한국지반환경공학회 논문집
제10권 제3호 2009년 4월 pp. 63～71

# The Analysis of Lateral Movement at the Top of Retaining Wall in the Downtown Area

# 도심지 옹벽 상단에서의 수평변위에 관한 사례분석

Bae, Yoon-Shin†
배 윤 신

**ABSTRACT :** The movement of in-situ walls has become very important as construction in large cities moves upward, instead of outward. Tall structures typically have deep excavations not on1y to provide extra space for parking, but also to reduce the potential settlement of the building. These large excavations require a robust bracing system to resist the lateral earth pressures as the depth increases. Methods to predict deflections of the retaining systems are of utmost importance because wall movements allow potentia1 settlement of adjacent structures. Case studies will be analyzed and measured wal1 def1ections will be compared with predictions from empirica1ly derived charts.

**Keywords :** Deep excavation, Retaining system, Settlement

**요 지 :** 대도시에서의 공사는 외부방향이 아닌 상부로 진행되기 때문에 현장의 벽면 움직임은 매우 중요하다. 고층 구조물은 주차장을 위한 여분의 공간 확보뿐 아니라 건물의 잠재적 침하를 줄이기 위하여 일반적으로 깊은 굴착을 수반한다. 이러한 대형 굴착은 깊은 심도에 따른 횡방향 지중압력에 견디기 위한 견고한 브레이싱 시스템을 필요로 한다. 벽체 움직임은 잠재적인 인접 구조물의 침하를 허용하기 때문에 sheetpile 이나 diaphragm wall과 같은 옹벽구조물의 변형을 예측하는 방법은 매우 중요하다. 사례들을 분석하고 측정된 벽체 변형은 경험적 도표로부터 예측된 값들과 비교되었다.

**주요어 :** 깊은굴착, 옹벽, 침하

## 1. Introduction

Permanent1y anchored (tied-back) earth retaining structures have been constructed in the United States since at least 1969. The principa1 types of structures are driven or pre-drilled steel soldier piles, reinforced concrete soldier piles, concrete diaphragm walls (slurry walls), or reinforced concrete slabs. Timber 1agging, facing, or some other method of horizontal reinforcement is also included (Pearlman and Wolosick, 1990). Documented failures of braced excavations have been seen in areas where soft, weak clay is the primary soil stratum at the base and 1ower portion of the wa1I (i.e. Islais Creek Basin in San Francisco). Fortunately, the Pacific Northwest area generally has competent soils at large depths that provide for stable retaining structures.

This paper discusses empirically based methods presented by C1ough and O' Rourke (1990) that can be used to predict horizontal wall movements of braced excavations in both stiff and soft soils. The charts introduced by C1ough and O' Rourke (1990) will be used to estimate wa11 deflections

Table 1. Case studies

| Building | Location | Type of structure | Depth of excavation |
|---|---|---|---|
| Pacific First Center | Seattle, WA | Tied back soldier pile and lagging wall | 22.86 m |
| Columbia Center | Seattle, WA | Tied back soldier pile and lagging wall | 34.14 m |
| Seattle First National Bank | Seattle, WA | Tied back soldier pile wall | 23.77 m |
| Bonneville Navigation Lock | Hood River, OR | Tied back reinforced concrete diaphragm wall | 15.24 m |
| Fox Tower | Portland, OR | Tied back soldier pile and lagging wall | 17.07 m |

† 정회원, 공학박사, 토질 및 기초기술사, 한국해양연구원 연안개발에너지연구부 연수연구원(E-mail : yoshbae@hanmail.net)

for case studies in the Pacific Northwest where wall movements were monitored. Comparisons wil1 be made between the predicted and measured values to determine the overall applicability of the empirically based methods to cases in the Northwest.

The case studies that will be presented are given in Table 1.

## 2. Design Procedure

The design of braced excavations must satisfy both the overall external stability of the retaining system as well as the stability of the internal structura1 elements. The horizontal deflection of the wall depends heavily on an effective design and efficient construction. Pear1man and Wo1osick (1990) outlined the general retaining wall design considerations

The most important part of the design procedure is the geotechnical investigation. Weak subsoils can 1ead to basa1 heave mat increases horizontal wall deflections sig1ificantly. If the soils are not properly identified and characterized, a wall failure could occur. The geotechnical investigation should include: 1) borings a maximum of every 30.5 meter along the wall; 2) borings behind the wall to identify anchor bond zone materials; 3) standard penetration testing in soils; 4) tests to determine soi1 strength and modulus parameters, such as triaxial shear, direct shear, consolidation, dilatometer and Menard pressuremeter; and 5) cores in rock where appropriate with tests to determine the shear strength of the rock (PearIman and Wo1osicki 1990).

## 3. Factor Affecting Wall Deflection

The horizontal movements of in-situ wall are a function of many factors. They include the soil and groundwater conditions, changes in groundwater level, depth and shape of excavation, type and stiffness of the wall and its supports, methods and quality of construction of the wall and adjacent facilities, surcharge loads, and duration of wall exposure (Clough and O'Rourke, 1990).

