MEASURED PERFORMANCE OF SLURRY WALLS ENG

by

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Abstract

This thesis evaluates the measured performance of 29 slurry wall supported excavations in Boston, Chicago, Washington DC, and San Francisco – most of which have been constructed since 1980. Each of these case studies includes data on the initial site conditions (soil profile and properties, groundwater conditions and location of adjacent facilities etc.) and designs for support of the excavations. The main goal is to relate construction records to the measured performance of the lateral earth support systems. The principal parameters of interest are the induced ground deformations (and their effects on adjacent structures) and observations of groundwater flows. The actual monitoring data always include inclinometer measurements of lateral deflections within the diaphragm wall and/or adjacent soil. However, other information such as surface settlements, building settlements, heave of the sub-grade or piezometric data were only archived for some of the projects (nearly all in Boston). Even fewer projects contain measurements of structural forces in either the diaphragm wall or bracing system.

These data have been grouped according to the soil profile, toe fixity of the wall and type of bracing system (tie-back anchors, prestressed cross-lot and top-down). Most of the projects have succeeded in allowing only small wall deflections, often less than 0.2% to 0.3% of the total excavation depth, and similar magnitudes of the maximum surface settlements. Larger wall movements did occur in several projects but have been linked to either inadequate bracing (poor tieback design or inadequate pre-stressing of rakers), lack of toe embedment or ground softening inside the excavation (installation of drilled caissons or load bearing elements). Unexpectedly large surface settlements in one project (Dana Farer) were clearly linked to ground loss during tieback installation. Several other reported cases of leakage (through panel joints and/or tiebacks) have been repaired by grouting.

Given the limited availability of archival data, the thesis has focused on the interpretation of lateral deflections. Wall deformations have been sub-divided into rigid body translation, rigid body rotation and bending modes. Empirical correlations have been proposed for estimating each of these components.

Thesis Supervisor: Prof. Andrew J. Whittle
Title: Associate Professor of Civil and Environmental Engineering

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Once again to my research supervisor at MIT, Prof. Andrew J. Whittle for picking me to work on such a practical and useful research, and for all the help and guidance. Also, to my parents and sister for their continuous support throughout my studies at MIT. I hope this research proves to be useful for the whole civil engineering community within and outside the United States. The world out there is waiting.

Dimitrios Konstantakos, 16- May- 2000.

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Introduction

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1.1 Introduction

The continuous diaphragm wall (also referred to as slurry wall) is a structure formed and cast in a slurry trench (Xanthakos, 1994). The trench is initially supported by either bentonite polymer based slurries. The term "diaphragm walls" refers to the final condition when the slurry is replaced by tremied concrete that acts as a structural system either for temporary excavation support or as part of the permanent structure. The term slurry wall is also applied to walls that are used as flow barriers (mainly in waste containment), by providing a low permeability barrier to contaminant transport. This thesis focuses only on the structural role of diaphragm walls when used for deep excavation support.

1.2 A Brief Historical Overview

Slurry wall technology hinges on specialized equipment for excavating slurry trenches. The simplest type of trenching equipment is the mechanical clamshell attached on a kelly bar. Individual contractors have developed their own specialized trenching equipment like hydraulic clamshells, fraise or hydromills (sample manufacturers: Icos, Bauer, Casagrande, Case Foundation, Rodio etc). A more detailed description of trenching equipment is given in section 2.2.

The first diaphragm walls were tested in 1948 and the first full scale slurry wall was built by Icos in Italy in 1950 (Puller, 1996) with bentonite slurry support as a cut-off wall. Icos constructed the first structural slurry wall in the late 1950s for the Milan Metro (Puller, 1996). Slurry walls were introduced in the US in the mid 1960s by European contractors. The first application in the US was in New York City [1962] for a 7m diameter by 24m deep shaft (Tamaro, 1990), that was followed by the Bank of California in San Francisco (Clough and Buchignani, 1980), the CNA building in Chicago (Cunningham and Fernandez, 1972), and the World Trade Center in New York (Kapp, 1969, Saxena, 1974). The majority of diaphragm wall projects in the US are located in six cities Boston, Chicago, Washington DC, San Francisco and New York.

Diaphragm walls are extensively used in the Central Artery/Tunnel project (CA/T) in Boston, Massachusetts (Fig. 1.1). Work in the CA/T involves many cut and cover tunnels constructed under the existing artery. Some of the deepest T-slurry walls, extending 120' below the surface have been constructed for the Central Artery (Lambrechts et al., 1998).

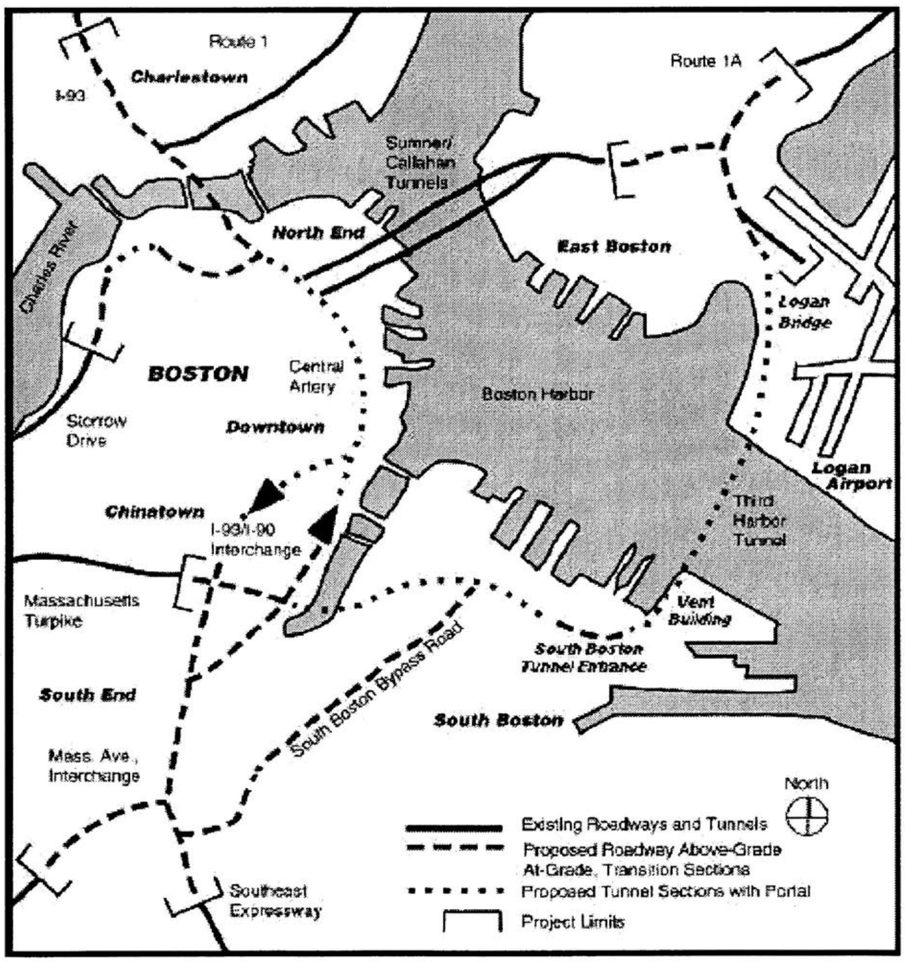

Figure 1.1: Central Artery Tunnel Project, Boston MA (Ladd et al., 1999)

1.3 Advantages & Disadvantages of Slurry Walls

The critical design criteria for diaphragm walls are: 1) structural strength and integrity, 2) permanence, and 3) impermeability (Millet and Perez, 1981). According to Puller [1996] diaphragm walls are generally efficient in cost and construction time where they are used for both permanent and temporary subsoil retention for walls of medium, and greater depth.

Some of the quoted advantages of diaphragm walls are (Puller, 1996; Tamaro, 1990; Hajnal et al., 1984):

1) They perform in multiple functions simultaneously, earth retention, groundwater flow control, and load bearing.
2) Minimization of excavation induced deformations since diaphragm walls are stiffer than sheet piling.
3) Minimization of water leakage and eliminate dewatering outside the site.
4) Eliminate underpinning of adjacent structures
5) The practical wall depth is limited by the properties of the excavating machinery
6) The method is mechanized and thus savings on labor can be achieved.
7) Slurry wall installation causes very little noise compared to traditional pile and sheet pile driving.
8) Layout arrangement is variable and can be adjusted to meet local conditions.
9) No major surface grading is required prior to trenching

The disadvantages of diaphragm walls are (Puller, 1996; Tamaro, 1990; Hajnal et al., 1984):

1) Except for precast panels, the surface quality of diaphragm walls depends on the equipment and the type of soil to be excavated.
2) Special precautions have to be taken to handel waste products of the slurry operations.
3) Inspection during construction is only possible by indirect methods since the wall is prepared under slurry.

4) Special precautions have to be taken when slurry walls are to be constructed a) in open water, b) in layers bearing artesian water, c) in poorly compacted fill, d) near existing buildings, or unknown utilities, e) in soil where a significant and rapid slurry loss is anticipated.
5) Generally more expensive than other methods.

However, the quoted advantages of slurry walls may not always materialize. For example, there have been very few cases where the effects of trenching and wall installation on deformations were considered on the overall performance (e.g., Koutsoftas et al., 2000). Soil movements during trenching may become crucial in cases where deformation tolerances are very strict. Slurry walls tend to be more watertight than sheet pile walls but total sealing can be a very difficult, if not impossible to achieve. Local defects in the diaphragm walls and in holes opened for tiebacks tend to control leakage. Much of the reasoning for using slurry walls for the permanent structure is relating to the advertised watertightness.

Furthermore, it is not clear if the stiffness of diaphragm walls is a big advantage since comparable stiffness can be obtained by built steel sections (e.g. sheet pile and H-pile combination). The actual magnitude of deformations depends on the used bracing system as much as the choice between a diaphragm wall and a sheet pile or other type of a retaining wall. Unfortunately, there have not been any cases of direct comparison of performance of slurry walls and sheet pile walls of comparable stiffness under the same conditions.

1.4. Goals of Thesis

This thesis provides an update (FHW, 1980) on the use and experience of slurry wall practice in the US, mainly for deep basement construction. This has been achieved by developing a database of well documented projects.

1.5 Research Methodology

The projects studied in this research were selected by a) reviewing existing literature, and by b) contacting companies for help in identifying well-instrumented case studies. Each case study provides information on:

a) Site conditions, soil profile etc.

b) Design of the wall and bracing system.

c) Summary of instrumentation.

d) Assessment of performance:

- Deformations, i.e. wall deflections, settlements.
- Construction problems (panel collapses etc.)
- Other data: water table & piezometric level readings, strut loads etc.
- Unexpected or unusual features.

Once a sufficient database was created then performance was compared on a city by city basis (Chapters 4, 5, 6, and 7). Finally, the performance of all the case studies were concluded in Chapter 8. When applicable and possible, the effect of evolving construction practices has also been discussed. Measured performance data is reported according to existing prediction methods (e.g. Clough et al., 1989, Peck, 1969).

1.6 Studied Projects

In the course of compiling projects, it became obvious that the applications of slurry walls are concentrated mainly in 5 major urban areas – Boston, Chicago, Washington DC, San Francisco, and New York. The main focus of this thesis is in the first three cities where the author was able to obtain relatively complete data from archived records. In San Francisco there is only one recent slurry wall project studied since local practice has shifted towards SPTC walls. Unfortunately, we were not able to locate any data on recent diaphragm wall projects in New York. The full data on each case study is presented in a separate data report (Konstantakos and Whittle, 2000). Tables 1.1 (a), (b), (c) provide the complete list of the studied projects.

Table 1.1 (a): Boston Slurry Wall Excavations

ID	Project Name	Year	Exc. Depth (ft)	Wall (inches)	Bracing & Wall	Level of Instrumentation or available data
B1 *	MBTA South Cove	1973	50	36	3-Levels CLB	Good
B2 **	60-State Street	1975	35	30	3 & 2 –Levels TB	Very Good
B3	State Transportation Building	1982	27	24	2-Levels TB	Good
B4 *	75 State Street	1983	65	30	6-Levels TD	No data
B5	Rowes Wharf	1984	55	30	5-Lev TD	Poor
B6	One Memorial Drive	1985	30	24	2 or 1 Levels TB	Good
B7	500 Boylston	1987	42	24	4 Levels TB, 1 Level TB & 2 R	Very good
B8	Flagship Wharf	1989	47	30	3-Levels CLB, PT	Very good
B9 *	125 Summer Street	1990	60	30	6-Levels TD	No data
B10	Post Office Square Garage	1991	75	36	7-Levels TD	Excellent
B11	Beth Israel Deaconess	1994	65	36	5-Levels TD	Very good
B12	Dana Farber Tower	1995	90	36	6-Levels TB	Excellent
B13	Millenium Place	2000	55	36	TD	Excellent

Note: TB -Tiebacks, CLB - Cross-Lot Bracing, TD - Top/down, R – Rakers, SB- Soil Berms, CB – Corner Bracing, PC – Precast, PT – Post Tensioned, SP – Soldier Piles

* Project only referenced from existing literature, not studied in detail

Table 1.1 (b): Chicago Slurry Wall Excavations

ID	Project Name	Year	Exc. Depth (ft)	Wall (inches)	Bracing & Wall	Level of Instrumentation or available data
C1**	CNA Building	1971	31	30	1 Level R, SB	Fair
C2 **	Sears Tower	1971	32	30	3 Levels R, SB	Poor
C3 **	Amoco Standard Oil	1973	23 44*	30	1 Level TB, SP, B	Good
C4 **	Water Tower	1974	44	24	1 Level TB & 1 Level R	Very good
C5	Loyola University Business School	1993	20 est.	---	1 Level Struts	Very poor
C6	Prudential Two	1986	25	27	1 Level. TB, & 1 Level R, CB	Good
C7 **	AT&T Corporate Center	1987	27	30	3 Levels R	Good
C8	Guest Quarters Hotel	1989	35	24	3-Levels TD	Good
C9	Northwestern University Memorial Parking Garage	1990	23	24	1 Level TB	Good
C10	Museum of Science & Industry	1997	34	30	3 levels permanent TB	Good
C11 *	311 South Wacker Drive	1987	35	24	TB, TD, CB, R	Good

Table 1.1 (c): Washington DC & San Francisco slurry wall excavations

ID	Project Name	Year	Exc. Depth (ft)	Wall (inches)	Bracing & Wall	Level of Instrumentation or available data
	Washington					
W1	World Bank	1991	60	30	5 Levels TB	Excellent
W2	Petworth Subway Station	1995	60 - 100	36	5 – 6 Levels CLB	Below average
W3	Washington Convention Center	2000	30 – 55	36 – 48 36	1 or 2 Levels TB, & 1 Level R 3 Levels TB,	Very good
W4 *	Metro Center II	1991	31	24	2 Levels TB	
	San Francisco					
S9	Yerba Buena Tower	1999	66	36	2 Lev. CB, 1 Lev. R or 3 Lev. TB, and 1 Level R	Excellent

Note: TB -Tiebacks, CLB - Cross-Lot Bracing, TD - Top/down, R – Rakers, SB- Soil Berms, CB – Corner Bracing, PC – Precast, PT – Post Tensioned, SP – Soldier Piles

* Project only referenced from existing literature, not studied in detail

** Summary of performance based on existing literature and/or revisited archived data.

Chapter 2

Slurry Wall Construction

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Slurry Wall Construction

2.1 General Methods of Slurry Wall Construction

Slurry wall design and construction requires careful consideration of many factors including panel size, slurry materials (i.e. processing), and excavating equipment. For example, the depth of the slurry wall may be dictated by the soil conditions in the site, while the site layout may constraint panel sizes. Adjacent buildings and existing utilities are always encountered in urban excavations and they always have to be protected or relocated. Waterstopping details should be given special attention since slurry walls are often part of the permanent structure. Working schedules can also be affected by requirements for traffic maintenance. Thus construction procedures should address such and other issues in order to optimize the whole construction.

A slurry wall is constructed by joining a series of slurry wall panels in a predetermined order. The panels are excavated to specified dimensions while slurry or another stabilizing fluid is circulated in the trench. Excavation equipment ranges from clamshell buckets, hydraulic clamshells to hydrofraises (Xanthakos, 1994, Parkison & Gilbert, 1991, Ressi, 1999, Bauer, 2000). Individual contractors have developed their own (typically) patented trenching equipment. Figure 2.1 shows a variety of trenching equipment used for slurry wall construction. The major types of excavating equipment are as follows:

I **Mechanical Clamshells**: Mechanical clamshells are simple and efficient devices that use mechanical power to move the buckets and excavate soil. They are not fixed with a crane and can work in mixed ground. A big advantage is that the clamshell can be changed with chiseling equipment very easily when rock has to be excavated. (Figs. 2.2 a, 2.3 a)

II **Hydraulic Clamshells**: Hydraulic clamshells are more productive than mechanical clamshells, but are more expensive and more difficult to set up. Hydraulic clamshells use hydraulic power to move the excavating buckets instead of mechanical power. Chiseling of rock is not easy with the same machine when hydraulic clamshells are used. (Fig. 2.2 b, Fig. 2.13 b)

III Hydromills: (Fig. 2.4). Hydromills have two counter rotating mills and work with reverse circulation. Excavated material is brought to the plant where coarse and fine materials are separated. They are more expensive than hydraulic clamshells and thus they are only used for large-scale projects (Ressi, 1999). In contrast to the hydraulic clamshell, the hydromill can handle rock chiseling. One design issue which arises when chiseling rock is the removal of the rock left between the two mills. Different contractors have developed techniques for knocking of this notch. One approach relies on vibration of the wheels to remove the rock notch. Initially, the hydraulic motors where situated in the wheels and that restricted the motor size and the available power. In contrast, Casagrande has designed a hydromill with which a toothed chain to drive the wheels, and also cut the rock notch. The chain cannot handle hard rocks (breakage of chain) and was changed such that the chain was dragged along and power was transmitted separately to the wheels. The German BAUER (http://www.bauer.de/index/htm) hydromill (Fig. 2.6) uses a set of kicking teeth at the bases of the wheels to knock of the rock notch, but rock the teeth can break in hard rock.

IV Hydrofraise: The Hydrofraise system was developed by Soletanche (Parkison & Gilbert, 1991), and was designed to excavate cohesionless soils as well as hard rock in a single pass without chiseling. It consists of a) a heavy-duty crawler crane of 100 to 150 ton capacity; b) a hydraulic power pack; c) the hydrofraise equipment; and d) the slurry treatment plant (Fig. 2.5). Three motors are located at the base of the system, two of which power the cutting drums while the third operates a special pump mounted centrally just above the cutting drums (Xanthakos, 1994). The panel width can be controlled by changing the cutting drums from 25" to 5ft. According to Bachy-Soletanche quotes the system can be used to excavate walls down to 400-ft (http://www.bachy.com/). Bauer has developed its own range of cutters (Fig. 2.6)

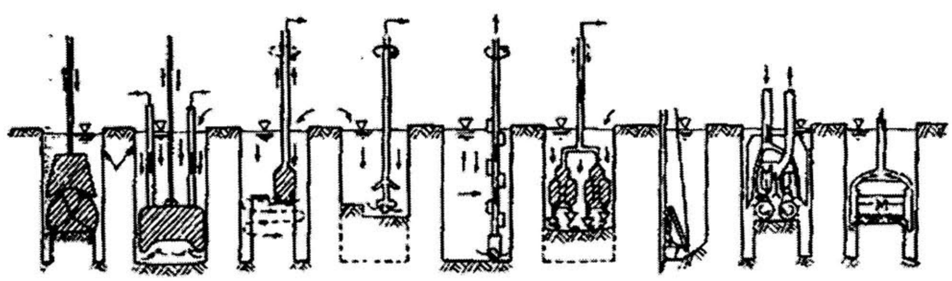
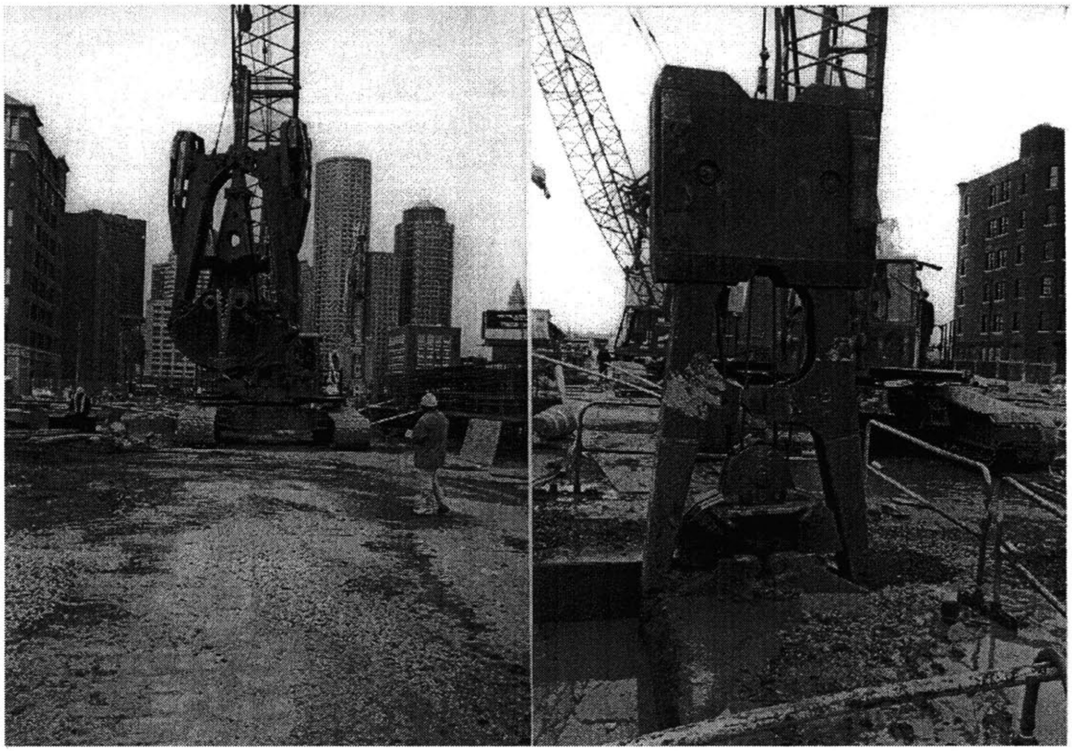
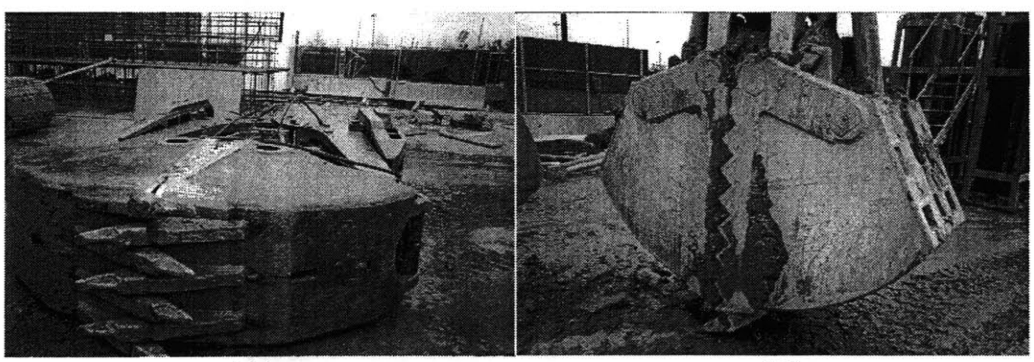


Figure 2.1: Trenching Equipment (Xanthakos, 1991), (a) Clamshell bucket attached to a kelly. (b) Vertical percussive bit with reverse circulation, (c) Percussive benching bit. (d) Rotary benching bit. (e) Rotary bit with vertical cutter. (f) Rotary drilling machine with reverse circulation. (g) Bucket scraper. (h) Bell-mouth suction rotary cutter with direct circulation. (i) Horizontal auger machine.



