Measured Performance of Full Scale Tieback Walls in Sand

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요 지

사질토 지반에서 앵커로 지지된 토류벽의 거동을 연구하기 위하여 텍사스 에이엔엠 대학에 소재한 미국 국립 토질 시험장에 계측기가 장착된 실물 크기의 앵커 토류벽을 시공하였다. 시공 기간 중의 단계별 계측과 시공 후의 계측을 실시하여 사질토 지반에서의 앵커 토류벽의 현장 거동을 분석하였다.

시공 단계별 토류벽의 수평 변형, 토류벽의 휨 모멘트, 토류벽의 연직 하중, 배면토의 침하와 앵커 하중을 계측하여 앵커 토류벽의 현장 거동을 분석하여 제시하였다. 사질토 지반에서의 앵커 토류벽의 거동 특성을 실측결과와 비교하여 분석하였다.

ABSTRACT

Two instrumented full scale tieback walls in sand were constructed at the National Geotechnical Experimentation Site located on the Texas A&M University Riverside Campus. Measurements were obtained from the one row anchor wall and from the two row anchor wall at different times during construction.

The measured performance of the tieback walls is presented and investigated. The behavior of these walls at different construction stage is evaluated with respect to lateral deflections of the wall, settlement of the ground, bending moment of the wall, axial load distribution in the wall, and anchor load variation. The fundamental mechanism of a tieback wall in sand is established and explained with the measurements.

Keywords : Tieback wall, Anchored wall, Excavation, Construction sequence, Permanent wall, Retaining wall

1. Introduction

Excavations represent major work in civil engineering. Excavations in urban areas and for

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highway facilities are normally made with vertical slopes for efficiency. The excavation causes stress relief in the ground and induces horizontal movement and settlement of the ground. Since most vertical slopes in soils and rocks are not stable, some type of supporting system is needed to ensure the overall stability of the excavation. Internal bracing has been used commonly for most temporary excavations.

A tieback wall or ground anchor wall is an innovative earth retaining system which uses tiebacks or ground anchors. A tieback functions as a load carrying element, consisting essentially of a steel tendon inserted into a suitable ground formation(Cheney, 1988). Retaining structures for transportation facilities, bridge abutments, deep excavation in urban area, underpinning of structures and stabilization of sliding soil or rock slopes are some applications(Weatherby, 1982).

A full-scale soldier pile and woodlagging tieback wall with two different instrumented sections have been built and monitored at the Texas A&M University National Geotechnical Experimentation Site. A series of in-situ tests and laboratory tests were performed to identify the soil condition and properties. The bending moments and axial forces in the soldier beams and the horizontal deflections of the wall were observed at different times during construction. The variation of the anchor loads with time, and the settlement of the wall at the final construction stage were also presented.

2. Site and Soil Condition

The soil at the location of the wall consists of a medium dense clayey sand or silty sand between the depths of 0 and 3.0m(10ft), a medium dense clean poorly graded sand between the depths of 3m(10ft) and 7.6m(25ft) and a medium dense clayey sand between the depths of 7.6

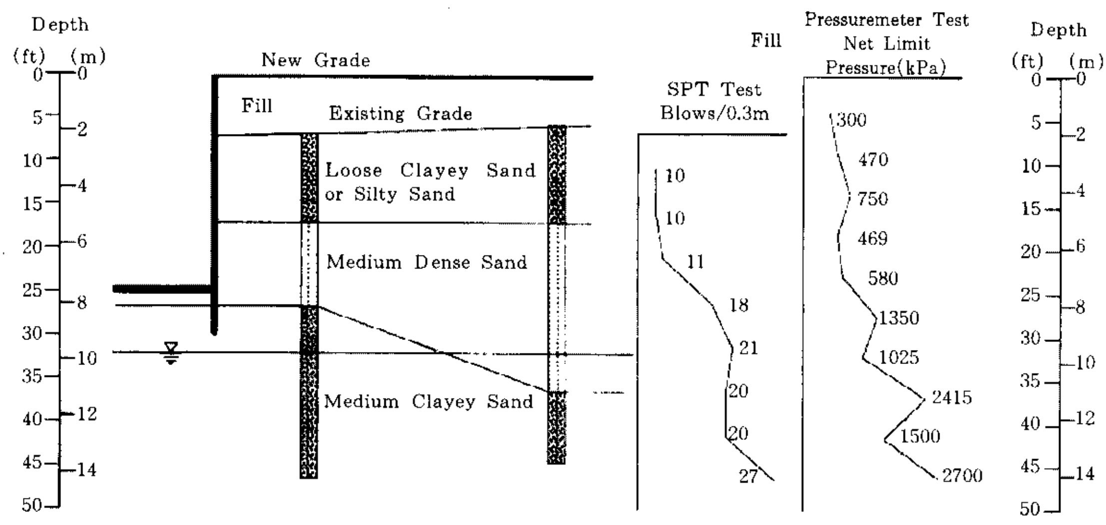


Fig.1 Soil condition at the test wall site

m(25ft) and 12.2m(40ft). The groundwater ranges from 7.5m(24.5ft) to 7.6m(25ft) below the ground level on May 10th, 1990 and September 20th, 1990. In order to ensure that the groundwater would be avoided during the construction, 2.1m(7ft) high fill was placed on top of the ground surface. A series of in-situ tests and laboratory tests were performed to investigate soil properties. In-situ tests include SPT, CPT, PMT and borehole shear test. The soil profiles at the test wall site is shown in Figure 1. Based on the borehole shear test, the angle of internal friction varies from 30 to 33degree and the cohesion varies from 0 to 4kPa.

3. Instrumentation

The instrumentation used in the tieback wall consists of strain gages, embedment strain gages, load cells and inclinometers. The data acquisition for strain gages and embedment gages was automated by using dataloggers and a personal computer. The inclinometer readings were taken by using an Inclinometer Reader(Geokon Model GK-601) and the data files were saved in a personal computer.

The strain gages used to measure strains in the soldier beams during the excavation are surface-mounted vibrating wire strain gages(Geokon Model VSM-4000, tolerence of ± 0.3 microstrain). Twenty eight strain gages were installed at every foot on each side of the soldier beam flanges. The lowest and topmost strain gages were located 0.3 m from the ends of the soldier beams (9.14m or 30ft). Only the second and third strain gages from the top of the beams were placed 0.45m(1.5ft) apart.

Sixteen inclinometer wells were placed on the instrumented soldier beams, between the soldier beams and behind the wall face. For the inclinometer wells on the instrumented drilled soldier beams, four inch diameter PVC pipes were securely attached to the soldier beams. The soldier beams with the PVC pipes were installed in predrilled holes. Then the inclinometer casing was installed inside the PVC pipe and secured with grout. The inclinometer casings on the instrumented driven soldier beams and behind the wall face, were installed in a 88.9 mm(3.5inch) outside diameter, 63.5mm(2.5inch) inside diameter flush-coupled casing driven close ended to a depth of about 13.72m(45ft) below the top of the wall. A Sinco Digitilt Inclinometer(Model 50325-E, tolerence of 2 to 10mm per 30m)was used. It was assumed that no movement occurs due to the excavation at that depth. The inclinometer casing with a plug at the end (48.3mm or 1.9inch diameter) was lowered into the driven casing and the casing was retrieved while filling with weak grout.

