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Online Database of Deep Excavation Performance and Prediction

Dimitrios C. Konstantakos
Deep Excavation LLC., Astoria, New York

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Dimitrios C. Konstantakos, P.E.
Deep Excavation LLC
Astoria, New York-USA, 11103

ABSTRACT

Deep excavations can comprise one of the most challenging design and construction geotechnical problems. Many factors affect their performance including soil strength, ground-water, building surcharges, construction methods, and construction techniques. Engineers are confounded with the complex problem of predicting behavior while producing a safe and economical design. Typical parameters of interest are design wall moments, bracing forces, wall displacements, and ground or building settlements. Many authors have compiled case histories that quantify maximum observed displacements in various publications. Often, these publications include benchmarking of observed behavior otherwise known as Class "C" predictions. Few authors have compiled Class "A" predictions where performance predictions are presented before the actual project is completed. This publication presents a recently developed online database of deep excavation prediction and performance available to engineers in an effort to make performance and modeling information more accessible. In addition, the performance and benchmarking of nearly 39 deep excavations is briefly presented, focusing mostly on inclinometer recorded wall deflections.

INTRODUCTION

Predicting the behavior of deep excavations is a very complex geotechnical problem. Factors affecting deep excavation performance include soil and site conditions as well as construction methods. Experience has shown that performance can widely vary even within the same project. In an effort to better understand deep excavation behavior, many authors have published a number of case studies. Occasionally, the case studies include class "C" predictions where measured excavation performance is matched with back analyses by means of finite element software.

Unfortunately, most case studies are dispersed amongst various publications that are difficult to find in every day engineering life. Today, in contrast to last decades, the internet enables us to almost instantaneously share and compare information.

In an effort to address these issues, the author has prepared an online database of deep excavation prediction and performance. The database includes close to thirty nine case studies and is continuously updated. Most case studies include class "C" back analyses and a few case studies include class "A" predictions. This publication describes the main features of the online database as well as main findings from individual case studies and associated back analyses. Main parameters of interest are wall deflections, surface and building settlements, and benchmarked internal wall moments

DATABASE DESCRIPTION

The database has been developed using html language, PHP dynamic html programming language, and my MySQL databases. Efforts were focused on making the database easily accessible, so that a user can easily compare similar projects and find interesting features. Once a user has found an interesting project, he or she can select it and a full description as well as other vital information is presented. The online database is available at:

www.deepexcavation.com/database.htm

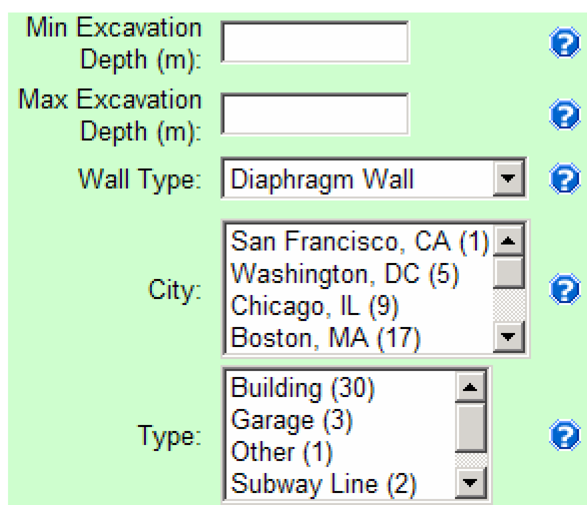

FIGURE 1(a): Online database basic search form

The database can be searched using basic and advanced search options. Individual projects can be searched according to the following information as Figures 1(a) and 1(b) show:

- Project location
- Wall type (i.e. diaphragm, sheet pile, soldier pile, etc)
- Support system (i.e. anchors, cross-lot struts, etc.)
- Project type (i.e. building, garage, subway, etc.)
- Maximum wall deflections
- Maximum settlements
- Basal stability index

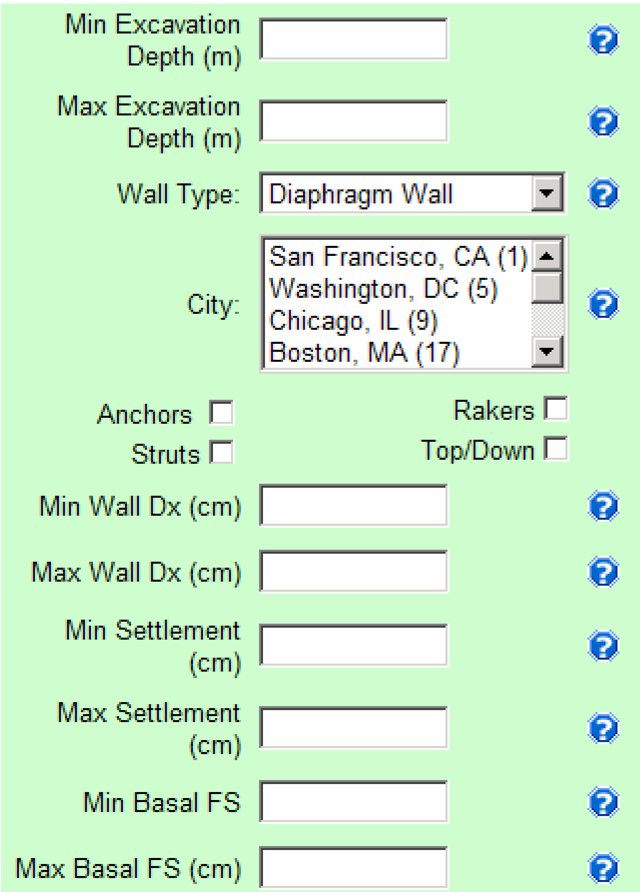

FIGURE 1(b): Online database advanced search form

When available, the database also includes the following archived information:

- Inclinometer displacement troughs
- Maximum measured wall displacements
- Maximum measured settlements
- Number of support levels
- Number of basement floor levels
- Wall thickness, Wall stiffness (i.e. moment of inertia)
- Benchmarking class (Class "A", Class "C", etc)
- Benchmarked wall bending moments
- Benchmarked wall deflections
- Link to a benchmarked model datafile (i.e. by commercial modeling software)

When a specific project has been benchmarked, the database includes a benchmarking class notation (Class "A", or Class "C"). Class "A" refers to a project where initial elastoplastic analysis has been performed prior to construction and after construction parameters were modified to match observed performance. Class "C" cases refer to benchmarking when project performance is already known.

Figures 2(a) and 2(b) show typical search results obtained from a database query for top/down projects in Boston. Once a search is submitted, the database generates a list of projects that have met search conditions. The generated list can be sorted according to excavation depth, construction year, and other vital project information (Fig. 2(a)). A link appears on the project name if a detailed project description is available.