(A) (B)

Figure 2.2: Trenching equipment, (A) Mechanical clamshell in front and hydraulic clamshell in the back, (B) Smaller size mechanical clamshell



(A) (B)

Figure 2.3: Close-up pictures of excavating buckets, (A) Mechanical clamshell, (B) Hydraulic clamshell

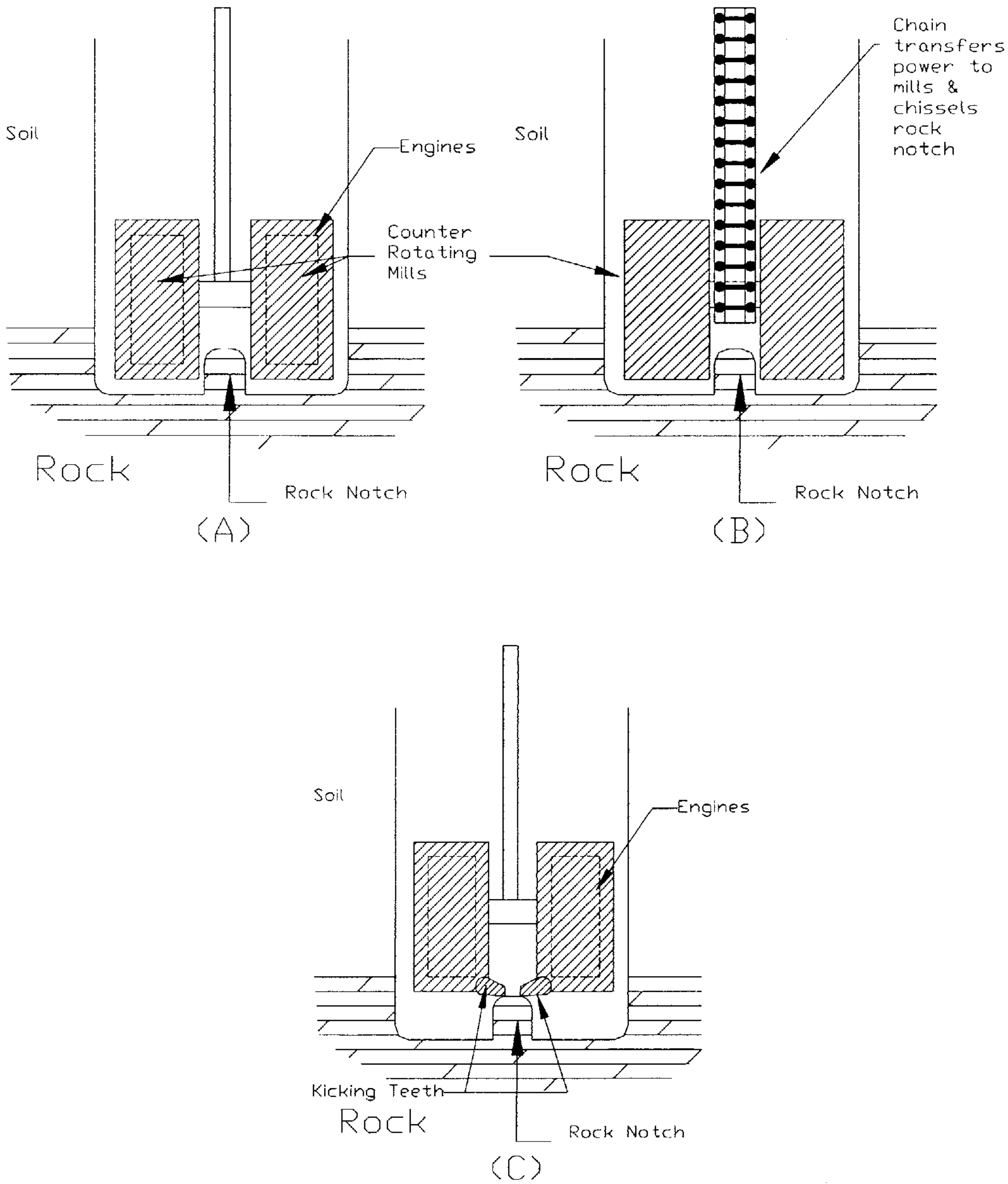


Figure 2.4: Hydromills, (A) Original hydromill with reverse circulation, powering engines are within the counter rotating mills (B) Casagrande design, (C) Bauer hydromill.

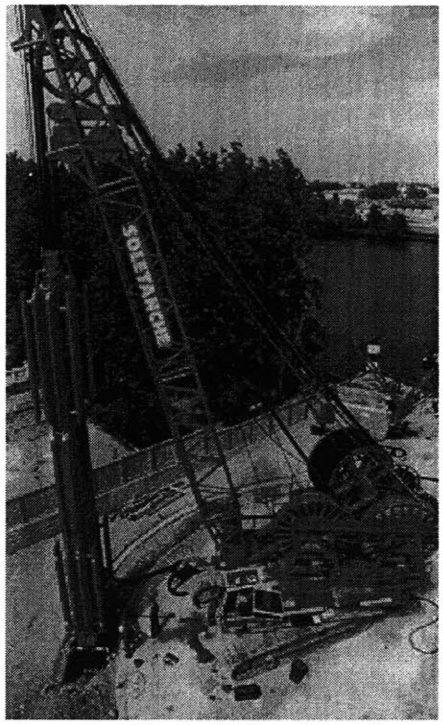
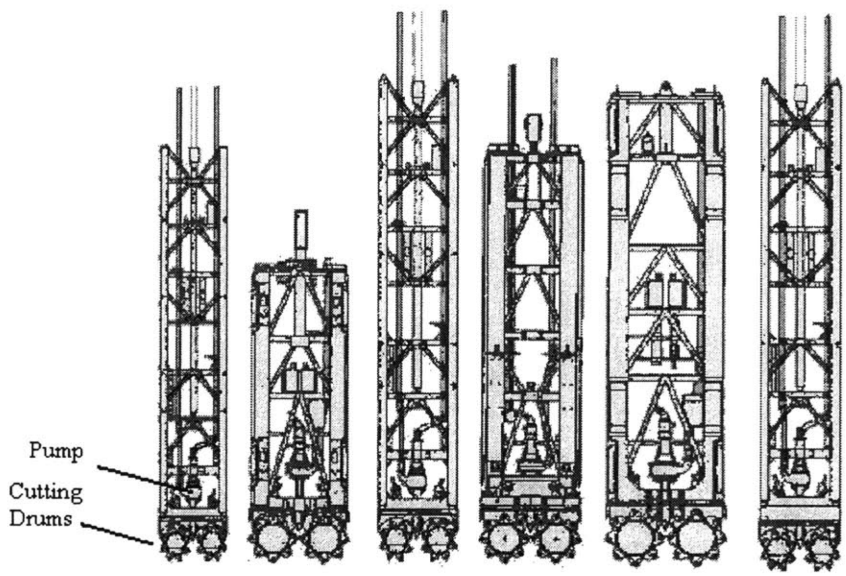


Figure 2.5: Soletanche Hydrofraise (http://www.bachy.com/)



	BC-20	BC-25	BC-30	BC-33	BC-40	BC-50
Torque Kn x m	2x81	2x81	2x81	2x81	2x100	2x135
Width (mm)	500-1000	640-1500	640-2400	640-1500	800-2100	1200-3200
Length (mm)	2200	2800	2800	2800	2800	2800-3200
Pump	6"	5"	6"	6"	6"	8"
Weight (tons)	12-20	25	25-35	20-35	30-45	40-50

Figure 2.6: Bauer trench cutters (http://www.bauer.de/index.htm)

Figure 2.7 illustrates the basic steps in typical slurry wall construction, while Figures 2.8 and 2.9 show selected pictures from construction of a new subway in Boston (MBTA South Boston Transit way). The first stage involves clearing the site of possible obstructions. Guide walls are then constructed to help stabilize the upper few feet of soil and guide the trenching equipment (controlling the verticality of the panels). End-stops are inserted into the panel when trenching has finished in order to help form water-tight joints connecting adjacent panels. The end-stops are withdrawn when the adjacent panel is trenched.

Once a panel is excavated to the specified dimensions, then a reinforcement cage is inserted into the slurry filled trench (Fig. 2.7). Occasionally reinforcement cages are spliced if the required cages are too heavy for lifting equipment. In some projects soldier piles have also been used as slurry wall reinforcement or as bearing elements of the slurry wall (Case studies C3 and C6).

The bottom of each panel is cleaned before concreting since sands and other soils may form intrusions that undermine the wall integrity (water-tightness, strength, and stiffness). Concrete is then carefully tremied into the trench and continuously displaces the slurry. Two or three tremie pipes are usually used to concrete each panel (typically 20ft long) but up to four pipes have been used to accelerate construction. The freshly tremied concrete is then given time to harden and construction progresses with the construction of another panel. The top few inches of the panel are always chipped as to expose fresh and competent concrete since slurry is trapped in the upper few inches of the panel.

One of the major issues during concreting is segregation of concrete aggregates during fast concreting. Occasionally slurry is entrapped within the tremied concrete and thus soft zones are created within the slurry walls. If the panel bottom is not adequately cleaned then the soil and the waste that may have accumulated in the bottom is displaced upwards during concreting. While part of this "waste" is carried to the top of the wall where it is later cleaned, another part flows into the bottom corners of panels and between the panel joints. Such problems can cause large leakage problems. Successful construction relies heavily on good quality control on site.

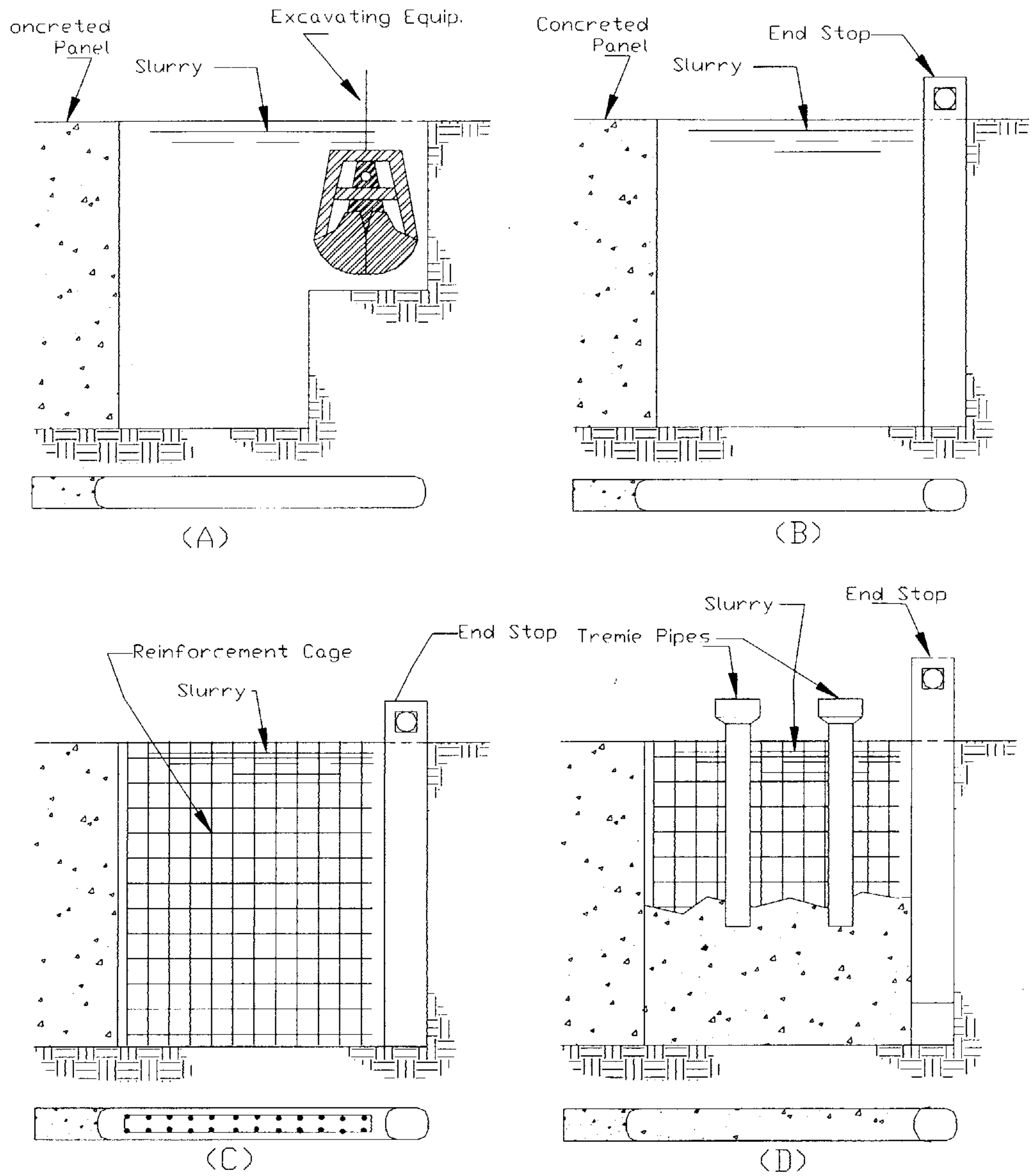


Figure 2.7: Typical construction sequence of slurry walls: (A) Trenching under slurry, (B) End stop inserted (steel tube or other), (C) Reinforcement cage lowered into the slurry-filled trench, (D) Concreting by tremie pipes.

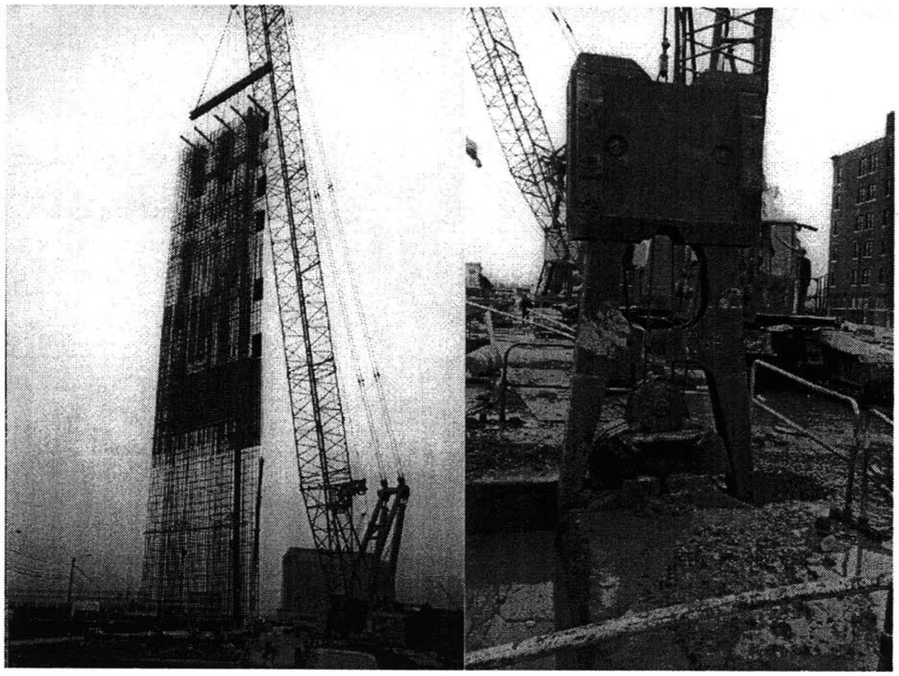
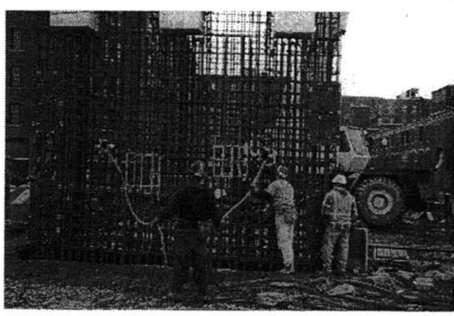
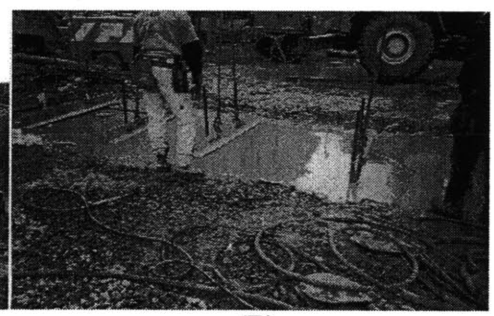


Figure 2.8: Slurry Wall Construction (Reinforcement cage left, scale given by worker near the cage, Trenching of a T-panel right)



(A)



(B)

(C)

(A) The reinforcement cage is Inserted into the slurry filled trench
(B) The reinforcement cage is tottaly inside the trench
(C) Concreted panel with end-stops still in place

Figure 2.9: Slurry wall construction for the MBTA Courthouse subway station in South Boston

Panels are typically constructed in an alternate sequence (Fig. 2.10) since freshly tremied concrete needs time to gain strength before the adjacent panel can be constructed. As Figure 2.10 illustrates, the intermediate panel is constructed after the two primary panels have gained sufficient strength. Continuous linear construction sequence is rather rare but it has been used in large-scale transportation projects (TreviIcos Company Brochure, 1999, Parkison & Gilbert, 1991).

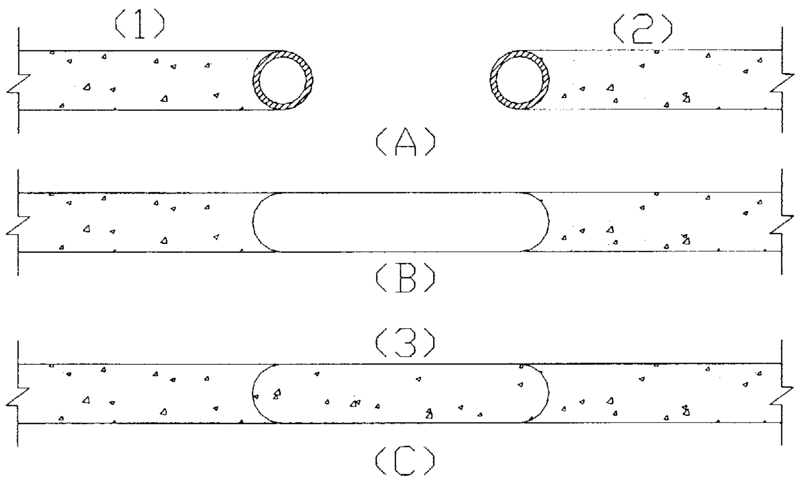


Figure 2.10: Alternate panel construction sequence, (A) Panels 1 & 2 concreted, (B) Middle panel trenched, end stops removed, (C) Middle panel concreted.

The panel trenching sequence obviously depends on the panel size and the type of trenching equipment that is used. In small panels, only one equipment pass is required, but more are required for longer panels. For a typical 20ft panel trenching is done in three bites (Fig. 2.11), beginning with the outer two bites and proceeding to the middle bite. This reduces the potential for trench instability by minimizing the time of full panel length excavation prior to concreting.

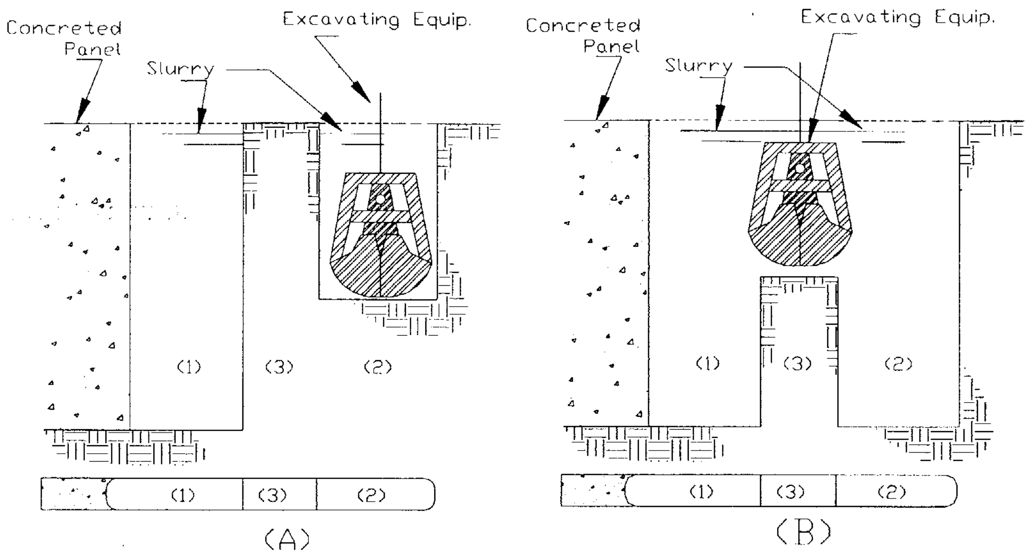


Figure 2.11: Typical trenching sequence, (A) outer bites excavated, middle bite left in place, (B) middle bite excavated typically on the same day when concreting is scheduled.

2.1.1 Panel Size

The selection of panel size is determined by the site conditions and by the excavating equipment. Smaller panel lengths enhance trench stability through arching, but increase the required number of construction joints. Longer panels on the other hand require fewer construction joints but are less stable during trenching. Panels have been constructed up to lengths of 30' feet, but the typical length is 20'. The wall thickness typically ranges from 2' to 4'. The excavating equipment determines the panel thickness as well as the minimum possible panel size, since a panel can not be smaller than one pass of the equipment.