### 3.1 Maximum Movements in Soft and Medium Clays

When soils beneath the base of the excavation are soft and weak, basal stability may be an issue. A basal heave failure is a global rotational failure of the base of the excavation wa1l, usually as a result of low-strength soils. Horizonta1 wall movements are subsequently dominated by the deflections occurring as a result of basal heave beneath the excavation.

Figure 1 shows the methods of basal analysis. The factor of safety against basal heave can be calculated by the following equation:

$$FS = \frac{(C_u)_B N_c (0.7B)}{(\gamma H + q_s)(0.7B) - (C_u)_s H} \tag{1}$$

where $(C_u)_B$ and $(C_u)_s$ are the undrained shear strengths along the base and side of the defined fai1ure surface, respectively, $N_c$ is the basal stability factor as given in Figure 1, B is the width of the excavation, $\gamma$ is the total unit weight of the soil, $q_s$ is surcharge, and H is the depth of the excavation. The given equation is based on a uniform clay backfill.

The presence of sand layers would require a Mohr-Coulomb shear strength calculation to determine the shearing resistance of any present sand layers. The maximum lateral wall movements have been defined as a function of the factor of safety against basal heave in Figure 2. It also shows the inf1uence that the wall system stiffness and support spacing can have on movements. When the factor of safety falls be1ow 1.5, horizontal movements increase rapidly. However, when the factor of safety is above 2, and basal stability is

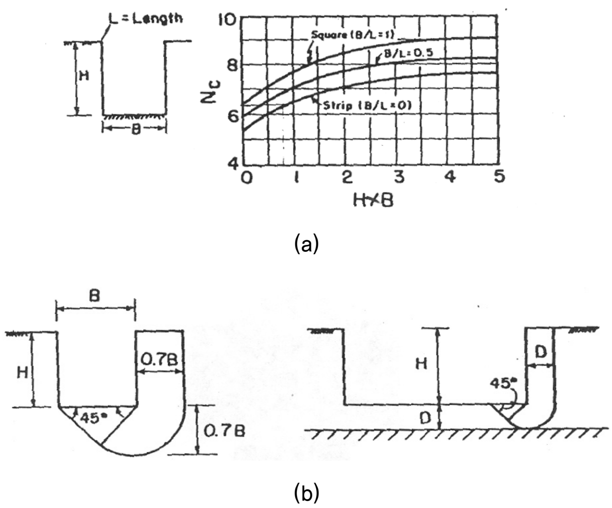


Fig. 1. Method of basal heave analysis: (a) for deep excavation with H/B>1 (b) for shallow of wide excavations with H/B <1 (Terzaghi, 1943)

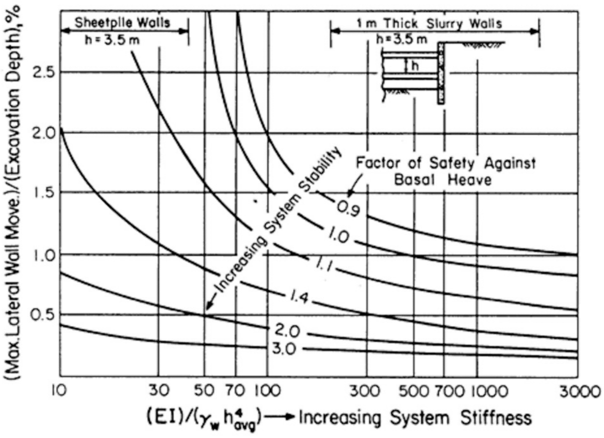


Fig. 2. Design curves to obtain maximum lateral wall movement for soft to medium clay (Clough and O'Rourke, 1990)

guaranteed, the maximum movements decrease below $\delta_m$/H = 0.5%. In conditions where the factor of safety is below 1.5, construction practices can have significant consequences on horizontal movements (Clough and O'Rourke, 1990).

## 3.2 Maximum Movements in Stiff Clays, Residual Soils, Sands

Basal stability issues rarely affect horizontal wall movements of braced excavations in stiff soils. All of the case studies analyzed in this paper had very strong, stiff soils at the base of the excavation and led to basal heave factors of safety greater than 2. Therefore, the calculations to predict the horizontal wall displacements for the case studies were based on the methods presented in this section. Peck (1969) showed through case study data that horizontal wall movement was usually less than 0.5% H in stiff soils. Figure 3 shows a plot developed by Clough and O'Rourke (1990) to show maximum horizontal wall movements as a function of the depth of the excavation. Some of the outlying data points show much larger movements with respect to depth. These are unique in that they were influenced by factors outside the basic excavation and support process.