Selecting a specific project will generate a complete project description as Figure 2(b) shows. This form displays detailed information regarding wall properties and performance, as well as benchmarking data when available.

A project can also be located from free online map services offered by Yahoo Inc (yahoo.com, maps.yahoo.com), if the project address is known.

The author anticipates both database format and content to be continuously updated and improved by the time of the conference in August 2008.

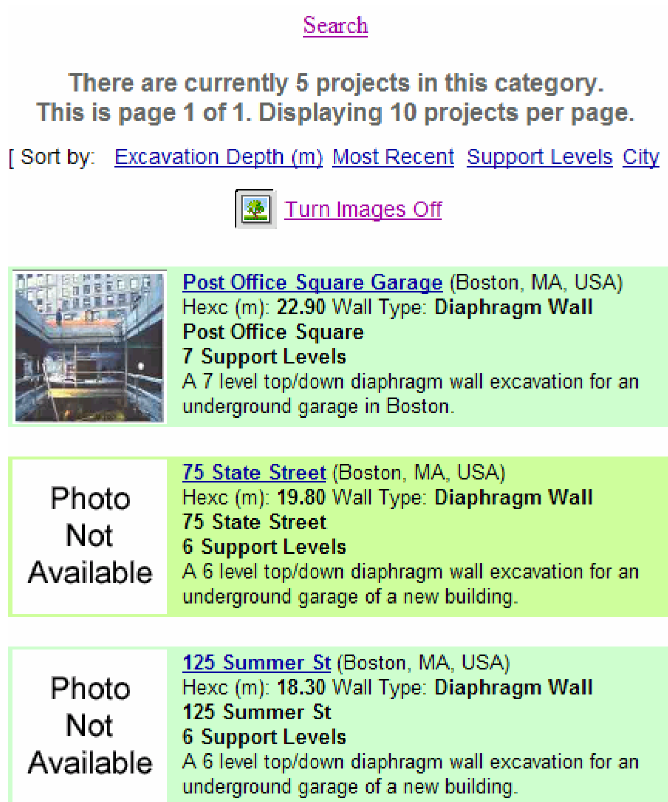

FIGURE 2(a): Typical search output from a database query listing all projects that met search criteria.

FIGURE 2(b): Typical detailed output for a specific project.

CASE STUDIES

At its current state, the database includes approximately forty projects with about twenty five case studies fully documented. Most projects are located in major cities in the US with remaining cases in Europe and Asia. A full project list is presented in Table 1. Most case studies were collected from previous research by the author (Konstantakos, 2000), while remaining case studies were collected from other publications and sources.

Most fully instrumented case studies were constructed with diaphragm walls. The near absence of other wall types likely reflects the need for instrumentation in critical projects, where mostly diaphragm walls have been used. Fully documented excavations included in the database have been constructed since the early 1970s. A few not fully documented projects utilizing steel sheet piles or soldier pile walls are also included in the database.

The author aims to balance the database with more sheet pile, soldier pile, and other wall type projects in the near future. Other fellow engineers are encouraged to submit their case studies for inclusion in the database. Submitted case studies by other fellow engineers will be acknowledged with a personal link on each project.

Wall bracing included soil/rock anchors, cross-lot struts, rakers, and internal floor slabs constructed with the top/down method. Excavation depths ranged from 6 to 31m.

Soil profiles shared many similarities across different cities. In particular, soft/medium stiff clay with high ground water profiles were encountered in most case studies. This “coincidence” likely reflects the engineer’s need for instrumentation and safety in critical projects.

Major factors affecting excavation performance are: soil and site conditions (e.g. stratification, abandoned structures etc.), ground water, excavation support wall stiffness, excavation support type and prestress levels, support construction methods, duration of construction, presence of permanent or temporary surcharges, and even temperature variations. Hence, case studies have been divided into four categories to list trends observed in the collected performance data:

A) Cross-lot, and internally braced walls: Excavations are typically braced by preloaded large diameter steel pipes spanning between opposing walls, in narrow excavations (<36m or 120ft). For wider excavations, the bracing usually is provided by a combination of rakers and corner strut systems. The site layout determines the arrangement of the cross-lot or the internal braces. Walls in most projects extend into a stiff stratum like glacial till or bedrock to limit movements and cutoff water within the excavation.

B) Keyed anchored walls: Excavations where the wall toe is embedded into a stiff stratum like glacial till or bedrock, with the wall braced by tiebacks in soil and/or rock anchors.

C) Floating walls: When the wall toe is embedded within a weak stratum that is subjected to basal movements, most of these projects were supported using tieback anchors, and/or in combination with rakers.

D) Top/Down (Up/Down) walls: Basement floors brace the walls as the excavation progresses downward to subgrade level. In most cases, the wall extends into a stiff stratum to support permanent vertical loads on the walls.

Excavation performance was typically monitored by inclinometers, piezometers/observation wells, surveying points, and more rarely load cells and strain gages. The evaluation is primarily based on measured inclinometer deflections for comparison, since such data are both widely available and more reliable than other measured data. Settlements were not available or reliable for all projects (because of subsequent construction activities). Therefore, general conclusions cannot be drawn from them, except for data from Boston projects where settlement data has been extensively recorded and archived.

The quality of available data varies from project to project. Generally, all projects include measurements of wall deflections with inclinometers. Fewer projects had settlement measurements archived. Support forces on struts were seldomly recorded. In one case study, internal wall moments were backfigured using strain gages.