In order to measure anchor loads, vibrating wire load cells(Geokon Model 4900, tolerence of ± 2 to 5%) were used on each of the anchors supporting an instrumented soldier beam. The load cells were installed by placing two bearing plates on the wale.

Optical settlement points were installed in order to measure the vertical movements of the ground surface behind the wall. The settlement points consist of a 1.5m(5ft) length of No.5 rebar driven 1.37m(4.5ft) into the ground. The settlement of the top of the wall face was measured by leveling the 127mm(5inch) length rebars welded at the top of the soldier beams.

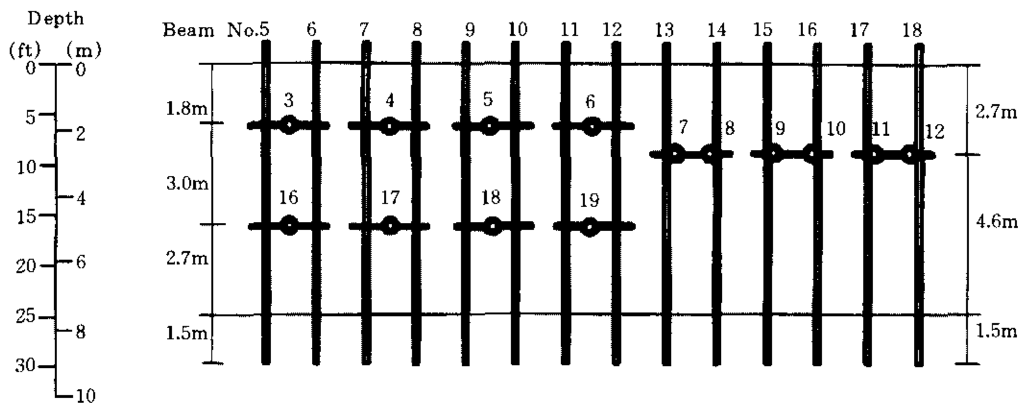
4. Construction

The wall is divided into a one row anchored section and a two row anchored section corresponding to soldier beams labeled 5 to 12 and 13 to 18, respectively. All beams were placed with 2.4m(8ft) center to center spacing. Beams No. 7 to 10(W6×25) were instrumented in the one row tieback section and Beams No. 13 to 16(HP10×57) were instrumented in the two row tieback section. The uninstrumented soldier beams were wide flange steel beams HP8×36 and HP10×42. The front elevation view of two sections are shown in Figure 2. The section view of one row and two row anchored wall is shown in Figure 3.

The excavated wall face between soldier beams was lagged with 305×2804×76mm(10×92×3inch) wood boards with a 304mm(1ft) center to center vertical spacing. Woodlaggings were cut to approximately 2.44m(8ft) long pieces and beveled at both ends. Then they were seated on 127mm (5inch) long bolts arc-welded on the surfaces of the soldier beams. The space between laggings was filled with hay to prevent the soil from being eroded away.

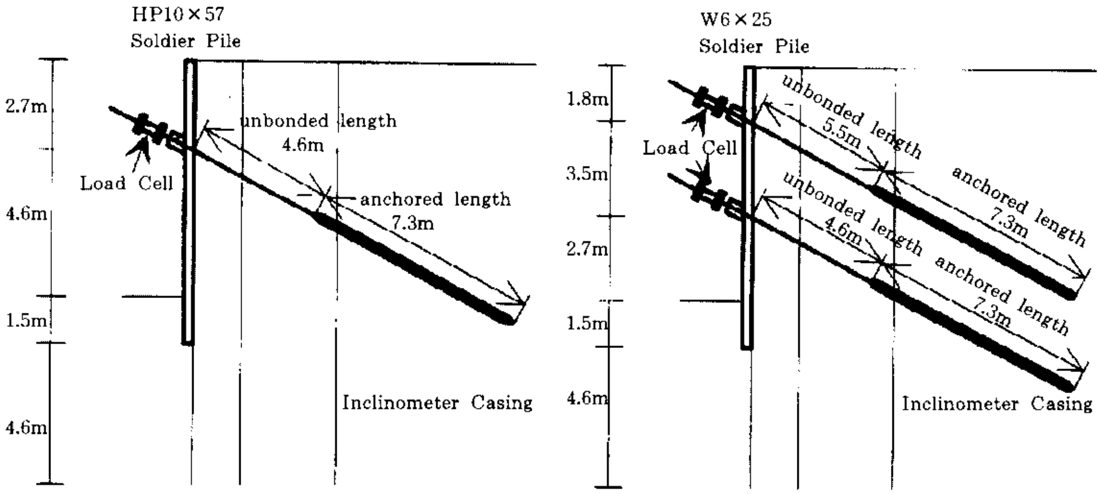
4.1 One Row Anchored Walls

The two instrumented driven soldier beams denoted as Beam No.15 and No.16 consist of a wide flange section(HP10×57) and two angle sections(L3×3×1/4) on each side of the flanges. For the two instrumented drilled soldier beams denoted as Beam No.13 and No.14, the same



Note : Instrumented Two Row Anchored Wall Section : Beam 7, 8(Driven)
Beam 9 (Drilled w/Structural Toe)
Beam 10 (Drilled w/o Structural Toe)
Instrumented One Row Anchored Wall Section : Beam 13(Drilled w/o Structural Toe)
Beam 14(Drilled w/Strucural Toe)
Beam 15, 16 (Driven)

Fig.2 Front elevation view of test tieback walls in sand



(a) One row wall section (b) Two row wall section

Fig.3 Section view of tieback walls in sand

steel sections as the instrumented driven beams were placed in 610mm(24inch) diameter holes with concrete backfill. The hole for the drilled beam No. 14 was backfilled with lean-mix grout. This lean mix grout was made of water and cement at the ratio of water 1 and cement 2.3 by weight. The average 28 day compressive strength of the lean-mix grout was 597.7kPa (86.7psi). The hole for beam No. 13 was filled with lean-mix grout above the excavation level and with class A concrete in the toe. This structural concrete was made of water, cement, sand and aggregate. The average 28 day compressive strength of the structural concrete was 3288 kPa (4770psi).

4.2 Two Row Anchored Walls

The two instrumented driven soldier beams marked as Beam No.7 and No.8 consist of an I-beam section(W6×25) and two angle sections(L3×3×5/6) on each side of the flanges. The two instrumented drilled beams denoted as Beam No.9 and No.10 consist of the same steel sections as the instrumented driven beams; they were placed in 457mm(18inch) diameter holes. The hole for the drilled beam(No. 9) was backfilled with lean-mix grout above the excavation level and with class A concrete in the toe. The hole for beam No.10 was filled with lean-mix grout. The concrete used for the two row anchored drilled beams were the same as those used for the one row anchored drilled beams.

