XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

BEHAVIOR OF ANCHORS FOR A TIEBACK-SUPPORTED EXCAVATION IN ALLUVIAL SOIL

TENUE DES ANCRAGES SUR FOUILLES ETANCONNEES EN SOLS ALLUVIAUX

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SYNOPSIS : Based on the test results of the ground anchors installed in alluvial soil, a critical fixed anchor length of 20 m was found for both the tension type anchors and the compression type single-bore-multiple-anchorage (SBMA) anchors in terms of the ultimate anchorage capacity. For fixed length longer than 20 m, a smaller increment in anchorage capacity per unit length of the fixed end was observed. According to the frictional resistance mobilized along the fixed length, it was most cost effective to place the internal anchorages within the fixed length of the SBMA anchor at a spacing between 3 m and 7 m. For anchors locked off at 50 ~ 60 % and 80 ~ 90 % of the limit anchor load, the difference in the average losses of the residual load is not significant, but the range of load variation for the former is much smaller.

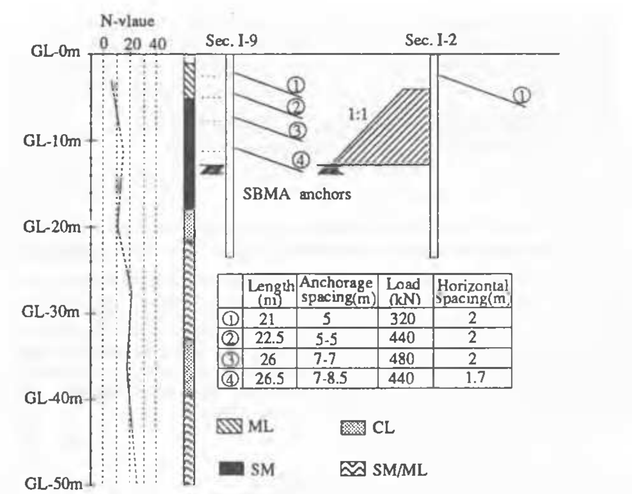
INTRODUCTION

Although braced excavation in alluvial soils is common practice for high rise buildings in Taipei city, an internal bracing system is usually not suitable for sites covering a large excavation area or with an irregular boundary shape. To cope with such size and shape problems, a tieback ground anchor system is often adopted to support the retaining walls during the excavation. In addition, vertical anchors or tension piles are sometimes required to tie down the basement against uplift ground water pressure or to counterbalance the eccentrical loads acting on the foundation mat. In particular, when anchor system is used for the permanent purpose, the long term behavior between soil and anchors should be carefully examined in order to ensure the safety of the entire structure. In this paper, an anchors supported excavation site with 12.4 m in depth and 50,000 m^2 in area will be studied. Totally, 1778 tieback anchors were installed to support the retaining walls and 1323 tiedown anchors were used to restrain the basement against the uplift of the ground water pressure. A series of on-site anchor load tests had been performed to determine the most suitable anchor length for the anchor systems. Moreover, variations in the residual anchor loads, as well as the deflections of the retaining walls, were also monitored during the excavation to safeguard the stability of the retaining systems.

SITE CONDITIONS AND TEST PROGRAM

The subsoil condition at the excavation site is typical of the upper level formation in the Taipei Basin. It consists of alternating layers of silty clay and silts interstratified with fine sand layers containing a high silt content (Fig. 1). The ground water level is located around the interface between the clayey silt layer and the silty sand layer. Dewatering outside the diaphragm retaining wall during the excavation was strictly prohibited. For the tieback anchors, the fixed ends were mainly located in the silty sand layer ranging from GL -5m to GL -18m with an average SPT value equal to 10. In comparison, the fixed ends of the vertical uplift resisting anchors were located in the alternate layers of silt and silty sand ranging from GL -22m to GL -50m with SPT values varying from 15 to 22.

