

DESIGN OF DIAPHRAGM WALLS FOR THE TRTS DEEP EXCAVATIONS

by

L.W. Wong, H. Lin and B.C. Patron

*Reprinted from Proceedings of
Symposium of Taipei Metropolitan Rapid Transit Systems,
Vol.B, pp.319-328, Taipei, 1993*

Design of Diaphragm Walls for TRTS Deep Excavations

L. W. Wong — Project Manager, Moh and Associates, Inc.
H. Lin — Deputy Director, Division 2, DORTS
B. C. Patron — Geotech. Engineer, Moh and Associates, Inc.

Abstract

The diaphragm wall design of more than 100 sections of the Taipei Rapid Transit Systems are reviewed and analyzed collectively with emphasis on wall depth, strut reaction, wall bending moment and wall deflection. The subsoil conditions, methods of analysis, methods of construction, ground improvement and groundwater pressure assumptions used in the design are discussed.

台北捷運系統深開挖之連續壁設計

黃立煌 — 亞新工程顧問公司計畫經理
林軒 — 台北市政府捷運局第二處副處長
B. C. Patron — 亞新工程顧問公司大地工程師

摘要

本文針對為數超過 100 個連續壁設計斷面作分析及系統性整理，內容包括貫入深度、支撐載重、壁體彎矩以及最大撓曲等項目，並對土層狀況、分析方法、施工法、地盤改良以及地下水壓力作簡單描述。

I. Introduction

Excavation depths for stations, cut-and-cover tunnels and ancillary structures of the Taipei Rapid Transit Systems (TRTS) vary typically between 12 to 28 meters but with a maximum of about 36 meters. Except in a few locations such as the southern section of the Hsintien Line, diaphragm walls with internal bracings have been designed as the retaining structure.

II. Hydro-geological Conditions

The TRTS Network covers almost the entire Taipei Basin. The geology of the basin comprises the Sungshan Formation, with thickness of 40 to 70 meters, overlying the Chingmei Formation of up to 150 meters in thickness. The area underlain by the Sungshan Formation has been subdivided into subzones [1] as indicated in Fig. 1. The Nankang, Panchiao, Hsintien, and Chungho Lines are mainly aligned within the T2 and K1 Zones. The T2 area typically consists of alternating sandy and clayey deposits while the K1 area is predominantly composed of thick clayey soils.

Groundwater observations indicate that the water pressure distribution within the Taipei Basin is non-hydrostatic. This characteristic is mainly attributed to the interbedded nature of the subsoil deposits and the past deep well pumping in the 1950's till the mid-1970's. As indicated in Fig. 2, the water pressure beneath the clayey Layer 4 within the Central Taipei area is currently about 10 m lower than that in the upper soil layers. This groundwater distribution is advantageous to diaphragm wall design due to the lower net water pressure and higher passive resistance.

III. Design Considerations

A. Design Criteria

Diaphragm wall design for the TRTS basically follows the criteria provided in the Civil Engineering Design Manual (CEDM) [2]. The major requirements are summarized in Table 1. Surcharge loads, up to 140 KN/m², are considered where diaphragm walls are located next to future developments. In fact, building protection plays an important role in the design.

B. Soil-structure Interaction Analysis

Soil-structure interaction analysis is normally required to determine the diaphragm wall thickness as well as the reactions of bracings. In designing the diaphragm walls, the Detailed Design Consultants (DDCs) use either the semi-empirical approach of the beam-and-soil spring model or the finite element technique.

Five different computer programs have been used as indicated in Table 2. Although these programs have each been calibrated against actual data, variations in design have resulted because of differences in modelling the soil stiffness, water pressure distribution, wall friction or adhesion, lateral load distribution due to surcharge, method of handling boundary conditions, and others.

Due to the empirical nature of analyses, monitoring of strut loads and wall deflections has been considered an essential part of design. The observational approach, as suggested by Peck (1969) [3] has been integrated in the project.