Table 1: List of Projects and Summary of Measured Performance

ID	Year	Cat.	Project Name	Soil Type	Wall Type	Bracing – Method	Exc. (m)	Wall Thick. (m)	δ_{Hmax} (mm)	δ_{Vmax} (mm)
B1	1969	A	MBTA South Cove	A	DW	3 Lev. CRLB	16.2	0.91	34	13
B8	1989	A	Flagship Wharf	C	DW	3 Lev. CRLB	14.3	0.76	46	43
B15	1949	A	J. Hancock Berkeley	A	SPTL	Rakers & Berms	13.7	-	200**	-
B16	1967	A	Accolon Way	C	SSP	5 Lev. CRLB	17.7	-	282**	-
B17	1972	A	J. Hancock Tower	A	SSP	Rakers & Berms	13.7	-	450**	-
C1	1970	A	C.N.A	CL	DW	1R.+ SB, CRLB Floors	9.4	0.76	84	127
C2	1971	A	Sears Tower	CL	DW	3 Lev. R, SB	9.8	0.76	152+	-
N2	1988	A	PATH, West & Morton	S/ML	DW	6 Lev. CRLB, IB	19.5	0.91	18	18
S9	1999	A	CB-1 Tower	CL	DW	2 Lev, IB, 1 Lev. R	20.1	0.91	20	12.7
TH4	1996	A	Bangkok	CL	DW	2 Lev. CRLB	12.7	0.80	25	-
TH5	1996	A	Business Complex	CL	DW	4 Lev. IB	15.5	0.80	80	-
W2	1995	A	Petworth Sub. Stat.	S, CL	DW	5-6 Lev. CRLB	18.3 to 30.5	0.91	19	-
W5	1973	A	Federal St Station**	S, CL	DW	3 Lev. CRLB	18.3	0.91	16	-
B2	1975	B	60-State Street	B, C	DW	3 or 2 Lev. TB,	9.8	0.61	33 – 23**	30
B12	1995	B	Dana Farber Tower	B	DW	6-Lev. RA	19.8 27.4	0.91	18 10	16 71
B18	1982	B	Four Seasons Hotel	B	SSP	TB – R	7.6	-	114**	-
B19	1986	B	International Place	C	SPTL	3L TB	16.7	-	18.4	-
N3	2001	B	9/11 WTC Collapse	O, S, GT	DW	5 Lev. TB	22.9	0.91	-63, 38***	-
NJ1	2001	B	30 Hudson Street	F, O, GT	DW	3 to 4 Lev. TB.	16.8	0.76	32***	-
W4	1990	B	Metro Center II**	S, CL	DW	2-L. TB	9.4	0.61	10	-
W1	1991	B	World Bank	S, CL	DW	5-Lev. TB	18.3	0.76	11 to -11	11
W3	1999 2000	B	Washington Convention Center	S, CL	DW	1-Lev TB, 1 Lev. R. 3-L. TB	9.1 16.8	0.91, 1.22 0.91	19 18	0
B3	1982	C	State Transp. Bldg.	A	DW	2 Lev. TB, R, CB	8.2 to 11	0.61	32	30
B6	1985	C	One Memorial Dr.	A	DW	2 Lev. TB	9.1	0.61	33	30
B7	1987	C	500 Boylston	A	DW	4 Lev. TB, R	12.8	0.61	84	114
B14	2001	C	Stata Center (MIT)	A	DW	3 Lev. TB, or 2 R	12.5	0.76	81	64
C3	1971	C	Amoco Standard Oil	CL	DW	1 Lev. Ties, SP, SB	7 to 13.4	0.76	117	-
C4	1973	C	Water Tower	CL	DW	1 Lev. TB & 1 R	13.4	0.61	64	38
C6	1987	C	Prudential Two	CL	DW	1 Lev. TB, 1 Lev R,	7.6	0.69	11	-
C7	1987	C	AT&T Corp. Center	CL	DW	3 Lev. R & CB	8.2	0.76	39	39
C9	1993	C	NW U. Mem. Parking	CL	DW	1 Lev. TB	7.0	0.61	39	-
C10	1997	C	Museum Science Ind.	CL	DW	3 Lev. TB	10.4	0.76	22	-
B4	1983	D	75 State Street	A	DW	6-Levels top-down	19.8	0.76	47	102
B5	1984	D	Rowes Wharf**	B	DW	5-Levels top-down	16.7	0.76	10	-
B9	1990	D	125 Summer St	A	DW	6-Levels top-down	18.3	0.76	15	10
B10	1989	D	Post Office Sq. Garage	A	DW	7-Levels top-down	22.9	0.91	55	70
B11	1994	D	Beth Israel Deaconess	A	DW	5-Levels top-down	16.8	0.91	22	18
B13	1998	D	Millennium Place	B	DW	5-Levels top-down	16.8	0.91	18	11
C8	1989	D	Guest Quarters Hotel	CL	DW	3-Levels top-down	10.7	0.61	17	-

Wall Types: **DW**: Diaphragm wall, **SSP**: Steel Sheet Pile wall, **SPTL**: Soldier Pile and Timber Lagging wall

Support Types: **CB**: Corner Braces, **CRLB**: Cross-Lot Braces, **IB**: Internal Braces, **SB**: Soil Berm, **SP**: Soldier Piles, **R**: Rakers, **RA**: Rock Anchors, **TB:** Tiebacks (soil anchors)

Soil Types: * A: Fill, Organic Silt, Boston Blue Clay, Glacial Till, Bedrock, B: Fill, Glaciomarine, Glacial Till, Bedrock, C: Fill, Moraine Deposits, Bedrock (after Johnson 1989), S: Sand, ML: Silt, O: Organic/Organic Silt, CL: Clays

ID: **USA Cities: B#:** Boston, MA **C#:** Chicago, IL **N#**: New York, NY **NJ#:** New Jersey, NJ **S#:** San Francisco, CA **W#:** Washington, DC. **International:** TH: Thailand

Category: **A=** Cross-lot/internally braced, **B=** Anchored walls keyed in stiff layer, **C=** floating anchored walls, **D=** Top down excavations.

Additional: ** Data & Deflections Referenced from existing literature, ***** Approximate Deformation Caused by Excavation, WTC Collapse Total movements> 30 to 60 cm, δ_{Hmax} , δ_{Vmax} = Maximum horizontal and vertical deformations.

MEASURED PERFORMANCE

Table 1 summarizes important excavation and performance data for each case study. Projects are categorized according to general wall type (cross-lot, keyed tieback, floating, & top/down). Information about soil conditions, wall thickness, excavation depth, bracing method, year of construction, and maximum deformations is also included.

Wall movements can be classified into three major categories: a) Cantilever stage rotations, b) wall flexure (bending), and c) translation of the wall base. Translation at the wall base was more important in floating walls where it amounted for movements up to up to 6.4 cm (2.5 in) towards the excavation when adequate embedment was not provided (C3, B7, and B14). In addition, floating walls supported by tiebacks tended to rotate more about the wall base than other types of walls. Flexural wall deflections were generally the largest in top/down and cross-lot excavations reaching 4.0 cm (1.6 in) at the B10 excavation whereas floating walls did not flex more than 1.3 cm (0.5 in). The B8 cross-lot project had the largest flexural deflection with roughly 4.8 cm (1.9 in) towards the excavation. Excavations constructed prior to 1980 generally measured greater displacements than more recent projects.

Surface settlements were typically in the same order as horizontal wall movements. In general, the maximum deformations summarized in Table 1 only accounted for a small percentage of all monitored data. The following sections discuss measured performance of documented excavations in more detail.