4.3 Tiebacks

Tieback tendons consisting of a 31.8mm(1.25inch) steel bar(ASTM A-722-75) for the instrumented beams were installed in 88.9mm(3.5inch) diameter holes filled with high pressure injected cement grout. The tendons consist of a 7.3m(24ft) anchored length and a 4.6 to 5.5 m (15 to 18ft) unbonded length sheathed in plastic with grease between the steel tendon and the plastic sleeve. The layout of the tiebacks is shown in Figure 2. The section views of the one row and two row tieback walls are shown in Figure 3.

The installation sequence was as follows. First, Wales and brackets were mounted on the wall face. Then, a 89mm(3.5inch) diameter casing was driven into the ground at 30 degrees downward to the horizontal by using an air track machine. The tendon was inserted in the casing. Then, while the casing was extracted, cement grout was pumped inside the casing under a pressure of 2759 to 4137kPa(400 to 600psi) for the anchored portion of the tendon and 1034 to 2068kPa (150 to 300psi) for the unbonded portion of the tendon.

The ratio of water and cement in the grout was approximately 1 to 2.3. The grout had a measured 28 day strength of 597.7kPa(86.7psi). After 3 days of grout curing time, the tendons were locked off to 75% of the design anchor load. For each anchor, performance tests and/or proof tests were carried out before the anchors were locked off.

4.4 Construction Sequence

The construction sequential stages are described as the following.

1. On February 14, 1991, two sets of initial readings of the instrumentation were taken: strain gages and inclinometers.
2. On February 26, 1991, the wall was excavated and lagged to a depth of 2.4 m(8ft).
3. On March 7, 1991, the one row tieback wall section was excavated and lagged to a depth of 3m(10ft).
4. On March 8, 1991, performance tests were performed on tieback no 3 and proof tests were performed on tiebacks no. 4,5, and 6. These are the upper tiebacks of the two row tieback wall. The tiebacks were locked off to 75% of the design anchor loads, 474.7kN(106.5kips).
5. On March 13, 1991, the anchor tests were performed on tieback numbers 7 to 12, the tiebacks of the one row tieback wall. The tiebacks were locked off to 75% of the design anchor loads, 400.3kN(90kips).
6. On March 20, 1991, the wall was excavated and lagged to a depth of 5.2m(17ft).
7. On March 26, 1991, the anchor tests were run on tieback numbers 16 to 19 and the tiebacks were locked off to 75% of the design anchor load, 427kN(96kips).
8. On April 3, 1991, the wall was excavated and lagged to a depth of 7.6m(25ft).

5. Observed Performance

The behavior of the one row and two row tieback walls is described in this section.

The bending moments and axial forces in the soldier beams and the horizontal deflections of the wall were observed at different times during construction. Three construction stages for the one row tieback wall and five construction stages for the two row tieback wall were chosen in order to plot the test results and are summarized in Table 1 and Table 2. The variation of the anchor loads with time, the axial strain distributions in the embedding concrete at each construction stage, and the settlement of the wall at the final construction stage are also presented.

Table 1. Construction stages for the one row tieback wall

Construction Stage	Date	Event
Stage 1	3-7-1991	Excavation to 3.1m(10feet)
Stage 2	3-13-1991	Stressing Anchor at 2.7m(9feet)
Stage 3	4-3-1991	Excavation to 7.6m(25feet)

Table 2. Construction stages for the two row tieback wall

Construction Stage	Date	Event
Stage 1	2-26-1991	Excavation to 2.4m(8feet)
Stage 2	3-8-1991	Stressing Anchor at 1.8m(6feet)
Stage 3	3-20-1991	Excavation to 5.2m(17feet)
State 4	3-26-1991	Stressing Anchor at 4.9m(16feet)
State 5	4-3-1991	Excavation to 7.6m(25feet)

5.1 One Row Anchored Walls

The one row tieback wall consists of a drilled beam section and a driven beam section. Each section has two instrumented soldier beams: Beams 13 and 14 for the drilled beam section and Beams 15 and 16 for the driven beam section. The location of these beams is shown in Figure 2. Figures 4 and 5 show the bending moments, axial forces, and horizontal deflections in the driven soldier beam numbers 15 and 16 at different times in the construction sequence. The behavior of drilled beams in the one row anchored wall is shown in Figures 6 and 7.

The measurements of anchor load during and after the excavation are presented in Figure 8. The final readings on the load cells were taken 75 days after the excavation was completed. Tieback numbers 7 and 8 were installed in the drilled beam section while tieback numbers 9 and 10 were installed in the driven beams. The tiebacks were locked off at 300.3kN (67.5 kips) or approximately 75% of the design anchor load of 400.3kN(90 kips).

The settlements of the ground surface behind the one row tieback wall at the final excavation stage are presented in Figure 9(a). The normalized plot of settlements of the ground surface is presented in Figure 9(b) in terms of the excavation height of the wall.

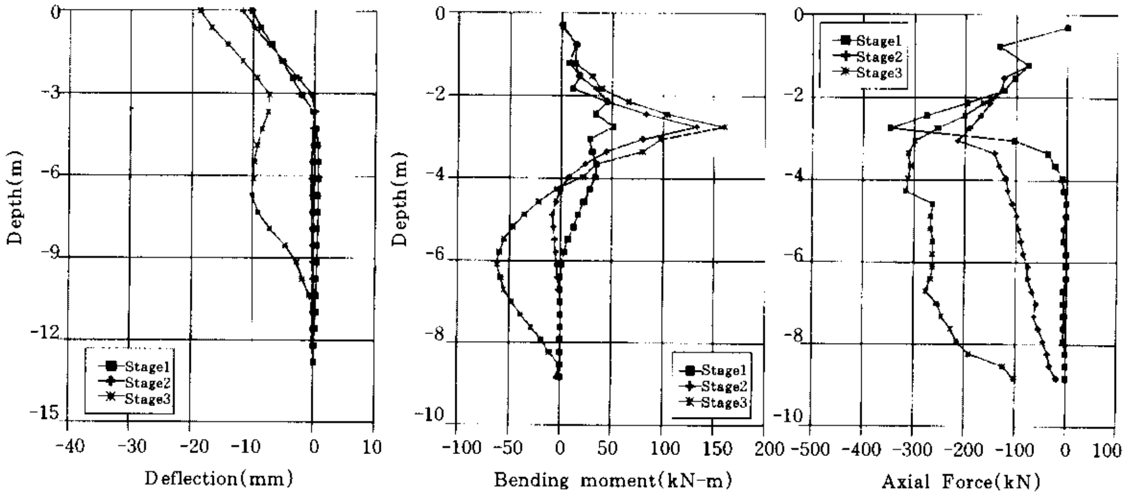


Fig.4 Deflection, bending moment and axial force of driven one row anchored wall(Beam No.15)