C. Wall Friction and Adhesion Assumptions

Estimation of earth pressures have adopted wall friction particularly when the Coulomb or the Caquot and Kerisel solution is used. The subject of wall friction has been deemed as "an area of significant uncertainty" [4] in diaphragm wall design. Other authors [5] have observed that the apparent angle of wall friction may be higher than the soil friction angle. Due to lack of sufficient information on wall friction for excavations using diaphragm walls in local soils, the TRTS design has been carried out using different assumptions of wall friction as illustrated in Table 2. Most design lots have adopted a wall friction value equal to half the soil friction angle. In DL173 and DL181, where the total stress analysis have been used, the values of wall adhesion have been assumed to be 0.45 and 0.67 of undrained cohesion, respectively.

D. Effect of Groundwater Pressure on Wall Penetration Depth

Among other design considerations, it is worthwhile to mention the effects of groundwater on diaphragm wall design. For most cases in the TRTS, the design wall penetration depths are governed by toe stability, blow-in and seepage control. As illustrated in Fig. 3, walls are sometimes extended further down in order to cut-off seepage flow or lengthen the path of the flow.

IV. Results of Diaphragm Wall Design

Conforming with the requirements of the CEDM and integrating other design considerations, the thickness of TRTS diaphragm walls generally vary between 0.8 to 1.2 meters. Walls for ancillary excavations such as station entrances and vent shafts are thinner, i.e. about 0.5 to 0.7 meters.

Except in DL173 where the top-down method is used, and in DL159 and DL172 which adopt the semi top-down method, all other excavations supported by diaphragm walls use the bottom-up method. The bracing systems for diaphragm walls usually consist of steel struts made of prefabricated H-sections, specified with preloading of at least 50 percent of design loads. Vertical distances between struts generally vary between 2 to 4 meters. Horizontal spacings are normally in the range from 3 to 6 meters. In some special cases where building protection is an extremely critical concern such as the excavation beside the South Gate, buried struts composed of treated ground have been specified in order to limit settlements. In areas where the subsoil is composed of very thick deposits of soft clay such as in the K1 Zone, jet grout slabs or transverse concrete diaphragm walls below the excavation base have been adopted.

Results of diaphragm wall design based on about 100 analysis sections in various design lots have been summarized. The wall depth, strut reaction, wall bending moment and wall deflection are provided in Figs. 4 to 7. These figures show that the designs vary from one site to another due to variations in the subsoil and groundwater conditions and the different assumptions made by the DDCs. However, a considerably narrow range can be defined after a more detailed analysis. The following sections present the results of the analyses.

A. Wall Depth

The plot of wall depth, L versus excavation depth, H for TRTS excavations in relatively sandy soils (e.g. T2 Zone) and predominantly clayey soils (e.g. K1 Zone) are shown in Fig. 4. The general range of wall depths for the TRTS are summarized as follows:

Sandy soils : $1.5 < L/H < 2.0$

Clayey soils : $1.5 < L/H < 2.5$

These cases exclude those where the wall penetrates into hard layer such as rocks or gravels. Furthermore, the plots exclude cases where diaphragm wall depths have been based on seepage cut-off.

B. Strut Reaction

The total strut load is plotted against excavation depth in Fig. 5 for excavations which have adopted the bottom-up method of construction and with no ground improvement. A simple parabolic function is used to define the relationship. The range of total strut load for excavations in both sandy and clayey soils are apparently the same. The design total strut load, TSL (in KN/m) for TRTS excavations using the bottom-up method of construction with no ground improvement generally falls within the range from 8 to 15 times the square of excavation depth, H (in meters).

C. Wall Bending Moment

Fig. 6A gives the plot of maximum bending moment versus excavation depth for excavations in sandy soils. As shown in the figure, the maximum bending moment (in KN-m/m) is within 50 to 100 times the excavation depth (in meters). Apparently, the method of construction has no clear influence on the magnitude of the maximum bending moment. This may be because the maximum bending moment in the diaphragm wall usually occurs near the maximum excavation depth where the early construction of the roof slab in a top-down construction may not have significant effect.