Category A: Measured Performance of Cross-lot and Internally Braced Excavations

Cross-lot bracing was preferred in relatively narrow sites where the opposite walls were 15 m to 36.5 m (50 ft to 120 ft) from each other. All of the studied cross-lot braced walls were keyed into a stiff stratum (glacial till or bedrock). Cross-lot excavations in Boston, New York City, and in Washington ranged from 14m to 30m (47 ft to 100 ft) deep. A complex internal bracing system was used in the 20 m - deep (66 ft) Yerba Buena Tower excavation, where the slurry wall was 35 m deep (115 ft). The two Chicago excavations are not representative of modern practice since they were amongst the first slurry wall excavations constructed in that city.

The database included the performance of three earlier excavations (B15, B16, B17) supported by soldier pile and timber lagging and steel sheet pile walls from Boston. These projects were constructed from 1949 to 1972 and are not fully documented. It is interesting to note though, that these projects experienced very large deformations and have likely led local practice towards diaphragm walls (when the technology became more widely available in the US).

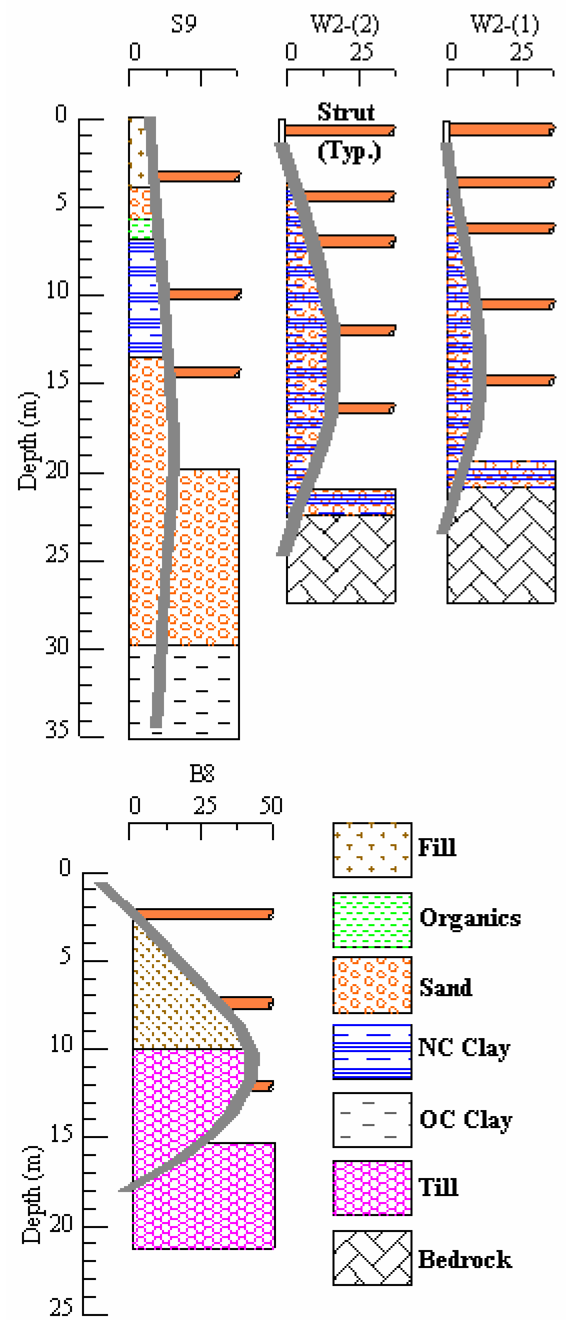

Figure 3: Typical sections and wall deformations δH (mm) of internally braced and cross-lot braced excavations.

Figure 3 plots maximum wall deflections versus depth for cross-lot and internally braced excavations. Wall deflections for the studied cross-lot braced excavations were moderate, ranging up to δH= 4.6 cm (1.8 in) (B8) (δH= Horizontal Deformation). In the deep Petworth WMATA Subway section, wall deflections were smaller than most projects and ranged up to δH= 1.9 cm (0.75 in) towards the excavation. In the Yerba Buena Tower project, the underlying clays were not able to fully restraint the base of the wall and thus soil and wall deformations occurred throughout the depth of the slurry wall. Nonetheless, wall deflections were small and ranged up to 2.0 cm (0.8 in) towards the excavation.

The range of wall deflections was almost double the maximum wall movement for B8 and W2, since some wall panels moved back towards the retained soil as much as other panels deflected towards the excavation. Most walls deflected less than 2.5 cm (1.0 in) or δH/H=0.19% (H = Excavation Depth). Flagship Wharf (B8) was the only project both inclinometer deflections at opposing walls and strut brace loads were measured. Load gages at B8 confirmed that thermal expansion and contraction can be very important when the bracing struts are very long.

Category B: Measured Performance of Anchor Supported Keyed Wall Excavations

In these projects, restraint at the wall toe (i.e. "fixity") is provided by keying the wall into a stiff stratum such as glacial till or bedrock. From the nine case studies available in this category, four were constructed in Boston (B2, B12, B18, B19), one in New Jersey (NJ1), one in New York City (N3), and the remaining three in Washington, DC (W1, W3, and W4). Excavation depths ranged from 9 m to 26.8 m (30 ft to 88 ft). For obvious reasons, the World Trade Center (WTC) collapse on 9/11/01 (N3) is not representative of typical excavations. Data from WTC is included in this paper as a case of slurry wall performance under severe conditions. Figure 4 (on the right) shows typical sections and wall deformations on selected projects.

As expected, there were no measurable movements at the wall toe. Excluding the WTC collapse, there were only small measurable wall deflections smaller than δH= 2.5 cm (1.0 in) towards the excavation (Figure 2). In these projects, anchor elongation due to creep and stress transfer mechanisms, and adjacent building surcharge appear to control wall deformations.

Small wall deformations can be a misleading indicator of settlement performance. Ground losses and disturbance during tieback installation can cause settlements that are significantly greater than wall deformations. This occurred in case study B12 where in a few locations settlements reached up to δV= 7.1 cm (2.8 in) while the diaphragm walls deflected only δH= 1.0 cm (0.4 in) towards the excavation.

Furthermore, case study B12 deserves special attention since it was the only studied project with strain gages installed within the slurry wall. The 0.91 m (3 ft) thick slurry wall at B12 project was supported with 4 to 6 levels of rock anchors with the excavation reaching to bedrock at 18m to 26.8 m (60 ft to 88 ft) beneath the ground surface. Wall strain gages can provide useful information despite being affected by concrete shrinkage and temperature changes. Interestingly, these strain gages revealed that practically all the vertical component of the anchor loads was transferred to the wall base. This finding suggests that there is little soil side support from friction acting on a wall when the toe of a slurry wall bears on a stiff stratum or bedrock. In a recent publication (2004), the author conducted finite element simulations for B12 that resulted in good agreement with measured performance.