The panel depth depends on design requirements and is limited by verticality tolerance limits 1/100th to 1/200th of wall depth (Xanthakos, 1994, ICE, 1996). Where it is economically feasible panels are keyed into stiff strata so that movements of the wall and of the retained are minimized during the excavation. In some cases panels have been constructed with stilts (i.e. where a fraction of the panel is extended deeper in a key fashion) that extend down to firmer strata (B11).

2.1.2 Bentonite and Polymer Slurries

Slurry does indeed possess remarkable properties which make it suitable for trenching (Parkison & Gilbert, 1994). The slurry has the following basic tasks: a) support the excavated trench, b) fill in voids in the trench, c) keep solids excavated in suspension. Slurry in a trench can be regarded as a plastic fluid due to the continuous movement of the trenching tool, while slurry infiltrating into the soil acts as a thixotropic fluid in a gel state.

The traditional bentonite slurry that has been used in most projects and only lately polymer mixtures have been developed. The use of additives in the blend remains limited, except for a few alchemists who concentrate on the slurry and ignore other aspects of the work (Tamaro, 1990).

During trenching an impermeable barrier is formed at the interface, which functions in two ways a) it separates the soil from the slurry, and b) it allows the slurry to exert its full hydrostatic thrust (Xanthakos, 1994). A seal is formed by colloid particles that are deposited along the interface according to "thixotropy", which describes the tendency of particles of the same nature to adhere upon contact. This deposition and accumulation of slurry particles forming a packed zone of solid materials, is commonly called "filter cake". The process of "filter cake" formation is affected by the permeability of the soil. A "filter cake" can not be formed in open grounds (high permeability), where there are large voids, or where the penetration is close to zero.

A typical bentonite slurry mix design is given in Table 2.1. Selection of the appropriate mix involves satisfying contradicting requirements. To facilitate adequate displacement by concrete, the slurry mix must have a low viscosity, but lowering the viscosity decreases the ability of the slurry to carry suspended solids. The initial fluid loss decreases with increasing bentonite content and increasing sand content (Hutchinson et al, 1974). However, the mix can become too viscous to use if the bentonite content is too high (Hodgson, 1977). A Marsh funnel viscometer is almost always used as an index test to assess the mix viscosity on site.

Polymer slurries on the other hand work by decanting the sand and solids to the bottom of the panel (Ressi, 1999). Panels trenched with polymers are easier to clean than panels trenched with bentonite but one has to wait for the sand in the slurry to settle down. Polymers do not form a filter cake as bentonite does and thus they can not be used in more permeable soils due to loss of polymer in the surrounding soil. Individual companies have developed their own types of patented polymer slurries, and as a result there is no published information regarding their properties.

Polymers are easier to dispose of than bentonite as they can be dissolved the easily and cheaply by using additives. However, the polymers can not be used in sites with organic soils since they can dissolve the organics, inducing settlements etc. Polymers have much smaller viscosity than bentonite and thus there is no need to chip the top of the wall, as there is a clean displacement by the concrete tremie. A cost-effective combination is to mix a little bit of bentonite with polymer so that a filter cake is formed (Ressi, 1999). In general selection of mixes is more like an art than a science and previous experience can be very valuable.

Table 2.1

Typical Slurry Mix Specifications & Observed Conditions (from case studies)

Parameter	Quantity
Mix	300 lbs. Sodium Bentonite per 600 gals of Water, (6% by weight)
Sand Content Prior to concreting	<5% by weight
Unit Weight	Controlled from 66 pcf to 75 pcf
Physical Purity	90% Montmorillonite minimum, 10% native sediment, maximum
Chem. Purity	60% Sodium montmorillonite min., 40% calcium and magnesium
PH	Controlled between 7 to 11
Viscosity*	40 sec max. with a standard Marsh Funnel Viscometer
Filtrate Loss	20cc maximum in 30 minutes**
Dry Fineness	80% minimum passing #200 mesh

Notes:

* Tests based on a suspension of 6% solids, by weight mixed in distilled water

** Other specifications control the loss from 15cc to 30 cc in 30 minutes.

2.1.3 End Stops

End-stops are used to form the construction joints between adjacent panels, in order to assure water-tightness. Until 15 years ago, end-stops were simple steel tubes used to form semicircular panel joints (Fig. 2.12). The end-stops were withdrawn to allow construction of the next panel, but with this configuration the concrete at the panel joints tended to spall off. Round end construction joints are difficult to seal.

In order to improve the situation, a second generation of end-stops included a key in the form of a thin sheet pile with a male notch to form the water-stop. The steel piece is knocked off in the process of excavating the adjacent panel. Another approach is to use a deformable sheet that can accept a chemical water-stop material in one side. In both cases the pulling of the end stops is a very tricky matter and experience is always helpful. Pictures from various keyed end stops can be seen in Figure 2.13.

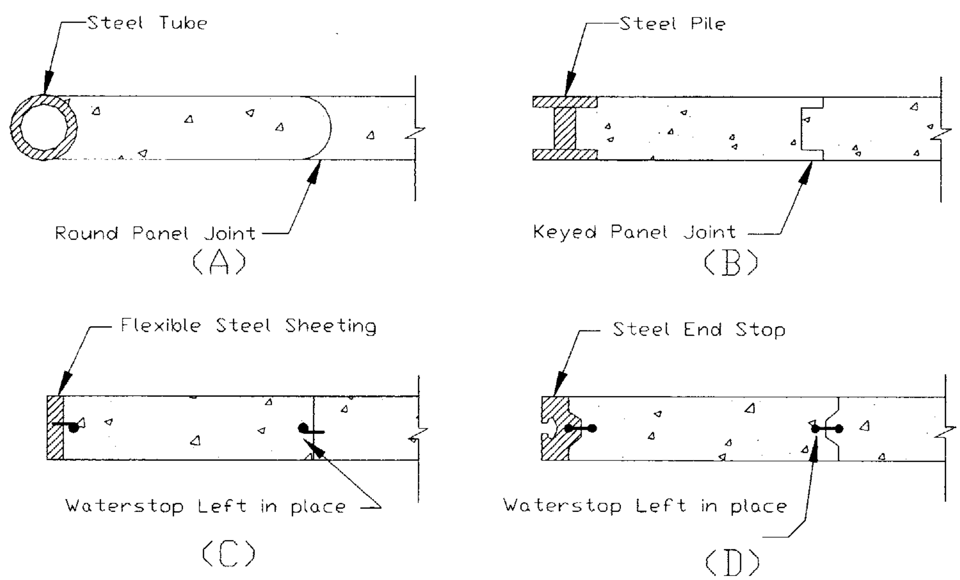


Figure 2.12: End stops and resulting panel joints, (A) Round tube, (B) Steel H-Beam, (C) Flexible sheet pile with male waterstop notch, (D) Keyed steel end stop with chemical waterstop. (Xanthakos, 1994, Parkison & Gilbert, 1991, Ressi, 1999).

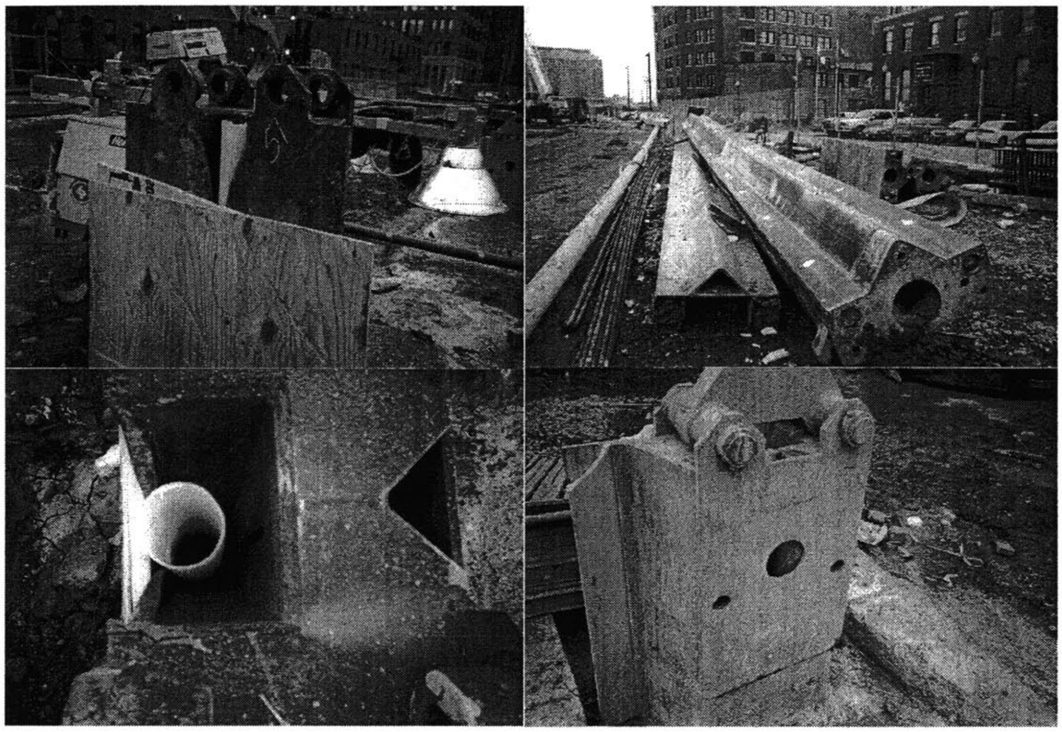


Figure 2.13: Top left: end stop in ground with wooden plank behind notch used to insert waterstop, Bottom left: details of same end stop (top view), Top right: two key end-stops resting on the site , Bottom right: Key end stop without notch (MBTA Courthouse Subway Station, Site visit, Nov. 1999)

2.1.4 Guide Walls

Guide walls are simple reinforced concrete sections of a suitable configuration built at grade and along the exact alignment of the trench (Xanthakos, 1994). They are constructed before any trenching and serve only for temporary construction. The exposed side of the guide wall is eventually dismantled, while the side in the retained part of the soil is usually left in place.

Some of the functions of guide walls are:

(I) They layout the plan of the slurry wall panel and control the range of movement of the trenching equipment.

(II) They help the verticality of the excavating equipment and thus aid vertical panel construction.

(III) They retain the typically unstable upper few feet of soil, and protect it from dynamic vibrations and loads from construction equipment.

(IV) They protect the upper section from the up-down passage of the trenching equipment

(V) They can allow for the excavation to start from a lower level and thus one can trench around buried utilities.

(VI) Together with the trench they function as a reservoir for the slurry.

(VII) They support prefabricated panels when used.

Figure 2.14 shows guidewalls in soft ground and typical rectangular and a corner guidewalls. Figure 2.15 shows an actual guide wall. The distance between guide walls is typically 2" to 4" wider than the required wall thickness. If guide walls are not constructed deep enough then the retained soil may become unstable and slide under the guide wall. During concreting, the tremie concrete may bulge just below the guidewalls requiring chipping at the excavation side.

In sites next to busy streets care should be given guide wall stability since traffic vibrations may cause the guide walls to collapse. Unreinforced guidewalls have been constructed but they have a tendency to break and collapse into the trench.

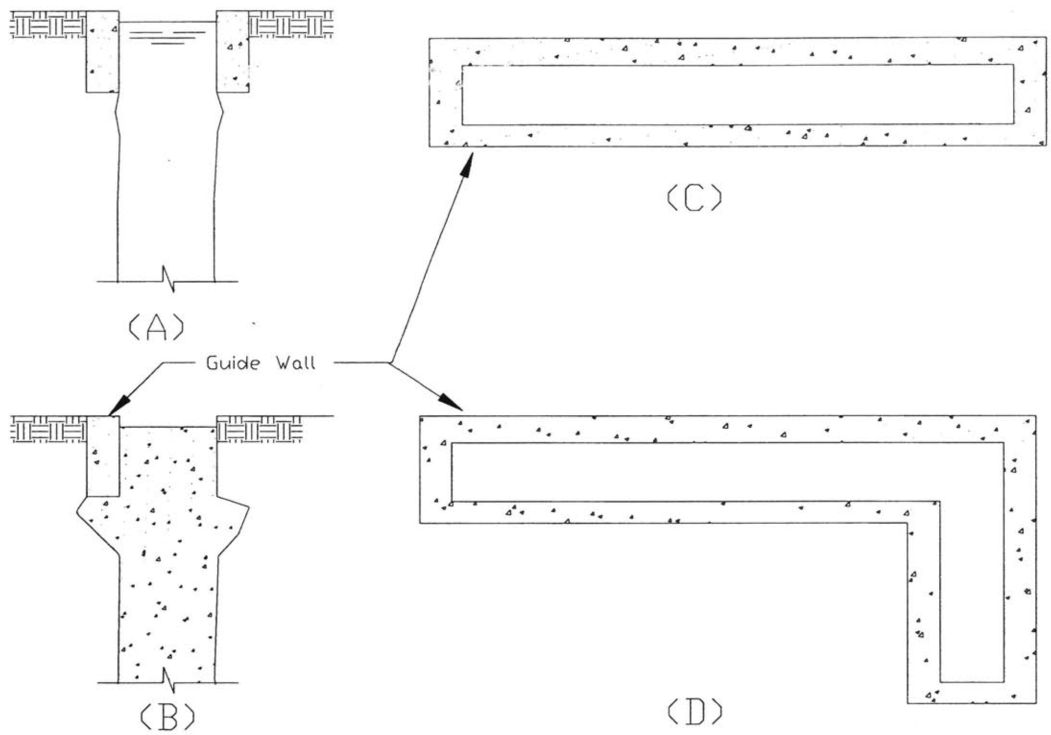
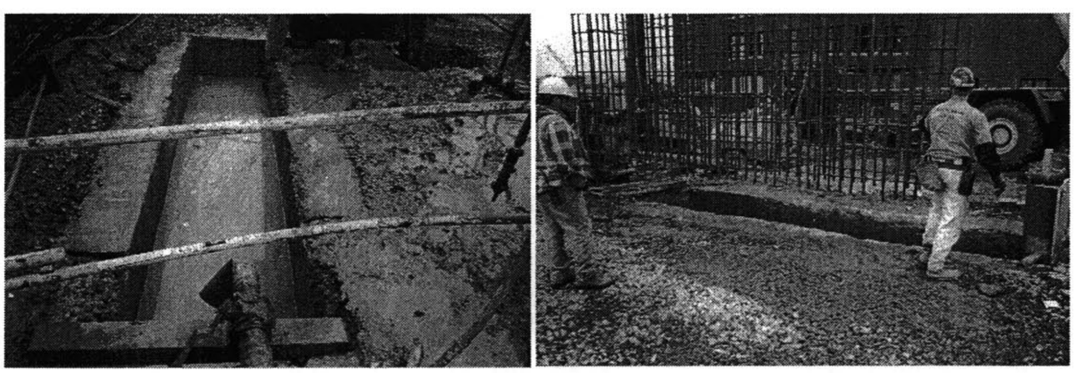


Figure 2.14: Guidewalls, (A) Guidewall and trench under slurry, (B) Guide wall and concreted panel, note the concrete bulges usually observed under the guidewalls, (C) Plan of a rectangular guide wall, (D) Plan of a corner guidewall



(A) (B)

Figure 2.15: Guidewalls, (A) corner guidewall during trenching, (B) rectangular guidewall prior to lowering of the reinforcement cage.

2.1.5 Tremied Concrete

The big issue with tremied concrete is the uncertainty of the quality and strength since the process is not standardized. Concrete in slurry wall panels cures under ideal conditions since it is constantly in contact with pore water in the surrounding soil. It has been claimed that the concrete strength obtained by testing core samples from slurry walls 2 years after the panels were concreted averaged f_c'=8000psi whereas the design strength was only f_c'=5000psi (Hosseini, 1999). Unfortunately, we have no data to verify this claim. Problems arise when slurry or soft material is entrapped between the concrete during the tremie process. These zones of soft material have very little to no strength and allow water leakage through if exposed. If the tremie process is done slowly and carefully then such problems can be avoided.

When concrete is tremied overpouring is required in order to push out the slurry contaminated concrete in the upper few feet of the wall. Despite the overpouring, the upper foot of the slurry wall concrete is still contaminated by slurry and thus it is chipped off. Slurry contaminated concrete can easily break with finger pressure (Fig. 2.16).

Figure 2.16: Top concrete layer, slurry contaminated concrete easily crumbles with finger pressure (author's hand), note fissures at the concrete

2.2 Alternative Wall Systems

While most of the projects use the standard design of reinforcing cage and tremied concrete, other wall designs have also been used:

2.2.1 Post-Tensioned Diaphragm Walls

The principles of prestressing have been applied to diaphragm walls in order to extend their effective unbraced structural depth. Prestressing introduces internal compressive stresses on the concrete and thus the whole concrete section is more effectively utilized (since concrete can not resist tensile stresses). Individual companies have developed their own proprietary systems (ICOS-Flex, B8). Figure 2.17 shows general schematic of a post-tensioned diaphragm wall (Fuchsberger, 1980).

Post-tensioning is a very common concept in the construction industry. A common procedure is to post-tension high-strength steel wire strands (similar to those used in tieback anchors), properly located in the panel, after the concrete has cured (Xanthakos, 1994). Because post-tensioning increases the wall stiffness larger unbraced lengths are allowed, and the expected wall deflections should be smaller.

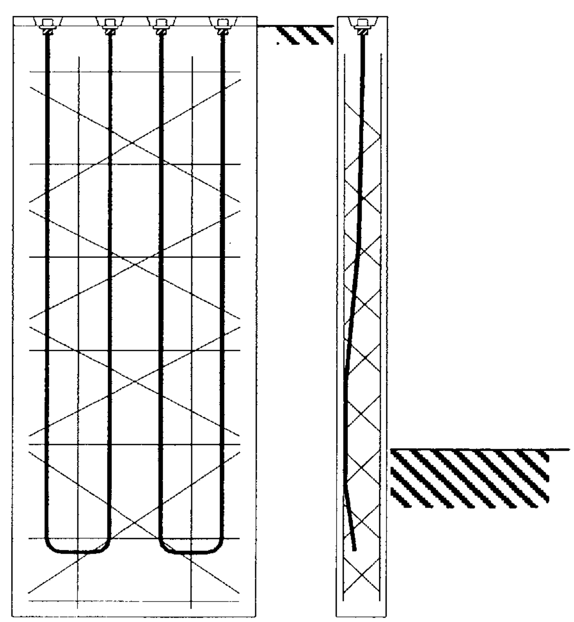


Figure 2.17: Post-tensioned diaphragm wall (Based on Fuchsberger, 1980)

2.2.2 Prefabricated Diaphragm Walls

Prefabricated slurry walls are constructed by inserting precast concrete panels in slurry trenches in place of the concrete tremie. The prefabricated panels are detailed so that they allow adjacent panels to interlock and form watertight joints. However, this might prove to be a difficult task. On several occasions in the Seaborn Hotel in Boston (Fig. 2.18), the tongues and grooves of the panels did not align properly, and the ill-fitting panels had to be removed, re-cut and reinstalled (BDC, 2000). Prestressing can easily be used in precast panels as was the case for the Pilot House Project in Boston (Kirmani et al., 1998).

The process requires more interaction between the slurry and the wall at the final configuration. In the single grout method the initial slurry that is used to support the trench is also used to form the final grout that seals panels and forms the interface between the precast panel and the retained soil. In contrast, in the displacement grout method the initial slurry is replaced by suitable bonding grout just before the precast sections are placed. In both methods the grout that remains in the excavation face of the panel is removed.

The big advantage of prefabricated walls is that they allow for better quality control through manufacturing. Also if constructed carefully the final wall finish of prefabricated walls is better than that of conventional diaphragm walls. Their major disadvantage is that the panel size is limited by the capacity of the lifting equipment and by handling limitations. It is also difficult to guarantee water-tightness between adjacent panels, especially when differential movements between panels occur.

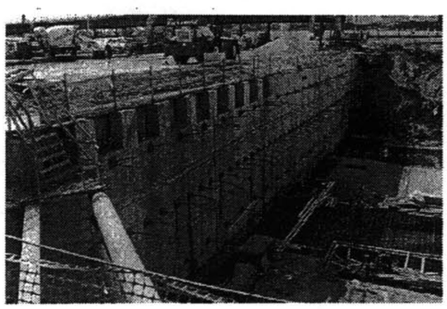


Figure 2.18: Precast slurry wall project in Boston (Seaborn Hotel, Boston), courtesy of TreviIcos Boston.

2.2.3 Slurry Walls Reinforced with Soldier Piles & SPTC Wall

Steel piles are used to reinforce slurry walls or to act as end bearing elements resting on a stiffer stratum. SPTC walls are standard in most temporary construction for the CA/T project in downtown Boston. Most panels in these types of walls contain three steel beams. As Figure 2.19 shows, the two outer steel piles are inserted in preaugered holes and then the trench is excavated in between under slurry. The inner pile is inserted into the middle of the slurry filled trench once the panel has been trenched into the desired dimensions. Figure 2.20 shows a cross-lot braced SPTC wall excavation (Soldier Pile and Tremie Concrete) (TreviIcos, 1999).

Concrete is then tremied in the same manner as in standard slurry walls construction. Special provisions have to be made to hold the soldier piles in their vertical position as the unbalanced horizontal stress conditions during concreting can displace them. In practice this is often achieved by using reinforcement cages are in combination with soldier piles.

Soldier Pile and Tremied Concrete walls make up the overwhelming majority of slurry walls installed in the West Coast cities in the last 15 years, since it is a more economical construction. The soldier piles are designed to be able to fully resist the horizontal pressures exerted on the retaining wall. The conservative assumption is that the tremied concrete acts only as a lagging system and that it doesn't contribute to the bending stiffness of the wall.

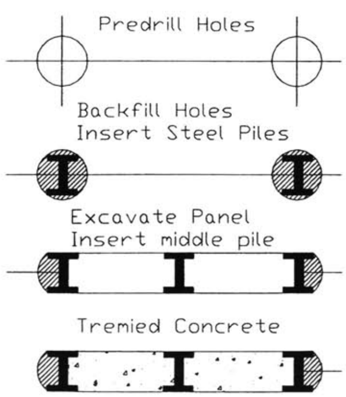


Figure 2.19: Construction Sequence

Figure 2.20: SPTC Wall Project (TreviIcos Boston)

2.2.4 Bored Piles or Caissons and Slurry Wall Panels

In some projects bored piles or caissons have been used in combination with slurry wall panels (Water Tower: C4, NU Parking Garage: C9, Chicago). The usual purpose of the caissons or the bored piles is to minimize settlement of the slurry wall when the panels do not bear on a stiff stratum. This can be a serious consideration when the slurry wall is to carry part of the vertical superstructure loads. Figure 2.21 shows a slurry wall and caisson combination.