The plot of maximum bending moment versus excavation depth shown in Fig. 6B for excavations in clayey soils, shows that the range of maximum bending moment (in KN-m/m) is relatively large, with about 50 to 150 times the excavation depth (in meters). It can be noted that the cases with ground improvement have been designed for higher maximum bending moments than those without ground improvement.

D. Wall Deflection

There is a limit in which maximum wall deflection can be allowed for TRTS excavations, as this is a major source of ground surface settlement during construction. Fig. 7 shows the plot of maximum wall deflection, d , versus excavation depth, H , for excavations in sandy and clayey soils. The analyses are summarized as follows:

Sandy Soils : $0.10 \% < d/H < 0.30 \%$

Clayey Soils : $0.10 \% < d/H < 0.40 \%$

As in the case of wall bending moment, it can be observed that the method of construction apparently has no major influence on the magnitude of maximum wall deflection. Grout slab below the excavation base reduces the magnitude of maximum wall deflection. However, the effect of ground improvement cannot be directly deduced from the plots. Grout slabs are used only in cases where deflections would be too large otherwise.

V. Summary

Analyses were made to define the general characteristics of the TRTS diaphragm wall design. Although there has been variations in design due to the variability of subsoil conditions within the very large area covered by the network, the TRTS wall design can be defined within certain ranges in relation to excavation depth. The results may be used as a guide for preliminary design of diaphragm walls in similar conditions.

Acknowledgements

The authors are grateful to the Department of Rapid Transit Systems of the Taipei Municipal Government for granting the permission to publish the data presented in this paper.

References

- [1] Moh and Associates, Inc., "Engineering properties of the soil deposits in the Taipei Basin", Report No. 85043, submitted to the Ret-Ser Engineering Agency and Taipei Public Works Department, 1987.
- [2] Department of Rapid Transit Systems, "Civil Engineering Design Manual", 8th edition, September 1991.
- [3] Peck, R.B., "Advantages and limitations of the observational method in applied soil mechanics", *Geotechnique* (19), 2, 171-187, 1969.
- [4] CIRIA, "Design of retaining walls embedded in stiff clay", Report 104, 1984.
- [5] Moh, Z.C. and Hwang, R.N., "Earth pressure on walls of a deep excavation", 3rd Int'l. Conference on Case Histories in Geot. Engrg., St. Louis, Missouri, USA, June 1993.

Table 1 Design Criteria for TRTS Deep Excavations

Failure Mode	Factor of Safety
Inward Yielding	1.50
Base Heave	1.50
Piping or blow-in	1.25
Flotation:	
During Construction	1.03
Permanent Condition	1.07

Table 2 Summary of Information on TRTS Diaphragm Wall Design

Design Lot	Zone	Analysis Method	Computer Program	Construction Method	Water Pressure Distribution on Passive Side Used in Stability Analysis	Wall Friction / Adhesion
DL159	T2	Effective Stress	Sheetpile 2	Bottom-Up(CH218) Semi-Top Down (CH219 & CH220)	Non-Hydrostatic	1/2 ϕ '
DL160	T2	Effective Stress	QWALLS	Bottom-up	Hydrostatic	1/2 ϕ '
DL161	H2	Effective Stress	FLAC	Bottom-up	Hydrostatic	1/2 ϕ '
DL162	H2	Effective Stress	RIDO	Bottom-up	Hydrostatic	2/3 ϕ '
DL171	T2	Effective Stress	FLAC	Bottom-up	Hydrostatic	1/2 ϕ '
DL172	T2/K1	Effective Stress	Sheetpile 2	Semi-Top Down	Non-Hydrostatic	1/2 ϕ '
DL173	K1	Total & Effective Stress	FREW	Bottom-up(CN257&CN259) Top-Down (CN258)	Non-Hydrostatic	1/2 ϕ ' & 0.45Cu
DL176	T2	Effective Stress	RIDO	Bottom-up	Non-Hydrostatic	1/3 ϕ '
DL177	T2	Effective Stress	Sheetpile 2	Bottom-up	Non-Hydrostatic	$\tan \delta = 0.3 \tan \phi$ '
DL178	T2	Effective Stress	RIDO	Bottom-up	Non-Hydrostatic	1/2 ϕ '
DL180	—	Effective Stress	RIDO	Bottom-up	Hydrostatic	1/2 ϕ '
DL181	—	Total & Effective Stress	RIDO	Bottom-up	Non-Hydrostatic	2/3 ϕ ' & 2/3Cu