Three types of ground anchors had been selected in this basement excavation project, namely, the single protection anchors which had a corrugated polyethylene tube within the fixed length seperating the tendons from the surrounding ground (TS type), the unprotected anchors which had no designed protection other than cement grout cover for the fixed length (TU type), and the steel tendon removable anchor. The single protection anchors were used as the permanent basement tiedown anchors, while the unprotected anchors, and the steel tendon removable anchors were both used simply as the temporary tieback anchors. Basically, the steel tendons removable anchors used on this project could be classified as the single-bore-multiple-anchor



	Length (m)	Anchorage spacing(m)	Load (kN)	Horizontal Spacing(m)
①	21	5	320	2
②	22.5	5-5	440	2
③	26	7-7	480	2
④	26.5	7-8.5	440	1.7

Fig. 1 Profiles of soils, typical excavation sections, and tieback anchors

(SBMA) type anchors (Barley, 1988) in terms of load carrying characteristics. They consisted of two to three load bearing anchorages and the same amount of steel tendon pairs whose entire length was sheathed with plastic tube in a single borehole (Fig. 2).

Totally, 22 anchors were tested to failure prior to the installation of working anchors. Of these, 6 were SBMA type anchors, 6 were unprotected anchors, and 10 were single protection anchors. The dimensions of the test anchors are given in Table 1. All of the single protection anchors were 37 m in length with 5 m free length and 32 m fixed length. The free length of the unprotected anchors was either 4 m or 5 m, and the fixed lengths varied from 12 m to 37 m. The spacing between each internal load bearing anchorage of the SBMA anchors varied from 3 m to 12 m. Generally, anchors were tested according to the on-site suitability test procedures recommended by FIP (1982). However, due to variations in the free length of each of the internal load bearing anchorages, the load testing procedures specified by FIP (1982) had been modified. As shown in Fig. 3, the steel tendon pair connected to each internal load bearing anchorage was prestrained individually during the initial loading stage. Typically, for the SBMA anchor having three load bearing

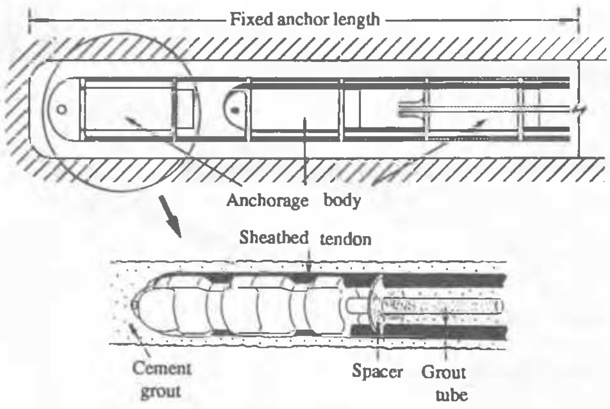


Fig. 2 Tendons assembly of the SBMA anchor

Table1. Dimensions of test anchors

	Fixed length (m)	Free length (m)	Anchorage spacing(m)	Anchor direction
TS1 - TS10	32	5	-	Vertical
TU1	32	5	-	Vertical
TU2	32	5	-	Vertical
TU3	37	5	-	Vertical
TU4	32	5	-	Vertical
TU5	12	4	-	Inclined
TU6	15	4	-	Inclined
C1(SBMA)	26.0 (25)*	-	7.0-7.0[3]**	Inclined
C2(SBMA)	26.0 (25)	-	7.0-7.0[3]	Inclined
C3(SBMA)	26.5 (20)	-	7.0-8.5[3]	Inclined
C4(SBMA)	26.5 (21)	-	8.5-12 [3]	Inclined
C5(SBMA)	21.0(9.5)	-	3.0[2]	Inclined
C6(SBMA)	30.0 (15)	-	5.0-5.0[3]	Vertical

*( ) indicates the length of anchor in sandy layer
**[ ] indicates the number of internal anchorage

anchorages, the steel tendon pair with the longest free length was first strained δ_1 alone; then it was strained an additional δ_2 together with the pair having the intermediate free length, and another δ_3 together with the pairs having the intermediate and the shortest free lengths. The values of δ_1, δ_2, and δ_3 were determined from the differences in the elastic elongation of the free length of each load bearing anchorage when the final pull-out load was applied to the SBMA anchor. Theoretically, by following the above procedure, each internal load bearing anchorage within the SBMA anchor would be subjected to the same load when the maximum test load, P_p, was reached.