Excavation of the bored piles or the caissons is carried under slurry in order to avoid soil movements. Care should be taken when excavating caissons since if oversized holes are drilled then soil movements can be very large despite the stiff slurry wall system. Caissons (or piles) may be constructed first and then the slurry wall panels may be trenched and concreted in between. The opposite construction sequence is also possible, but in both cases special attention should be given to the waterstopping details in the joints between caissons and slurry wall panels.

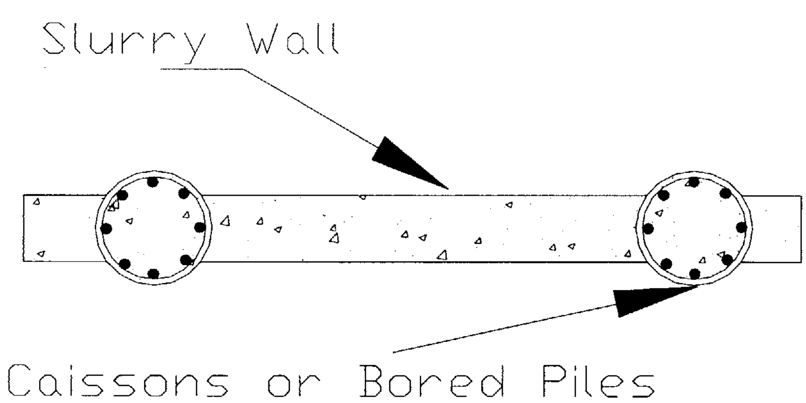


Figure 2.21: Slurry wall panel and caisson combination (or bored piles)

2.2.5 T-Walls

Buttressed or T-walls are typically analyzed as stiffened cantilevers and not as gravity retaining structures (Xanthakos, 1994). Some of the deepest T-walls are been used in the CA/T project in Boston (Lambrechts et al., 1998), and in the Dam No. 2 Hydropower Project on the Arkansas River (Berger & Tryon, 1999). Figure 2.22 illustrates the basic arrangement of a cantilever T-panel, where the flange acts in compression, while part of the stem is in tension. Stability analyses may include the skin friction at the stem yielding a more economical design. The extra rigidity allows for greater unbraced lengths but the wall stability is eventually controlled by its embedment. Figure 2.23 shows a reinforcement cage for a T-panel for the MBTA Courthouse Subways Station in Boston.

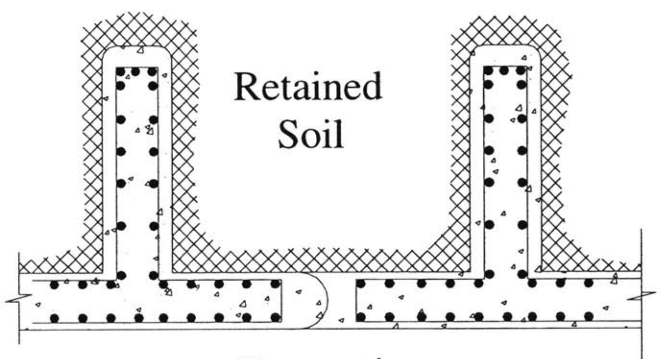


Figure 2.22: T-Slurry wall

Figure 2.23: Reinforcement cage for a T-Slurry wall, (MBTA Courthouse Subway Station, South Boston)

2.3 Bracing Systems

2.3.1 Tiedback Walls

Anchored walls have become popular in braced excavations because of a) the substantial progress in the technology and availability of high-capacity anchor systems, and b) the absence of interior obstructions that permit uninterrupted earth moving and thus improve the construction conditions of the underground portion of a building (Xanthakos, 1994). Figures 2.24 and 2.25 show a photo and a schematic of tied-back slurry wall excavations. In some projects tiebacks have been used in combination with rakers and soil berms and/or corner braces (Gnaedinger et al., 1975, B7). Tieback anchors comprise a barrel anchorage located either in a bearing layer (Fig. 2.25) which is tensioned at the front face of the wall. The part of the anchor that transfers the force to the surrounding soil is frequently called the "fixed length", while the "free length" transmits forces from the fixed length through the anchor head to the slurry wall (Fig. 2.26).

In order to minimize wall movement and ground settlement, tieback anchors are designed to achieve the highest stiffness possible within economical considerations. In urban cities like Boston, Chicago, New York, and Washington where land is precious such deep excavations are more common. Tieback capacity depends on the vertical and horizontal spacing of anchors and on surcharge conditions. Prestress levels typically range from 40 to 250 kips when the grouted portion of tiebacks is within soil, higher loads are used when the ties are located in bedrock. Typical tieback spacing ranges from 7ft to 13ft in the vertical, and from 5ft to 15ft in the horizontal direction (from the current database). Tieback capacity is reduced if the spacing is too close due to interference between adjacent grouted zones.

Often the tiebacks are used only for temporary excavation support, while the basement floors provide permanent lateral earth support. In such projects the tiebacks are detensioned when the basement floors have gained sufficient strength. The basement floors should be designed to resist permanent lateral earth pressures, since stress transfer from the tiebacks to the floor system will take place when the ties are detensioned. This stress transfer has reportedly caused long-term cracking of many the basement floors where slurry wall excavations were braced with temporary tiebacks (B3).

Tieback installation follows a predetermined sequence as to minimize soil movements and speed the excavation construction (Fig. 2.27). The excavation is carried a couple of feet below the tieback to enable access for the drill rig. Further excavation occurs only after prestressing and proof-testing of the anchors. As Figure 2.28 illustrates, the process can be repeated for additional levels of tiebacks. Building codes require that all tiebacks are proof-tested to an excess percentage of their final lock-off load, which usually ranges from 120 to 150% of the final lock-off load. Regroutable tiebacks are most commonly used because their capacity can be increased by regrouting (to meet test requirements) without having to drill a new anchor hole.

A tieback is made by first drilling a hole with an auger and then placing a bar (tendon) in the hole, concrete is then poured in the hole and the connection with wall is made (Figure 2.26). Different types of augers are used to drill the tieback holes. The choice of the drilling method depends on the soil/rock conditions on the site. The main types of drilling equipment fall under the following categories:

I. Percussive Drills: These accomplish penetration by the action of an impulsive blow, usually exerted from a chisel or wedge-shaped bit (Xantakos, 1991). Mawdsley [1970], reports three main types:

   a) Type A, with a compressed air-powered drifter driving standard coupled rods.
   b) Type B, with a compressed air-powered drifter driving special coupled drill rods which also act as the anchor.
   c) Type C, with an independent rotation compressed air-powered drifter simultaneously driving coupled drill tubes and drill rods, also known as "Atlas Copco overburden method" .

II. Rotary Drills: They work by combining axial thrust and rotational torque. The main types are:

a) Auger driving with coupled flight augers. Augers are most commonly used in self-supporting materials like stiff to hard clays, marls and soft rocks. They can be further subdivided to 1) continuous flight augers with hollow couplings, that allow water, bentonite, or grout to be pumped into the hole, 2) standard continuous flight augers for open hole drilling, most commonly used in cohesive soils, and 3) hollow-stem augers with a removable center bit allowing sampling during drilling.

b) Normal rotary drills with flush coupled drill rods and usually a drag bit as the cutting component.

III. Rotary-Percussive Drills: These combine the characteristics of the two previous types, with: a) axial thrust and torque but lower than rotary drills, b) impact like the percussive type but smaller in magnitude.

Drilling should be done carefully since inadequate procedures can cause significant soil losses. The biproduct of drilling is removed by flushing the hole with either air, water, or slurry. Air is most efficient in dry ground, but it requires special attention because it can become entrapped during drilling, building up zones of high pressure in the soil that can eject material for several feet and at high speeds (potentially injuring workers). Water flushing is best used in sticky clayey soil, and it also cleans the sides of the hole by its sweeping action, providing a stronger bond at the grout-anchor interface. Bentonite slurry flushing works the best since it keeps particles in suspension, while the sealing action keeps the hole from collapsing.

Significant soil losses through the tiebacks cause significant settlements even if the retaining walls do not move towards the excavation. In granular soils the drilled hole must be cased to avoid collapse.

Some tieback creep can be expected especially if the ties are very short and the fixed length of the tie is within soft ground. For stability reasons, the fiexed anchor should be located beyond the active zone of movements. As a result, tieback anchors may not be an option at sites congested where there are adjecent underground utilities or when adjacent owners do not grant permission to drill them under their properties.

Special attention should be given to the waterproofing details at the anchor heads and at the tieback holes. Significant leakage can be caused by inadequate waterstopping details at these locations (see section 2.3).

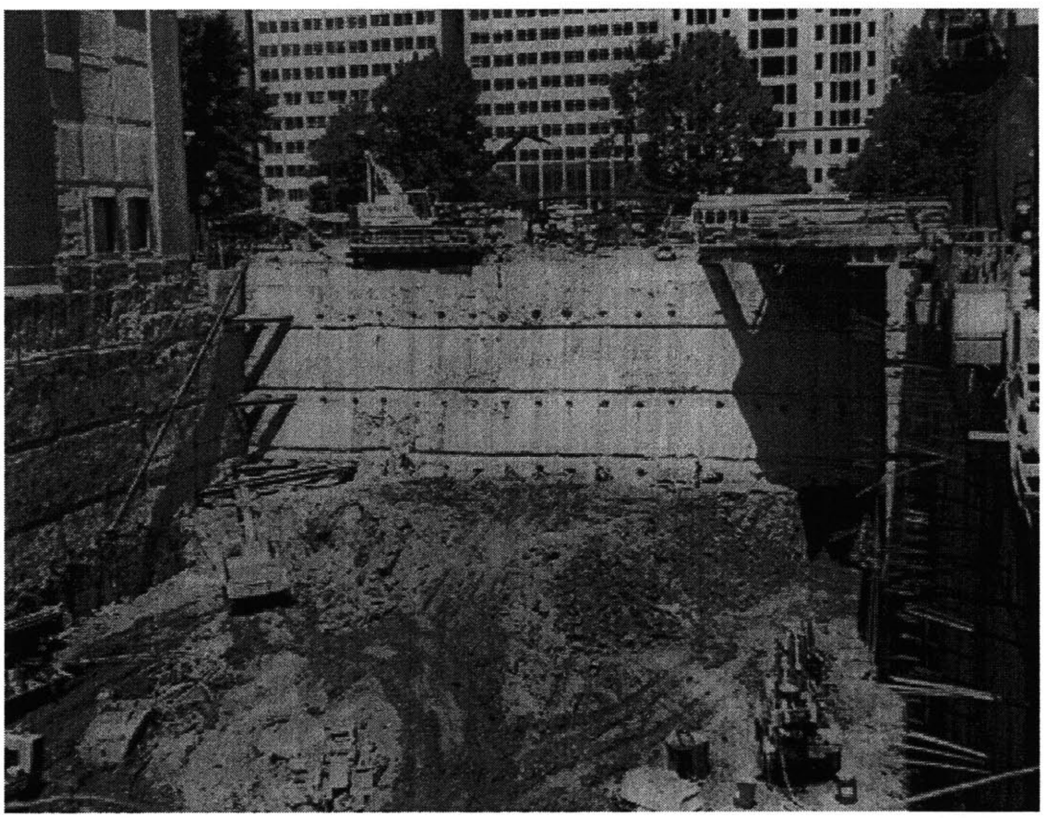


Figure 2.24: Picture from a tieback slurry wall excavation (World Bank Project Washington, Case Study W-1)

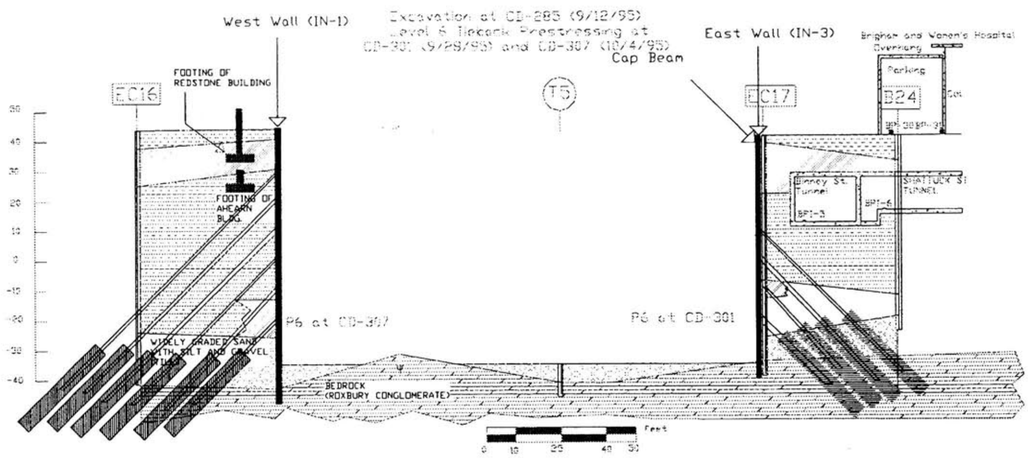


Figure 2.25: Tieback slurry wall excavation (Dana Farber Tower, Boston: Case Study B-12).

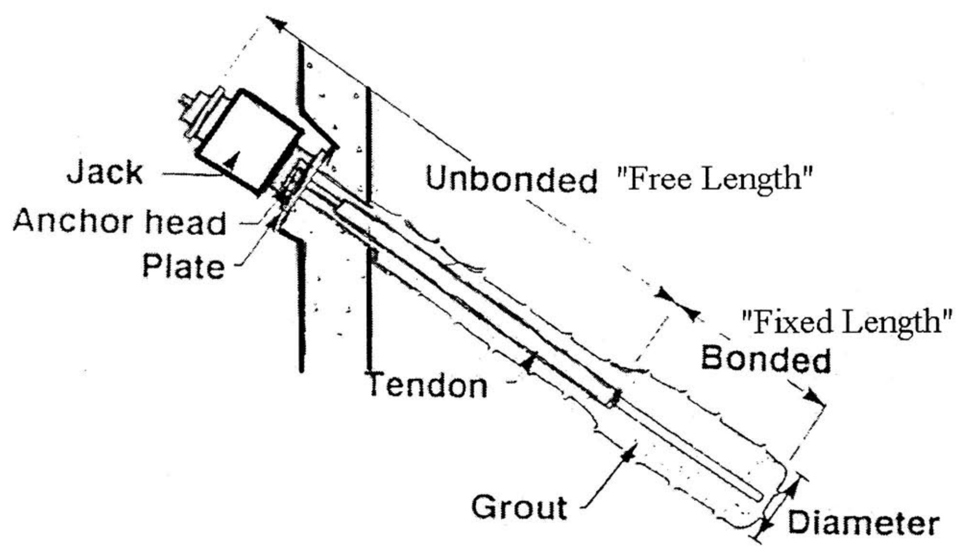


Figure 2.26: Tieback configuration, free and fixed lengths (Schnabel, 1982)

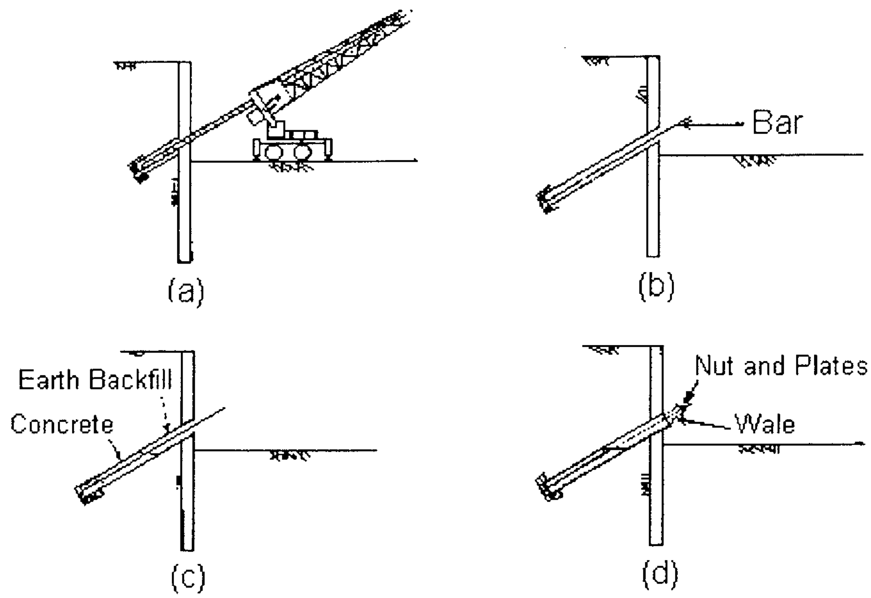


Figure 2.27: Steps in making a tieback: (a) hole drilled; (b) bar placed in hole; (c) concrete poured for anchor; (d) wall connection made (Schnabel, 1982).

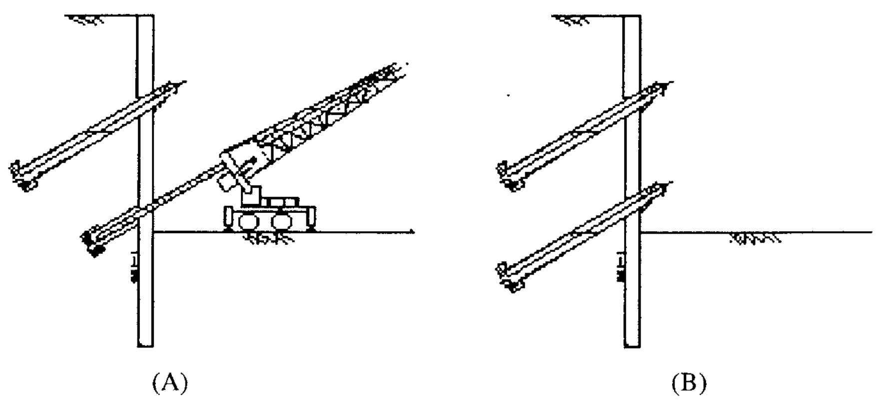


Figure 2.28: Steps in making a multilevel tieback excavation, (A) first level of tiebacks installed and second level of tiebacks drilled, (B) second level of tiebacks installed.

2.3.2 Cross-Lot Braced Slurry Wall Excavations

Cross-lot bracing transfers the lateral earth (and water pressures) between opposing walls through compressive struts. Typically the struts are either pipe or I- beam sections and are usually preloaded to provide a very stiff system. Installation of the cross-lot struts is done by excavating soil locally around the strut and only continuing the excavation once preloading is complete. A typical sequence of excavation in cross-lot braced excavations is shown in Figure 2.29. The struts rest on a series of wale beams that distribute the strut load to the diaphragm wall.

Pre-loading ensures a rigid contact between interacting members and is accomplished by inserting a hydraulic jack as each side of an individual pipe strut between the wale beam and a special jacking plate welded to the strut (Fig. 2.30, Xanthakos, 1994). The strut load can either be measured with strain gages or can be estimated using equations of elasticity by measuring the increased separation between the wale and the strut. Figure 2.31 shows the basic arrangement for the wedging, and the telescoping preloading methods.

In some earlier projects the struts were not preloaded, and as a result when the excavation progressed deeper the soil and the wall movements were large (C1). Thus it has become standard practice to preload struts in order to minimize wall movements.

Cross-lot bracing makes sense in narrow excavations (60ft to 120ft) when tieback installation is not feasible. The struts can bend excessively under their own weight if the excavation spacing is too large. In addition, special provisions have to taken to account for thermal expansion and contraction of the struts.

The typical strut spacing is in the range of 15ft, both in the vertical and the horizontal direction. This is larger than the typical spacing when tiebacks are used, because the pre-loading levels are much higher. A clear benefit of using struts is that there are no tieback openings in the slurry wall, thus eliminating one source of leakage.

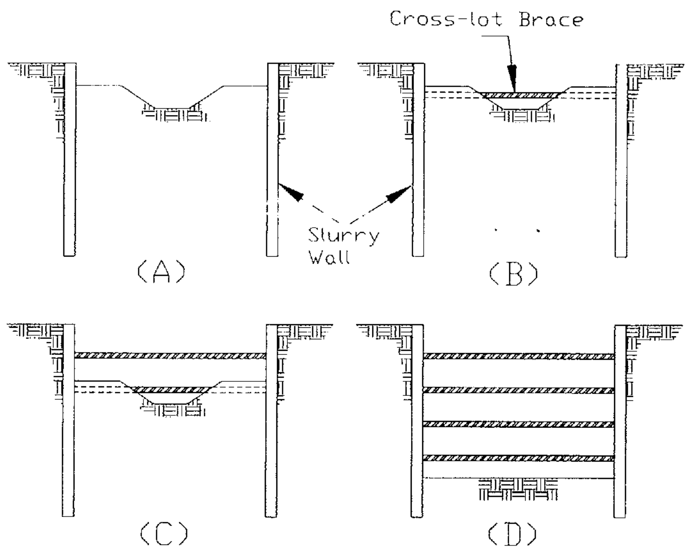


Figure 2.29: Typical excavation sequence in cross-lot excavations: (A) V-cut initial cantilever excavation, (B) Strut installation and pre-loading in small trenches in soil berms, (C) V-cut excavation to next level and strut installation, (B) Final grade.

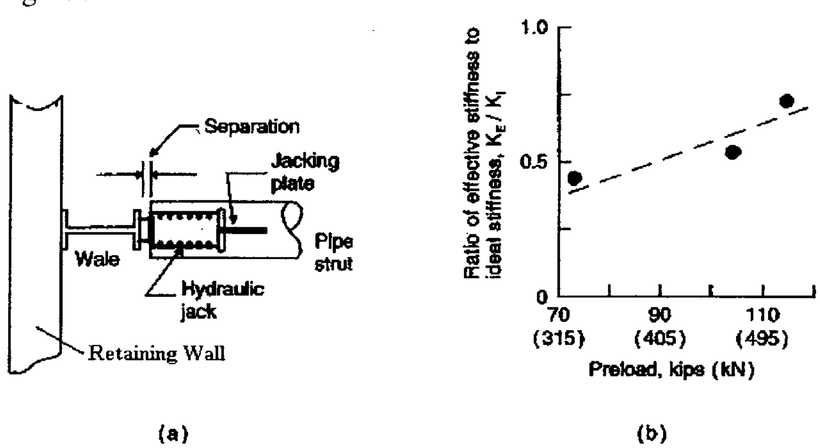


Figure 2.30: (a) preloading arrangement, and (b) measured brace stiffness (Xanthakos, 1994)

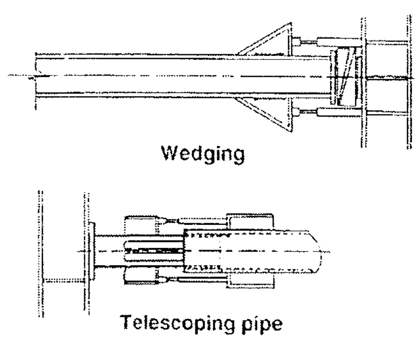


Figure 2.31: Methods of preloading struts; Wedging (top), Telescoping pipe (bottom)

2.3.3 Top/Down Slurry Wall Excavations

Top/down or up/down construction methods are another method for constructing deep excavations. In this case the basement floors are constructed as the excavation progresses. The top/down method has been used for deep excavation projects where tieback installation was not feasible and soil movements had to be minimized. Figures 2.32 through 2.33 show construction photographs from two top/down excavations in Boston (B11, B13). The general top/down construction sequence is shown in Figure 2.34 (B11). The Post Office Square Garage in Boston (B10) (7-levels deep) is one of the best-instrumented and documented top/down projects in the US (Whittle, et al., Whitman et al., 1991).

