

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Performance of Columbia Center shoring wall

Fonctionnement d'étré sillonnement du mur de Columbia Center

W. PAUL GRANT, Senior Principal Engineer, Shannon & Wilson, Seattle, Washington, USA

SYNOPSIS This paper presents the geotechnical design recommendations and observed performance of a 37 m deep tieback shoring wall for the 76-story Columbia Center office building in Seattle, Washington, USA. The building excavation encountered very stiff fissured clay, sands and gravels, glacial till, and interbedded sand, silt, and clay. The geotechnical design of the shoring wall was complicated by the great depth of the excavation, large loads on the soldier piles, and physical constraints of the urban site. A finite element study was performed to evaluate the wall performance considering restricted anchor lengths on two sides of the excavation and nearby structures. The wall was instrumented at selected locations to monitor loads on the tiebacks and soldier piles, as well as movement of the soldier piles. Actual wall performance was better than predicted. Specific conclusions were reached regarding the no-load zone, apparent earth pressures, anchor spacing, and finite element studies.

INTRODUCTION

Tieback shoring walls are a popular means of temporary excavation support as this system provides an unobstructed working area within the excavation. The design of tieback shoring walls is largely based on empirical procedures that have been successfully used on excavations with depths typically on the order of 10 to 20 m (Goldberg, et al., 1976).

Use of these empirical procedures on the 37 m deep Columbia Center tieback shoring wall was complicated by several factors. First, existing or proposed building development along two sides of the excavation precluded use of the conventional no-load zone for anchor construction. Secondly, we questioned the applicability of the empirical design procedures, as little performance data exist for tieback shoring walls with depths in excess of 30 m. Additionally, very little data exist on the behavior of shoring walls in very stiff, fissured clays (Goldberg, et al., 1976) such as the soils existing at Columbia Center. Finally, details of the soldier pile and tieback layout created high vertical loads on the soldier piles.

Innovative solutions were required to develop design recommendations considering the above limitations. A modified no-load zone was developed and used to permit anchor construction in zones where tieback lengths were restricted by nearby development. A finite element study was performed to evaluate wall performance considering the restricted anchor lengths and adjacent development. The following discusses our design recommendations and the measured performance of the shoring wall.

PROJECT AND SOIL CONDITIONS

The Columbia Center project occupies all but the north-west corner of a city block (Figure 1) where the existing five-story Columbia House structure remains. The site is located on a hillside where the ground generally slopes downward from east to west, with surface eleva-

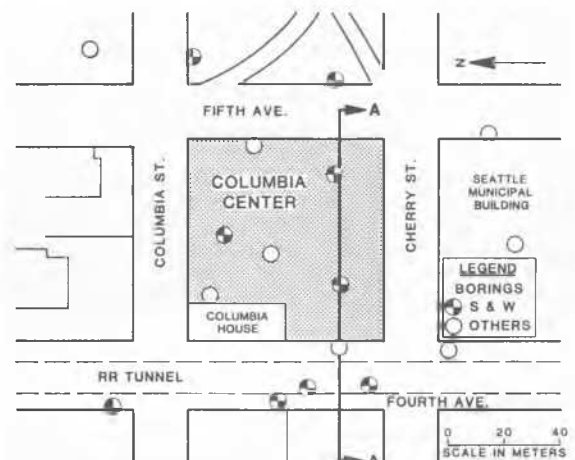


Figure 1 Site Plan

tions of 45 m to 48 m along Fifth Ave. and 32 m to 35 m along Fourth Ave. The base of the general excavation for the basement is at elevation 13 m. Local excavations were made to elevation 8 m for the three main 15 by 15 m footings for the building, effectively creating a 37 m deep excavation.