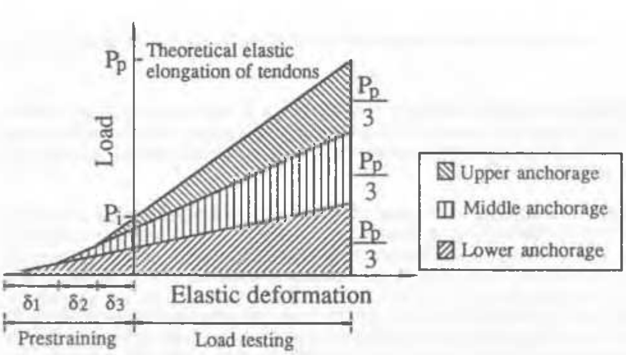


Fig. 3 Prestraining procedures for tendons of SBMA anchors during load test

BEHAVIOR OF ANCHORS

The main purpose of this anchor testing program was to determine the reliability of the anchors installed in the alluvial deposit. So, both the plastic deformations and the elastic deformations of the test anchors were measured and analyzed. In addition, since the fixed lengths of the test anchors were considerably longer than the upper limit of the fixed length recommended by some guidelines for the anchor design (FIP, 1982), the critical fixed length in terms of the ultimate anchorage capacity for anchors constructed in the alluvial soils had been determined statistically from the results generated from this test program and test results gathered from other areas around the Taipei Basin.

Tension Type Anchors

The load-elastic deformation relationships of the single protection (TS) anchors is shown in Fig. 4. It can be found that due to the possible slip between the cement grout and the steel tendons and the cracking of the cement grout caused by the tensile force, the elastic deformation per unit load increment increases gradually with the applied load. An approximately 5 m increase in the free length was observed per 500 kN load increment. Since any increase in the free anchor length will reduce the bond length between the tendons and the cement grout, the effective fixed anchor length will decrease gradually as the applied load increases. Although certain debonding of tendons within the fixed length is allowed (FIP, 1982), the effect of shortened effective fixed anchor length should be taken into account when determining the fixed length for the anchors. However, it ought to be pointed out that since the bond among the tendons, the cement grout, and the corrugated polyethylene tube depends upon several complex and not fully understood phenomena (Hanna, 1982), the free lengths determined from Fig. 4 were based on the theoretical elastic elongation of steel tendons and can only be treated as the "apparent" free length, which may be somewhat different than the actual free anchor length.

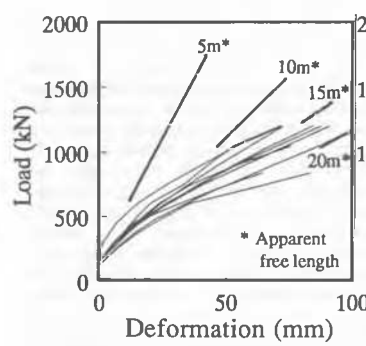


Fig. 4 Elastic deformation of TS type anchors

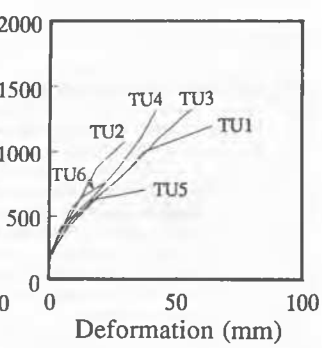


Fig. 5 Plastic deformation of TU type anchors

Since the plastic deformation of the tendons during anchor load testing was negligible, the plastic deformation of the anchors occurred mainly at the fixed anchor end. The plastic deformation of the unprotected (TU) anchors, which had fixed lengths varying from 12 m to 32 m, is shown in Fig. 5. Prior to the yielding of the anchors with fixed lengths of 12 m (TU5) and 15 m (TU6), all of the curves of plastic deformation virtually follow the same path, despite the difference in the fixed length. The plastic deformation increases at the rate of 0.42 mm per 10 kN load increment. For those with a fixed length varying from 32 m to 37 m, the similarity in the plastic behavior continued until the yielding loads were approached. This implies that the load transfer along the fixed anchor started at from the front of the fixed end and progressed gradually to the rear of the anchor. Although larger anchorage loads can be mobilized with longer anchors, only a very limited gain in the anchor capacity was observed when the fixed length exceeded 32 m.