When using this chart as a design tool to predict horizontal wall movements, it is difficult to predict on which of the two curves (0.2%H or 0.5%H) a particular project will fall. For those cases with movements that fall within the scatter around the 0.2%H trend line, the data follow an approximately linear relationship. This suggests that the soil masses are behaving approximately as elastic materials

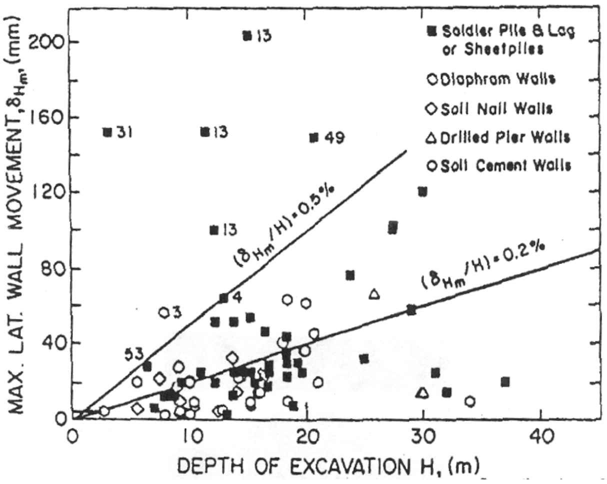


Fig. 3. Observed maximum lateral movements for insitu walls in stiff clays, residual soils, and sands (Clough and O'Rourke, 1990)

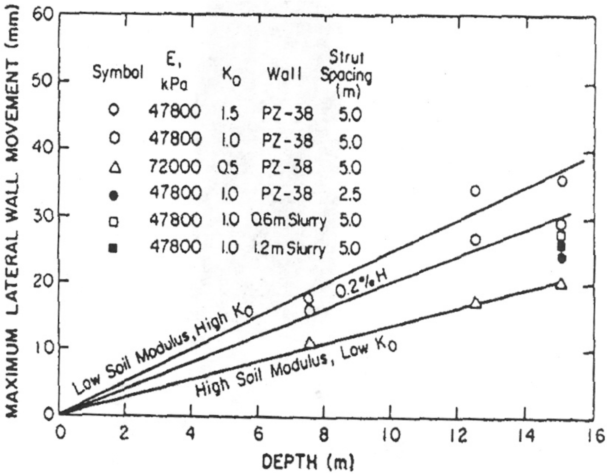


Fig. 4. Predicted maximum lateral wall movements by finite element analysis modeling stiff soil conditions (Clough and O'Rourke, 1990)

(Clough and O'Rourke, 1990). Using the assumption of elastic soil behavior, Clough and O'Rourke performed a series of finite element analyses with soil stiffness modulus values typical of those for stiff soils. Figure 4 shows that the predicted lateral wall movements followed an approximately linear relationship with depth centered on the 0.2%H trend line. They found that the predicted response was consistent with the average behavior shown in Figure 3. The wall stiffness and strut spacing were found to have only a small influence on predicted movements because the soil in these cases is stiff enough to minimize the need for the structure. However, soil modulus and coefficient of lateral earth pressure had a more significant impact (Clough and O'Rourke, 1990).

# 4. Pacific Northwest Case Studies

## 4.1 Pacific First Center

The Pacific First Center is a 44-story structure built in 1988 in Seattle, Washington. At the time of its construction, the Pacific First Center was one of the largest excavations ever in Seattle. The depth of the excavation reached a maximum of 25 meters and required removing more than 114,750 cubic meters of soil. City streets surrounded the building site on all sides except one where a 10-story building occupied a comer. The excavation was supported by a tied-back soldier pile and lagging wall (Winter, 1990).

### 4.1.1 Soil Profile

The soil profile at the site was estimated with the use of eight hollow stem auger borings; seven around the perimeter and one in the middle. The subsurface conditions were typical for that of the Seattle/Puget Sound area. The primary support soils are glacial in origin and consisted of lacustrine clay. The clay is heavily overconsolidated with Standard Penetration Test blow counts in the upper 20's to 40's and an undrained shear strength of 287.1 to 358.9 kPa as measured by both unconfined compression and unconsolidated undrained triaxial compression. The clay was overlain by clean to silty sand ranging from about 3 to 9 meter thick. SPT blowcounts in the sand were 45, with an estimated angle of internal friction of 38 degrees. Loose fill with scattered debris was present near the ground surface to a depth of 3 to 4.5 meter (Winter, 1990). Figure 5 shows the soil profile used in the wall deflection analysis.

