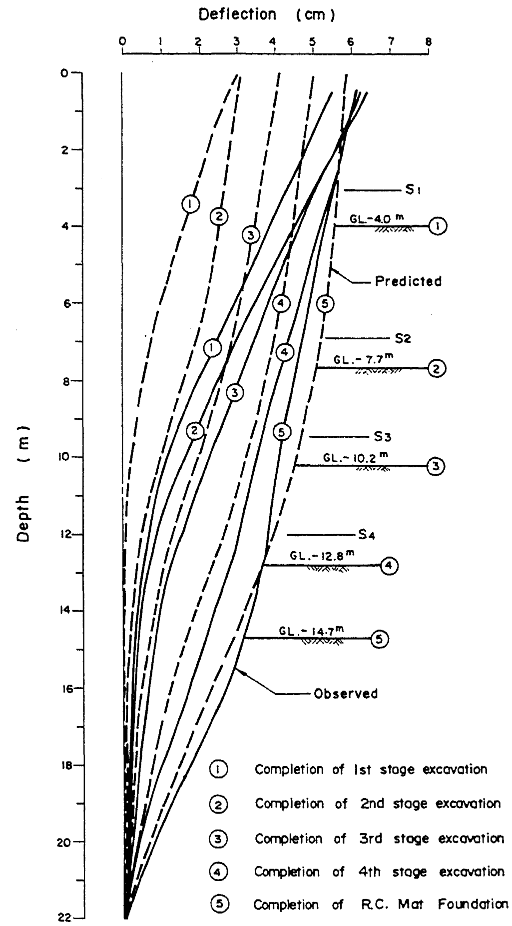



Fig. 5. Deflection Curves of Diaphragm Wall of the TPC Site

after each stage of excavation as measured by inclinometer casing installed inside the wall. Comparison of the curves shown in the figure clearly indicate that the actual measured curvatures of the diaphragm wall were always larger than the predicted value based on a simplified soil-structure model. The differences are particularly large at shallow depths and during early stages of excavation. It is significant to note that after the first stage of excavation, the top of the diaphragm wall moved more than 50 mm which was about 70 per cent of the total movement after completion of the excavation work. Due to this large lateral movement, considerable ground settlement has occurred in the area surrounding the excavation. Cracks were observed in streets and buildings located about 8 m southwest of the site. The amount of lateral displacement of the wall increased with excavation and the rate of increase was larger near the bottom of excavation. It was also found that the lateral movement of the long side of the diaphragm wall was more than that of the short side. Settlement records of ground surface surrounding the excavation indicate that the rate of ground settlement decreased significantly after casting of the foundation slab. The measured settlements are compared with predicted values (according to PECK 1969) in Fig. 6. The maximum settlement was equivalent to about 0.3 per cent of the depth of excavation.