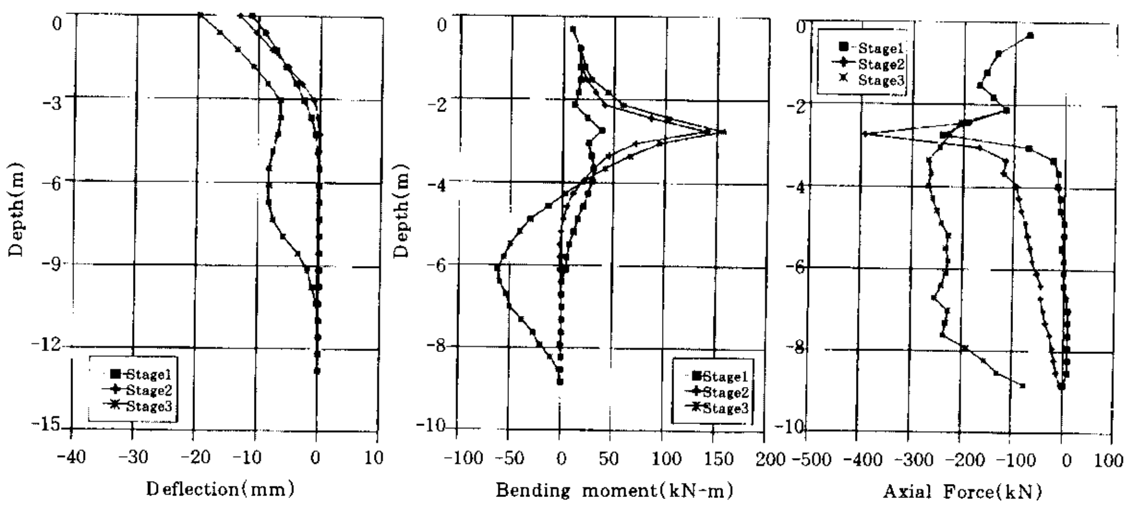


Fig.5 Deflection, bending moment and axial force of driven one row anchored wall(Beam No.16)

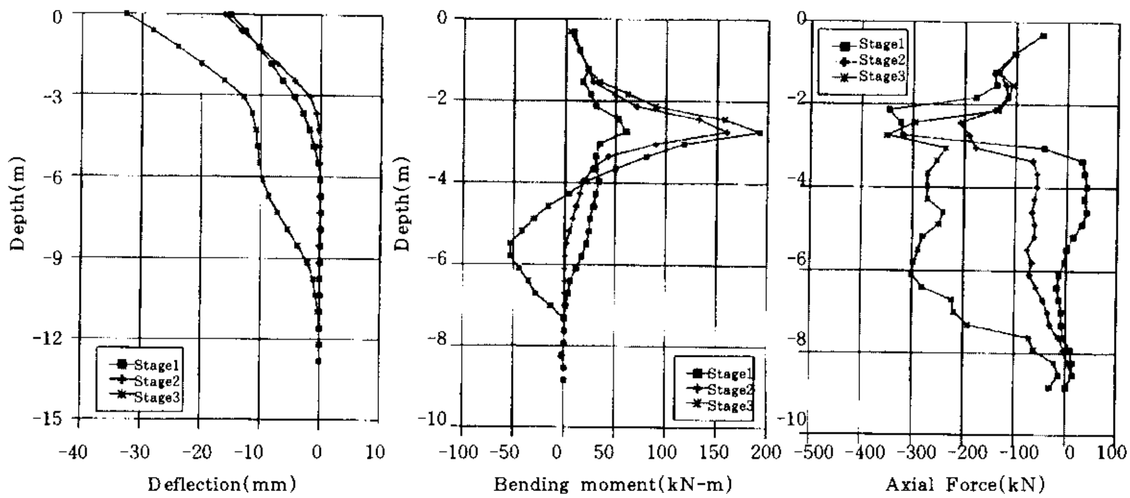


Fig.6 Deflection, bending moment and axial force of drilled one row anchored wall(Beam No.13)

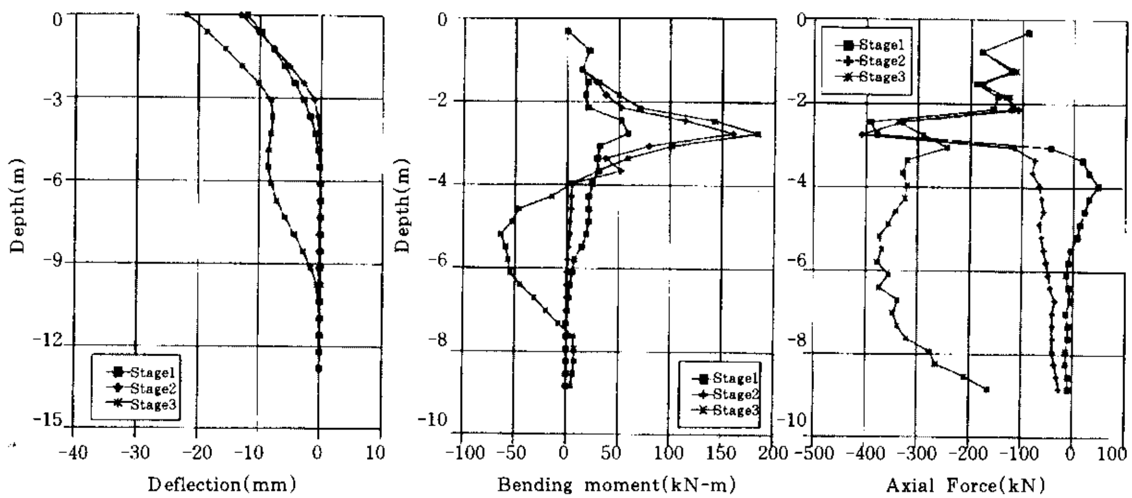


Fig.7 Deflection, bending moment and axial force of drilled one row anchored wall(Beam No.14)