The sequence construction begins with slurry wall installation and then load-bearing elements that will carry the future super-structure. The basement columns (typically steel beams) are constructed before any excavation takes place and rest on the load bearing elements. These load bearing elements are typically concrete barrettes constructed under slurry (or caissons). The top few feet of a barrette with a steel beam can be seen in Figure 2.33. Then the top floor slab is constructed with at least on construction (glory) hole left open to allow removal of spoil material (Figs. 2.33, 2.34).

The excavation starting at the glory hole begins once the top floor has gained sufficient strength. Soil under the top basement floor is excavated around the basement columns to slightly lower than the first basement floor elevation in order to allow for the installation of the forms for the first level basement slab. Glory holes are left open within each newly formed basement floor slab and the procedure is repeated. Each floor rests on the basement columns that were constructed earlier (Fig. 2.33).

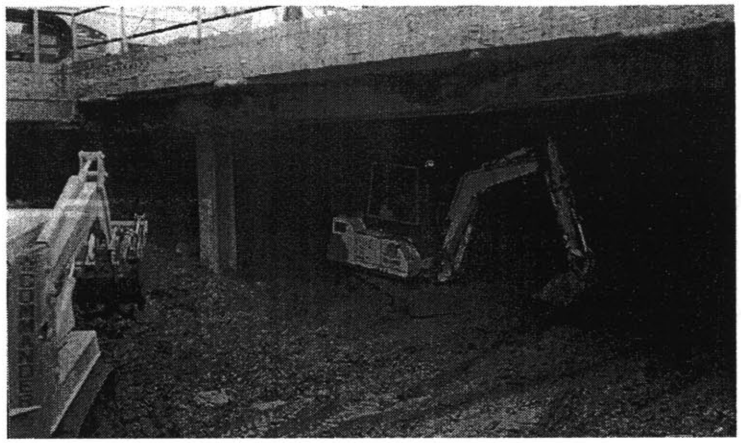
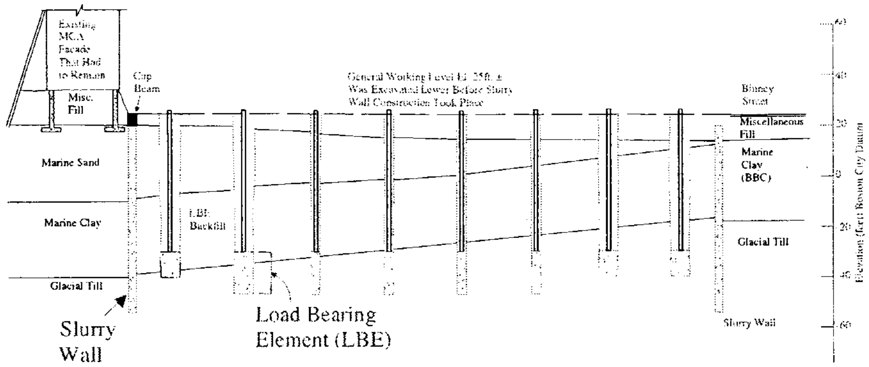
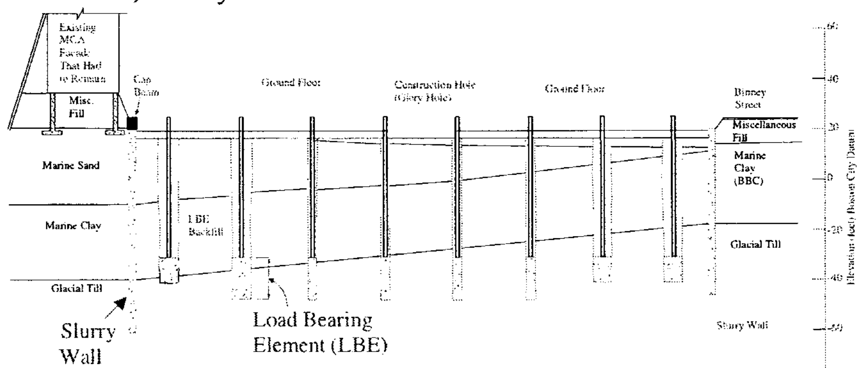


Figure 2.32: Top/Down Excavation (Beth Israel Deaconess Hospital, Boston, Case study B-11)

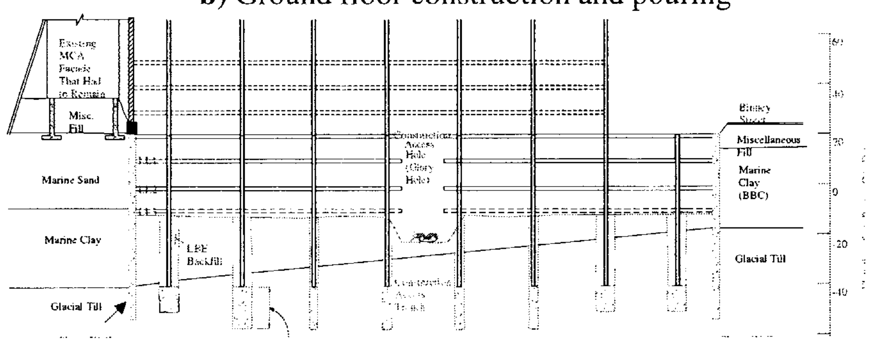
Figure 2.33: Millenium Place excavation. Left: Looking up at a glory hole, Top right: author in the lowest most level note LBE on the left and the barrette, Bottom right: close up view the same barrette (LBE) and steel beam (B13).



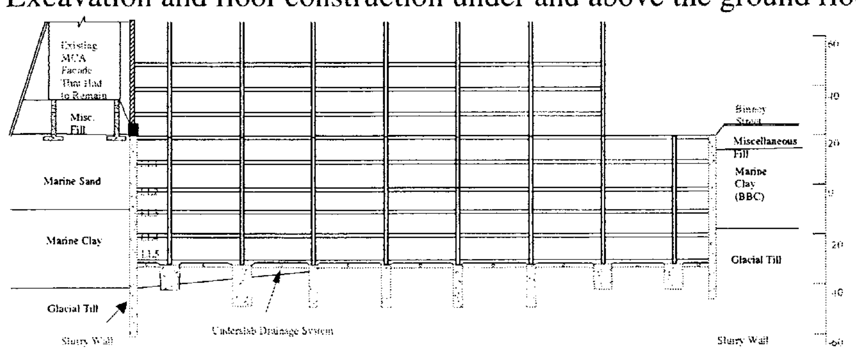
a) Slurry wall and basement column construction



b) Ground floor construction and pouring



c) Excavation and floor construction under and above the ground floor



d) Excavation & lowest basement floor completed

Figure 2.34: Top/down basic construction (B11)

2.4 Watertightness

Watertightness is very important especially since slurry walls are either used as part of the permanent underground basement structures or are selected in part to control groundwater flow during excavation. Although minimal water can flow through the mass concrete, significant leakage can occur at joints, in openings for tiebacks or when there is a breach in the wall integrity. The risk of leakage is increased when the number of construction joints is increased. Therefore, water leakage is usually controlled by minor defects on the wall. Filling with hydraulic cement and grouting are the most common methods of repairing leakage. Typical grouts used are either cement grouts, chemical, or a combination of both.

Special attention should be given to the waterstopping details at the interface of tiebacks with slurry walls (Fig. 2.35) (B12). When tiebacks are permanent waterstops at the concrete and at the anchor head should be designed for multiple safety. Figure 2.36 (B12) shows the water-stopping details for a permanent tieback.

Differential movement at panel joints usually results in dampness around joints. In order to avoid such leakage contractors have developed waterstops that are inserted with the help of the end stops (Fig. 2.12) (Xanthakos, 1994, Parkison & Gilbert, 1991, Ressi, 1999). These waterstops maybe chemical or plastic but in both cases care should be given during construction. The usual remedy for excessively leaking joints is to grout behind the joint once the movements of the wall have stabilized stopped. However, additional differential deflections can occur between adjacent panels if too much grout is inserted behind the wall, and thus sealing will not be effective.

Improper cleaning of the panel bottom after trenching can result in leaking construction joints. After trenching has finished, soil cutting material can accumulate at the bottom of the panel. If this material is not cleaned then it is pushed up during concreting. Part of the material reaches the top of the panel where it is later chipped and removed, but some can get entrapped between the panel joints. When exposed, water can easily leak through these contaminated joint sections.

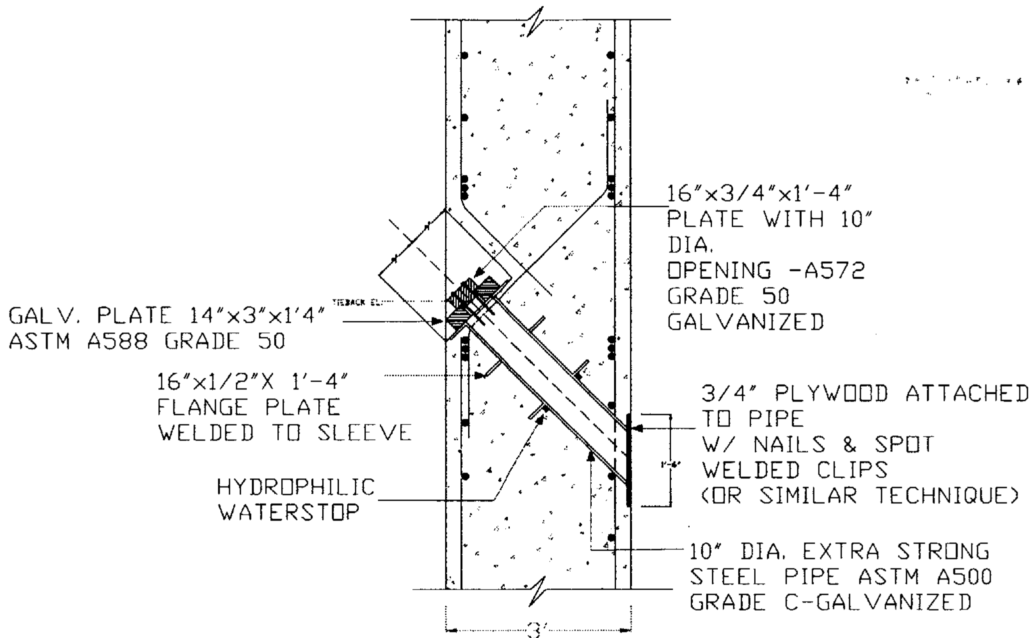


Figure 2.35: Typical waterstopping details at the tieback slurry wall interface , Dana Farber Tower (B12)

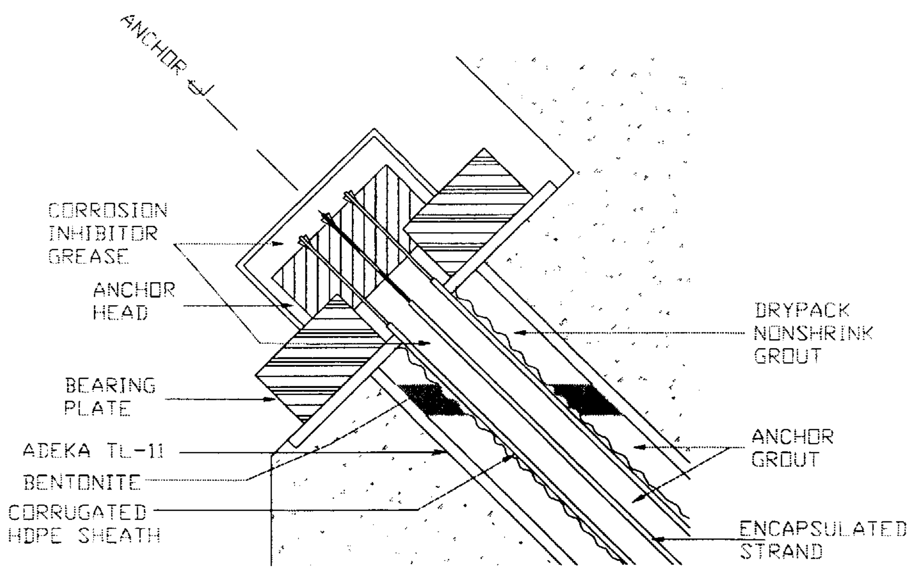


Figure 2.36: Permanent tieback and sealing details, ADEKA is a hydrophilic material that expands when exposed to water forming a seal (B12)

2.5 Wall Finish

With the exception of precast panels, slurry walls never have a perfect wall finish. The final wall finish contains irregularities and protrusions from the average wall face. The usual overbreak tolerance is 3" to 4" from the specified wall thickness but this depends on the soil conditions in the site, excess concrete beyond the specified tolerance is chipped away. The tendency for overbreaks increases in loose sands and heavily fissured clays. Large excess concrete volumes are sometimes observed at locations where strata change like from soil to bedrock (Figure 2.37). Excess concrete is occasionally observed in corner panels (Fig. 2.38).

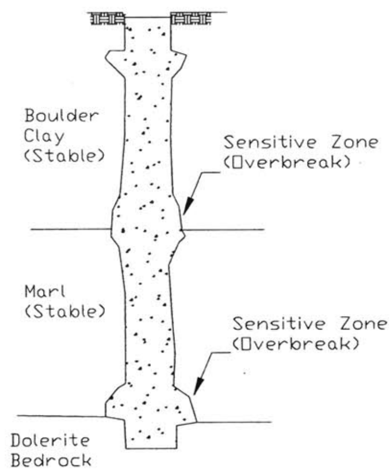


Figure 2.37: Concrete overbreaks due to strata change (after ICE, 1976)

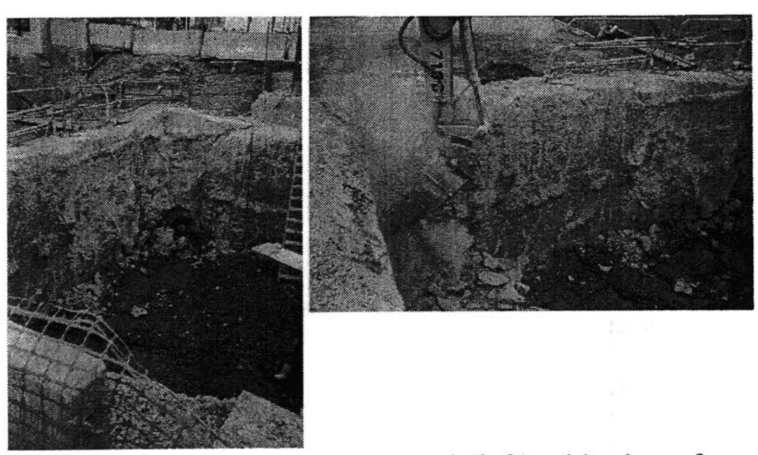


Figure 2.38: Excess concrete at a corner panel (left), chipping of excess concrete at a corner panel (right)

Chapter 3

Instrumentation for Performance Monitoring

Chapter 3

Instrumentation for Performance Monitoring

3.1 Instrumentation in Deep Excavations

The principal parameters of interest in monitoring are: (1) wall and soil deformations, (2) stresses acting on structural elements (wall and bracing), and 3) ground water pressures and inflows. Table 3.1. summarizes the parameters and methods of measurements used for slurry wall excavations.

Inclinometers, piezometers and optical surveys have been used to monitor the majority of the projects compiled in this database. Table 3.2 lists the performance monitoring measured and what archived data could be retrieved for each case study. Generally, the quality of the available instrumentation was dictated by the project requirements. The quality of data in more recent projects tends to be better since computers have facilitated data acquisition and archiving.

Clearly, inclinometers and surface settlement surveys have been used in almost all the projects but the latter were not archived in many projects. Thus, inclinometer deflections have been more widely used for comparisons between cases due to their availability in almost all the archives. Water levels and piezometric levels were measured in most projects but were not always available in the archives.

In a few cases, strain gages or load cells were used to monitor bracing loads but these cases tend to be the exception rather than the rule. A truly rare case is the Dana Farber Tower (B12) project in Boston, where strain gages and embedment gages where used to deduce moments and axial forces in the slurry wall.

Sometimes, single point extensometers have been used to measure subsurface settlements, but multipoint borehole extensometers have only rarely been used. Earth pressure cells have been used only in test programs.

Monitoring strategies are not significantly different from city to city but the extent of instrumentation does depend on local practices. For example, in Boston, the majority of the inclinometers were installed within the diaphragm walls while in Chicago they have been installed within the retained soil. In Washington, inclinometers have been installed both within the walls and within the surrounding soil.

Table 3.1

Instrumentation used in slurry wall construction (after Xanthakos 1991)

Geotechnical Problem	Parameter of Interest	Possible measurement methods
Trench stability	Width of trench	• Trench width gage • Inclinometer
Trench verticality	Alignment of sides of trench	• Inclinometer in temporary casing
Guide wall stability	Horizontal and vertical movement of guide wall	• Optical survey
Ground and wall movement	Horizontal movement of ground or wall	• Optical survey • Inclinometer in wall or in soil • Horizontal multipoint extensometer (HMPBX) • Tape extensometer across the excavation
	Vertical movement of ground or wall	• Optical survey • Subsurface settlement gage ie. Settlement rods or Multiple Point Borehole Extensometers (MPBX)
	Movement of adjacent structures	• Photography • Optical survey • Tiltmeter • Crack gage
Cross-lot bracing	Loads in struts or braces	• Strain gage
Tieback bracing	Tieback load	• Strain gage • Load gage
	Anchor movement	• Telltale
Basal stability	Bottom heave or horizontal ground movement	• Heave gage • Inclinometer
Groundwater	Groundwater level	• Observation well
	Pore pressure	• Piezometer
Advanced information	Wall moments, axial forces, stresses in wall or reinforcement	• Strain gages
	Earth stresses	• Earth pressure cells

Table 3.2: Measured and archived performance monitoring instrumentation used in slurry wall excavations (B) Boston, (C) Chicago, (W) Washington DC, (S) San Francisco.

ID	Project Name	Year	IN		HoS		SS, VoS		SbS, MPBX, EX		PZ - OW		SG - LC		TM		RsG - EG		Quality
	Boston		M	A	M	A	M	A	M	A	M	A	M	A	M	A	M	A	
B1 *	MBTA South Cove	1973	✓	✓			✓	✓			✓	✓	✓	✓					Good
B2 **	60-State Street	1975	✓	✓			✓	✓			✓	✓	✓	✓					Very Good
B3	State Transportation Building	1982	✓	✓			✓	✓			✓	✓	✓	✓	✓	✓			Good
B4 *	75 State Street	1983	✓	✓			✓	✓											No data
B5	Rowes Wharf	1984	✓								✓								poor
B6	One Memorial Drive	1985	✓	✓	✓	✓	✓	✓			✓	✓	✓	✓					Good
B7	500 Boylston	1987	✓	✓			✓	✓			✓	✓							Very good
B8	Flagship Wharf	1989	✓	✓			✓	✓			✓	✓	✓	✓					Very good
B9 *	125 Summer Street	1990	✓				✓				✓								No data
B10	Post Office Square Garage	1991	✓	✓			✓	✓	✓	✓	✓	✓							Excellent
B11	Beth Israel Deaconess	1994	✓	✓			✓	✓			✓	✓							Very good
B12	Dana Farber Tower	1995	✓	✓			✓	✓			✓	✓	✓	✓			✓	✓	Excellent
B13	Millenium Place	2000	✓	✓			✓	✓			✓	✓							Excellent

ID	Project Name	Year	IN		HoS		SS, VoS		SbS, MPBX, EX		PZ - OW		SG - LC		TM		RsG - EG		Quality
	Chicago		M	A	M	A	M	A	M	A	M	A	M	A	M	A	M	A	
C1**	CNA Building	1971			✓	✓	✓	✓			✓								Fair
C2 **	Sears Tower	1971	✓		✓		✓				✓		✓	✓					Poor
C3 **	Amoco Standard Oil	1973	✓	✓			✓	✓			✓								Good
C4 **	Water Tower	1974	✓	✓	✓	✓	✓				✓		✓	✓					Very Good
C5	Loyola University Business Scho	1993	✓	✓			✓				✓								Very Poor
C6	Prudential Two	1986	✓	✓			✓				✓								Good
C7 **	AT&T Corporate Center	1987	✓	✓			✓				✓								Good
C8	Guest Quarters Hotel	1989	✓	✓			✓				✓								Good
C9	Northwestern University Memori	1990	✓	✓			✓				✓								Good
C10	Museum of Science & Industry	1997	✓	✓			✓				✓								Good
C11 *	311 South Wacker Drive	1997	✓	✓			✓				✓								Good

ID	Project Name	Year	IN		HoS		SS, VoS		SbS, MPBX, EX		PZ - OW		SG - LC		TM		RsG - EG		Quality
			M	A	M	A	M	A	M	A	M	A	M	A	M	A	M	A	
	Washington																		
W1	World Bank	1991	✓	✓	✓	✓	✓	✓			✓	✓			✓	✓			Excellent
W2	Petworth Subway Station	1995	✓	✓			✓				✓								Poor
W3	Washington Convention Center	2000	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓	✓					Excellent
W4 *	Metro Center II	1991	✓	✓			✓				✓								Not avail.
	San Francisco																		
S9	Yerba Buena Tower	1999	✓	✓	✓	✓	✓	✓			✓	✓							Excellent

Note: **M – Measured, A – Archived**, * Only Referenced, ** Referenced and revisited archives, **IN**-Inclinometers, **HoS** – Horizontal Offset Surveys (Optical), **SS - VoS**– Surface Settlements & Vertical offset Surveys (Optical), **PZ OW**- Piezometers and Observation Wells, **SG** – Strain Gages, **LC** - Load Cells, **EG** – Embedement Strain Gages (Concrete), **RsG** – Reinforcement strain Gages, **SbS** – Subsurface Settlements, **EX** – Extensometers, **MPBX** – Multiple Point Borehole Extensometer

3.1.1 Optical surveys

Optical surveys are always used to monitor ground, guide wall, and slurry wall movements. Surface and building settlements are most commonly monitored by optical surveys, while wall deflections calculated from surveys are generally used as a back-up reference. The measured movements are, in most cases, referenced to preconstruction surveys that determine the initial conditions of adjacent buildings. Unfortunately, preconstruction survey data was not archived for most projects in this study.