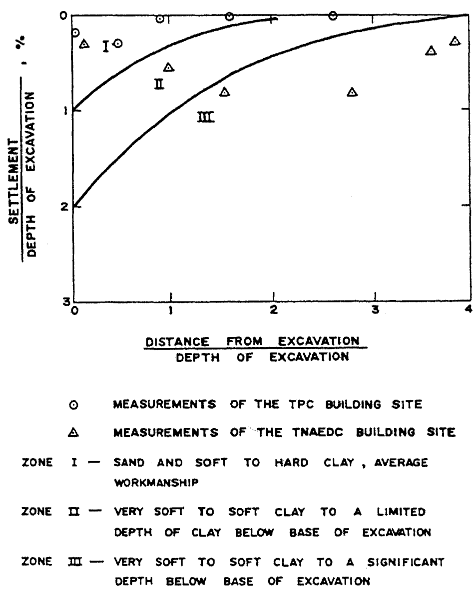



Fig. 6. Comparison of Predicted Ground Settlement with Measured Values

As described in previous section, two types of retaining structure were used for the TNAEDC project. YSP III sheet piles were used as temporary retaining system in Zone A whilst 60 cm thick diaphragm wall was used in Zone B. Both types of retaining structure penetrated into the subsoil to a depth of 16 m. That means the retaining systems have not penetrated into the underlying hard stratum. Figure 7 presents the deflection curves of the diaphragm wall. During the first three stages of excavation, the diaphragm wall moved more or less parallel to the excavation surface. In the last two stages of excavation, in addition to inward parallel movement, the lower part of the wall has yielded inward more than the upper part due to reduction of the penetration depth of the wall. The preidcted deflection curves were obtained by multiplying an empirical factor of 1.5 to the theoretically calculated values using finite element method.

Deflection curves of the steel sheet piles in Zone A of the project site are shown in Fig. 8. Due to extreme softness of the subsoil,

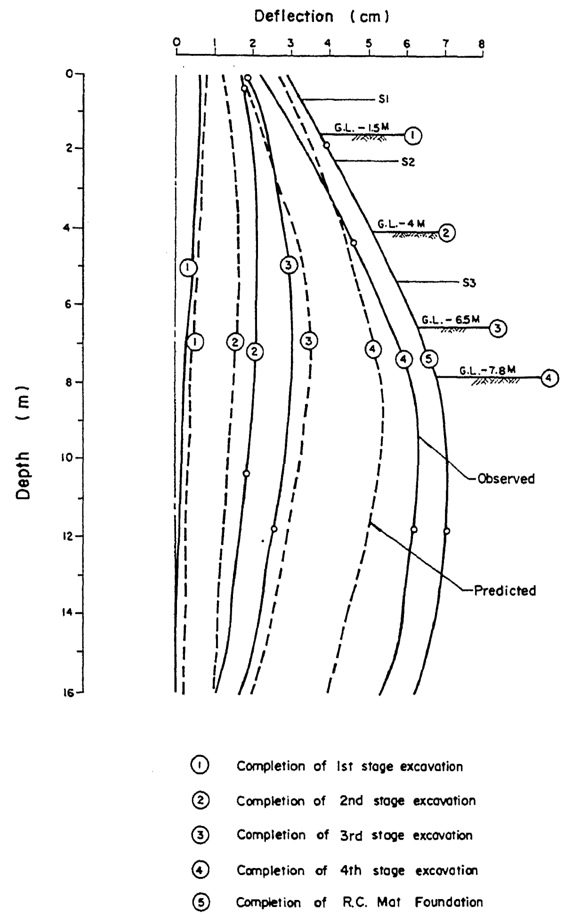



Fig. 7. Deflection Curves of Diaphragm Wall at the TNAEDC Site

maximum deflection of the steel sheet pile after the 4th stage of excavation developed near the bottom of the excavation surface and was increasing at the rate of 20 mm per day. This increase in deflection was arrested soon after the pouring of a 20 cm thick layer of plain concrete at the base. At the depth of 9 m below the ground surface the sheet pile has developed very large distortion, with the ratio of maximum deflection to distance reaching a value as high as 1/105. This large distortion indicates that a plastic hinge has developed in the sheet pile.

For this project, since dewatering was not allowed prior to or during excavation, settlement of the surrounding area can only be attributed to loss of ground caused by lateral displacement of the earth retaining system. The most important structure in the vicinity of the excavation work was the Transmission Station located on the north side of the site. The effect of the excavation of Zone A has caused only minor settlement in the Transmission Station area with a maximum value of only 0.55 cm, even though the steel sheet piles had undergone significant amount of lateral movement. On the other hand, the distance between the excavation of Zone B and the outer wall of the Transmission Station was only 9 m. The maximum measured settlement