Based on the plastic behavior at the fixed anchor end (Fig. 5), it can be deduced that there is a critical fixed anchor length for anchors constructed in the sandy alluvial deposit, and this critical length seemed to be considerably higher than the 10 m limit specified by the FIP's (1982) recommendation for ground anchors. The data shown in Fig. 6 give the statistical relationship between the fixed anchor length and the ultimate anchorage capacity for the 6 compression type SBMA anchors and 15 tension type test anchors tested in this site and 25 anchors constructed in other areas of the Taipei Basin. Generally speaking, the relationship between the ultimate anchorage capacity (P_u) and the fixed anchor length (L) can be represented by the bilinear curves.

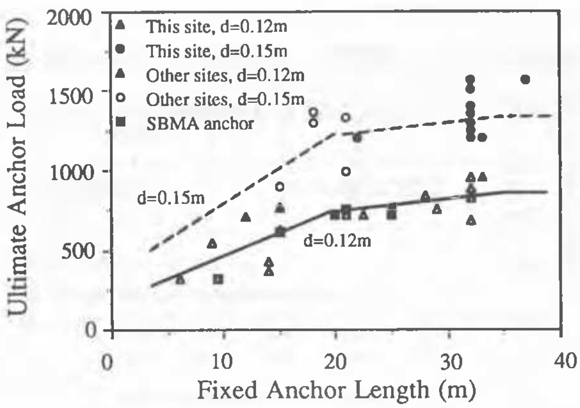


Fig. 6 Relationship between fixed anchor length and ultimate anchor capacity

For anchors with diameter = 0.12 m:

$$P_u\,(kN) = 26\,L\,(m) + 175, \qquad 3\,m < L \leq 20\,m \qquad (1)$$
$$P_u\,(kN) = 7\,L\,(m) + 625, \qquad 20\,m < L < 35\,m \qquad (2)$$

For anchors with diameter = 0.15 m:

$$P_u\,(kN) = 42\,L\,(m) + 350, \qquad 3\,m < L \leq 20\,m \qquad (3)$$
$$P_u\,(kN) = 7\,L\,(m) + 1100, \qquad 20\,m < L < 35\,m \qquad (4)$$

For anchors with fixed length shorter than 20 m, the ultimate anchorage capacity increases more significantly with the increasing fixed anchor length than those with fixed anchor length longer than 20 m. Furthermore, no obvious increase in the ultimate capacity was found for those anchors with fixed anchor lengths longer than 35 m. Although a similar relationship between the fixed anchor length and the ultimate capacity was observed for anchors with different diameters, a 1.6 times increase in the ultimate capacity increment per unit length of the fixed end was found when the anchor diameter increased 1.25 times from 0.12 m to 0.15 m (Eq. 1 and Eq. 3). This implies that an increase in the borehole diameter is relatively more effective in increasing the anchorage capacity than an increase in the fixed anchor length. However, the benifit of increase in the anchor diameter becomes negligible for anchors with the fixed length longer than 20 m (Eq. 2 and Eq. 4).

Compression Type Anchors

The deformations of SBMA type tendon removable anchors have been measured and shown in Fig. 7. Although the bond stress distribution within the SBMA type anchors is rather complicated, the plastic deformation curves of C1 to C4 anchors generally follow the same path on the figure, while the one (C5) with the shorter length exhibits a slightly larger increment in plastic deformation per unit load increment. These test results indicate that there also exists a critical fixed length for the SBMA type anchors. Similar to the friction type anchors, the bilinear relationship represented by Eq. 1 and Eq. 2 can be used to describe the relationship between the ultimate capacity of the SBMA anchors and the fixed anchor length (Fig. 6).