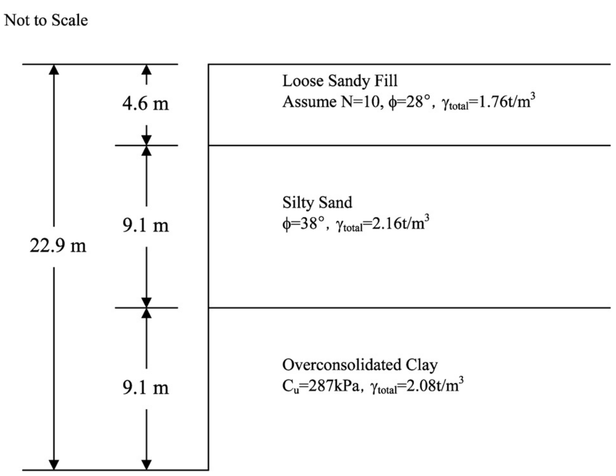


Fig. 5. Soil Profile for Pacific First Center

### 4.1.2 Shoring System Component

The retaining wall consisted of 101 soldier piles installed on a center-to-center spacing of about 3 meter. Each pile was a wide-flange section and extended about 7.6 meter beneath the base of the excavation. The tieback anchors were drilled at the center of the soldier piles at a vertical spacing of about 1.5 meter. The tight vertical spacing was mainly the result of high loads and street and adjacent property right-of-way restrictions. Finally, timber lagging 15.2 cm thick was installed between each pair of soldier piles (Winter, 1990).

### 4.1.3 Basal Heave Calculations

The soils at the site exhibited high shear strength and were stiff so basal heave was not believed to be an issue. However, a basal heave calculation was performed on this first case study to ensure this was the case. The calculation was completed according to the procedure presented earlier in this paper. The factor of safety against basal heave was just over 4, which guarantees basal stability. The rest of the case studies have similar soil conditions and will be assumed to be stable against a base heave failure. Therefore, the wall deflection estimates are simply made by applying Figure 3 to each case study.

### 4.1.4 Wall Deflection Analysis

The wall deflection was estimated using both the 0.2%H and 0.5%H curve on Figure 3 to see which had the best correlation to actual wall deflection. Figure 3 gives the maximum horizontal wall deflection that the wall would experience, but does not give any indication as to where that maximum would occur or the overall deflection profile.

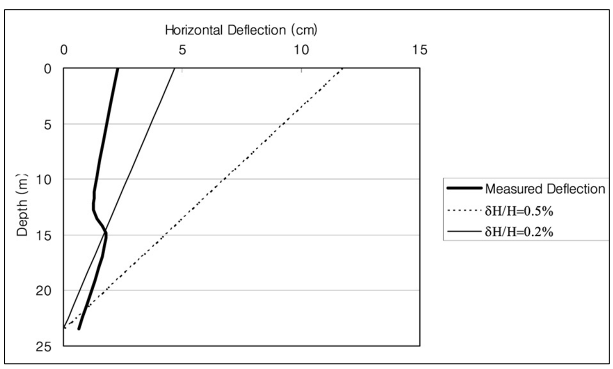


Fig. 6. Measured vs. predicted wall deflections for the Pacific First Center (depth of excavation = 23.5 m)

Based on the finite element analysis performed for stiff soils by Clough and O'Rourke (1990) shown in Figure 4, the maximum deflection was assumed to be at the top of the wall and decrease linearly to a value of zero at the base of the excavation. Figure 6 shows the predicted and actual wall deflection profiles for the Pacific First Center. The maximum predicted value was 4.8 cm for the 0.2%H curve and 11.9 cm for the 0.5%H curve. The actual maximum wall deflection was 2.3 cm as measured by inclinometers.

## 4.2 Columbia Center

The Columbia Center office building in Seattle is a 76-story structure that required a 34 meter deep excavation though very stiff fissured clay, sands and gravels, glacial till, and interbedded sand, silt, and clay. The site occupied all but the northwest corner of a city block except for a small 5-story structure in one corner. The retaining mechanism was a tied-back soldier pile wall with timber lagging (Grant, 1985). The site was very similar to that of the Pacific First Center previously discussed.

### 4.2.1 Soil Profile

The subsurface soils at the site are glacially consolidated and are very stiff. A total of 19 borings were made around the proposed structure to determine the soil profile. The bearing layer is a combination of stiff clay, silt, and sand with blow counts greater than 100. The bearing layer is overlain by a 7.6 meter thick layer of glacial till that also had blowcounts greater than 100. The uppermost layer is a very stiff clay which had blowcounts ranging from 40 to 70 (Grant, 1985). Figure 7 shows the soil profile used for the horizontal wall deflection analysis.

### 4.2.2 Shoring System Components

The retaining wall was composed of 82 soldier piles driven 3.96 meter on center-to- center spacing. The piles consisted of two 36.6 cm wide flange beams placed in a 1.2 meter diameter hole and filled with concrete. Wood lagging was installed between the soldier piles to retain the soil. Tiebacks were placed in me center of the soldier piles with vertical spacing ranging from 0.9 meter to 2.4 meter. The tiebacks had design loads ranging from 444 to 1,000 kN, depending on the earth pressures at each location (Grant, 1985).

### 4.2.3 Wall Deflection Anlaysis

Figure 8 shows the wall deflection profiles for both the predicted and measured values for the Columbia Center office building. The measured deflection profile was obtained through the use of deep inclinometers installed along the length of a soldier pile. The maximum predicted deflection was 0.07 meters for the 0.2%H curve and 0.18 meters for the 0.5%H curve. The actual maximum horizontal deflection was 0.015 meters (Grant, 1985).