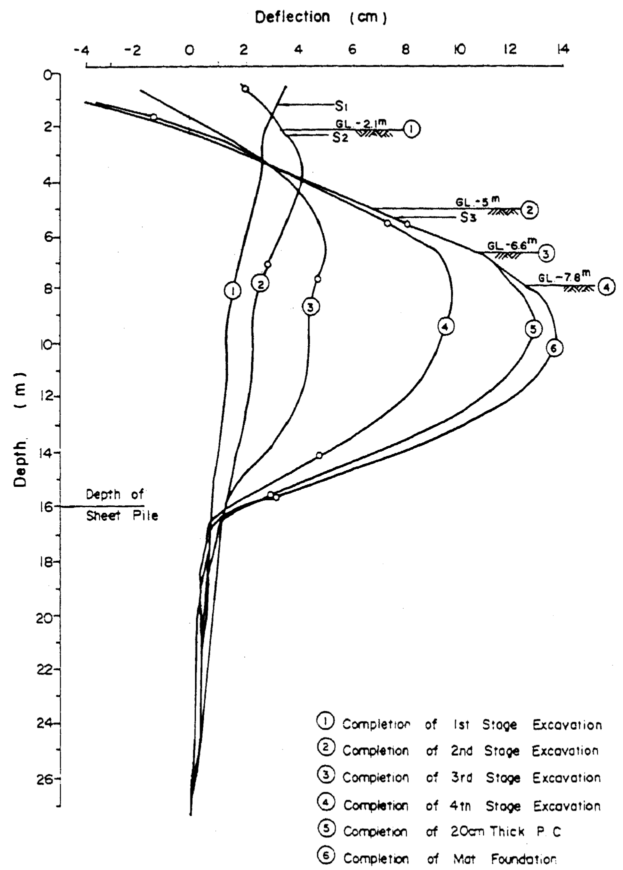



Fig. 8. Deflection Curves of Steel Sheet Pile Wall at the TNAEDC Site

reached 6.5 cm. However, the differential settlement was still within the allowable limit for continuous steel frame structure of the Transmission Station with angular distortion of less than 1/500. From Fig. 6 it can be seen that the extent of influence of excavation on settlement is equal to about 4D where D is the depth of excavation, whilst the maximum settlement would occur within a distance of 1.5 D to 3.0 D. From all the measured data, it could be concluded that the installation of a row of continuous bored piles immediately adjacent to the Transmission Station had developed its function in reducing the potential differential settlement caused by excavation.

## Heave of Excavation Surface

Heave stakes were installed at both sites for checking the magnitude and rate of heave of the bottom of excavation. Although heave stake is the most simple type of instrument employed in the project, it is generally most vulnerable to damage by construction equipment. At the TPC site, due to this problem, monitoring records of heave measurement were not complete. It was estimated from the records that the total heave ranged from 50 to 60 mm which was slightly less than the predicted value of 67 mm. At the TNAEDC site, it was estimated that the total heave would be about 74 mm. The measured values in Zone A were 70 mm and 75 mm and that in Zone B were 69 mm and 94 mm. The high value of heave in Zone B was most likely caused by a heavy typhoon rain when the excavation reached the depth of 7 m. For a period of several days, more than 3 m of water was accumulated in the excavation which had probably caused softening of the subsoil. As illustrated in Fig. 9, the actual rate of heave was less than that predicted, which was more or less uniform, when the excavation depth was less than 6 m. For excavation, below the depth of 6 m, the measured rate of heave exceeds that predicted. This tends to indicate that the excavation was approaching the critical depth.