Subsurface soil conditions at the site have been generalized along an east-west cross section facing Cherry Street, as shown in Figure 2. Essentially all of the soils at the site are glacially consolidated with near horizontal bedding planes. Very stiff clays with some slickensides were typically encountered to elevation 29 m, overlying a 3 m thick stratum of outwash sand and gravel. Glacial till was encountered between elevations 26 m and 21 m, overlying interbedded sands, silts, and clays. The water table was encountered at elevation 8 m. Properties of the subsurface materials are summarized in Table I.

Disturbed soil conditions exist adjacent to the site at

TABLE I
Soil Properties

Soil	Elevation (m)	N-Value blows/30 cm	Water Content %	Liquid Limit	Plastic Limit	Wet Unit Weight kN/m ³	Effective Friction degrees	Effective Cohesion kPa	Elastic Moduli MPa
Clay	>29	40-70	20-40	55-75	23-30	18.9	31 ¹ 13 ¹	57.5 ¹ (peak) 47.9 ¹ (residual)	345 ³
Sand/Gravel	26-29	>100	4-12	--	--	21.2	40 ²	0 ²	585 ²
Till	21-26	>100	10-15	--	--	21.2	40 ²	0 ²	585 ²
Sand/Clay/Silt	<21	>100	10-30	31-51	29-40	19.6	37 ¹	0 ¹ (peak-silt)	585 ³

Notes: 1. Consolidated-undrained triaxial test.
2. Estimated.
3. In situ self-boring pressuremeter test.

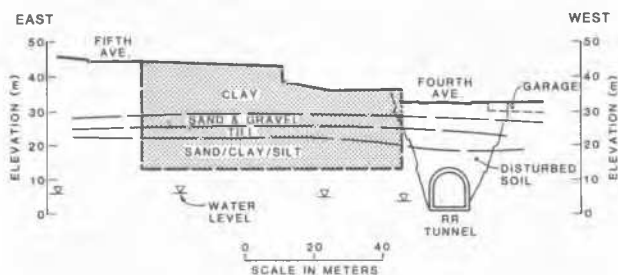


Figure 2 Subsurface Profile A-A

two locations. First, they exist above the railroad tunnel along Fourth Ave. as the streets in this area settled about 0.5 m following tunnel construction. Secondly, soils in the vicinity of Cherry Street and Fifth Ave. were disturbed as a result of an excavation failure during the construction of the Municipal Building.

SHORING WALL

In addition to providing temporary support during construction, the soldier piles of the shoring wall were later incorporated into the permanent basement wall as structural members. The majority of the 82 soldier piles installed at the site were spaced at 4 m intervals and consisted of two 36 cm wide flange beams placed in a 1.2 m diameter hole and filled with concrete. Wood lagging 10 and 15 cm thick was used to retain the soil between the soldier piles, except adjacent to Columbia House, where cast-in-place concrete lagging was used. Friction anchors, with a nominal diameter of about 31 cm, were used for approximately 90 percent of the tiebacks. Typical anchor design capacity was 670 kN, although capacities of 450 to 1,000 kN were specified in selected areas. Grouted anchor lengths were typically on the order of 6 to 9 m, although grouted lengths of up to 14 m were used along the west wall where disturbed soil conditions were anticipated. Vertical spacing of tiebacks on the soldier piles ranged from slightly under 1 m to as much as 2.4 m.

DESIGN PARAMETERS

Existing or proposed development opposite the east and south walls of the excavation precluded using a conven-

tional no-load zone for locating the tieback anchors. Figure 3 illustrates that use of a conventional no-load zone would result in anchors being located within the Municipal Building. Similar restrictions apply along Fifth Avenue. To resolve this situation, the modified no-load zone shown in Figure 3 was developed. This modified no-load zone was based upon failure planes assumed in the design and observed in experimental studies associated with reinforced earth retaining walls (McKittrick, 1978).