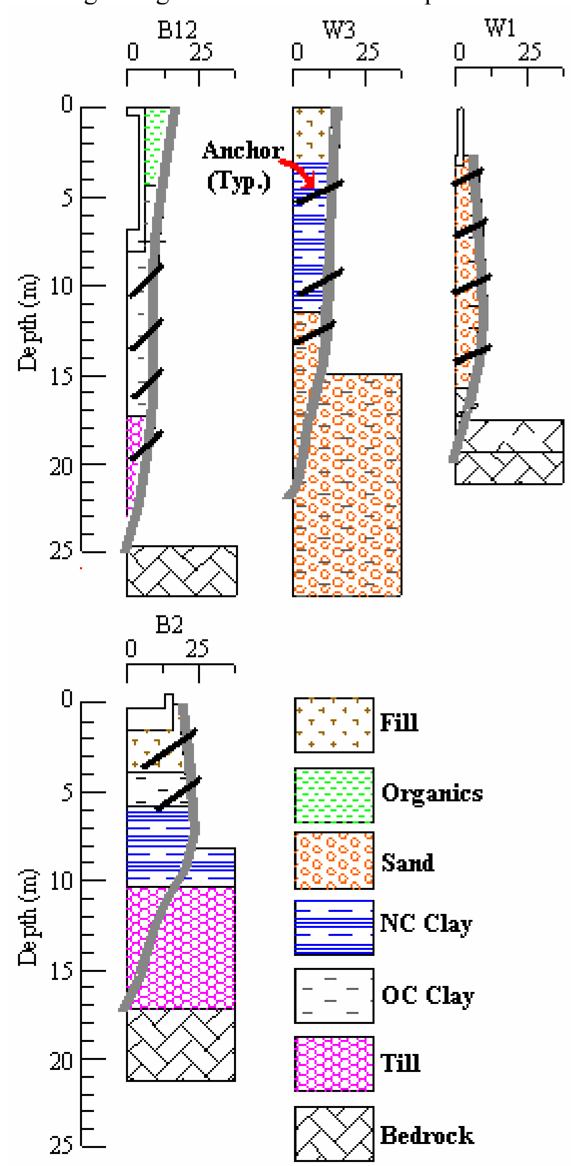

Figure 4: Typical sections and wall deformations δH (mm) of anchor supported keyed wall excavations.

As previously discussed, the World Trade Center collapse is not representative of typical excavation staging and construction. Portions of the permanent basement slabs crushed when the twin towers collapsed into the WTC basement. The result was a drastic reduction in the effective lateral support of the perimeter slurry wall. Consequently, the slurry wall experienced deformations in excess of 0.60 m (2 ft) into the WTC basement. The slurry wall withstood both the large deformations and the large unsupported lengths. The stability of the WTC slurry wall immediately after the collapse is mainly attributed to the ability of poorly or non-supported individual panels to cantilever from the base, to benefit from any residual floor diaphragms, and to span adjacent panels that had adequate lateral support. Nonetheless, the diaphragm wall stability had to be ensured by proper anchoring before the recovery crews could "safely" reach the old subgrade 22.9 m (75 ft) beneath the Hudson River. The general philosophy of the redesign was that the old anchor system had to be replicated in some fashion. Most importantly, the first level of rock anchors was prestressed with considerably excess force to account for the possibility of further collapses in the basement. Relatively small deformations occurred after the first level of anchors was prestressed. Many panels were pushed back towards the retained soil as anchor installation proceeded deeper (considering the position of the wall before the first level of anchors was installed as initial). As expected, individual panel exhibited considerably different deformations. Moskowitz and Tamaro (2002) discuss this subject in more detail.

Category C: Measured Performance of Floating Wall Excavations

These excavations are relatively shallow, with most being 10.5 m (35 ft) deep or less and supported by two or three levels of bracing. These walls could not be keyed because stiffer strata are not available within economically reasonable depths below final subgrade. Thickness of diaphragm walls (when used) ranged from 0.60 m to 0.76m (2 ft to 2.5 ft), and all of them are embedded into clay. In Chicago projects (C3, C4, C6, C7, C9, and C10), the designs called for the walls to extend by a minimum of 1.5 m (5 ft) into a stiff clay stratum (1.0 tsf to 4.0 tsf) that underlies softer clays and fills, at 15 m to 16.7 m (50 ft to 55 ft) beneath the surface. In Boston projects, the designs extend the diaphragm walls into either the desiccated Boston Blue Clay crust, or in lower sand lenses between the crust and the lower Boston Blue Clay (BBC). Tiebacks were the preferred bracing type for these excavations, but some projects also used rakers. Figure 4 plots selected maximum deflections versus depth for floating walls. Larger deformations were mainly caused by:

1. Short free or bonded tieback length.
2. Tieback creep and load loss.
3. Inadequate toe-embedment resulting in deep basal movements.
4. Influence of other construction activities (i.e. caisson installation, pile extraction).

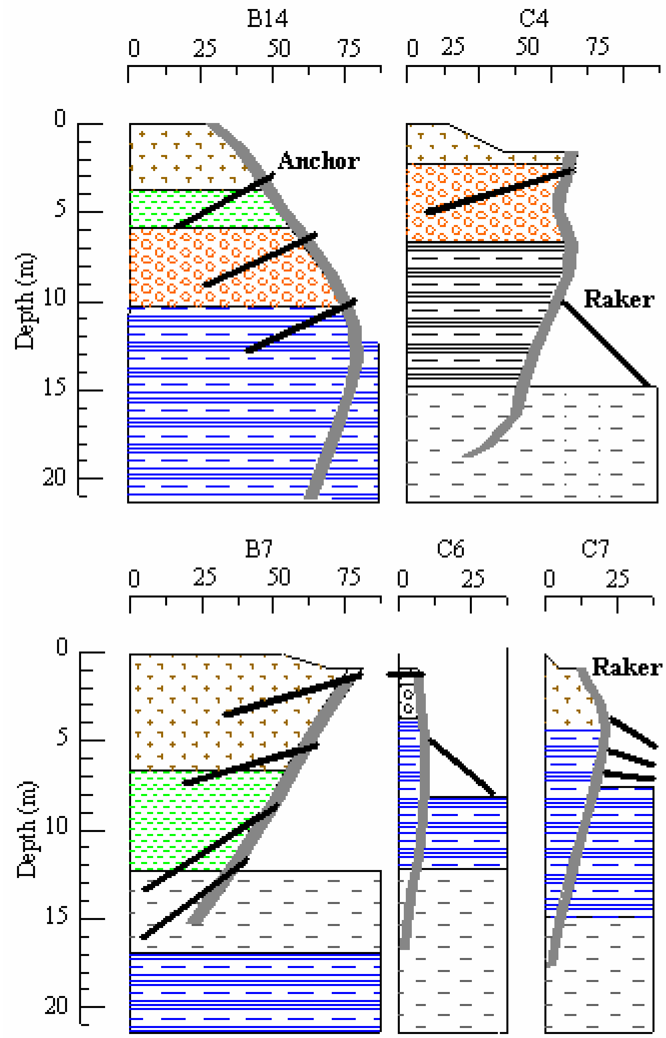

Figure 5(a): Typical sections-wall deformations δH (mm) for floating excavations with soil anchors and inclined rakers