## 4.3 Seattle First National Bank

The Seattle First National Bank, at the time of its construction (1969), was the city's tallest structure (50 stories) and required the deepest excavation ever at a depth of 23.8 m. The site is surrounded on all sides by city streets without

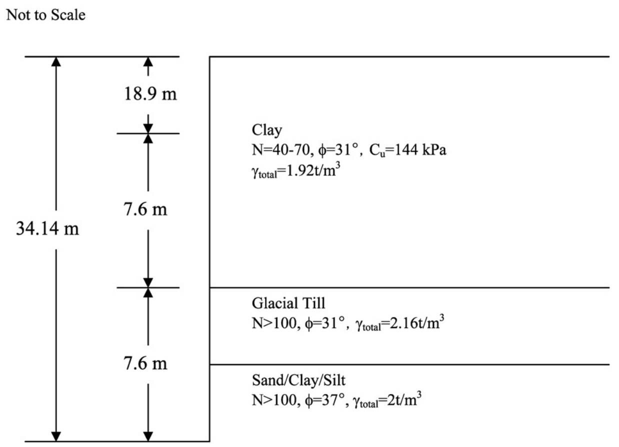


Fig. 7. Soil Profile for the Columbia Center

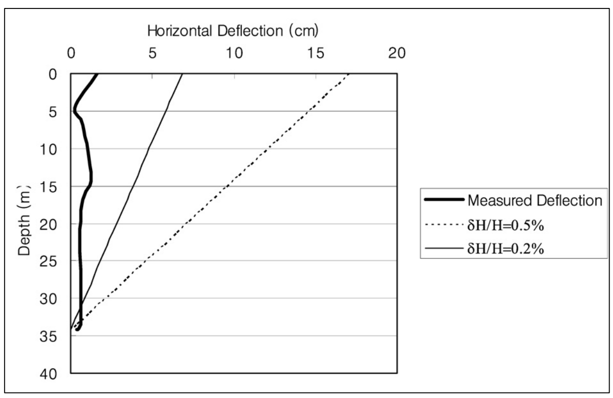


Fig. 8. Measured vs. predicted wall deflections for the Columbia Center (depth of excavation = 34.14 m)

any structures occupying space immediately adjacent to the excavation. The retaining wall was unique in that it was a tied-back soldier pile wall, but it did not use lagging. Instead, a binder material was applied in between the piles that retained the soil. Tied-back walls were just being introduced as replacements to rakers for deep excavations when the Seattle First National Bank was being constructed (Shannon & Strazer, 1970).

#### 4.3.1 Soil Profile

Based on 13 borings, the subsurface profile was determined as illustrated in Figure 9. Once again, the soil at this Seattle site was determined to be highly overconsolidated as the result of ice that was estimated to be 914.4 m thick during glaciation. Without specific strength data for the soils observed by Shannon & Strazer, assumptions were made regarding soil properties based on the case studies previously presented in this paper that were located in the Seattle area. The top 7.6 m of the excavation was characterized by a very stiff gray, silty clay. The underlying layer was a dense sand and the bearing material was a hard, gray, silty clay.

#### 4.3.2 Shoring System Components

The bracing of the excavation was done with the use of 312 soldier piles (8 WF 35) placed in 40.6 cm augured holes extending 1.5 m below the base of the excavation. The bottom 1.5 m was encased with structural concrete and the remaining depth was grouted with a sand-cement mixture. The piles were placed on 0.9 m center-to-center spacing leaving only 50.8 cm of exposed soil in between piles. A coating of binder was sprayed on the area in between piles to prevent spalling of the exposed soil. The binder performed well enough to eliminate the need for horizontal timber lagging during the 10- month excavation. Tie-backs were placed in the middle of a pair of soldier piles with anchor tension being transferred to the piles by a welded truss. Each tie-back had a design value of 427 kN (Shannon & Strazer, 1970).

#### 4.3.3 Wall Deflection Analysis

The measured wall deflections were from rectilinear extensometer and pile surface data. Shannon & Strazer do not provide a profile of horizontal wall deflection as a function of depth, but they simply mention that the face of the soldier pile moved the greatest (7.62 cm) at the top and decreased to approximately 3 cm over the bottom two-thirds of the wall. This information was interpreted into the rough horizontal deflection profile shown in Figure 10. The maximum predicted deflechon was 5.1 cm for the 0.2%H curve and 12.4 cm for the 0.5%H curve.