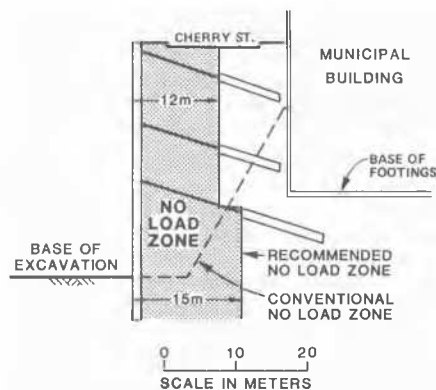


Figure 3 No Load Zone

Development of an apparent lateral earth pressure diagram for the Columbia Center shoring wall was complicated by the fact that little data exist for excavations with depths over 30 m and for excavations in very stiff fissured clay (Goldberg, et al., 1976). However, based upon our review of available literature, performance of other local tieback projects and conditions at the site, the two pressure distributions shown on Figure 4 were recommended for design. The higher pressure distribution was recommended for use along Fourth Ave., where the adjacent soils have been disturbed by construction of the railroad tunnel. Pressure distributions of Schnabel (1982) and Terzaghi and Peck (1968) are also presented on Figure 4 for reference. Compared to local projects, the Columbia Center shoring pressure is 30 percent lower than the value used for the nearby Sea-First Fifth Ave. plaza shoring wall. This is particularly significant considering the Columbia Center excavation is 12 m deeper.

The relatively large soldier pile spacing of 4 m com-

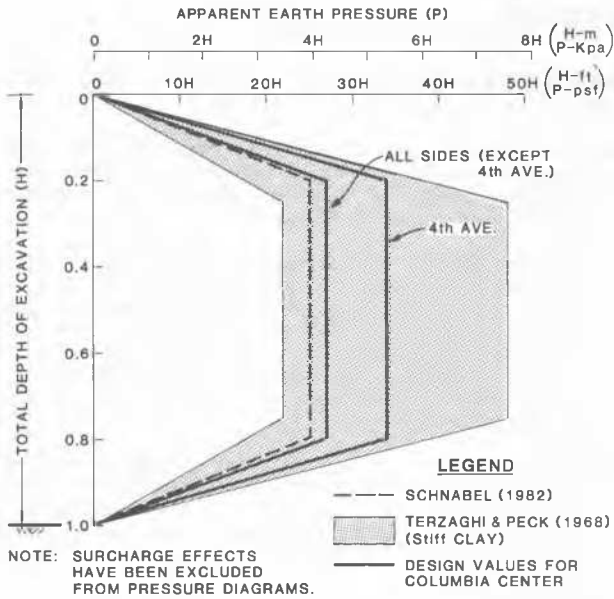


Figure 4 Apparent Earth Pressure Diagrams

bined with the 37 m depth of excavation resulted in 20 to 30 tiebacks being required on each soldier pile. These tiebacks, drilled at an angle of 20 degrees to the horizontal, produced vertical loads of about 5,500 kN on the soldier piles. This magnitude of loading is about four times greater than that normally experienced on local excavations about 15 m deep. It was recommended that the capacity of the soldier piles to resist these vertical loads be based on skin friction values of 95 to 145 kPa depending on soil types and on an end bearing resistance of 720 kPa. These parameters are typically twice the magnitude of values used on other local projects.

Estimates of horizontal and vertical movement of the shoring wall were made using empirical procedures and data gathered from the performance of other excavations in similar soils (Goldberg, et al., 1976) as well as from a finite element analysis. From the results of both the empirical and analytical studies it was estimated that both vertical and horizontal movement at the top of the shoring wall may amount to 2 to 5 cm.

INSTRUMENTATION AND MEASUREMENTS

As a number of design recommendations developed for the Columbia Center shoring wall departed from conventional practice, it was important to measure the performance of the wall during construction to identify potentially critical conditions, and to develop appropriate remedial measures on a timely basis.