Building reference points typically comprise steel bolts socketed into an existing building or surface (Fig. 3.1). Surface reference points are steel nails or markers inserted into the desired surface. Slurry wall offset surveys can be made using an arrangement like the one shown in Figure 3.2. Occasionally, wall movements are measured at the top of wall reinforcement (Case study B-6: One Memorial Drive).

Measurements are made with the aid of an optical or laser gun and an optical reflector mounted on a steel rod (Fig. 3.3). Optical surveys can give very accurate results if they are performed carefully. However, the precision can be affected by inadequate setting up of the tripod construction details and by the care that is taken during measurements. Also, measurements should be taken against steady reference points that are not influenced by the excavation.

The problem with optical surveys is that reference points can easily be destroyed or influenced by construction equipment that passes nearby. For example, a heavy crane may pass on top of a surface settlement marker and thus the resulting settlement will not be associated with the excavation. In addition, street markers can show a lot of variations since traffic conditions vary. For these reasons, the scatter in the offset data can be larger than the actual settlement when the magnitude of the settlements is small. On the other hand, advances in electronics, computers, and software can make measurements, data reduction, and presentation more efficient and economical.

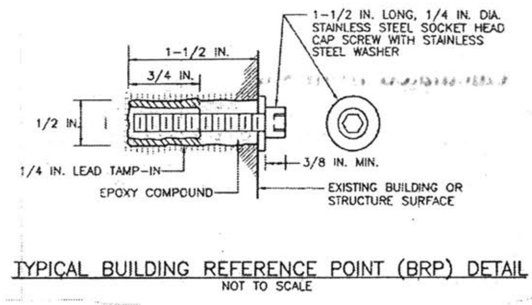


Figure 3.1:

Building reference point detail (Case Study B-13, Millenium Place)

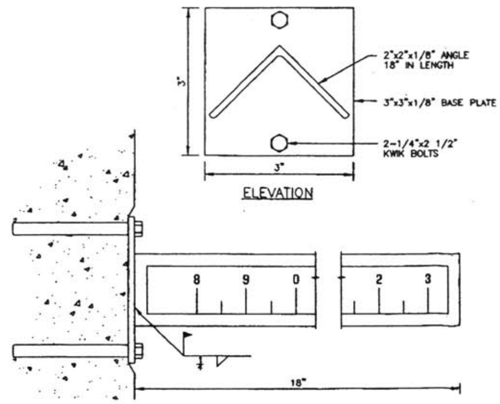
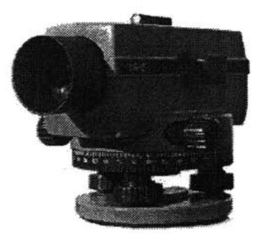


Figure 3.2:

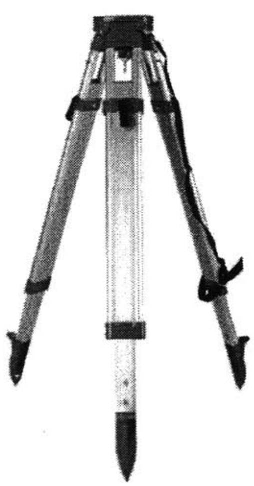
Horizontal offset monitoring survey detail (Case Study W3: Washington Convention Center).

(A)

(B)



(C)



(D)

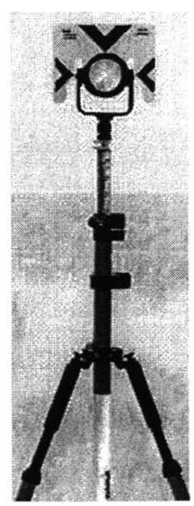


Figure 3.3: (A) Theodolite (TopCon), (B) Automated level, (C) Tripod, (D) Reflector.

3.1.2 Inclinometers

Inclinometers are the most important source of geotechnical data during construction (Erikson et. al., 1992). Inclinometers are widely used to measure lateral soil and wall movements in slurry wall and deep excavations (Fig. 3.4). They fall into two categories, 1) probe inclinometers and 2) fixed- in-place inclinometers (Laplante, 1998). Only probe inclinometers have been used in all the studied projects. In either type, a casing with four orthogonal grooves is installed in a borehole in the ground or within the slurry wall (Fig. 3.5). The grooves are designed to fit the wheels of the inclinometer probe (Fig. 3.6). The angle of the probe from the vertical axis is measured in both directions with the use of a sensitive gravity pendulum, tiltmeters, or a servo accelerometer. The deflections are calculated automatically from this angular measurement and from the distance between the wheels (which is known). Deflections are always compared to an initial reading since the casing may not be installed in the vertical position.

Typically measurements are reported at two foot intervals and a process requires up to 15 minutes (Fig. 3.7). The reduced data maybe erroneous and certainly will be less accurate if the measurements are taken at steps greater than the typical two-foot distance between the probe wheels (W3). For better results it is advisable that the inclinometer casing extends to a firm stratum (or to a great enough depth) so that the inclinometer base does not move. For example, walls in floating excavations tend to translate at their bases (B3, B7) and these movements will not be picked up if the inclinometers do not extend beneath the base of the wall (B6). Movements during slurry wall construction can not be measured when all the inclinometers are installed within the diaphragm wall. Thus in order to get a more realistic view of soil movements during slurry wall construction it is suggested that a few inclinometers be installed within soil. In ideal conditions, soil inclinometers should be installed before any construction takes place.

The limitation of inclinometers is that they measure only horizontal ground deformations. Deformation data is necessary to appease abutters worried about potential damage to their property (Laplante, 1998).

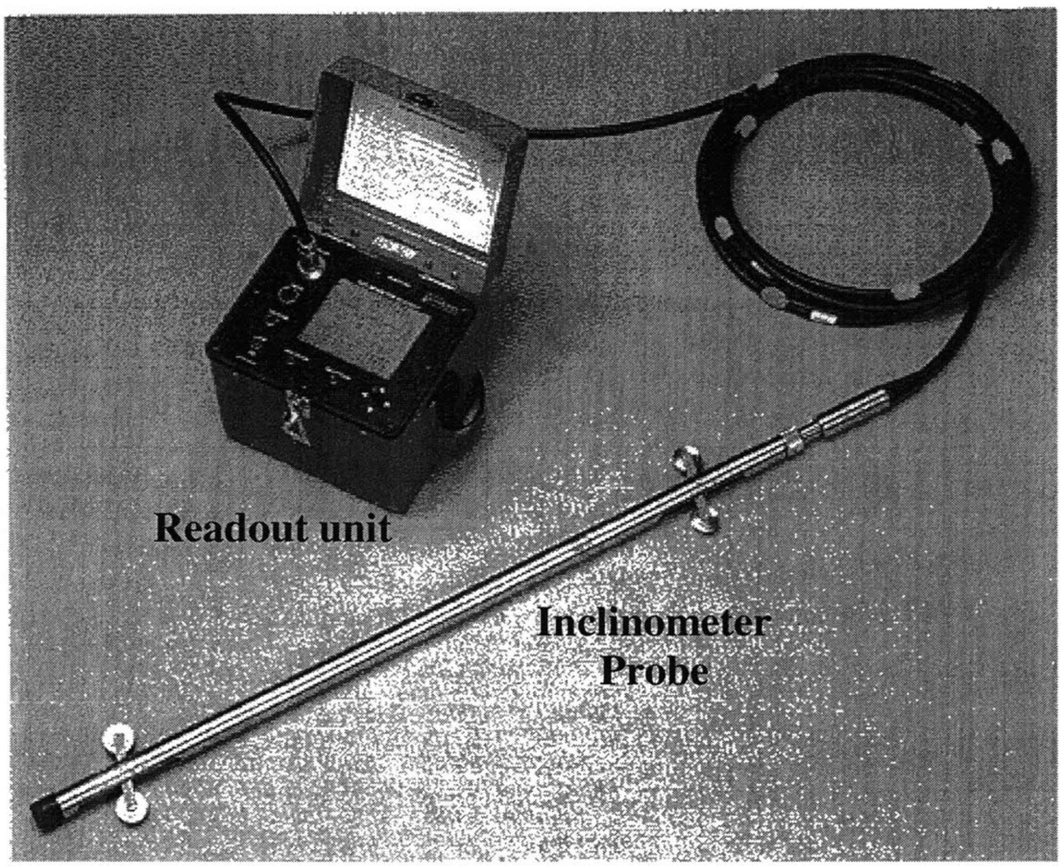


Figure 3.4: GeoKon Model 6000 Inclinometer probe and readout device.

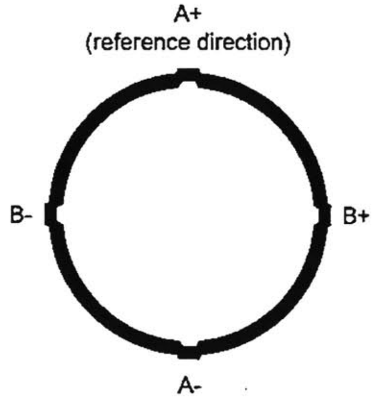


Figure 3.5: Inclinometer grooves, A+A- is the primary direction, B+B- is the secondary direction

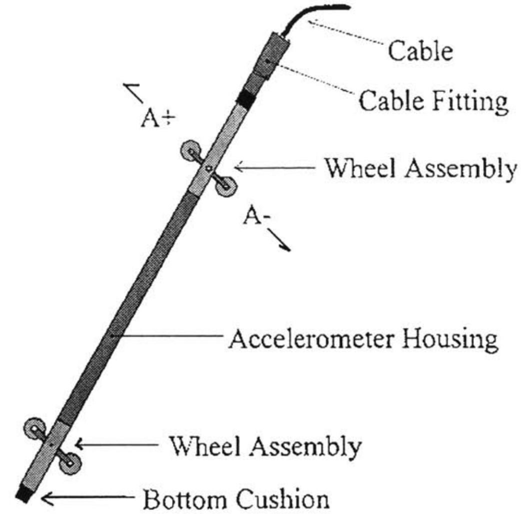


Figure 3.6: Inclinometer probe

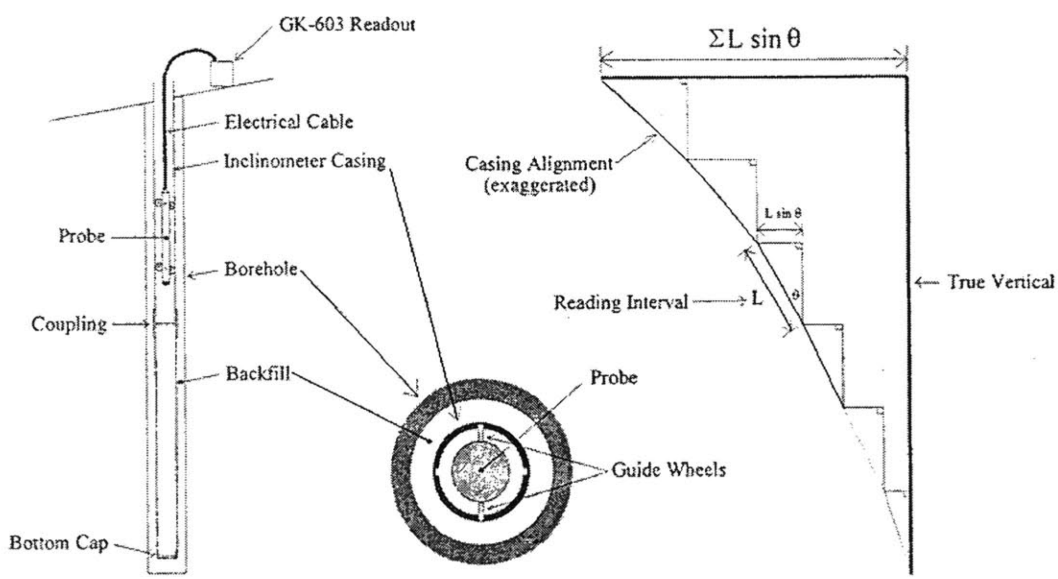
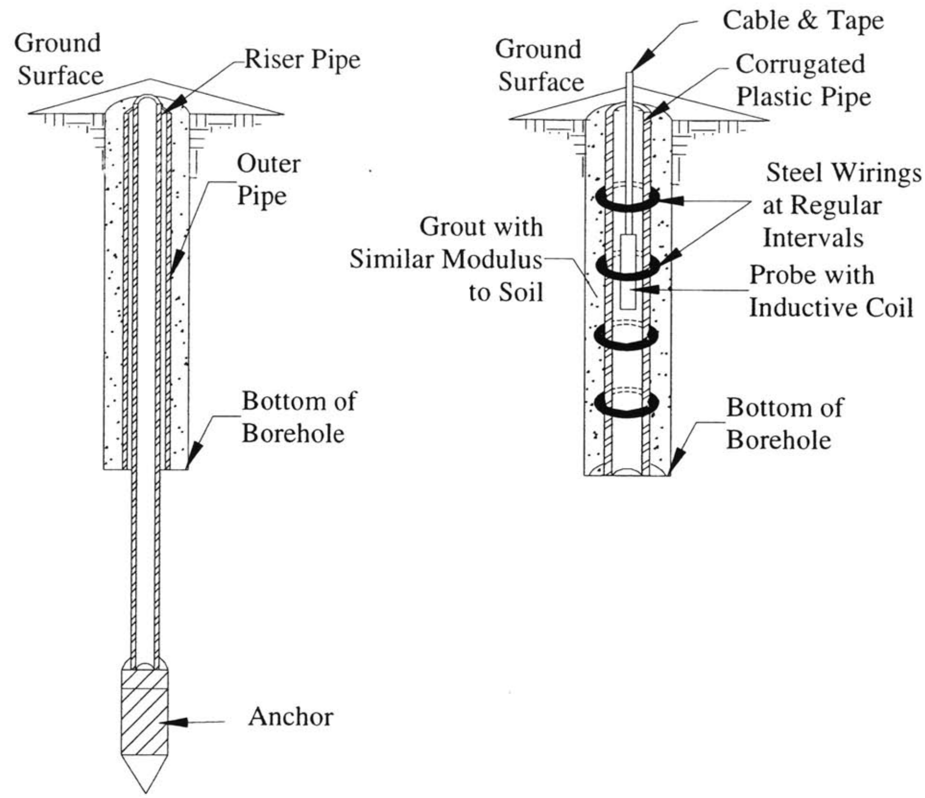


Figure 3.7: Inclinometer survey description, (Geokon manual)

3.1.3 Subsurface Settlement Gages

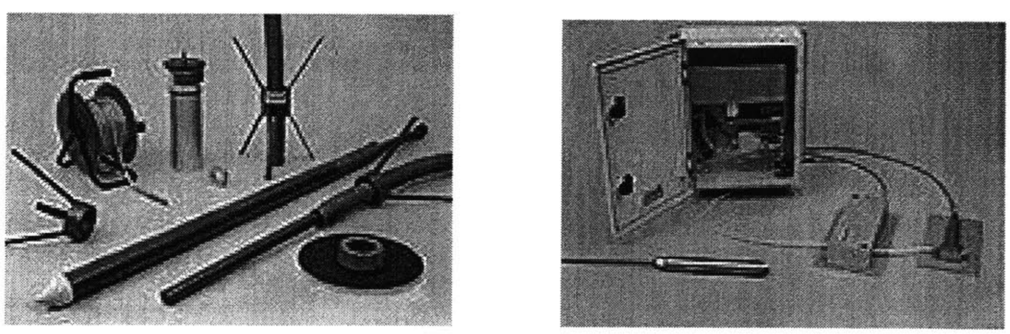
Subsurface settlement gages fall under two general categories, single-point and multipoint gages. As the name indicates they are used to measure subsurface settlements at one or more reference points. The "Borros" anchor point is the most commonly used single-point gage and it consisting of a steel anchor mechanically set at the bottom of a borehole (Fig. 3.8a). A heave/settlement point consists of a three-pronged anchor, a 1/4" inner pipe, and a 1" outer pipe, both steel. The inner pipe is attached to the anchor and is free to move inside the 1" pipe. An optical survey is used to determine the elevation of the top of the inner pipe. Changes in its elevation indicate an equivalent amount of settlement or heave at the anchor. Single-point settlement gages are sometimes used in deep excavations (Case Study B-3: State Transportation Building). Their accuracy is in the order of 0.1" (Xanthakos 1994).

Multipoint settlement gages are rarely used in common projects but they are widely used in test programs and for research purposes (Case Study B-10: Post Office Square Garage). A corrugated plastic pipe is installed in a nominally vertical borehole with stainless-steel wire rings around the pipe placed at intervals of 5' to 10' (Fig. 3.8b). A grout with a modulus similar to that of the surrounding soil is inserted in the annular space between the pipe and the borehole wall. Settlements are measured by lowering a probe containing an inductive coil within the pipe on the end of an electrical cable and a survey tape .The standard accuracy is in the order or 0.1" but it can be improved by modifying the basic arrangement. Another type uses magnetic rings instead of the inductive coils and a magnetic probe (Fig. 3.9a). With careful sizing and detailing it is possible to combine an inclinometer and a multipoint settlement gage in the same borehole (Xanthakos).



(A) (B)

Figure 3.8: (A) Single point "Borros" anchor, (B) Multi-point settlement gage.



(a) (b)

Figure 3.9: (a) Magnetic settlement anchors, (b) Settlement meter.

3.1.4 Multi Point Borehole Extensometers

Multi-Point Borehole Extensometers are installed vertically to monitor subsurface settlement, but they can also be installed horizontally to monitor horizontal soil or wall movement (Fig. 3.10). They consist of up to six separate anchors placed at various depths in a borehole (Pagani: http://www.pagani-geotechnical.com/english/extenso.htm), with a sleeve rod connected to each. The rods pass through a head set at the top of the borehole, and the relative displacement between each anchor rod and the head gives the movement (Xanthakos 1994). The quoted accuracy is in the order of ±0.06". In our database, they were used only in the Post Office Square Garage (Case Study B-10).

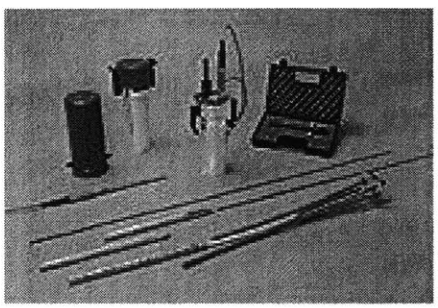


Figure 3.10: Borehole multipoint Extensometer (Pagani D220,).

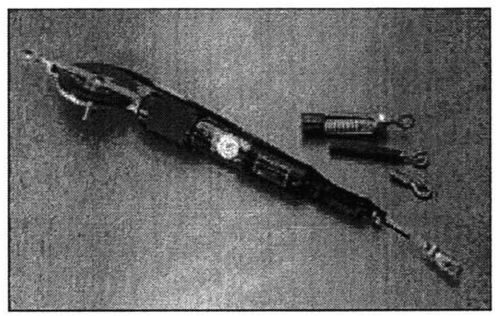


Figure 3.11: Tape extensometer (SlopeIndicator Company website)

3.1.5 Heave Gages

Heave gages are used to monitor the base heave in an excavation and thus give an indicator of basal instability problems. There are two basic types of heave gages: (1) gages that require the lowering of a probe in a hole to locate a buried component, (2) electrical gages that measure differences with respect to a deep anchor reference point. These arrangements are similar to those used for subsurface settlement measurements (Section 3.1.3). The accuracy of the first type is in the order of ±0.1" while the second type has a quoted accuracy of ±0.05". No data from heave gages were recovered in the archives of the studied projects.

3.1.6 Tape Extensometers

Tape extensometers are used to detect and monitor changes in the distance between two reference points (Fig. 3.11). The change in distance between two points can be obtained by comparing the current and initial readings. They are easy and cheap to get repeatable measurements up to spans of 66'. Reference points are typically stainless steel eyebolts that are economical and easy to keep clean. The eyebolts can be bolted or welded to the structure or threaded into groutable or expansion-type anchors (Slopeindincator: http://slopeindicator.com/). The resolution can be as high as 0.01 mm (Pagani: http://www.pagani-geotechnical.com/).

3.1.7 Crack Gages

Crack gages are commonly used to measure the behavior of existing or newly formed cracks in adjacent structures during the excavation (Fig. 3.12) (Case Study B-12: Dana Farber Tower). Sometimes they are even used to measure the trend of cracks in slurry wall joints (Case Study B-3: State Transportation Building). They typically consist of a mechanical or electrical gage that bridges the crack.

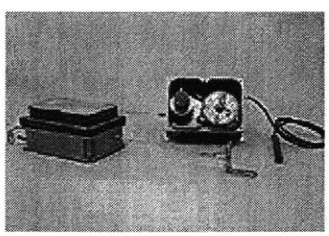


Figure 3.12: Pagani D240 Wire Crackmeter (Pagani website).

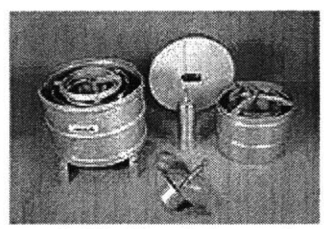


Figure 3.13: Pendulum tiltmeters. (Pagani website)

3.1.8 Tiltmeters

Tiltmeters are used to measure tilts of adjacent buildings or surface during the excavation (Fig. 3.13). They usually consist of a ceramic plate attached to the surface where the tilt is to be measured, and the readings are made with a portable tiltmeter. Tiltmeters are quite common in excavations but their accuracy is relatively limited. In the studied projects they have been used in the World Bank (W1), in the State Transportation Building (B3), and in the 500 Boylston (B7).

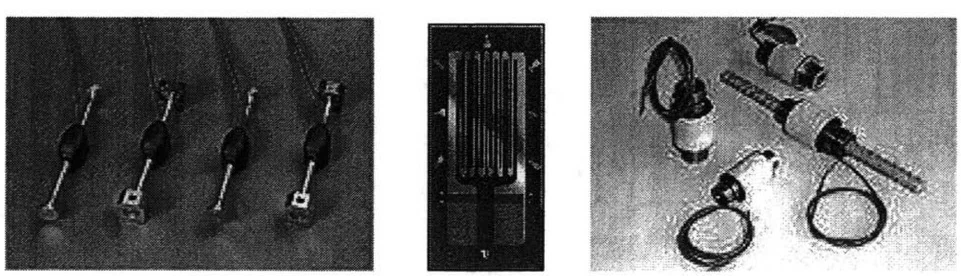
3.1.9 Trench width gages

Trench width gages can be used only to measure distance changes between opposing trench faces. Their application has been very limited with only a few research programs having used them. Their sensitivity is in the order of 0.1" (Xanthakos 1994). They have not been used in any of the studied excavations.