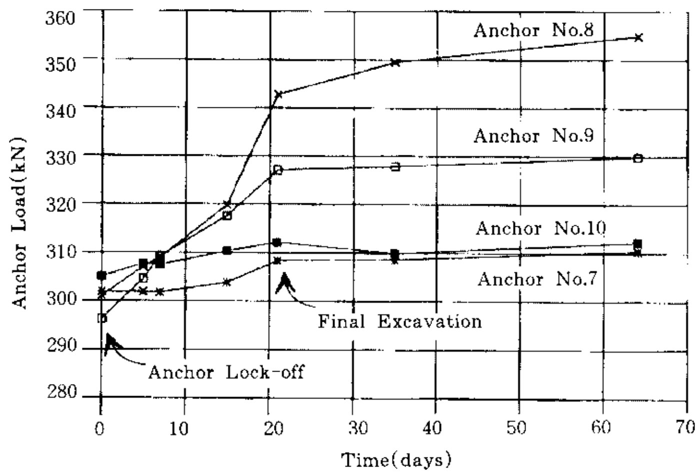
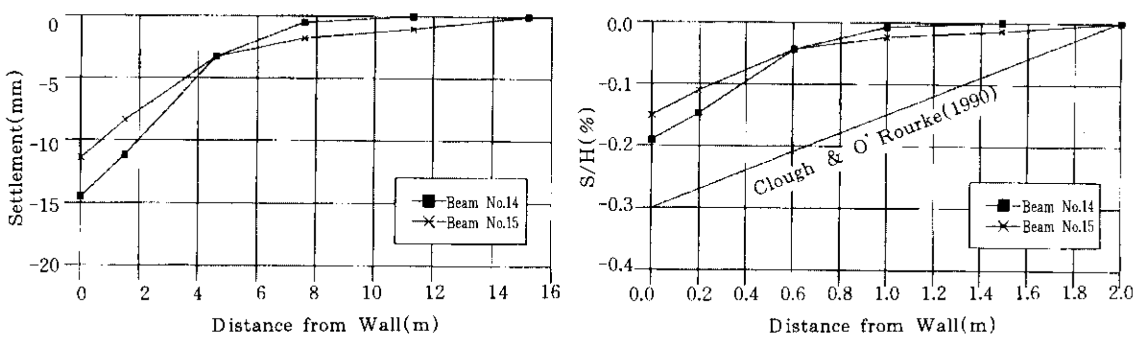


Fig.8 Anchor load variation of one row anchor wall



(a) Measured settlement

(b) Normalized settlement

Fig.9 Settlement of one row anchor wall

5.2 Two Row Anchored Walls

The two row tieback wall is made of 4 instrumented soldier beams. Beams 7 and 8 are driven beams and Beams 9 and 10 are drilled beams. The location of the soldier beams is shown in Figure 2. The bending moments, the axial forces and the horizontal deflections in the driven beam numbers 7 and 8 at each construction stage are shown in Figure 10 and Figure 11. The performance of the drilled beams in the two row anchor wall is presented in Figures 12 and Figure 13.

The measurements of anchor load in the two row tieback wall with time are presented in

Figure 14. The tiebacks were locked off at 364.7kN(82.2kips) or approximately 75% of the design anchor load of 473.7kN(106.5kips) for upper anchors and 427 kN(96kips) or approximately 320.3kN (72.0kips) for lower anchors.

The settlement of the ground surface of the two row tieback wall for the final construction stage is presented in Figure 15a. The normalized plot of the settlment is presented in Figure 15b in terms of the excavation height.

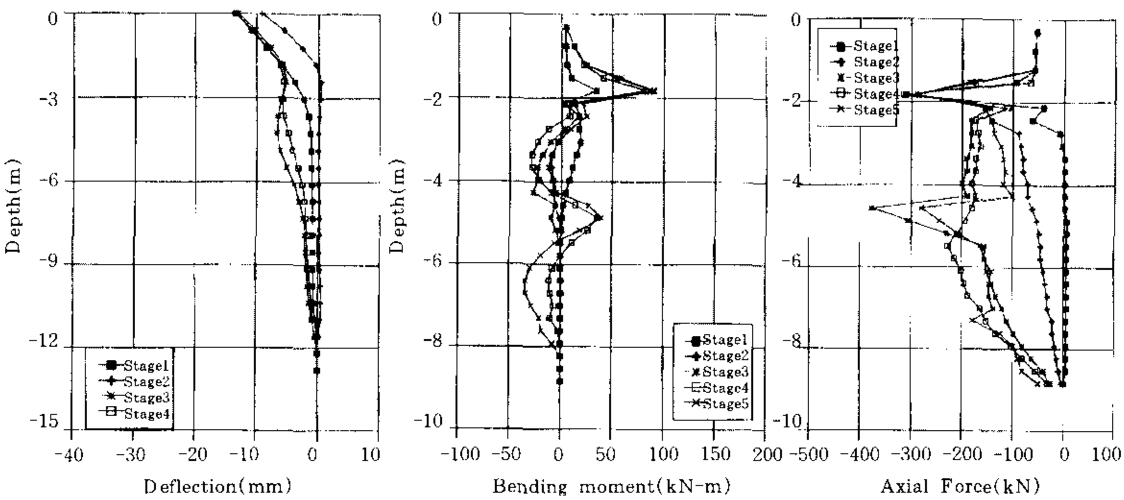


Fig.10 Deflection, bending moment and axial force of driven two row anchored wall(Beam No.7)

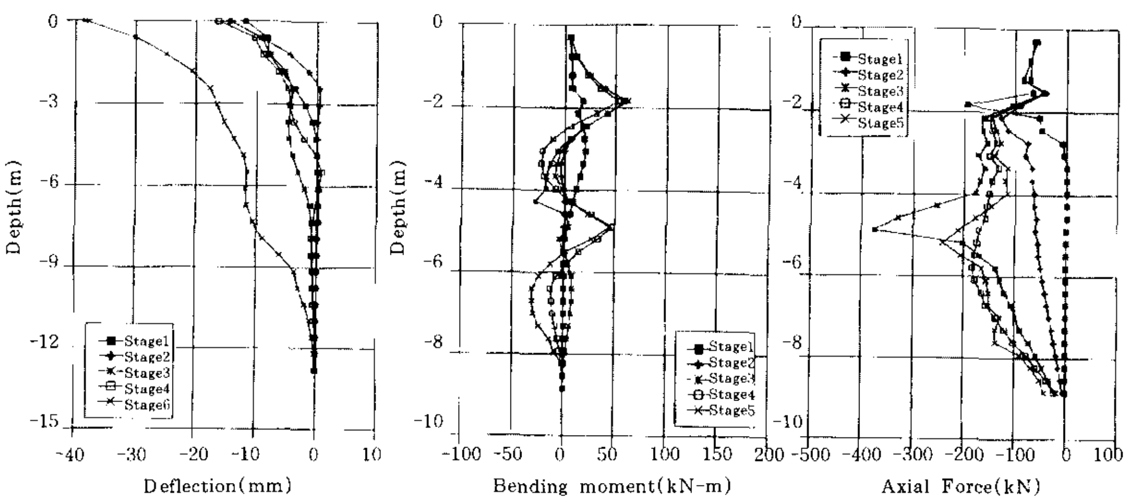


Fig.11 Deflection, bending moment and axial force of driven two row anchored wall(Beam No.8)