The locations of geotechnical instruments installed on the shoring wall are indicated in Figure 5. Two soldier piles, E-10 on Fifth Ave. and S-14 on Cherry Street, were instrumented with deep inclinometers (extending 15 m below the base of the excavation) to measure horizontal deflection along the length of the soldier pile, load cells on each of the tiebacks to monitor the tendon load, and strain gages and extensometers to evaluate the vertical load reaching the tip of the soldier pile. These soldier piles were instrumented considering the depth of the excavation and potential influence of

nearby structures. Other inclinometers were installed to the base of the soldier piles at locations shown on Figure 5. Additionally, optical surveys were made to monitor the deflection and settlements of the soldier piles and adjacent sidewalks and streets.

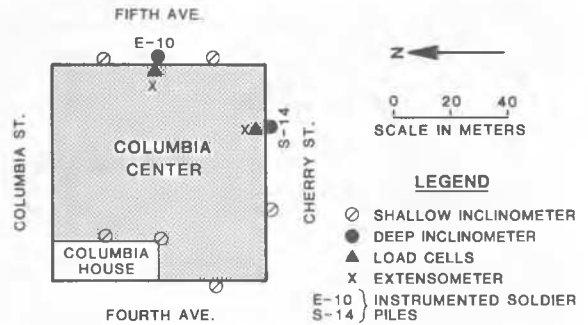


Figure 5 Geotechnical Instrumentation

Typical results from the instrumentation are presented in Figures 6 through 9. Plots of the tieback loads at various stages of the excavation are presented in Figure 6. It is noted that the tiebacks were locked off at 90 percent of the design load to provide a basis for assessing the conservatism of the design. The distribution of vertical load within soldier pile E-10 is presented in Figure 7. The total vertical load was based on the vertical component of the tieback load as calculated from the load cell readings. Strain gages located 1 m above the pile tip provided data for the tip load. The observed load distribution is generally consistent with the design values considering that skin friction was probably not fully mobilized along the pile, as total settlement at the pile top was less than 0.5 cm. Horizontal surface movements at the edge of the excavation, street centerlines, and sidewalks are shown on Figure 8 at the time the excavation was completed (3/83) and five months later (8/83). Finally, observed deformations and computed movements of soldier pile E-10 are shown in Figure 9.