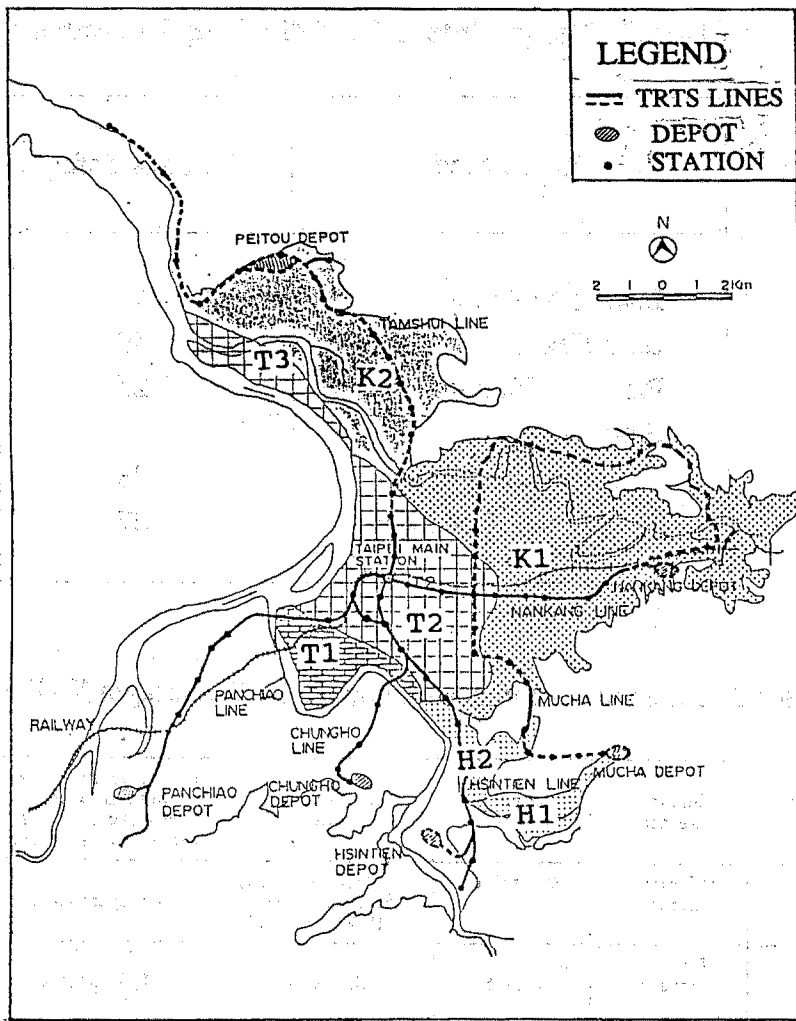


Fig. 1 Sub-divisions of the Sungshan Formation in the Taipei Basin

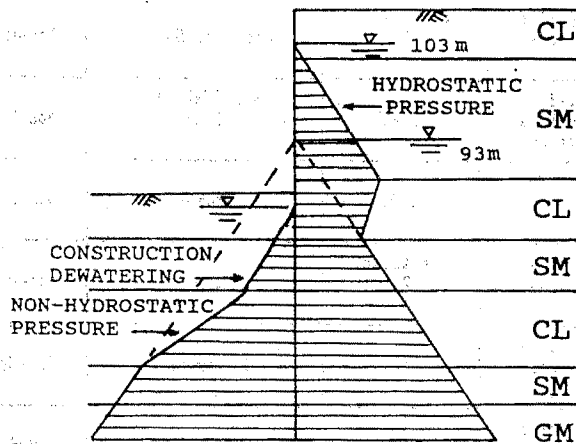