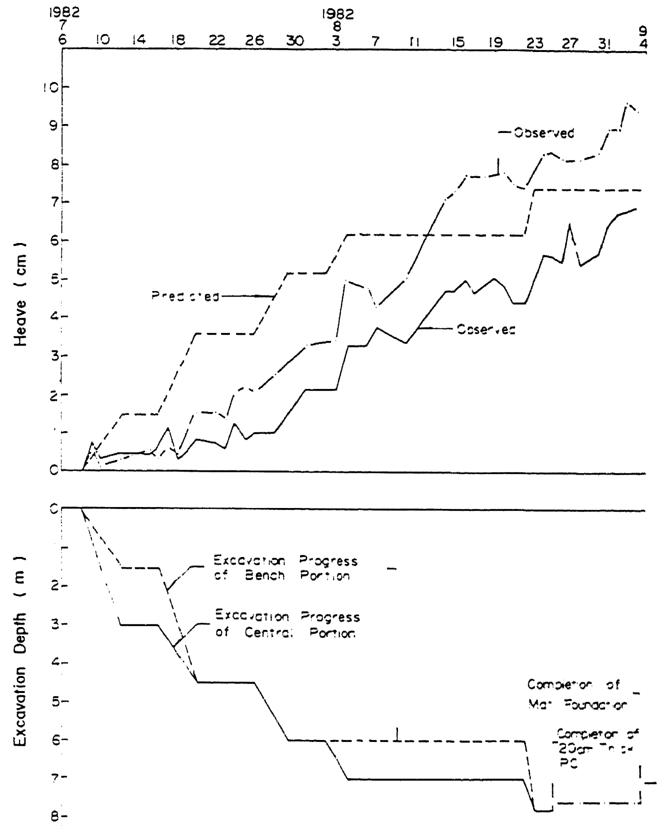
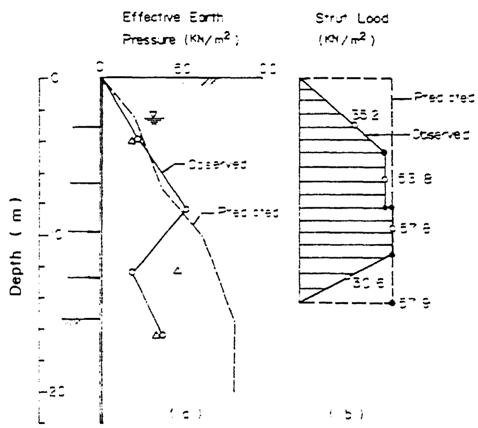



Fig. 9. Comparison of Predicted Heaves with Measuremed Values at TNAEDC Site

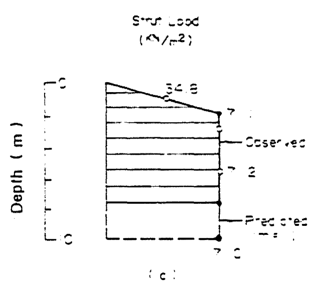
## Earth Pressures and Strut Loads

Many studies have been carried out in regard to the distribution of earth pressures acting on earth retaining structures. A number of empirical approaches based on comparison of field measured data and theoretical analysis has been proposed. Among them, PECK's (1969) apparent earth pressure diagram has been most widely used.

At the TPC site, oil-filled earth pressure cells were installed at four depths, i.e. at 4, 8, 12 and 16 m, in two of the diaphragm wall panels to monitor the earth pressures. The actual distribution of earth pressure acting on a retaining system is closely related to the deformation of the system. TERZAGHI (1936) suggested that in sandy soils, the earth pressure distribution would be in arching active condition when the midheight of the retaining structure moved outward about 0.05 per cent of the wall height. The pressure distribution curve will not be in a triangular form but depends upon the amount of yield and tilt of the wall. The measured values of deformation at midheight of the diaphragm wall at the TPC site at all stages of excavation were more than 30 mm which exceeded the 0.05 per cent considerably. Due to the arching action of the sand layer, the measured earth pressures above 8 m depth where the first sand layer existed were close to active condition whilst the pressures below 8 m depth were much lower than the active pressure. Figure 10(a) compares the measured earth




TPC Building Site




TNAEDC Building Site

Fig. 10. Earth Pressure Diagrams

pressure with the predicted active pressure at the last stage of excavation. Vibrating wire strain gauges were installed on the web of steel H struts for monitoring of strut loads during excavation. The gauges were installed in pairs in order to check eccentricity of strut loads. Each strut was preloaded immediately after installation to a load equal to approximately 15 to 20 per cent of the estimated strut load. A comparison of the measured load with the predicted value is shown in Fig. 10(b). The lower load monitored near the excavation surface is likely due to the arching effect of the soil.