### 4.4 Bonneville Navigation Lock

The Bonneville Lock and Dam is located on the Columbia River, 67.6 km east of Portland, Oregon. In 1988, a new navigation lock needed to be constructed between an existing lock on the north and the Union Pacific Railroad's trans-continental rail line on the south (Munger et al., 1990). A braced excavation needed to be implemented because a typical open cut excavation would not be possible with the previously described obstacles. Both permanent and temporary

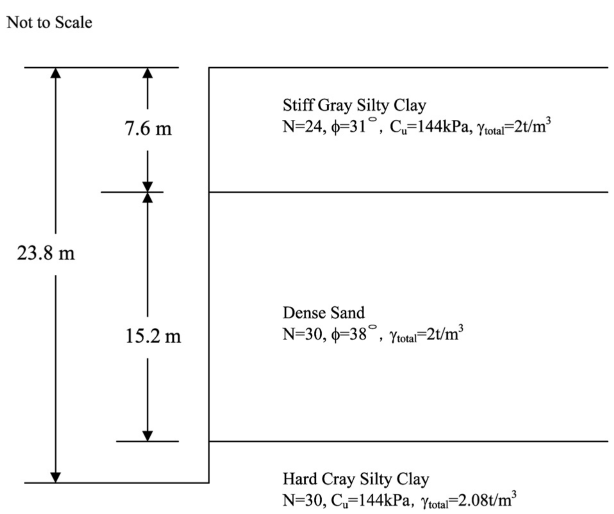


Fig. 9. Soil Profile for the Seattle First National Bank

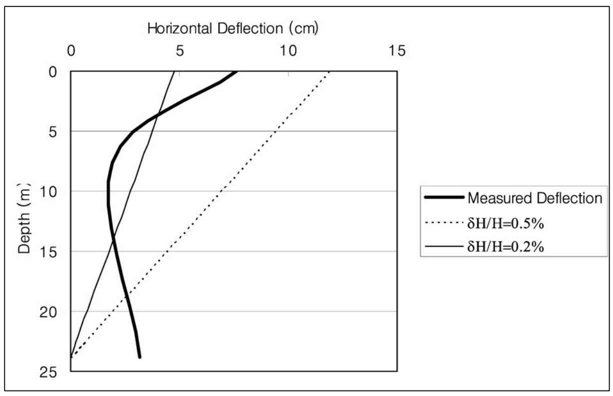


Fig. 10. Measure vs. predicted wall deflections for the Seattle First National Bank (depth of excavation = 23.8 m)

tied-back walls were built during this project, but only the temporary wall was installed to measure horizontal def1ections. The temporary 15.2 m high structure was a tied-back reinforced concrete diaphragm wall.

### 4.4.1 Soil Profile

The soils at the site are the result of massive ancient landslides, regional volcanics, and alluvial deposits from large floods. The top layer of soil is reworked slide debris and is approximately 6.7 m thick. It is a heterogeneous mixture of alluvial silts, sands, gravels, cobbles and boulders, mixed with angular rock fragments of old landslide masses. Beneath the slide debris is the Weigle Formation-a mixture of fine-gained, volcanic-derived, sedimentary rocks. At the base of the excavation is a diabase bedrock which certain1y removes the threat of basal heave. Figure 11 shows the soil profile.

### 4.4.2 Shoring System Components

The braced excavation consisted of a reinforced concrete diaphragm wall with a series of tie-back anchors. The wall was 0.9 m thick, approximately 14.6 m tal1, and consisted of nine 6.1 m long structural panels. Four tie-back anchors were placed at a vertical spacing of 3.35 m over the height of the wa11. The allowable design loads for the tendons ranges from 1,045.3 to 1,592.5 kN (Munger et al., 1990).

### 4.4.3 Wall Deflection Analysis

Horizonta1 wall def1ections were measured with embedded inclinometers, strain gauges on the wall of the reinforcement steel, tiltmeters, and horizontal multi-position bore hole extensometers (Munger et al., 1990). Figure 12 shows the estimated and measured def1ection profiles of the wall. The wa1l

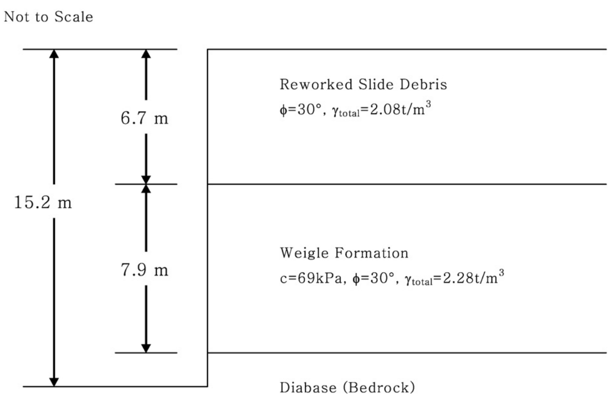


Fig. 11. Soil Profile for the Bonneville Navigation Lock

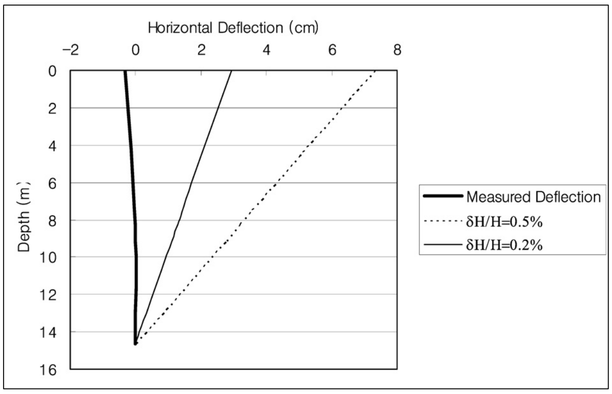


Fig. 12. Measured vs. predicted wall deflections for the Bonneville Navigation Lock (depth of excavation = 14.6 m)