Larger basal movements may take place in floating wall systems especially when deep soft clay deposits exist beneath the excavation. In such cases, the active zone of movements extended considerably beyond the influence line of $45^\circ - \varphi/2$ taken from subgrade. This appears to be the case in project B7, where tiebacks did not have sufficient free length to extend beyond the active zone of movements or did not gain sufficient capacity. More tiebacks were installed when poor performance of the initial bracing was realized. However, this action was not fully effective in restraining additional movements because the tieback proximity (4 ft to 5 ft horizontal spacing in some cases) resulted in interaction between the fixed zones of adjacent tiebacks. In addition, the wall embedment of 3.3 m (10 ft) did not appear to provide adequate restraint as the wall rotated as a rigid body despite being braced by four levels of tiebacks.

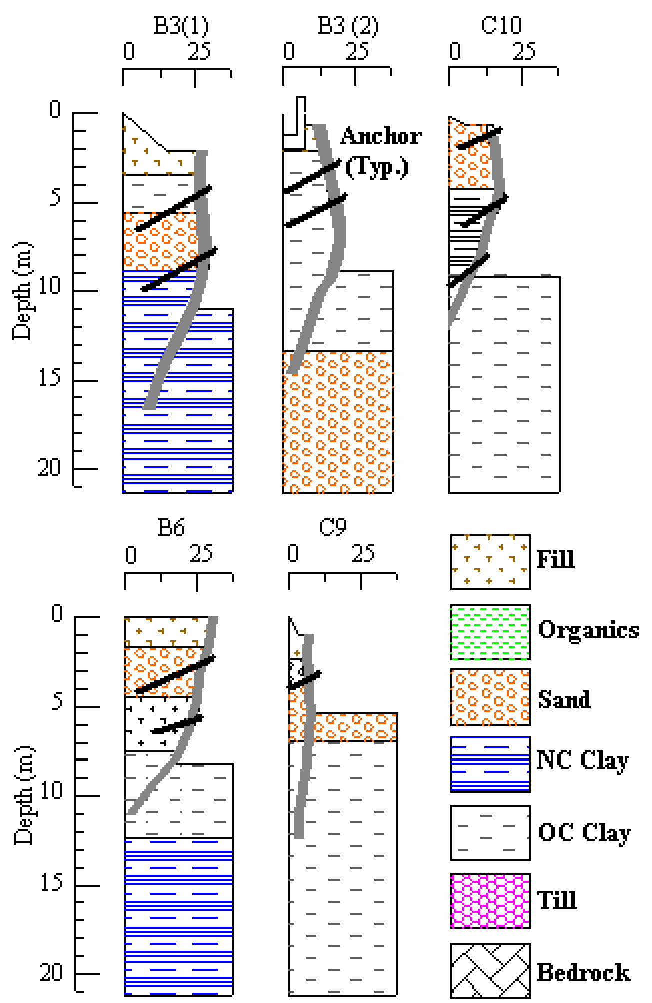

Figure 5(b): Typical sections-wall deformations δH (mm) for floating excavations with soil anchors and inclined rakers continued from 5(a)

Placement of tiebacks within the active zone of movements occurred in the Stata Center project (B14). Olsen (2001), reported maximum wall movements of δH= 5.0 cm to 8.3 cm (2.0 in to 3.25 in) with the translation at the wall base ranging from δH= 3.3 cm to δH= 6.4 cm (1.3 in to 2.5 in) despite a 9.1 m (30 ft) toe embedment into Boston Blue Clay. In this project, the lowest two levels of tiebacks were located within the active zone of movements. During excavation, grade beams were installed in many locations to prevent further wall movements and eliminate the potential for basal failure. Olsen (2001) also reported that these grade beams were only marginally effective in reducing further horizontal wall deformations.

With the exception of projects B7, B14, C3, & C4, other projects measured smaller wall deflections and surface settlements that reached up to δH= 4.0 cm (1.55 in). Shallow excavations in Chicago (excavation depth less than 9.1 m or 30 ft) showed very little wall bending with the wall base translating slightly towards the excavation (0.5 cm to 1.3 cm or 0.2" to 0.5"). In most monitoring locations, the wall primarily bent between the lowest bracing level and the final excavation grade, with very little to no bending between the bracing supports. Maximum ground settlements were in the same order as maximum wall deflections. The C3 and C4 projects were early diaphragm wall excavations in Chicago in the 1970s, and thus experience had not fully accumulated at that time. Caisson construction caused soil remolding in these projects.

Special attention must be paid to re-entrant diaphragm wall corners. In these cases, increased potential exists for panel separation and cracking, as panels near a re-entrant corner tend to move in different directions. This was observed in project C10 where re-entrant wall panels experienced considerable cracks and joint separation.

The database study of wall performance clearly demonstrates that the two most important factors in restraining movements for floating walls are:

I) Sufficient toe embedment, and
II) Sufficient tieback length beyond the active zone of movements.

Category D: Measured Performance of Top/Down Excavations.

The top/down construction method was primarily utilized when it was desired to expedite construction of the superstructure or when site constraints did not permit the use of alternative bracing systems. Six out of the seven examined top/down projects are in Boston and one is in Chicago. Excavations in Boston are 16.5 m to 22.9 m (55 ft to 75 ft) deep, while project C8 in Chicago is only 10.7 m (35 ft) deep. All the walls in Boston projects are keyed into either glacial till or bedrock.

Maximum wall deflections ranged up to 5.6 cm (2.2 in) (Figure 4), while maximum settlements induced by the excavation only where roughly in the same order as maximum wall deflections. However, most monitored sections deflected less than δH = 2.5 cm (1.0 cm) towards the excavation or less than δH /H= 0.2%. In most cases, the walls bent in a concave shape with deflections increasing as excavation progressed deeper. Some of this movement can be attributed to shrinkage of the bracing concrete floor slabs.