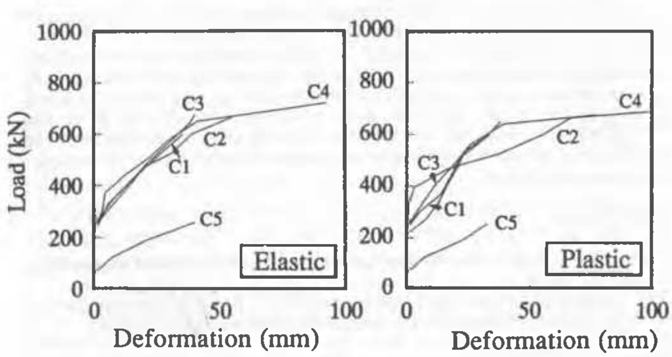


Fig. 7 Deformations of SBMA type anchors

Despite the prestraining procedure followed during the initial loading stage, it is practically impossible for all of the internal anchorages within the SBMA anchors to fail simultaneously. Instead, the failure of the SBMA anchor was quite often caused by the breaking of cement grout around one of the internal anchorages at the loads ranging from 250 kN to 300 kN if the spacing between the internal anchorages was larger than some critical value. Ideally, the anchorages within the SBMA anchors should be carefully arranged to have the anchor pulled out as a whole rather than failed by the local failure of individual anchorages. Therefore, a series of tests was performed on the SBMA anchors with the internal anchorage spacings varying from 3 m to 7 m, or greater than 7 m (Table 1) to study the influence of spacing. For all of the SBMA anchors tested here, only the anchor with a 3 m (25 d, d is the anchor diameter) spacing between each anchorage was pulled out as a whole. Others with 5 m (42 d), 7 m (58 d), or greater than 7 m spacings were failed by the local breakage of the cement grout around one of the individual anchorages. In other words, if the spacings of the internal anchorages were up to 5 m ~ 7 m, the frictional resistance mobilized along the fixed length by the individual anchorage tended to exceed the compressive strength of the cemnet grout. So, any increase in the internal anchorage spacing beyond 5 m ~ 7 m might prove to be ineffective in increasing the overall pull out resistance of the SBMA anchor. However, even though not all of the SBMA anchors were pulled out as a whole, the preliminary results showed a linear relationship between the ultimate overall anchorage capacity and the fixed anchor length was maintained for anchors with a fixed length up to 20 m and spacing between anchorages up to 7 m. So, it can be suggested that the most cost effective spacings between the internal anchorages of the SBMA anchors in the sandy alluvial deposit would be larger than 3 m (25 d) but not exceeding 7 m (58 d).

Since the ultimate pull out resistance of the SBMA anchor was a combination of that of each internal anchorage, it is of interest to determine the relationship between the overall ultimate capacity of the SBMA anchor and the individual capacity of each internal anchorage. To compare the load-deformation curves between each internal anchorage and the SBMA anchor as a whole, the internal anchorages of the C6 anchor were loaded individually to the yield point and then the applied load was released. After completion of the individual test, the anchor was loaded as a whole in accordance with the procedure described in Fig. 3. It is quite obvious that each internal anchorage has its own load-deformation behavior and each does not reach its ultimate capacity at the same deformation (Fig. 8). Actually, the ultimate capacity of the SBMA anchor is a complicated combination of bond stress distribution and the strength homogeneity of the cement grout body, and it is so far not fully understood. Therefore, the overall ultimate anchorage capacity of the SBMA anchor can not be treated simply as the summation of all of the internal anchorages. The preliminary test results show that the former is only about 75 % of the latter for the SBMA anchors with three internal anchorages and installed in the alluvial soil.