actually moved into the soil as opposed to the excavation. The cause of this could be that the tie-back anchors were designed to resist earth pressures that were higher than actually present in the field. Much researches are needed for this movement. The maximum predicted def1ection into the excavation was 3.3 cm for the 0.2%H curve and 7.62 cm for me 0.5%H curve.

## 4.5 Fox Tower

The final case study is the Fox Tower located in Portland, Oregon. The excavation is one of the deepest in the downtown area extending to a depth of 17.1 m. The site occupies an entire city block on all sides and is surrounded by city streets. Large buildings stand on the opposite side of the streets at all locations. A tied-back soldier pi1e and lagging wall braced the excavation. The information presented in this case study is limited because the project is sti1l under construction.

### 4.5.1 Soil profile

The soil profile for the Fox Tower was presented in Figure 13. No information was obtained about the unit weight of soils. The base material is made up of gravel and cobbles in a matrix of sand and silt. This bearing layer is very stiff with blowcounts greater than 50. A sandy silt layer that has blowcounts ranging from 6 to 34 overlies the base material. The surface is a thin, 2.13 m deposit of si1t fi1l with blowcounts varying from 7 to 15. The values of soi1 properties used to calculate earth pressures were averages of a1l three layers. The unit weight of the soil was 18.84

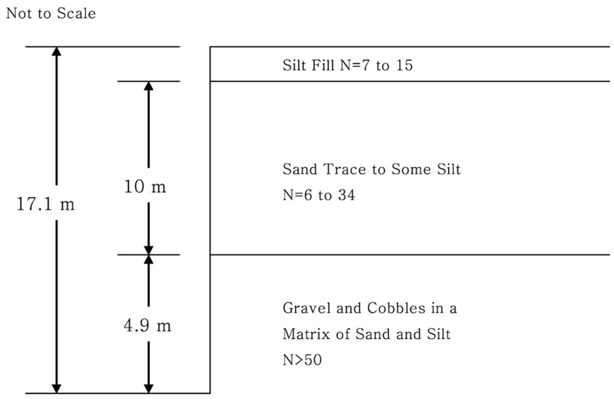


Fig. 13. Soil Profile for the Fox Tower

kN/m$^3$ and the angle of internal friction ($\phi$) was 33°.

### 4.5.2 Shoring System Components

The soldier piles (W16 x 67-89) were placed in 76.2 cm diameter, pre-drilled holes on 2.4 m center-to-center spacing. They were secured beneath the base of the excavation with structural concrete and above the base with a lean grout mix. Five tie-backs were used on each pile as anchors, and the distances between each varied. The loads on the tie-backs ranged 307 kN to 823 kN.

### 4.5.3 Wall Deflection Analysis

The estimated and measured wall deflection profiles are as shown in Figure 14. The actual wall deflections were obtained from an inclinometer that was placed in the middle of the north wall. The curve represents the cumulative horizontal deflection of the wall during the observation period. The maximum predicted deflection was 3.5 cm for the 0.2%H curve and 8.9 cm for the 0.5%H curve.

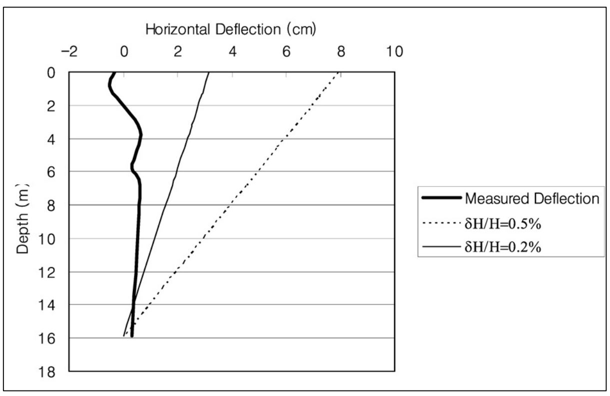


Fig. 14. Measured vs. predicted wall deflections for the Fox Tower (depth of excavation = 15.8 m)

## 5. Comparison between Measured and Predicted Wall Deflections for Case Studies

The chart presented by Clough and O'Rourke (Figure 3) for predicting the maximum horizontal deflection for braced excavations in stiff soils has generally overestimated the maximum movement. Figure 15 shows the location of the estimated maximum horizontal displacement of each case study as a function of excavation depth when plotted on a reproduction of Figure 3. All of the case studies experienced less maximum movement than predicted except for the Seattle First National Bank. The 0.2%H curve yielded the closest estimates while the 0.5%H curve greatly over-predicted deflection. Excluding the Seattle First National Bank case, the 0.2%H curve overestimated wall deflection by an average factor of five.