Fig. 2 Typical Subsoil Profile and Groundwater Distribution

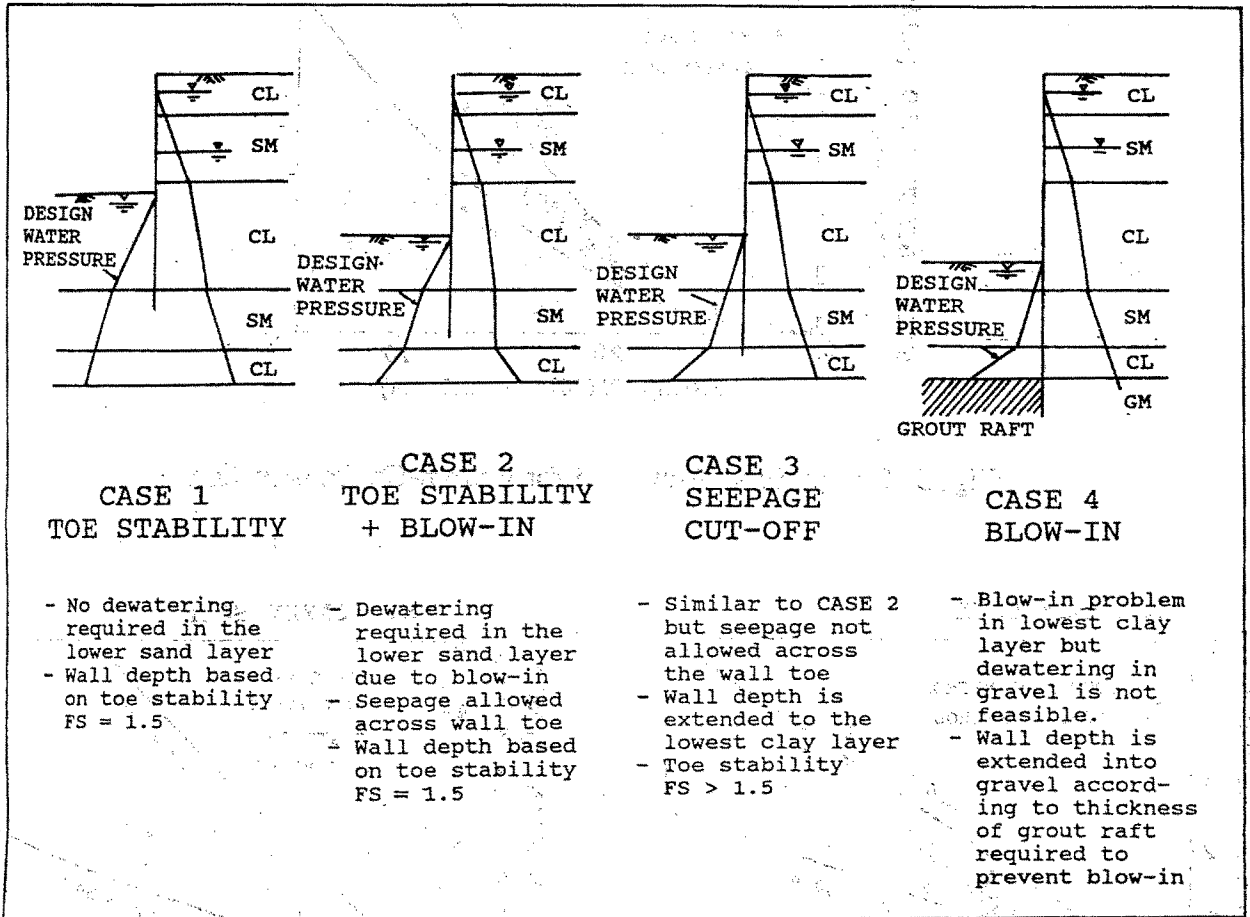


Fig. 3 Effect of Water Pressure on Wall Penetration Depth