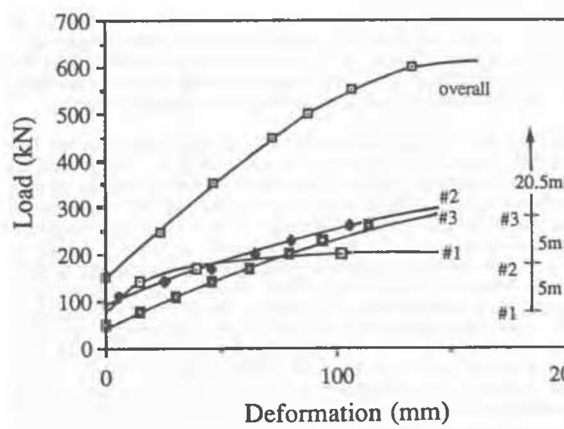


Fig.8 Comparison of overall and individual anchorage behavior of SBMA type anchor

In general, the dimensions of the working anchors and the test anchors were kept the same. The allowable loads of the working anchors were determined from the creep behavior of the test anchors. A typical way adopted for determining the allowable loads of the working anchors is illustrated in Fig. 9. The creep deformations of a SBMA anchor (C2) at various observation times are plotted against the different loading levels (Fig.9a). The limit loads for different observation times were decided according to the creep behavior of the test anchor (Fig. 9b) and the procedures proposed by FIP (1982). Although the limit loads of the anchor are found to vary with the observation time, it

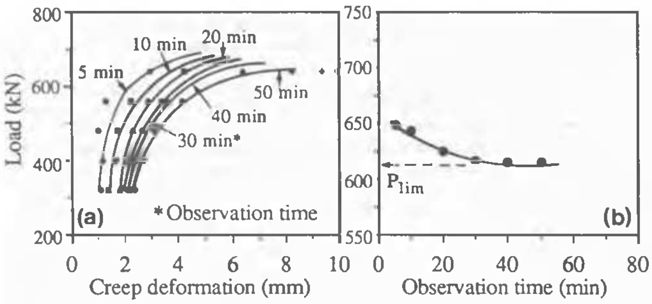


Fig. 9 Determination of the limit anchor load from the creep deformation of anchor

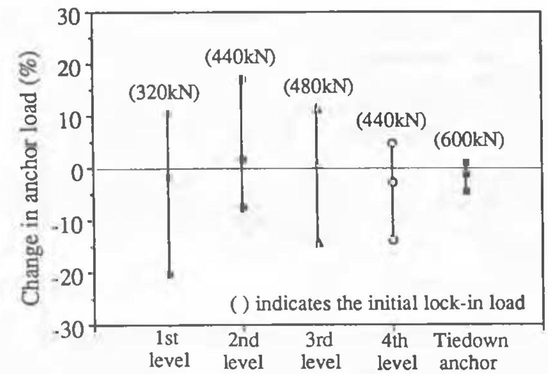


Fig. 10 Changes in lock-in anchor load

tends to approach a constant value (P_{lim}) after certain elapsed time, say 50 minutes. For the temporary tieback anchors, the allowable load of working anchor were set at the 80 to 90 % of the P_{lim}, while a working load equal to 50 to 60 % of the P_{lim} was chosen for the permanent tiedown anchors.

Since the change in the residual anchor loads is a complex matter and depends on the nature of the excavation induced retaining wall deflection and the bond between the anchor and the soil, an anchor load monitoring program involving 64 tieback and tiedown anchors, respectively, was carried out throughout the entire excavation period. The monitoring results indicate that the changes in the residual anchor loads vary from + 10 % to - 20 % (with an average value equal to - 2.0 %) of the lock-in load for the anchors at the first teiback level, while a change between + 5 % to - 14 % (with an average value equal to - 3.0 %) in lock-in anchor load was measured for the anchors at the fourth tieback level. In comparison, the change in the residual anchor load for the tiedown anchors ranges from + 1.0 % to - 4.5 % (with an average value equal to - 1.5 %) of the initial lock-in load (Fig. 10). In other words, for anchors locked off at 80 to 90 % of the P_{lim}, an average loss of anchor load equal to 3 % was observed, and a 1.5 % decrease in the lock-in anchor was measured for anchors locked off at 50 to 60 % of the P_{lim}. Although the difference in the average values is not significant, the range of load variation for the latter is much smaller (Fig.10).

Deflections of Tiebacked Retaining Wall

This basement excavation was supported on three sides by 60 cm thick diaphragm walls and 4 levels of the SBMA tieback anchors and on the fourth side by one level of SBMA tieback anchors and an earth berm (Fig.1). The reason for using only one level of anchors on one side was simply due to being unable to get permission from neighbors to install anchors under their properties. The deflections of the diaphragm retaining wall were constantly monitored with inclinometers as the excavation proceeded downward.