Similar internal bracing system was used at the TNAEDC site, except that preloading of the strut was in the range of 22 to 57 per cent of the estimated apparent value. Comparison of the measured and estimated strut load in Zone B at the final stage of excavation is shown in Fig. 10(c). The estimated values based on TERZAGHI and PECK (1967) for cohesive soils were almost identical to the measured loads. In Zone A, steel sheet piles were used with same system of internal bracing. The measured strut loads as comparied with the corresponding strut in Zone B were, load in the first layer of strut was 71% of that in Zone B, second layer strut was 76% and third layer strut was 101%. The lower strut loads in the upper struts in Zone A can be attributed to the higher flexibility and therefore larger yield of the retaining system in Zone A.

## Stresses in Reinforcing Bars and Bending Moments of Diaphragm Wall

Monitoring of the stresses in reinforcing bars is a reliable method to check safety of the diaphragm wall during construction. For the two projects described in this paper, a series of reinforcing steel transducers were welded onto the primary reinforcing bars of the reinforcing steel cage of selected diaphragm wall panels.

Monitoring records clearly indicate that stresses in the primary reinforcement increased with depth of excavation. At the completion of excavation at the TPC project, it was found that maximum tensile stress developed in the reinforcement at approximately 12 m depth. After removal of the fourth level of struts, the strut load varied between 700 $kg/cm^2$ and 1160 $kg/cm^2$ (6.87 $KN/cm^2$ and 11.38 $KN/cm^2$). Monitoring records of the inclinometer casing also indicate that maximum yield of the diaphragm wall occurred at that depth and the maximum moment was about 19 t-m/m (186 KN-m/m).

For the TNAEDC project, the maximum tensile stress in the diaphragm wall reinforcement was 790 $kg/cm^2$ (7.75 $KN/cm^2$) and occurred at 4.3 m depth after completion of the third stage excavation and near the surface of excavation at depth of 7.8 m when the excavation was completed. The maximum bending moment in the wall was approximately 14 t-m/m (137.3 KN-m/m). Figures 11 and 12 compare the predicted bending moments with bending moments calcualted from measured stresses in the reinforcing bars. The actual moments were very close to that predicted at the TPC site but were lower than the theoretical value at the TNAEDC site. However in both cases, the moments developed in the wall are within allowable limits.

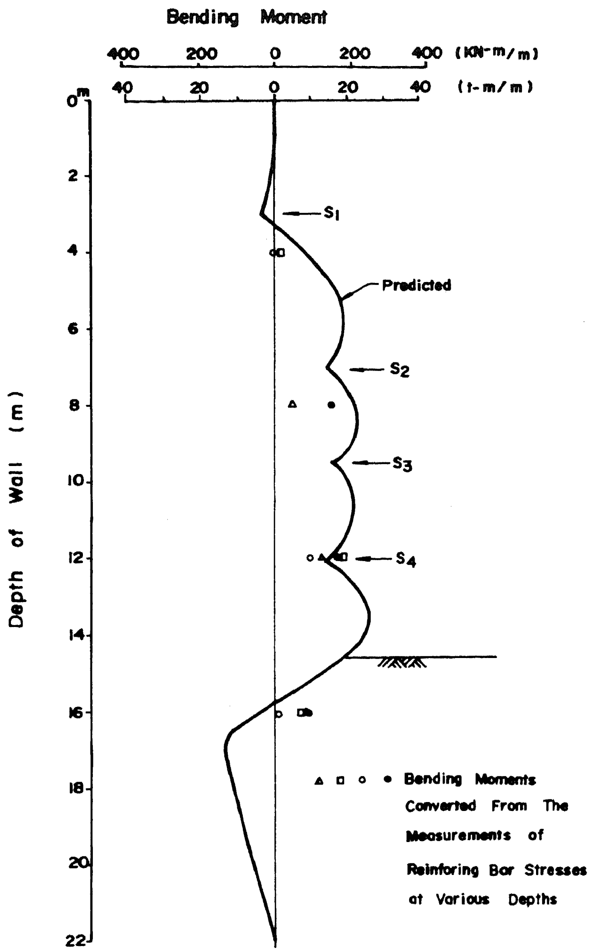



Fig. 11. Comparison of Predicted Bending Moments with Measurements of the TPC Diaphragm Wall