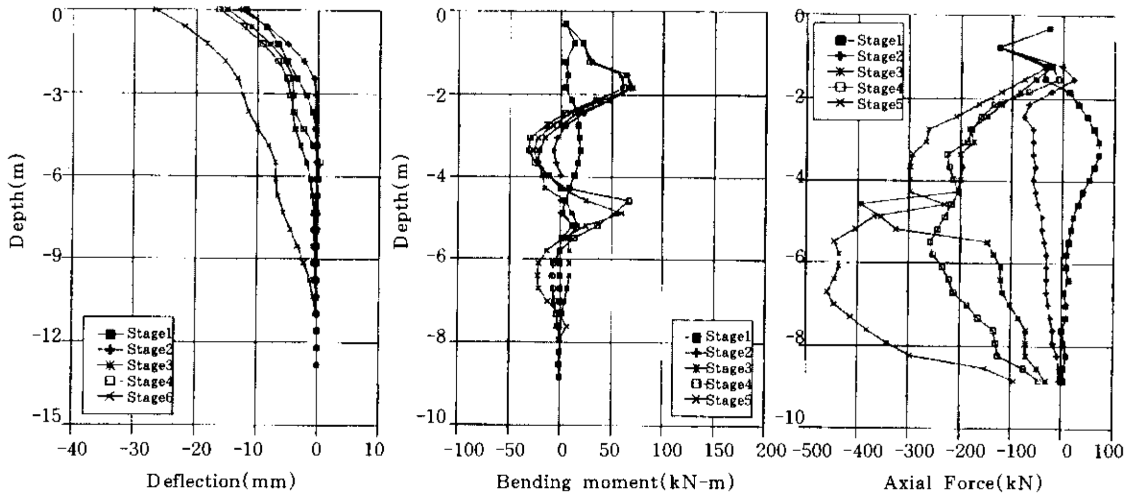


Fig.12 Deflection, bending moment and axial force of driven two row anchored wall(Beam No.9)

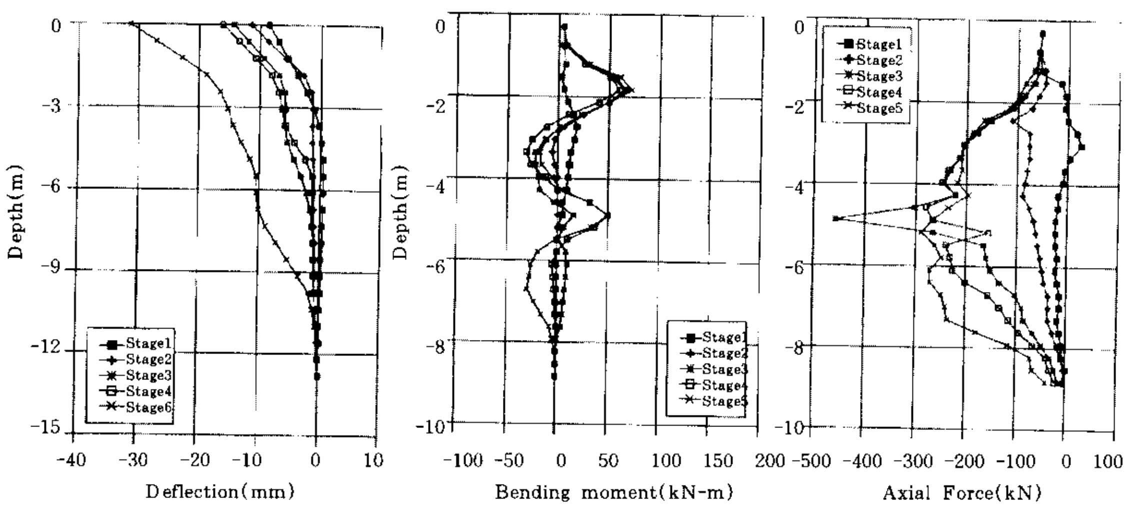


Fig.13 Deflection, bending moment and axial force of driven two row anchored wall(Beam No.10)

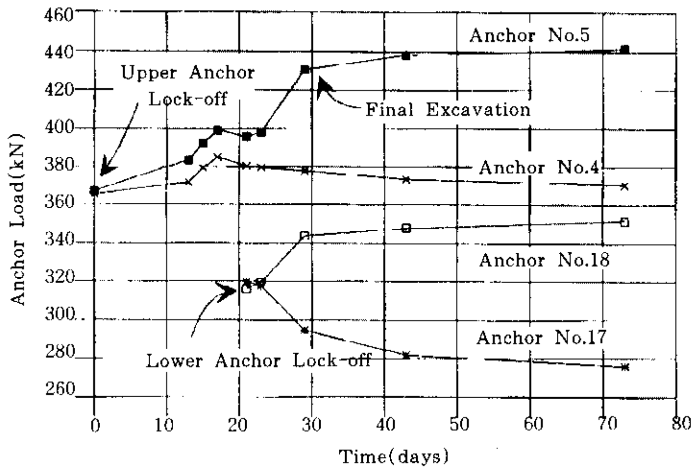
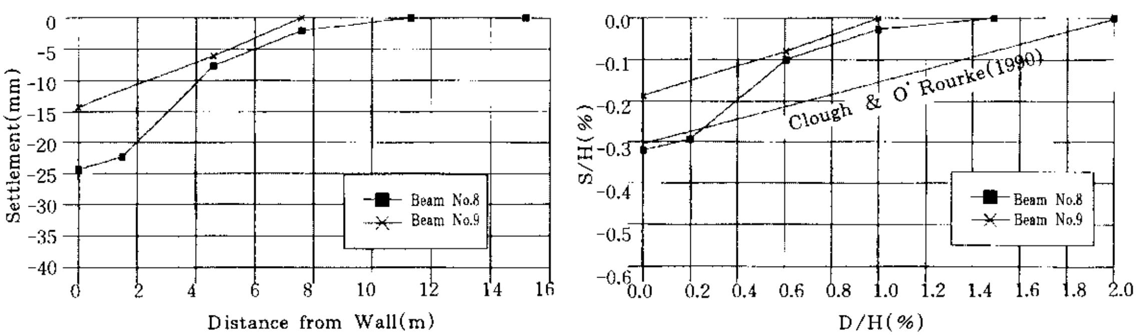


Fig.14 Anchor load variation of two row anchor wall



(a) Measured settlement (b) Normalized settlement

Fig.15 Settlement of two row anchor wall

6. Discussion of Results

6.1 Horizontal Deflections

The maximum lateral deflections of the one row anchor walls were measured and varied from 19.1mm(0.75inches) to 31.8mm(1.25inches) at the top of the beams. The average lateral deflection of the one row anchor wall was 22.9mm(0.9inches). The maximum lateral deflections of the two row anchor wall were measured and varied from 26.4mm(1.04inches) to 38.4mm

(1.51inches) at the top of the beams. The average deflection of the two row anchor walls was 32.0mm(1.26inches).