One explanation for the overestimated wall deflections could lie in the earth pressure and tie-back design load calculations. Typically, designers will estimate soil properties on the side of conservatism to incorporate a factor of safety. In particular, the angle of internal friction ($\phi$) used may be lower than what is actually present insitu. These low $\phi$ values result in larger design earth pressures and, in turn, larger design loads for the tie-back anchors. The tie-backs are locked off at design forces that are higher than actually needed which reduces the expected horizontal movement of the wall into the excavation. The Bonneville Lock project actually experienced movement of the wall back into the soil and away from the excavation.

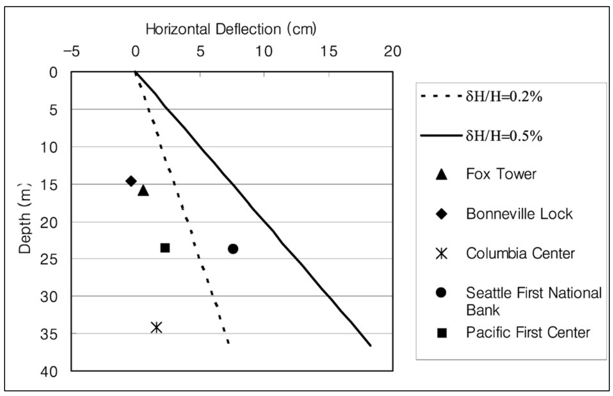


Fig. 15. Location of Pacific Northwest case study data points when plotted on the design chart by Clough and O'Rourke (1990)

In Figure 3, Clough and O'Rourke do not explicitly mention when the measurement of the wall deflection was initiated in the case studies used to develop the figure. If the measurement began as soon as the excavation began, excessive deflections could have occurred as a result of the construction practices on the project. If tie-backs or struts are not placed immediately, the wall could deflect much more than predicted. This figure could be skewed by wall deflections that occurred as a result of poor construction practice. On the other hand, the case studies analyzed for this paper also do not mention over what time in the construction process the wall deflections were measured. If the measurements commenced after the wall was built, the measured "net" deflection would be smaller than the cumulative value. If this hypothesis is true, it could partially explain the discrepancy between the measured and predicted deflections.

Finally, the predicted profile of the horizontal deflection of the braced excavations were assumed to decrease linearly with depth, following the profiles established in the finite element analysis performed on stiff soils by Clough and O'Rourke (Figure 4). However, the FEM calculation was done assuming a homogeneous soil profile over the depth of the excavation. One could expect an approximately linear deflection profile with the maximum occurring at the top of the wall if the soil stratum was uniform. The presence of non-uniform soil profiles behind the retaining structures would have led to a non-linear earth pressure distribution. This could lead to the random deflection profiles measured in the case studies.

## 6. Conclusion

A general design procedure for braced excavations was presented along with factors affecting the horizontal deflections of the walls. Design charts for predicting the horizontal defections were presented, and their applicability to Pacific Northwest case studies was analyzed. The design charts were shown to overestimate horizontal wall movement by a factor of five and many researches are required for an optimized design chart. However, they do provide a useful, conservative estimate of the maximum wall deflection. The procedure is very quick and is a viable alternative to more expensive and time consuming finite element analyses.

## References


1. Clough, G. W., and O'Rourke (1990), Construction Induced Movements of In-situ Walls. *Geotechnical Special Publication* #25: Design and Performance of Earth Retaining Structures. pp. 439～470.
2. Grant, W. P. (1985), Performance of Columbia Center Shoring Wall. *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*. Volume 4, pp. 2079～2082.
3. Munger, D. F., Jones, and Johnson (1990), Temporary Tieback Wall, Bonneville Navigation Lock. *Geotechnical Special Publication* #25: *Design and Performance of Earth Retaining Structures*. pp. 778～794.
4. Pearlman, S. L., and Wolosick (1990), Anchored Earth Retaining Structures-Design Procedure and Case Studies. *Geotechnical Special Publication* #25: *Design and Performance of Earth Retaining Structures*. pp. 525～539.
5. Shannnon, W. L., and Strazer (1970), Tied-back Excavation Wall for Seattle First National Bank. *Civil Engineering*. Volume 40. Number 3, pp. 62～64.
6. Winter, D. G (1990), Pacific First Center-Performance of the Tieback Shoring Wall. Geotechnical Special Publication #25: Design and Performance of Earth Retaining Structures. pp. 764～777.
7. Terzaghi, K. (1943), *Theoretical Soil mechanics*, John Wiley & Sons, New York.
8. Peck, R. B. (1969), Advantages and Limitations of the Observational Method in Applied Soil Mechanics. *Geotechnique*, Vol. 19 (2), pp. 171～187.


(접수일: 2009. 2. 23 심사일: 2009. 2. 27 심사완료일: 2009. 3. 13)