The deflection profile of the diaphragm wall during excavation has been monitored and some of the results are shown in Fig. 11. In general, the deflection of the diaphragm wall was well under control. Less than 3 cm of wall movement (relative to the bottom of the wall) toward the excavation was found throughout the entire excavation stage for the section (I-9) supported with 4 levels of tieback anchors. In comparison, the section (I-2) which was tied back with only one level (the first level) of anchors deflected up to 10 cm relative to the bottom of the diaphragm wall. However, since the bottom of the wall might have moved inward, it is believed that the maximum deflection of the wall could have exceeded 3 cm and 10 cm, respectively. For the I-2 section, no significant lateral wall deflection was observed until the third excavation was reached. However, prior to the fourth excavation stage, a sudden and significant wall deflection was noted on the I-2 section during a heavy rainfall which lasted for three days. Piezometer measurements detected an average of 2 m to 3 m raise in the ground water level outside the excavation during that period. Meanwhile, it was also found that the rain water was seeping into the cracks of the shotcrete cover protecting the surface of the berm and consequently reducing the passive resistance of the berm. The combined effect of the rising ground water table and the reduced passive resistance of the berm caused an excessive lateral movement of the diaphragm retaining wall. Since some of the supporting load was transferred from the softening berm structure to the tieback anchors, several local failures of the internal anchorages were found within some of the SBMA tieback anchors at the first tieback level of the retaining wall. Although the wall deflection was arrested after supplemental anchors were installed to replace the failed anchor and virtually no additional wall movement was observed as the excavation proceeded downward, it should be cautious when the prestressed tieback anchors and an earth berm are used jointly as the supporting system for a deep excavation in alluvial soil.

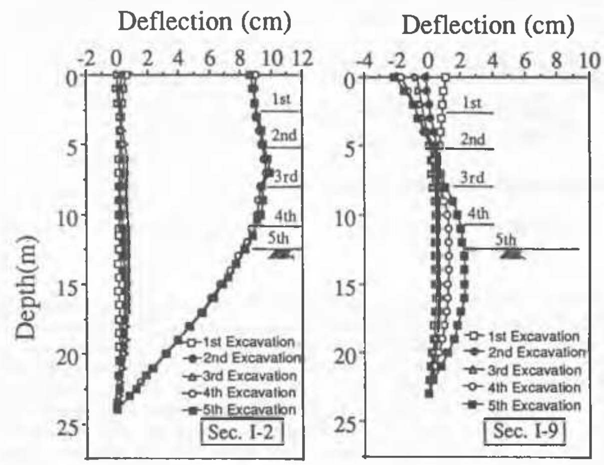


Fig. 11 Deflection profiles for diaphragm retaining wall

CONCLUSIONS

Based on the results of on-site anchor pull-out tests involving ground anchors installed in the alluvial soils of the Taipei Basin and the monitored behavior of the retaining system, the following conclusions can be advanced:

1. The anchorage capacity of tension type anchors can be increased more effectively by enlarging the anchor diameter than by increasing the fixed anchor length. A critical fixed length of 20 m was found in terms of ultimate anchorage capacity; for fixed anchor lengths longer than 20 m, a smaller increment in anchorage capacity per unit length of the fixed end was observed.
2. Similarly, a critical fixed length of 20 m was found for the SBMA anchors. For those with a fixed length shorter than 20 m, the recommended spacing between the internal anchorages is between 3 m and 7 m, and the overall ultimate capacity of the SBMA anchor is about 75% of the summation of the individual anchorages for those with three anchorages.
3. A loss of the anchor load equal to 3 % was measured for the inclined tieback anchors locked off at 80 to 90 % of the limit load which was determined from the creep behavior of anchor. In comparison, a 1.5 % loss of the anchor load was monitored for the vertical tiedown anchors locked off at 50 to 60 % of the limit load. Although the difference in the average values is not significant, the range of load variation for the latter is much smaller.

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