3.1.10 Strain Gages

Strain gages are quite commonly used to deduce loads in cross-lot struts or braces. The load is deduced from the measured strain, assuming a modulus of elasticity for the member that is tested. The actual load can be deduced only if the gages are installed before the struts are pre-loaded. The strain gages should have a modulus of thermal expansion approximately equal to that of the member they are attached to avoid any strain measurements due to temperature change. The gages should also be installed at a distance greater than 5' from the member end in order to avoid measuring the combined effect of bending and axial strains on struts. Strain gages have also been used to deduce tieback loads.

Strain gages have also been used to deduce loads, moments, and stresses in the reinforcement and in the slurry wall (Fig. 3.14c). Gages attached to the reinforcement cage can be used to deduce the stress at the reinforcement. Embedment gages in the concrete can be used to monitor the stress within the concrete. Sister bar gages are short lengths of reinforcement in a bridge resistance circuit that are cast in the concrete. They can be used to deduce the wall moments (Dana Farber case study: B-9).

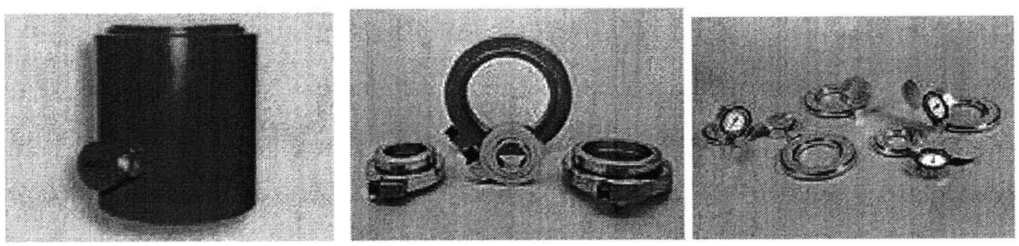


(A) (B) (C)

Figure 3.14: (A) Vibrating wire strain gages, (B) Strain gage electric resistor, (C) Rebar strain gages.

3.1.11 Load Cells

Load cells have been used in many projects to measure tieback loads and thus give an idea about the performance of the tieback bracing system through (Fig. 3.15). Load cells have a circular opening in the middle that allows the anchor to pass. The strain measurement can be made by mechanical, hydraulic, or electrical gages.

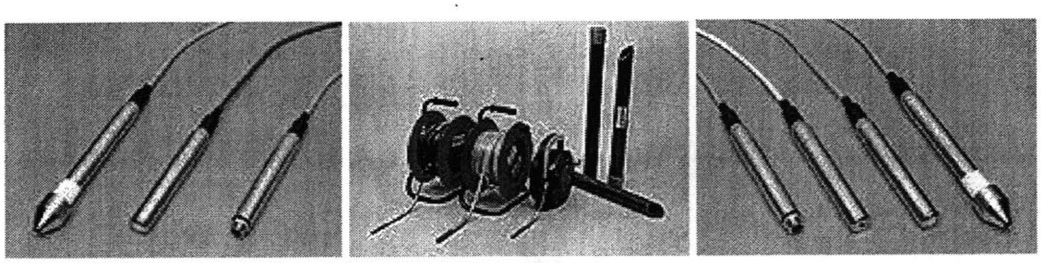


(a) (b) (c)

Figure 3.15: Load cells by Pagani, (a) Strain gage load cell, (b) Electric anchor cell, (c) Hydraulic anchor cell (http://www.pagani-geotechnical.com/)

3.1.12 Observation Wells and Piezometers

Observation wells are used to monitor the water level, and they are always used in excavations since water drawdown outside the excavation is a major issue. Piezometers are used to monitor pore water pressures at elevation where they are screened. Measurements are usually made by vibrating wire, standpipe, or electrical piezometers (Fig. 3.16). Both piezometers and observation wells are installed within drilled boreholes. The observation wells can easily function as dewatering wells if it is required.

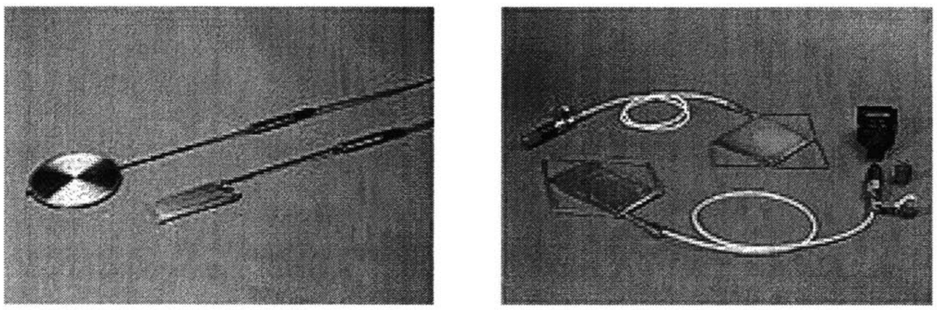


(a) (b) (c)

Figure 3.16: Piezometers by Pagani, (a) Vibrating wire, (b) Casagrande standpipe piezometer and water level meter, (c) Electrical piezometers (Pagani, 2000)

3.1.13 Earth Pressure Cells

They have been used only in research projects to measure earth pressures and compare them with theoretically calculated pressures (Xanthakos, 1994). A piezometer must be placed in proximity if effective stresses are to be measured. Pressure cells can also be used to measure pressured within concrete. Earth and concrete pressure cells can be seen in Figure 3.17.



(a) (b)

Figure 3.17: Pressure cells by Pagani, (a) Earth pressure cells, (b) Concrete pressure cells.

3.2 Previous Performance Data for Slurry Wall Excavations

3.2.1 Wall & Ground Movements

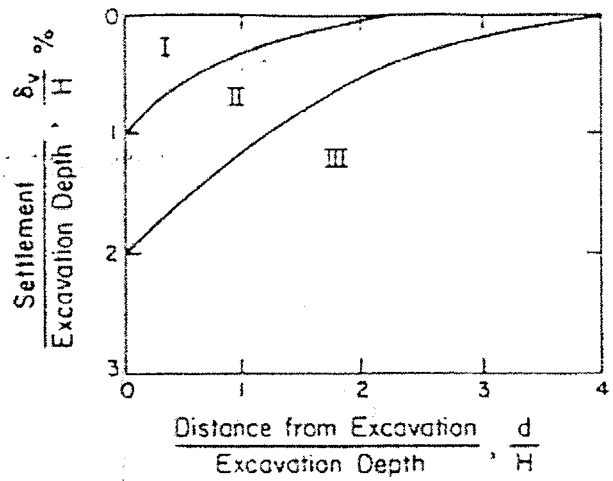
The first major compilation of excavation field data was published by Peck [1969]. He compiled ground deformation measurements from excavations in performed in San Francisco, Chicago, St. Louis, and Oslo (Norway). All of these excavations were supported by either soldier piles and lagging or sheet pile walls. A variety of wall supports were included in the case studies ranging from cross-lot bracing, pre-stressed rakers, H-pile tiebacks, and anchors. In addition to the variation in wall support system, the excavations took place within wide range of soils – soft to medium clay, stiff clay, cemented sand, and cohesionless sand. The excavation depths, H, range from 20' to 63' [6-m to 19m]. With this database, Peck generated a summary of the expected normalized limits and normalized magnitudes of the settlement troughs as a function of soil type, excavation depth, and “workmanship”; both the limits and the magnitudes were normalized with respect to the total excavation depth, H.

Figure 3.18 shows the design chart presented by Peck (1969) together with data compiled by O’ Rourke (1976) from projects in Chicago. Peck subdivides the movements into three categories according to the dominant soil type, and excavation conditions. Zone I soils are described as sands and soft to hard clays, with an average workmanship. Zone II conditions have very soft to soft clays to a limited depth below the excavation, while Zone III conditions have very soft clays to a significant depth below the excavation.

The maximum predicted settlement even in Zone I is quite large if we have a deep excavation. For example, for zone I the chart would predict a maximum settlement equal to 1% of the excavation depth. This implies up to 7.2” of settlement for 60’-deep excavation, while in zone III the chart suggests in excess of 14”, with the zone of disturbance extending up to 240ft (44m) from the excavation.

Currently, there are no standard design methods for estimating ground deformations caused by deep excavations. The current prediction methods fall under two basic categories: 1) empirical, and 2) numerical. However, neither of the two approaches can satisfactorily account and quantify the influence of every factor which contributes to ground movements because of the inherent complexities in staged excavations.

It is very diffucult to determine the influence of the individual factors from empirical data due to the limited number of excavations under similar soil and construction conditions. On the other hand, existing numerical solutions tend to be site-specific and not available to design recommendations. Non the less, both these general approaches can help in understanding the problem of excavation- induced deformations.



(a) Peck 1969

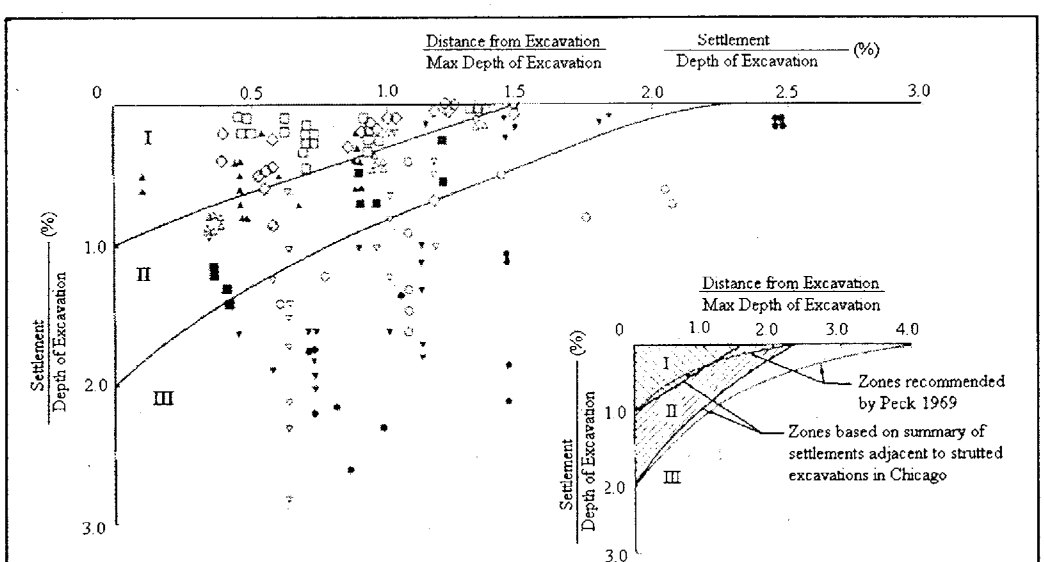
I - Sand and Soft to Hard Clay, Average Workmanship

II - Very Soft to Soft Clay
1. Limited Depth of Clay Below Excavation Bottom
2. Significant Depth of Clay Below Excavation Bottom But $N_b < N_{cb}$*

III - Very Soft to Soft Clay to a Significant Depth Below Excavation Bottom and $N_b > N_{cb}$

*N_b = Stability No. Using S_{ub} Below Base Level = $\frac{\gamma H}{S_{ub}}$

N_{cb} = Critical Stab No. for Basal Heave



Notes:
(1) Zone I - Well-braced excavations with slurry wall or substantial berms left permanenly in place
(2) Zone II - Excavations with temporary berms and raking support
(3) Zone III - Excavations with ground loss from caisson construction or insufficient wall support

(b) O'Rourke (1976) – Data for Chicago Projects

Figure 3.18: Empirical settlement data; (a) Peck 1969, and (b) O'Rourke 1976

Mana and Clough [1981] presented a more detailed analysis of observed wall movements for braced cuts in clay. Their study also focuses on excavations supported by sheetpile walls or soldier piles (mostly with cross-lot bracing). A total of 11 case histories were included in their study. However, a further 100 case studies were eliminated because large deformations were caused by "unusual construction effects" such as consolidation due to dewatering. They report a definite correlation between the measured normalized lateral wall deflection (δ_w / H) and the factor of safety against basal heave greater than 1.5 as defined by Terzaghi [1943] (Fig. 3.19). For factors of safety against basal heave greater than 1.5, the expected maximum wall movement is in the range of $0.2\% \leq \delta_w /H \leq 1.0\%$. However, much larger wall deflections can occur for deep excavations in soft clay where factors of safety against basal heave are usually less than 1.5. Uncertainties in the estimates of δ_w are generally much larger than are needed in design. Since damage to adjacent structures is related to surface settlements and horizontal displacements, Mana & Clough [1981] also attempted to relate maximum lateral wall movement to maximum settlement. Figure 3.20 shows that the maximum settlement, δ_v , is generally between 0.5 to 1.0 times the maximum wall displacements, δ_w. Currently settlements by Peck [1969] and Mana & Clough [1981] findings are based on observations from excavations supported by soldier piles or sheet piles, and therefore, cannot be relatively extrapolated to other support conditions easily.

Clough et. al. [1989], updated the existing database by incorporating the performance of excavations supported by diaphragm walls and relating the deformations to the support conditions and soil profiles. They attempted to correlate the wall movements, δ_w/H, with the stiffness of the lateral earth support system through a stiffness parameter, $EI/(\gamma_w h^4_{ave})$. In this definition EI is the elastic bending stiffness of the wall, h_{ave} the average vertical spacing between supports, and γ_w is the unit weight of water. Their proposed design method (Fig. 3.21) was guided by results of FE analyses and suggests that wall movements decrease significantly with system stiffness. Although the limited experimental data (Fig. 3.20 from Terzaghi, Peck, & Mesri [1996]) provide only minimal justification for the proposed design chart, the results are widely quoted and have been used to justify the selection of stiff diaphragm wall sections to control ground movements.

It is clear that the big limitation of the system stiffness approach is the generic assumption that wall deflections are primarily related to deformations occurring between support levels. In individual projects, there may be several length scales affecting the wall deflections depending on the toe fixity of the wall, the depth to bedrock, the wall embedment below the base of the excavation, the width of the excavation, the size of berms, and the initial unsupported excavation depth. Furthermore, the proposed method of Clough et. al. [1989] takes not account of the stiffness profile in the retained soil.

Recently, a much more detailed finite element study has carried out by Jen [1998]. These finite analyses consisted of three main groups of parametric analyses to quantify the effects of excavation geometry, soil profile, and support system. The parameters she studied included: I) Geometry (wall length, excavation width, depth to bedrock), II) Soil Profile (overconsolidation ratio of the clay, cohesionless layer over a clay stratum, presence of clay crust over low OCR clay stratum), and III) Support System (stiffness of support wall, and bracing components).

Jen [1998] found that walls basically undergo three phases of deformation: i) Unsupported cantilever deflections; ii) bulging (subgrade bending); and iii) toe kickout. She concluded that the actual deformation phase was determined by the wall embedment depth. Wall stiffness was more effective in reducing deformations for soft soils but had a smaller effect in stiffer soils. She also found that the depth to bedrock had a significant impact on the surface settlement at a distance from the excavation.

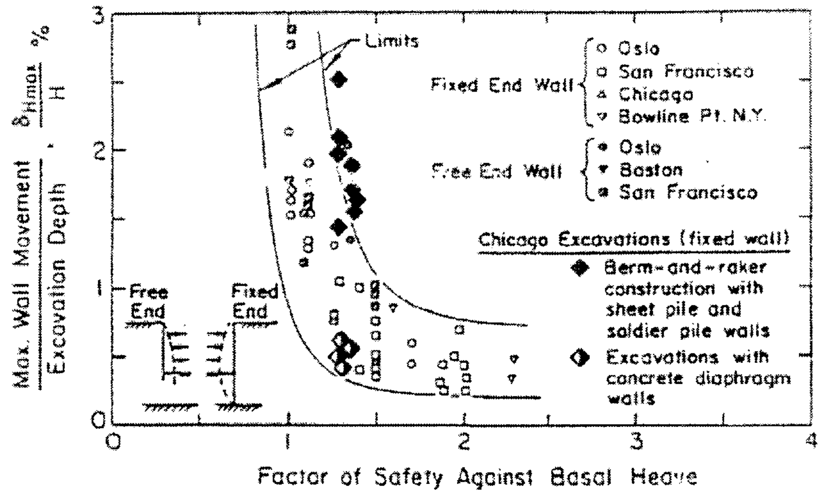


Figure 3.19: Correlation between basal heave stability and measured wall deflections (Mana & Clough, 1981).

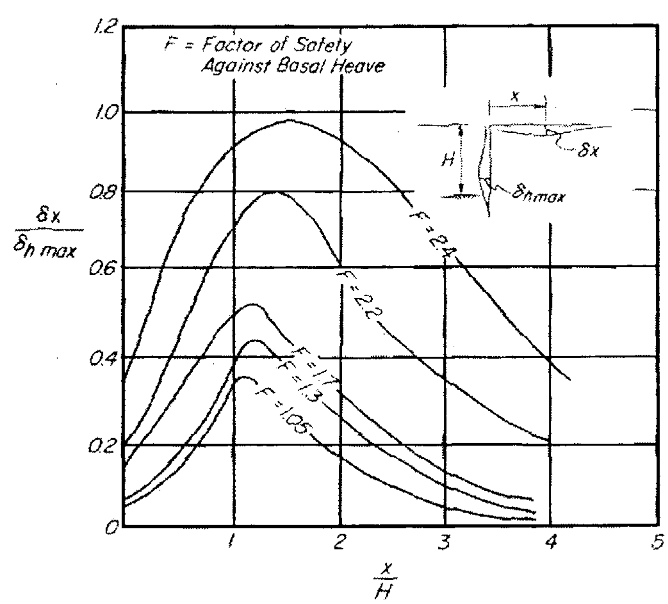


Figure 3.20: Horizontal movement δ_x toward wall of braced open cut in clay, at distance x from face of cut, for various values of factor of safety *F* against heave of bottom cut, as determined by finite element calculations (after Clough et. al. 1989). Values of maximum lateral movement δ_{hmax} to be determined from Figure 3.19.

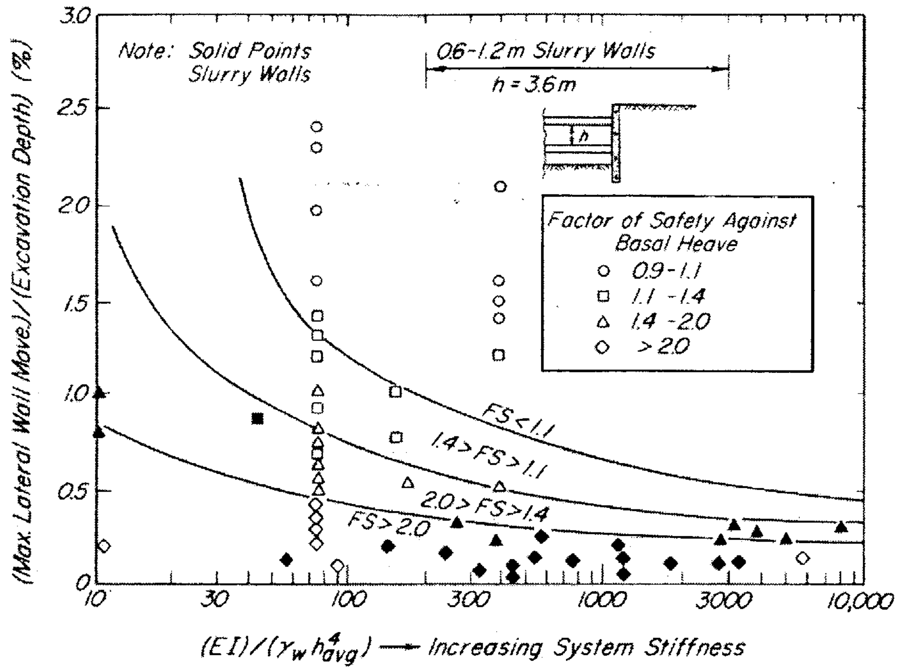


Figure 3.21: Wall deformations/ Excavation depth versus system stiffness and factor of safety against basal heave (Clough et. al. 1989). Terzaghi , Peck & Mesri [1996] data plotted.

Table 3.3: Limit equilibrium calculations of base stability (after Terzaghi, 1943).

$FS=\dfrac{N_c\, s_{ub}}{\gamma - s_{ub}/D}$	For $D<(\sqrt{2}/2)\,B$
$FS=\dfrac{N_c\, s_{ub}}{\gamma - \dfrac{2\, s_{uu}}{\sqrt{2}\, B}}$)	For $D>(\sqrt{2}/2)\,B$
s_{ub} = undrained strength of basal clay s_{uu} = undrained strength of clay above the excavation grade B = Breadth of excavation D = Depth from excavation grade to firm stratum	

Despite the limitations of the system stiffness approach, the limited experimental data reported from slurry wall supported excavations (Fig. 3.20), often show $\delta_w/H \leq 0.5\%$. Apart from these excavation induced movements, several authors (Cowland & Thorley, 1984) have reported data on ground movements due to slurry wall installation. Figure 3.22 shows settlements induced by slurry wall construction at five international sites. These settlements are all smaller than 0.12% of the trench depth at the trench face and decrease to near 0% at distances of 1.0 to 2.0 times the trench depth. Ng and Yan [1998] compared computer FE results with centrifuge test and field measurements settlements and reported settlements generally smaller than 0.05% of the wall depth near the face of the trench (Fig. 3.23). They also reported that the maximum settlement during slurry wall construction occurs within 0.2 times the trench depth behind the wall.

Poh and Wong [1997], reported on a field trial in Singapore where a 1.2m thick, 55.5m-deep slurry wall panel was constructed in mixed soil with thick soft marine layers. The 6-m long test panel was constructed in three bites. They reported that lowering of the slurry level caused the trench wall to move towards the excavated trench side, while raising the slurry level caused the opposite movement (similar behavior also reported by Tamaro et. al., 1996. Trenching caused settlements generally up to 15mm (0.6"), while concreting of the panel decreased the settlements slightly. As expected, they found that maximum horizontal soil movements behind the trench decrease as the distance behind the trench increases (Fig. 3.24).

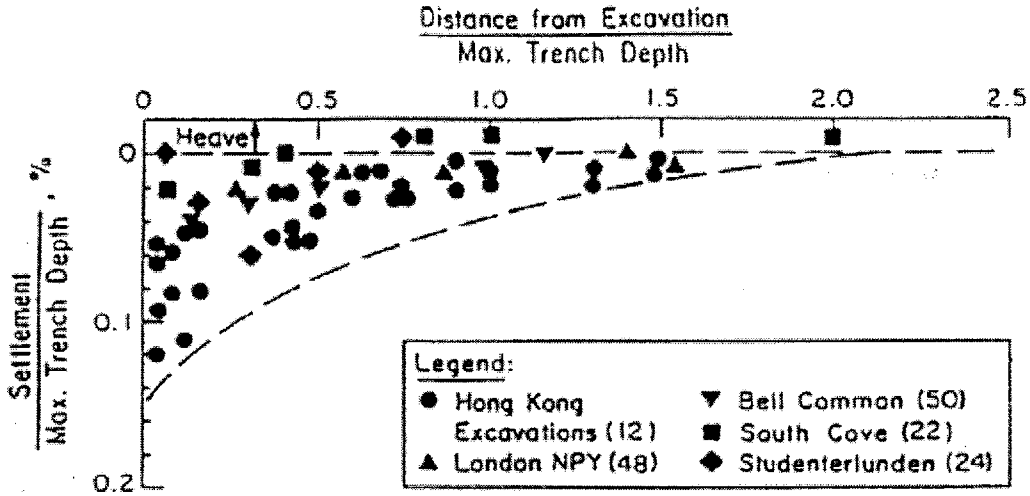


Figure 3.22: Summary of measured settlements caused by the installation of concrete diaphragm walls.