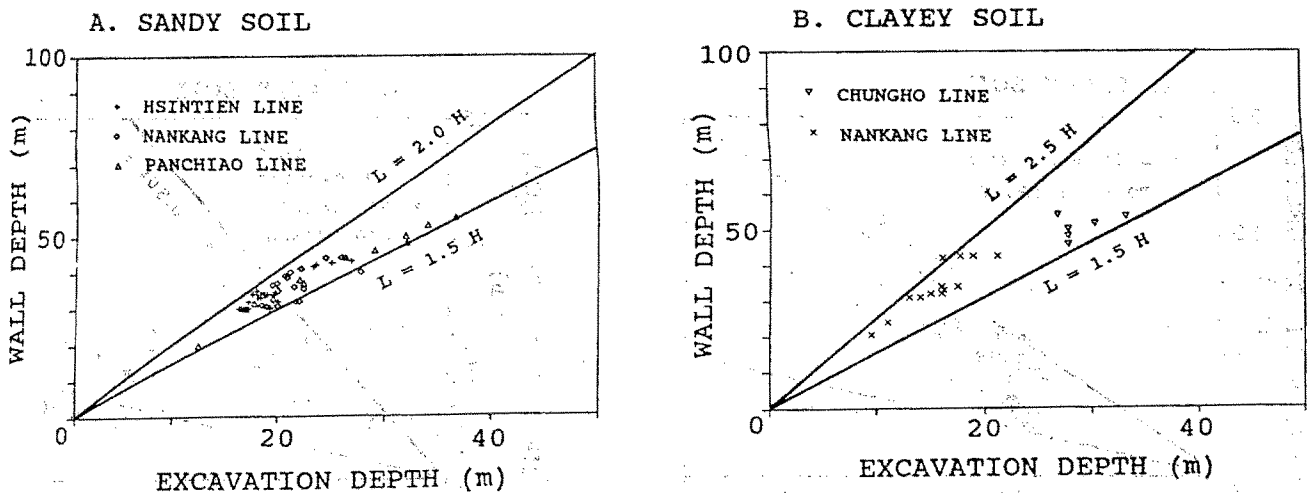


Fig. 4 Design Wall Depth

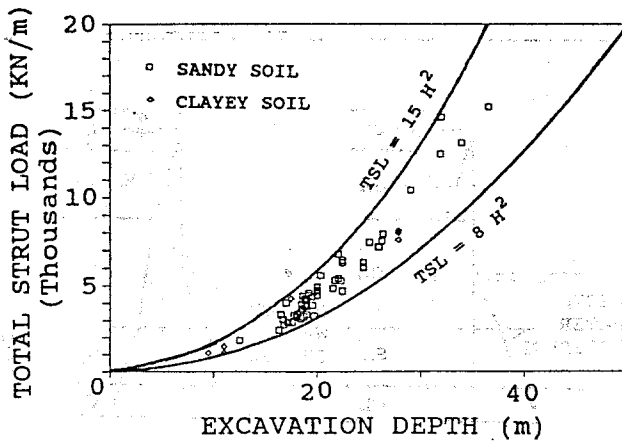


Fig. 5 Design Total Strut Load for Bottom-Up Construction