The lateral deflection of supported walls has been related to the stiffness of the wall which is the inverse of the flexibility of the retaining wall (Goldberg, et al, 1976). The flexibility of the retaining wall system can be defined by Rowe's flexibility number (Rowe, 1952) as follows,

$$\rho = \frac{L^4}{EI}$$

where L is the distance between supports or between an anchor and an excavation bottom grade, and EI is the lateral stiffness of the wall. With an analogy of simply supported beam with overhang, the deflection of the wall depends on the flexibility number. The Rowe's flexibility numbers for one row anchor and two row anchor wall are $1.6 \times 10^{-2} m^2/kN (1.1 \times 10^{-2} m^2/lb)$ and $7.5 \times 10^{-3} m^2/kN (5.2 \times 10^{-2} in^2/lb)$ respectively.

Based on Rowe's flexibility number, the one row anchor wall is more flexible than the two row anchor wall. However, the lateral deflection of the one row anchor wall was smaller than the deflection of the walls. Therefore it is not always true that more flexible wall has more lateral deflection in tieback walls in sand.

6.2 Bending Moments

The maximum bending moments of the one row anchor walls varied from 158.2kN-m (1.4million lb-in) to 192.0 kN-m (1.7million lb-in) at the anchor locations. The average bending moment of the one row anchor walls was 172.9 kN-m (1.53million lb-in). The maximum bending moments of the two row anchor walls varied from 65.5 kN-m (0.58 million lb-in) to 90.4 kN-m (0.8million lb-in) at the topmost anchors. The average bending moment of the two row anchor walls was 73.4kN-m (0.65million lb-in).

The maximum bending moments occurred at the anchor location for the one row anchor wall and at the upper anchor location for the two row anchor walls. It is quite common in practice that the design bending moment may be calculated with an analogy of simply supported beam with overhang. The measured bending moments were within the calculated moment of 249kN-m (2.2million lb-in) for the one row wall and of 110kN-m(0.97million lb-in) for the two row wall.

It can be concluded that the analogy of simply supported beam for the design bending moment calculation is an appropriate method and the shorter distance between supports or between an anchor and an excavation bottom grade causes the smaller maximum bending moment of anchored walls in sand.

6.3 Settlement

The settlement behind the wall for the one row anchor wall and for the two row anchor wall is shown in Figure 9 and 15, respectively. The maximum settlement of the ground behind the one row anchor wall occurred at the wall face and was approximately 14.7mm or 0.58inch

(0.2percent of the excavation height) in drilled beam section and approximately 11.4mm or 0.45inch (0.15percent of excavation height) in driven beam section.

The maximum settlement of the ground behind the two row anchor wall occurred at the wall face and was approximately 14.2mm or 0.56inch (0.2percent of the excavation height) in the drilled beam section. The maximum settlement of the ground behind the two row anchor wall occurred at the wall face. and was approximately 24.4mm or 0.96inch (0.3percent of the excavation height) in the driven beam section.

The settlements at the top of Beam No.8 (driven) and Beam No.9 (drilled with a structural toe) of the two row anchor walls were 24.4mm (0.96inches) and 14.2mm (0.56inches), respectively. The settlement of the two row anchor wall with the driven soldier beam was 1.7 times larger than the settlement of the wall with drilled beams which used a structural concrete at the toe of the beam.

Therefore, the proper toe design of the drilled soldier beam may reduce the settlement of the wall by resisting the axial component of the anchor load and downdrag force. And also, since anchors are commonly inclined, the settlement of the wall face due to the verticai component of anchor load causes the additional lateral deflection of the wall. The mechanism is shown in Figure 16.

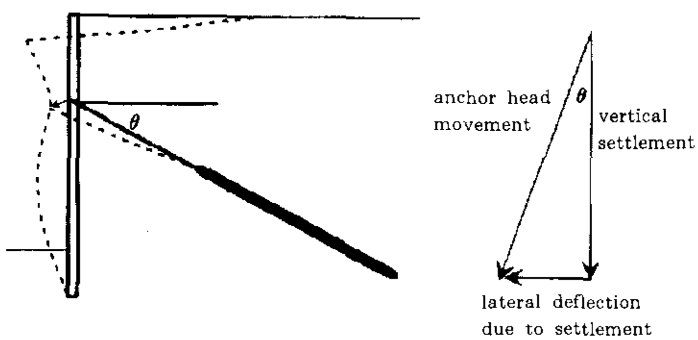


Fig.16 Lateral deflection due to the settlement

6.4 Axial Loads

The maximum axial loads of the one row anchor walls varied from 266.9kN(60kips) to 378.1kN(85kips) and occurred between the anchor and the excavation level. The average axial load of the one row anchor walls was 315.8kN(71kips). The maximum axial loads of the two row anchor walls varied from 373.6kN(84kips) to 467.0kN(105kips) and occurred between the lower anchor and the excavation level. The average axial load of the two row anchor walls was 418.1kN(94kips).

In the axial load distribution of Beam No.16 shown in Figure 6, the maximum axial load measured at the final construction stage was about 267kN(60kips) right below the anchor. The actual anchor lock-off load was 302kN(68 kips) which gives 151kN(34kips) of vertical load. The additional portion of this vertical load may come from the downdrag of the soil on the wall.

If the pile is placed in a soil where there is no downdrag movement of the soil mass, the axial load distribution will be due only to the vertical component of the anchor load as shown in Figure 17(a). As the excavation proceeds, the soil mass moves toward the excavation and downward. Since the pile is much more rigid than the soil, the soil mass moves downward more than the pile and downdrag takes place on the pile as shown in Figure 17(b). It shows the axial load from downdrag only.

If the anchor is stressed in the second construction stage, the axial load on the pile increases as much as the vertical component of the anchor lock-off load as shown in Figure 17(c). In the case of a multi-anchor wall, the probable axial load distribution is shown in Figure 17(d).