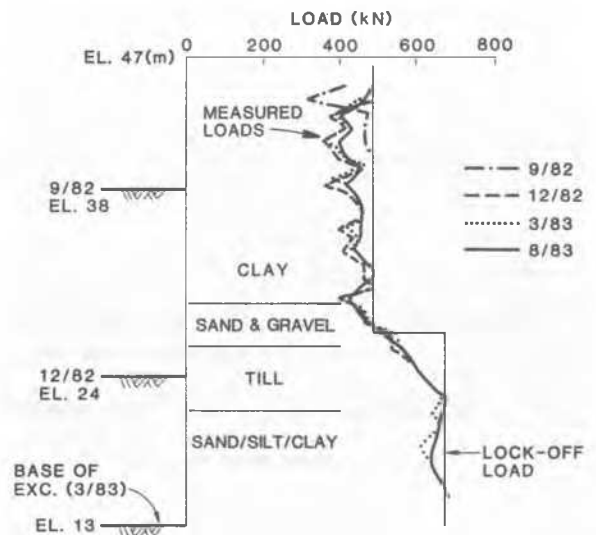


Figure 6 Tieback Loads on Soldier Pile E-10

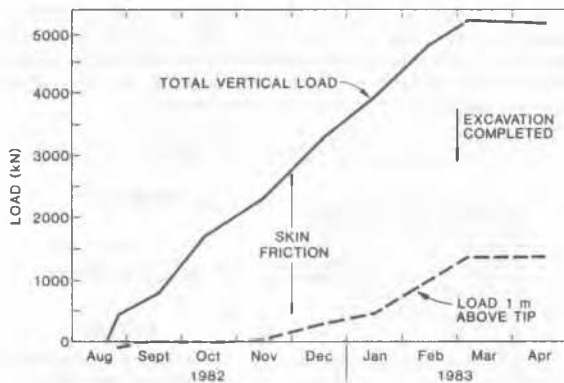


Figure 7 Vertical Load on Soldier Pile E-10

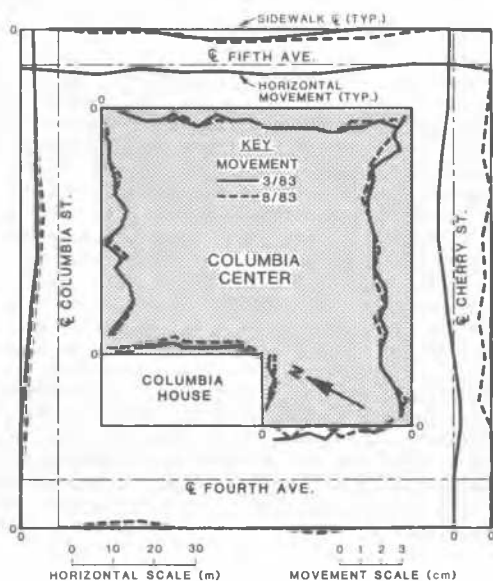


Figure 8 Horizontal Surface Movements

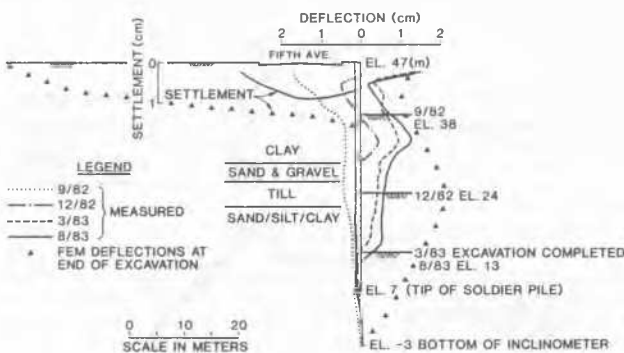


Figure 9 Wall and Street Movements

CONCLUSIONS

Based upon the data presented in Figures 6 through 9, we arrived at the following conclusions:

1. The modified no-load zone developed for the south and east walls of the excavation provided satisfactory stability for the shoring wall without excessive horizontal movement.
2. The recommended lateral earth pressure distributions for the shoring wall, although based on data for shallower excavations, provided satisfactory performance. In fact, the measured tieback loads were less than 90 percent of the design values, which is useful information considering the materials down to elevation 29 m consisted of very stiff clay with some slickensides.
3. The consistency of the load cell readings indicate that there was negligible group action or interaction among the tiebacks. This is particularly relevant considering that some of the anchors were constructed at spacings as close as 3 times the anchor diameter.
4. The observed deformations of the shoring wall and streets were typically half the values estimated from the empirical studies and the finite element analysis. Use of higher moduli values would have improved the predictions of deformation in both instances.

ACKNOWLEDGEMENTS

The author is grateful to Professor G. Wayne Clough and his graduate students at Stanford University for their assistance in performing the finite element study for the excavation. The author also thanks Mr. George Yamane of Shannon & Wilson in Seattle for his helpful comments and suggestions in his review of this paper.

REFERENCES

- Goldberg, D. T., Jaworski, W. E., and Gordon, M. D. (1976), *Lateral Support Systems and Underpinning*, FHWA-RD-75-128, Report to the Federal Highway Administration, Washington, D.C., Vol. I-III.
- McKittrick, D. P. (1978), *Reinforced Earth: Application of Theory and Research to Practice*, Symposium on Soil Reinforcing and Stabilizing Techniques, Sydney, Australia.
- Schnabel, H. (1982), *Tiebacks in Foundation Engineering and Construction*, 120 pp., McGraw-Hill, New York.
- Terzaghi, K., and Peck, R. B. (1968), *Soil Mechanics in Engineering Practice*, 2nd Ed., 729 pp., Wiley, New York.