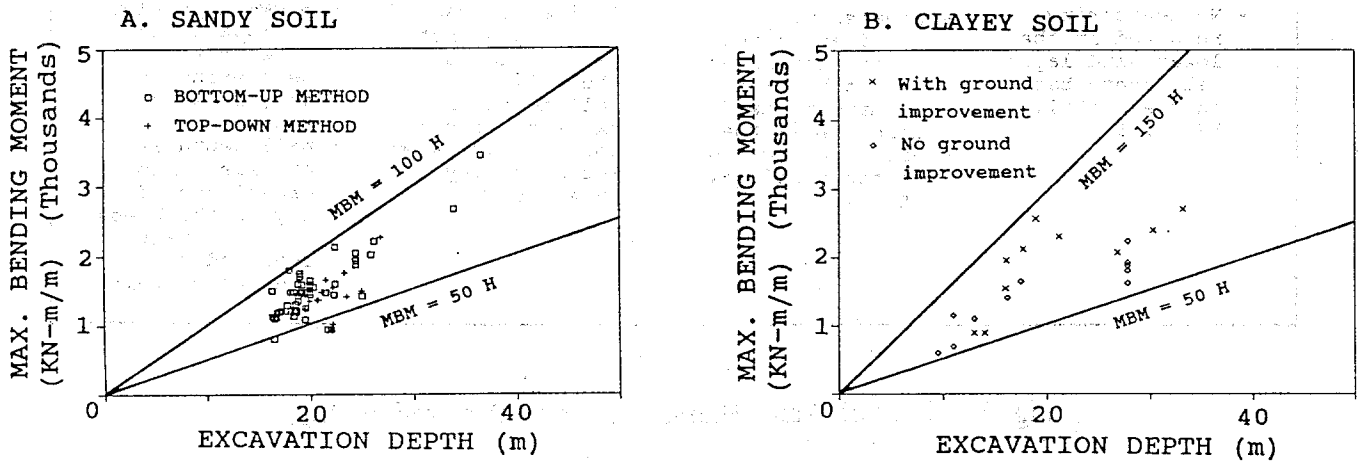


Fig. 6 Design Maximum Wall Bending Moment

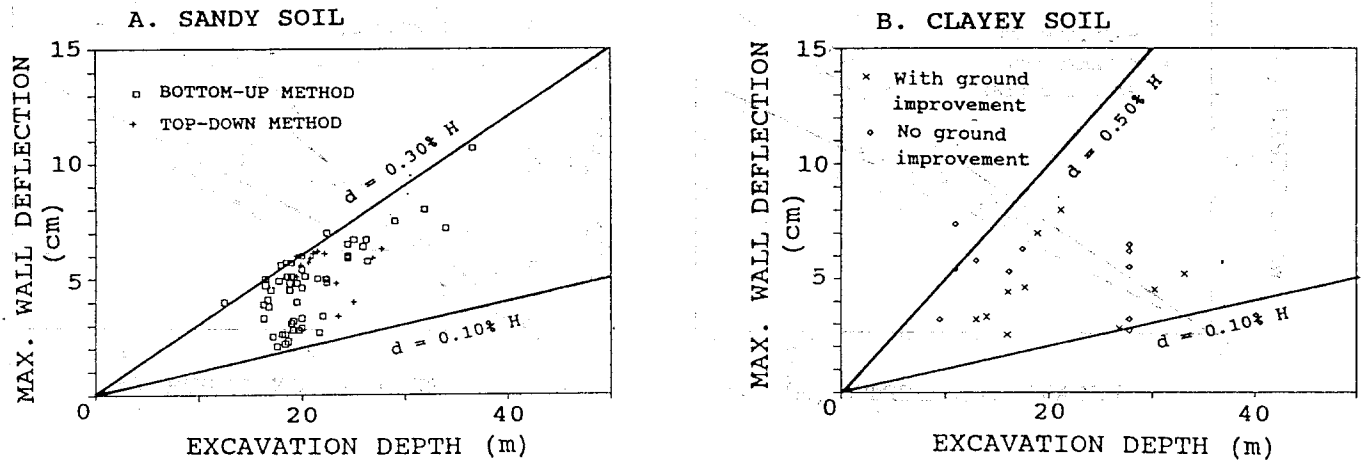


Fig. 7 Design Maximum Wall Deflection