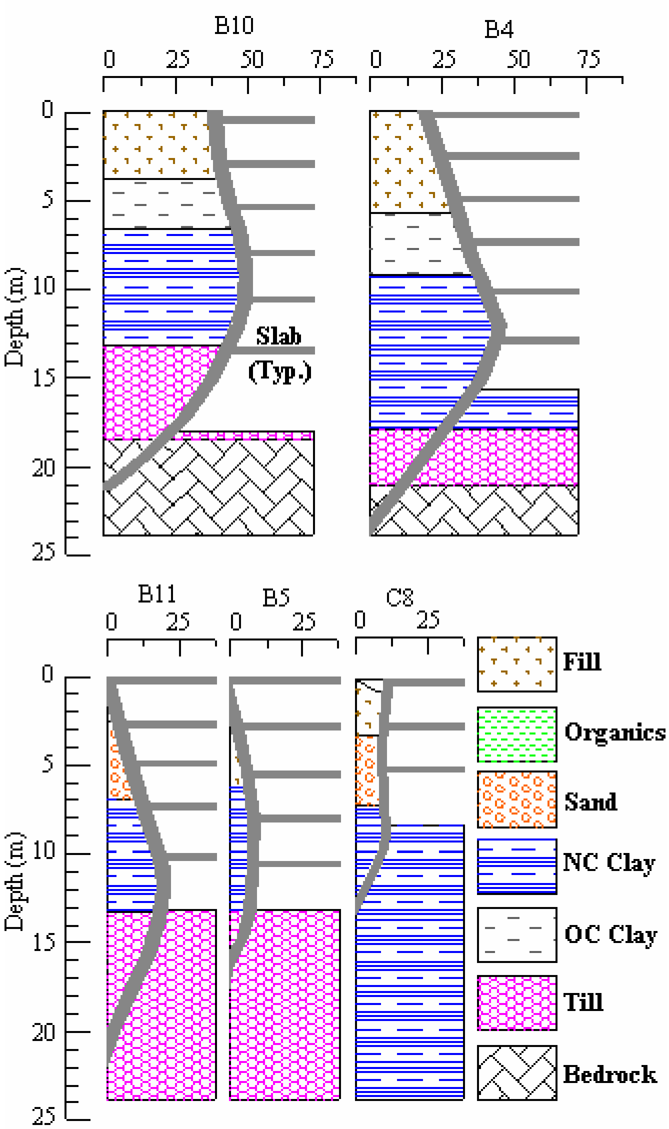

Figure 6: Typical sections and wall deformations δH (mm) for top-down excavations.

In project B4, pile extraction appears to have caused ground softening and thus surface settlements were larger. In project B10, larger than expected wall deflections at some locations were attributed to poor control on the backfilling for Load Bearing Elements (LBE's). In other projects, maximum wall deflections and settlements were typically less than 2.5 cm (1.0 in).

In Boston projects, where soft Boston Blue Clay strata dominated the profile, walls mostly bowed towards the excavation with maximum wall deflection taking place within the clay (B4, B10, B11). In these excavations, wall deflections were larger from excavations where glaciomarine soils dominated (B5, B13: soil profile B). Walls constructed within glaciomarine soil profiles did not have as significant bending as walls in soil profiles where Boston Blue Clay dominated.

BENCHMARKING APPROACH

The online database contains a number of case studies where excavation performance has been benchmarked. The purpose of the benchmarking is: a) To assess the ability of modeling options to capture measured performance, b) To judge what soil parameters are most influential in our prediction methods, c) To gain an insight into the generated wall moments, and d) to create a database of successful benchmarking to be used in future projects.

Currently, the database contains mostly Class "C" predictions (performance matched after project was completed). One Class "A" case study is under progress since the specific project has not been completed to date. Another Class "A" case study included in the database has been presented by Whittle & Hashash (1993).

Two software programs have been utilized for the benchmarking: a) DEEP 2007 Contractor by Deep Excavation LLC, USA, and b) PLAXIS V8.2 by Plaxis B.V., Netherlands. DEEP 2007 performs multistage non-linear elastoplastic soil spring analyses (active/passive soil elements modeled as springs), while PLAXIS performs full soil structure finite element analyses. Both software programs offer non linear hyperbolic elastic soil models. As apparent, the "soil spring" solution can not truly capture horizontal basal movements beneath the wall base as well as vertical surface settlements

Unfortunately, many case studies lacked essential data or information to make benchmarking more accurate. Missing information sometimes included strut or anchor sizes, and almost always elastic soil properties. In cases where support structural data were absent, the author estimated "reasonable" properties from design line loads, apparent earth pressure diagrams using allowable stress design and engineering judgment. In some projects where adjacent buildings were present, the author had to make some guess about the building surcharge based on available information.

BENCHMARKING RESULTS

One of the first case studies to be analyzed was project B12. This excavation involved a 20m diaphragm wall excavation anchored with permanent rock anchors inclined at 45 degrees from the horizontal. Konstantakos et. al., 2003, had benchmarked this case study with Plaxis. The author was able to closely replicate the benchmarking with DEEP, using the same soil strength and elasticity properties. Figures 7.A and 7.B show results of this comparison.

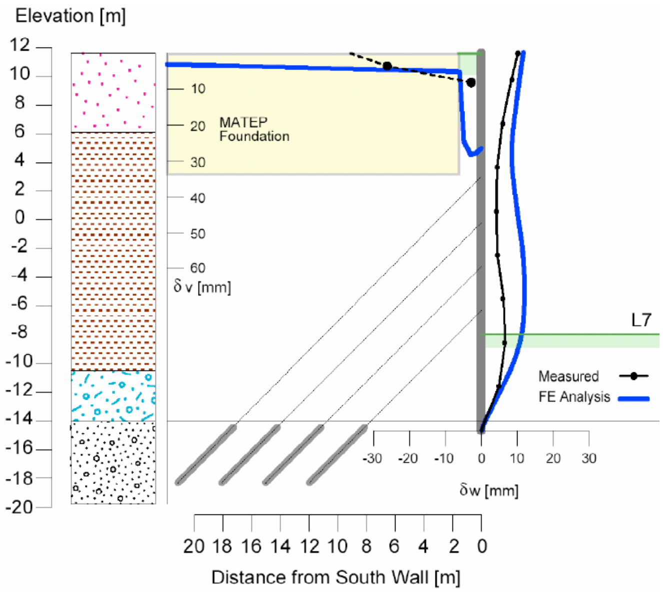

Figure 7(a): Project B12 with measured and benchmarked performance by PLAXIS

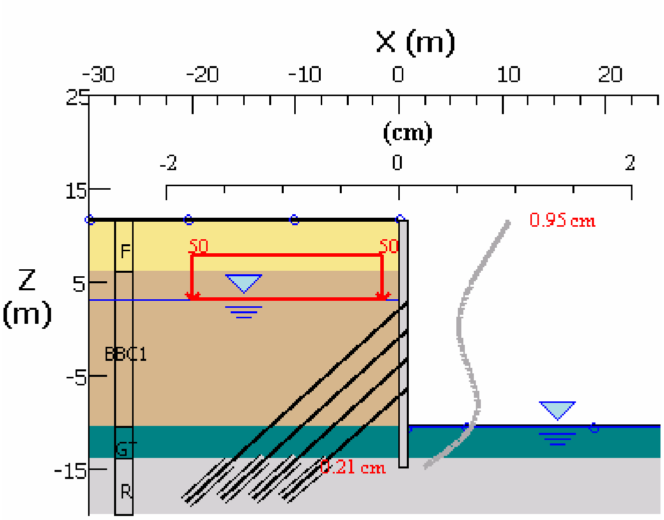

Figure 7(b): Project B12 with benchmarked performance by DEEP 2007 Contractor.