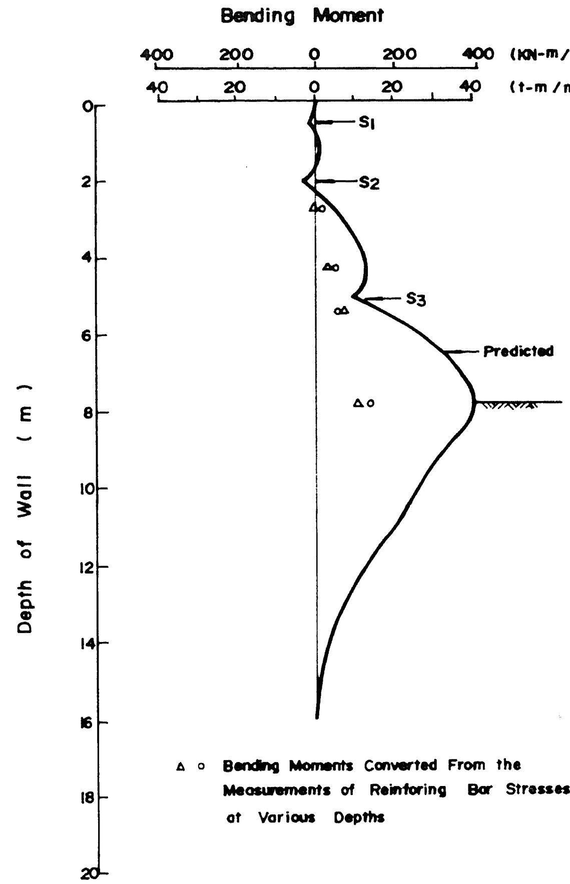



Fig. 12. Comparison of Predicted Bending Moments with Measurements of the TNAEDC Diaphragm Wall

CONCLUSIONS

The economic value and engineering feasibility of using diaphragm walls as both temporary and permanent earth retaining structure for deep excavations in high density urban areas are well recognized. From the two case records described in this paper the following conclusions can be drawn in regard to the behavior of diaphragm walls:

(1) In sandy soil, the diaphragm wall tends to rotate around the wall base towards the excavation surface and behaves like a cantilever. As the excavation progresses, the upper portion of the wall deflects more towards the excavation and stresses in the wall increase gradually due to the arch action of the sand. Stresses in the mid section of the wall start to decrease and gradually reaches equilibrium as soon as the base mat is constructed.

(2) In the very soft silty clay stratum in Taipei, when the diaphragm wall is not penetrating into any hard stratum deflection of diaphragm wall continues to increase with progress of excavation. In the early stages of excavation, more deflection occurs in the upper part of the wall. The lower portion of the wall yields rapidly as the excavations is near completion.

(3) Elastic model of soil-diaphragm wall system can be used in predicting the wall behavior In the analysis when average value of active earth pressure and at rest pressure is used, the factor C for calculating the modulus of subgrade reaction of sandy soil is taken at 10, the predicted values of deflection and moment are very close to the field measured values. In soft cohesive soils, when the value of facto C is taken at 6 to 10, the calculated lateral deformations are slightly lower than the measured values, however, the estimated curvature of the wall agrees well with the measurement.

(4) Behavior of diaphragm wall in deep excavation work is closely related to the stiffness of the bracing system, magnitude of preload on the struts, magnitude of surcharge surrounding the wall, construction procedure and selection the magnitude of coefficient of subgrade reaction.

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