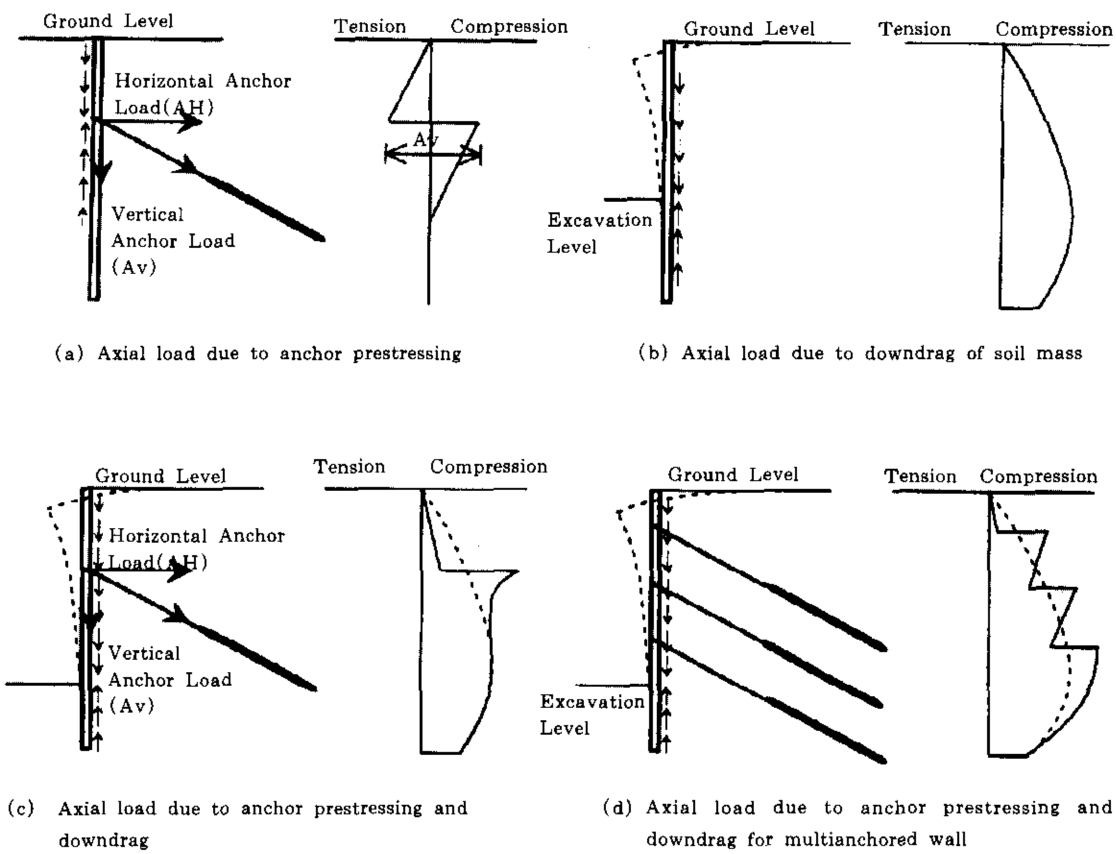


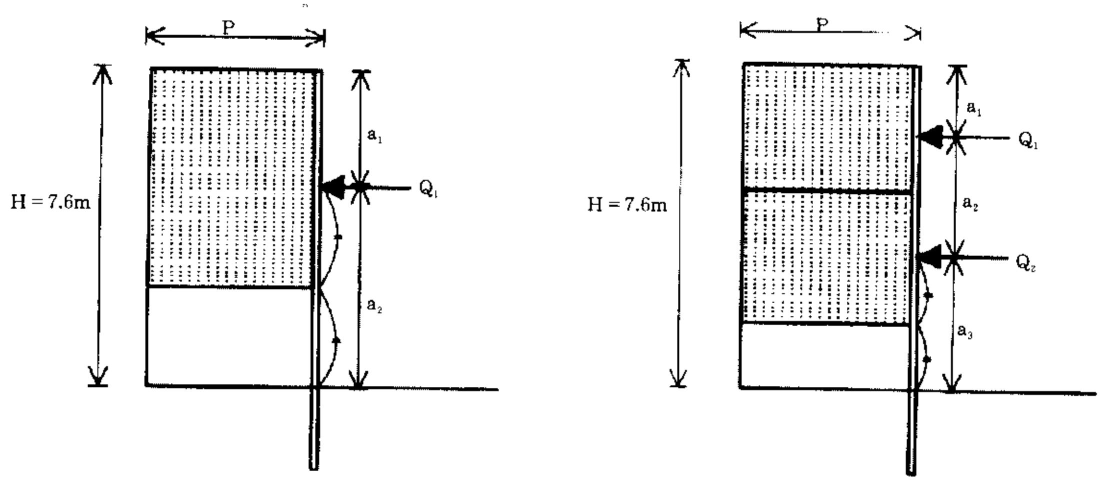
Fig.17 Axial load due to anchor pretressing and downdrag

6.5 Anchor load Variations

The lock-off load on one row anchor wall was approximately 301 kN(67.7kips), 261 kN (58.6 kips) of horizontal force at the stage of 3m(10feet) excavation. As excavation proceeds up to 7.6m(25feet), anchor loads were increased up to an average of 322kN(72.5kips), 279kN(62.8 kips) of horizontal force. The lock-off load on the two row anchor wall was approximately 367 kN(82.4 kips), 317kN(71.3kips) of horizontal force and 318kN(71.5kips), 275kN(61.9kips) of horizontal force for the upper anchor and lower anchor respectively at the stage of 3m(10feet) excavation. As excavation proceeds up to 7.6m(25feet), the anchor loads were increased except the lower anchor for driven beams (Tieback No.17) up to an average total anchor load of 404kN(90.9kips), 350kN(78.kips) of horizontal force and 319kN(71.8kips), 277kN(62.2 kips) of horizontal force for the upper and lower anchor respectively.

Anchor loads decreased with time on tieback No.4 after final excavation and on tieback No.17 after lock-off. The probable reasons may be that the lock-off loads were higher than what it should be and the clayey pockets or lenses of clay existing around the anchor bonded zone cause the creep behavior of anchors.

The apparent earth pressures were developed from the load measurements on anchors for the one row anchor wall and the two row anchor wall with the same analogy by Peck (1942) and by Terzaghi and Peck (1967) and are shown in Figure 18. The apparent earth pressures are 82 and 77percent of the earth pressure by Terzaghi and Peck for the one row and two row anchor walls respectively. These results are slightly higher than the most probable earth pressure of 75 percent of the apparent earth pressure by Terzaghi and Peck.



$$P = \frac{Q_1/S}{(a_1 + 1/2a_2)}$$

(a) One row tieback wall

$$P = \frac{(Q_1 + Q_2)/S}{(a_1 + a_2 + 1/2a_3)}$$

(a) Two row tieback wall

Fig.18 Apparent earth pressures from anchor load measurements

7. Conclusions

An instrumented full scale tieback wall was built at the sand site of the National Geotechnical Experimentation Site. The behavior of the wall was measured at each construction stage. The measurements consisted of the lateral deflection profile of the wall, the bending moment profile for the wall, the settlement of the ground behind the wall, and the axial load distribution in the soldier beams.

1. The more flexible anchored wall defined by Rowe's flexibility number does not necessarily have more lateral deflection on anchored wall in sand.
2. The shorter distance between supports or between an anchor and an excavation bottom grade causes the smaller bending moment of anchored walls in sand. The analogy of simply supported beam with overhang is appropriate to calculate the initial design bending moment.
3. The proper toe design of the anchored wall can reduce the settlement of the wall member and lateral deflection of the wall.
4. The axial load on the wall member of anchored wall is from the vertical component of the anchor force and from the downdrag force due to the settlement of the ground behind the wall.
5. The apparent earth pressures estimated from the anchor load measurements were compared to the apparent earth pressure for sand by Terzaghi and Peck. The apparent earth pressures were 82 and 77 percent of the earth pressure by Terzaghi and Peck for one row and two row anchored wall respectively.

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