In project B7, the only way to reasonably capture measured performance was to reduce capacities for the top two levels of anchors sufficiently so that yielding would occur. Making such progressive adjustments enabled the author to capture the cantilever wall behavior seen in Figure 4.

Modeling the effect of construction events or other site factors also proved challenging. For example, in project B8, there was a sudden jacking box failure that caused an additional movement of about 1.2 cm (0.5 in). Accurate benchmarking with both PLAXIS and DEEP proved impossible.

Top/down projects in Boston initially proved very challenging to benchmark. The greatest difficulties where encountered in properly capturing the equivalent stiffness of below grade floor slabs. The author has generally discovered that floor slab stiffness should be reduced to about 10 to 15% of the theoretical value. This reduction was necessary to capture additional horizontal movements associated with concrete shrinkage. Figure 8 shows typical benchmarking results from case study B10 constructed with the top/down method. It is interesting to note that some walls in top/down excavations appear to have experienced considerable bending moments. While the author is not aware of the reinforcements used in most case studies, it is likely that bending wall moments may have exceeded the theoretical elastic capacity and as a result a plastic hinge might have formed.

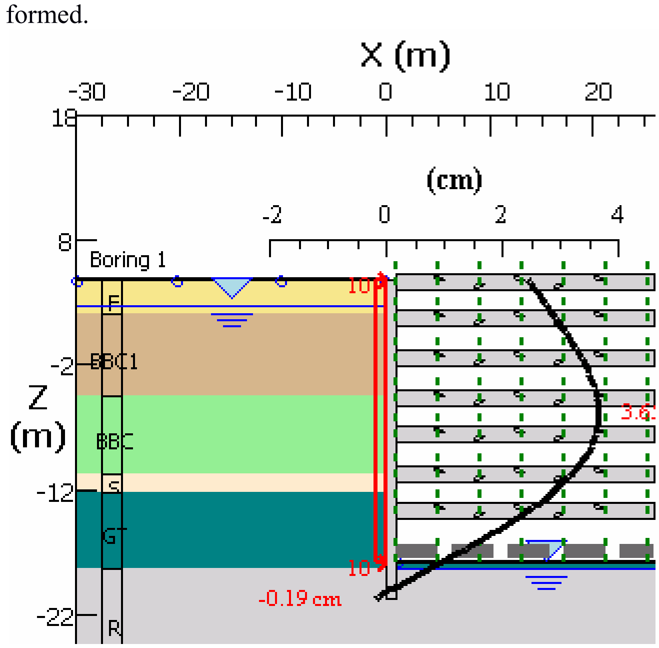

Figure 8: Typical benchmarking model (Case B10)

Where conditions were similar within the same city, the author was able to reasonably capture wall behavior by reusing soil properties from similar benchmarked cases.

This benchmarking exercise also revealed that capturing the initial horizontal state of stress correctly is as important as using proper elastic soil properties.

Figure 9 plots standardized maximum wall displacements (by excavation height) against the benchmarked basal stability safety factor. The benchmarked basal stability safety factor is calculated directly from benchmarked soil strength parameters and not from original design values. As it can be easily seen, most excavation succeeded in allowing smaller than 0.25% wall deflections. Currently, case studies included in the database seem to indicate that basal movements affect wall deformations when the basal stability safety factor is smaller than 1.8.

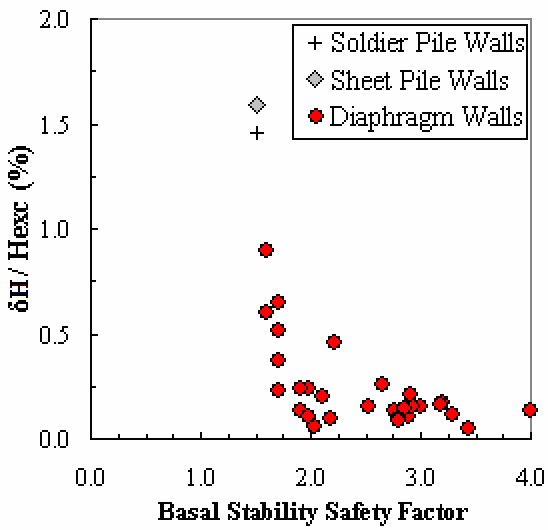

Figure 9: Maximum wall horizontal displacement δH divided by excavation depth Hexc vs. basal stability safety factor.

Finally, the author would like to stress that it is likely that another analyst may match measured performance using different assumptions.

CONCLUSIONS

An online searchable database of deep excavation performance and prediction is briefly presented. At its current state, the database includes 39 case studies, where, mostly diaphragm walls have been used. It is anticipated that the number of case studies will increase with time. Archived information includes soil conditions, wall type, and lateral support system. Furthermore, projects have been categorized according to general construction method and toe embedment characteristics. Most case studies are accompanied by detailed project descriptions, available only online.

As expected, performance of documented excavations has been affected by numerous factors. A key factor affecting performance though proved to be the choice of construction methodology. In many instances where larger deformations were recorded, project performance has been affected by construction induced displacements such as soil losses during anchor installation. In some cases where basal stability was a consideration, the fixed length of ground anchors has been placed within an expanded zone of basal movements. As a result, in some cases, ground anchors did not provide adequate wall restraint.

Back-analyses of excavation performance using 2-D finite element analyses were able to give generally consistent estimates of measured wall deflections on many projects. However, precise benchmarking down to the last millimeter proved an almost impossible task. In some cases, even reasonable benchmarking proved impossible.

This benchmarking exercise revealed one very interesting finding that warrants further investigation. The author discovered that in very few diaphragm wall supported excavations where considerable bow shaped wall displacements where recorded, wall moments might have exceeded possible theoretical wall yield limits and a plastic hinge might have locally formed. Despite this finding, none of the documented diaphragm walls exhibited any structural distress. Furthermore, as the WTC recovery efforts indicated, diaphragm wall systems are remarkably capable of sustaining previously unimaginable displacements without collapse.

The author aims to expand the database and encourages other fellow engineers and corporations to contribute to this effort.

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