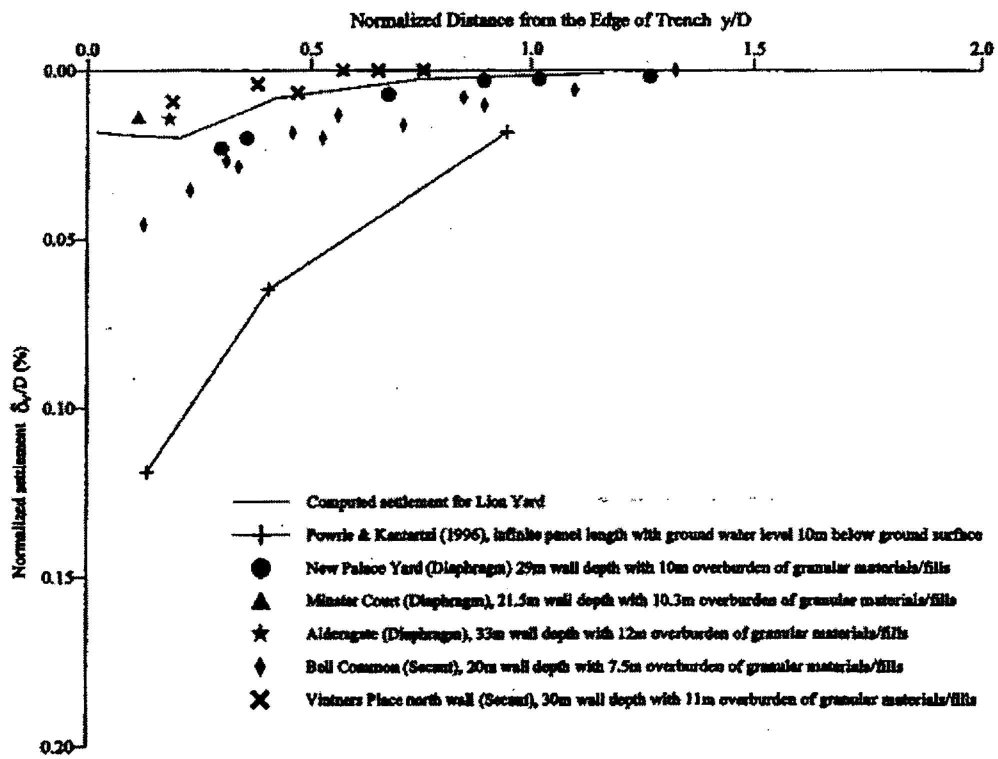


Figure 3.23: Comparisons of computed results with centrifuge test and field measurements (Ng & Ryan, 1998).

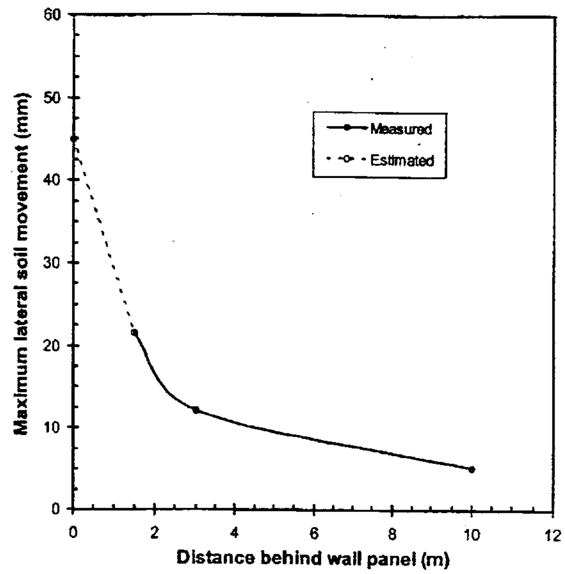


Figure 3.24: Maximum horizontal soil movements versus distance behind wall panel (Poh, & Wong, 1997).

DiBiagio and Roti [1972, 1979], report on a diaphragm wall excavation where lateral earth pressures where measured during and after construction. They reported that the total force on the wall decreased as the excavation proceeded but the total resultant force was always located between 0.44 and 0.45 of the wall height from the bottom. The lateral earth stresses showed a tendency to increase long term. Figures 3.25 and 3.26 show the lateral earth pressured measured at the reported excavation.

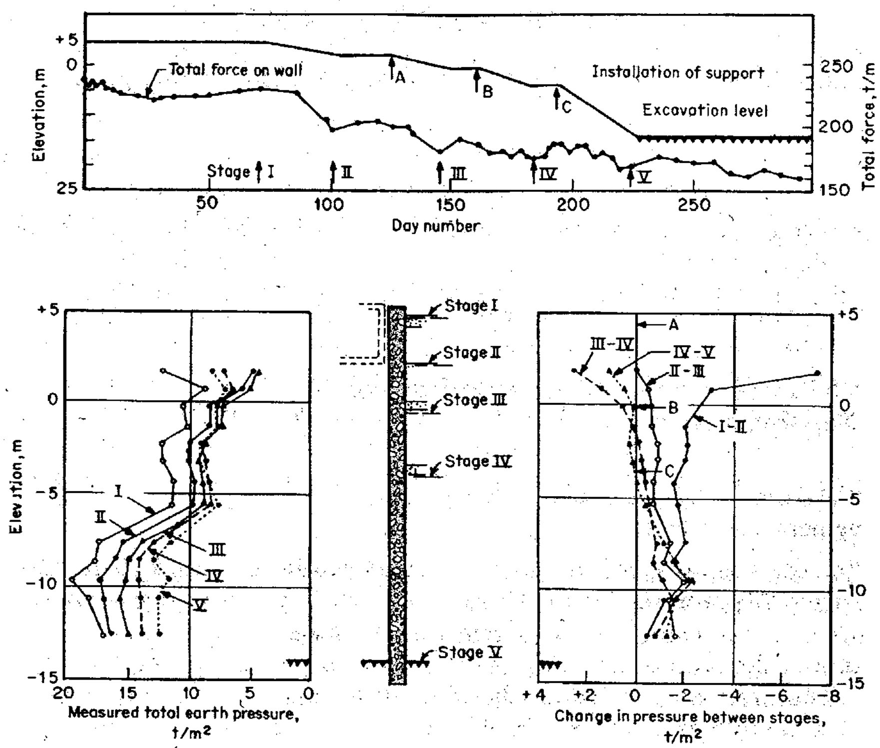


Figure 3.25: Magnitude and changes in total lateral force (DiBiagio & Roti,1972)

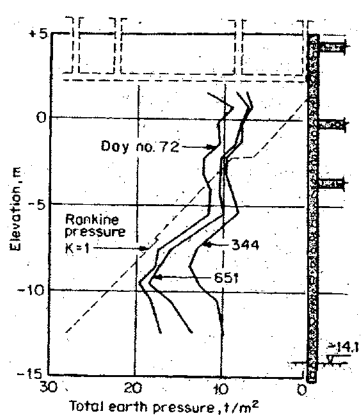


Figure 3.26: Long-term measurements and changes in lateral earth pressures for a wall in Oslo (DiBiagio & Roti 1979)

3.3 Classification of Inclinometer Deflections

Figure 3.27 classified inclinometer deflections according to their mode of deflection. Inclinometer deflections fall under six categories, which are explained in more detail within the figure. These types of deflections will be used in the following chapters to give quick comparisons to horizontal soil and wall deflections.

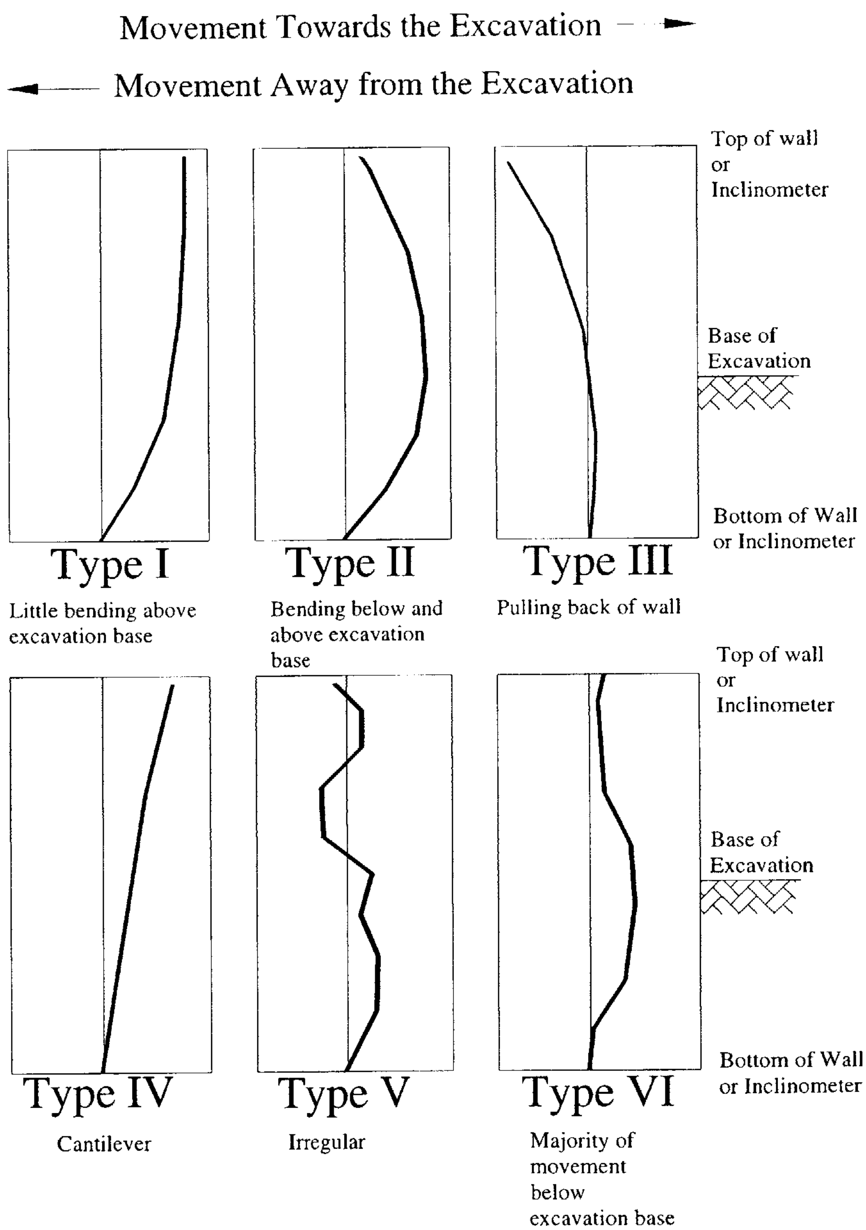


Figure 3.27: Classification of inclinometer deflection shapes.

Chapter 4

Boston Slurry Wall Projects

Chapter 4

Boston Slurry Wall Projects

4.1 Introduction

Boston is one of the cities in the U.S. with the largest concentration of diaphragm wall supported excavations. Slurry wall excavations were first introduced in the city of Boston in the early 1969 with the South Cove cut-and-cover tunnel (Lambe et. al, 1972, D'Appolonia, 1973), and the 60-State Street Office building in 1975 (Johnson, 1976). More diaphragm wall projects were carried as expertise increased, and today major parts of the Central Artery Tunnel (CA/T) project are supported by slurry walls (often SPTC type). Currently, there are several ongoing projects using slurry walls as well as three proposed projects on the MIT campus, i) the Stata Center, and ii) the Media Center. Table 4.1 lists the projects that have been studied in this research, while Table 4.2 lists other known diaphragm wall excavations in Boston.

Boston does present some relatively unique issues regarding deep excavations:

- The soil profile is dominated by post-glacial deposits. The depth to bedrock can exceed 150' in some locations (notably south Boston), and the dominant Boston Blue Clay (BBC) is a soft and sensitive marine clay.
- Many old structures, some historic and supported by wooden piles like the Trinity Church in the 500-Boylston St. case study (B7), have to be protected from excavation induced damage.
- Lowering of the groundwater table in the upper fill can cause decay of the old wooden pile foundations.
- Groundwater inflow is generally not a problem given that the profile often includes clay. However, there is a high water table and the fill/till/ and rock strata are more permeable thus problems with inflow from these layers can result.

Slurry walls are generally more expensive than other methods of lateral earth support. Diaphragm walls were used as part of the permanent structure (for the Post Office Square Garage, B10), while ground deformation control was the primary reason they were adopted in most of the projects. Movement control was important in the 60-State St. project (B2) because the excavation was next to the MBTA orange line and other historic structures. The same was true for the 500-Boylston Street excavation (B7), where the historic Trinity Church and the MBTA Green Line had to be protected. The 60' to 90'-deep Dana Farber excavation (B12) was right next to an operating power plant and other structures that had to be protected. In the Millennium Place (B13), the 55'-deep excavation is abutted by several buildings and the MBTA Orange Line.

Requirements in the Dana Farber project (B12) called for the building to be isolated from vibrations of an adjacent power plant. The adopted design used permanent diaphragm walls extending into bedrock isolated from the rest of the basement structure. Slurry walls were also used in waterfront projects like the Flagship Wharf (B8), Rowes Wharf (B5), and One Memorial Drive (B6), to minimize potential water inflow.

In the CA/T project, huge T-walls have been used in cut & cover tunnels to minimize damage to adjacent structures and to avoid basal stability problems. The extended use of diaphragm walls in the CA/T project probably reflects the confidence of the local geotechnical consultants in recommending this technology.

Table 4.1: List of studied slurry wall excavations in Boston

ID	Project Name	Year	Depth(ft) H	Depth(ft) D	Thick (inches)	Soil Type*	Bracing	Wall Type	Toe Fixity
B1*	MBTA South Cove	1969	50	30	36	A	3-Lev CLB	RCDW	√
B2	60-State Street	1975	35	27	30	A, B	3, 2 –Lev TB	RCDW	√
B3	State Transportati on Building	1982	27	19	24	B	2-Lev TB	RCDW	
B4 **	75 State Street	1984	65	35	30	A	6-Lev TD	RCDW	√
B5	Rowes Wharf	1984	55	15	30	B	5-Lev TD	RCDW	√
B6	One Memorial Drive	1985	30	11	24	A	2, 1-Lev. TB	RCDW	
B7	500 Boylston	1987	42	14	24	A	4 Lev. TB or 1-TB, 2 R	RCDW	
B8	Flagship Wharf	1989	47	13	30	C	3-Lev CLB	PT	√
B9 *	125 Summer Street	1990	60	?	30	B	5-Lev TD	RCDW	√
B10	Post Office Square Garage	1989	75	12	36	A	7-Lev TD	RCDW	√
B11	Beth Israel Deaconess	1994	65	24	36	B	5-Lev TD	RCDW	√
B12	Dana Farber Tower	1995	60 90	2	36	B	4-Lev TB 6-Lev TB	RCDW	√
B13	Millennium Place	1999 2000	55	41	36	B	5-Lev TD	RCDW	√

Note: * According to Johnson 1986, **H** – Excavation Depth, **D**- Embedment depth, **TB** - Tiebacks, **CLB** - Cross-Lot Bracing, **TD** - Top/Down, **R** – Rakers, **SB**- Soil Berms, **CB** – Corner Bracing, **IB**-Internal Bracing, **PC** – Precast, **PT** – Post Tensioned, **SP** – Soldier Piles, RCDW – Reinforced Concrete Diaphragm Wall, **SPTC** – Soldier Piles & Tremie Concrete.

Table 4.2: Other known slurry wall excavations in Boston

Project	Year	Project	Year
MBTA Washington St. Subway Station	1980s	Cambridge Hospital	1998
Harvard Sq. Station	1988	World Trade Hotel	1999
Pilot House Extension	1996	Trinity Place	1999
Seaborn Hotel	1998	MBTA Silver Line, South Boston Station	2000

4.2 Soil Conditions in Boston

The principal soils in Boston are primarily post-glacial deposits or manmade fills. When the city was first colonized in the 1600s it was situated on a peninsula that became an island during high tide (Fig. 4.1 shows the colonial shoreline). Subsequent infilling of the shallow marshlands and bays begun in the 1820s and continues to this day (Fig. 4.2).

The predominant upper bedrock in Boston is the Cambridge Argillite (Johnson, 1989). When it is fresh and unweathered, the argillite is typically a hard, blue-gray, finely laminated rock. Locally layers of tuff and sandstone are also typical of this formation as well as numerous intrusive sills and dikes of diabase, diorite or basalt. However, in many areas the argillite is highly weathered or altered, containing zones of clay-like kaolinized material. The thickness of these clay-like zones may vary from a few inches to hundreds of feet (Humphrey & Soydemir 1991). Humphrey and Soydemir point out that kaolinization has been encountered and reported in many areas in Boston, and that the alteration has been found in all rock types including the conglomerate.

Conglomerates may be encountered locally in southern and western portions in Roxbury and Brookline. Conglomerate is a very hard and durable rock, usually molted brown in color, with embedded round to angular pebbles, and resembles a dense concrete material (Johnson, 1989).

Johnson [1989] proposes three typical soil profiles in the greater Boston area:

- Profile A is widely distributed and is typical of Back Bay and marginal waterfront areas
- Profile B is representative of intermediate areas adjacent to the original Boston Peninsula.
- Profile C comprises moraine deposits over bedrock found typically within the limits of the original colonial shoreline of the Boston Peninsula

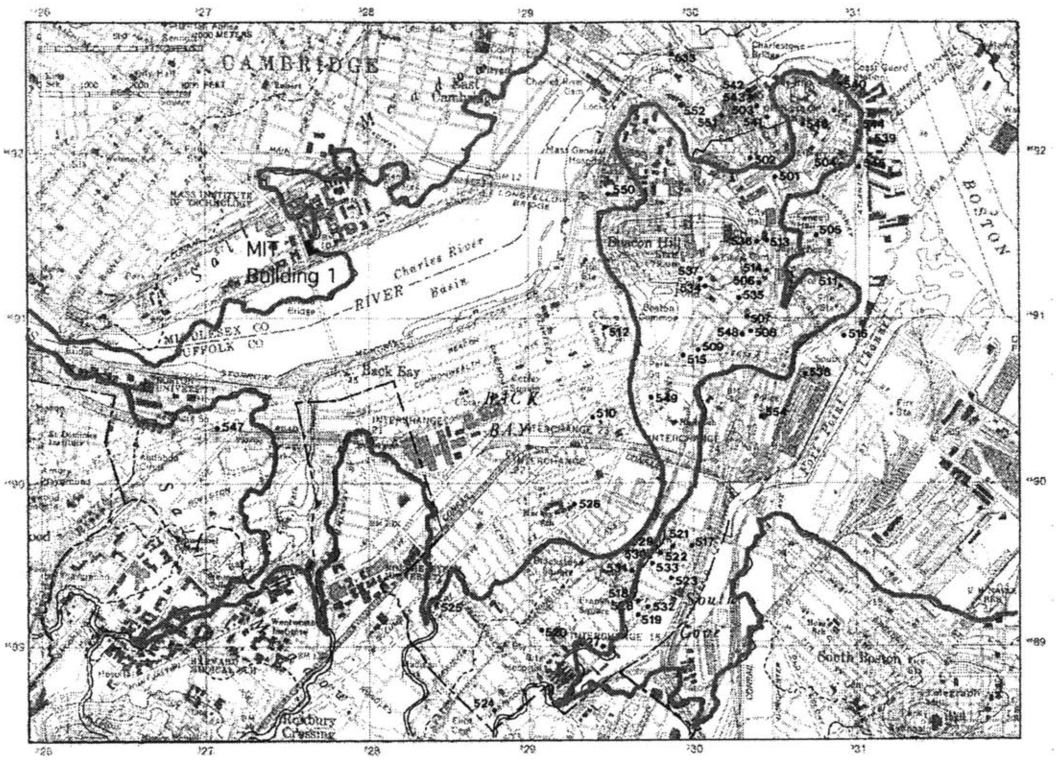


Figure 4.1: Colonial shoreline superimposed over current map of Boston.

The Glacial till (VI) is a very compact, unsorted layer comprising of either a clayey or sandy matrix containing cobbles and boulders. Engineering properties of the till are highly variable and SPT N-values are typically used to describe the strength of this layer. Glacial till overlies the bedrock throughout most of greater Boston. Zones of softer material are also can be found within the glacial till.

Outwash deposits (Unit V) are of glaciofluvial origin and consist of medium dense, stratified sands and gravels. Outwash deposits are typically discontinuous and overlie the till layer.

The marine clay or Boston Blue Clay (BBC) is the predominant layer throughout most of the infilled areas. The thickness of the clay layer varies widely within the Boston area. In the Back Bay the clay thickness is in the order of 100', in East and in South Boston it is up to 150ft thick. Engineering properties of BBC have been thoroughly investigated for foundation design. The BBC deposits typically have a relatively stiff crust overlying nearly normally consolidated clay, such that net loadings can cause significant long term settlements and deep excavations can pose severe problems due to bottom instability and movement of adjacent ground (Ladd et. al., 1999). The crust is overconsolidated with maximum OCR= 4 – 6 and has much higher undrained shear strengths (s_u = 2ksf) compared to the lower portion of the clay (s_u = 0.6 – 1.2ksf). Discontinuous sand layers and lenses are often found within the upper clay.

Area	Size (acres)	Dates
1. West Cove	80	1803-1863
2. Mill Pond	70	1807-1829
3. South Cove	86	1806-1843
4. East Cove	112	1823-1874
5. South Boston	714	1836-Present
6. South Bay	138	1850-Present
7. Back Bay	580	1857-1894
8. Charlestown	416	1860-1896
9. Fenway	322	1878-1890
10. East Boston	370	1880-Present
11. Marine Park	57	1883-1900
12. Columbus park	165	1890-1901
13. Logan Airport	150	1922-Present.

Figure 4.2: Map of the Boston Metropolitan Area showing the history of land filling that expanded the city's original 783 acres to over 3000 acres today, (after Boston Society of Architects, 1976).

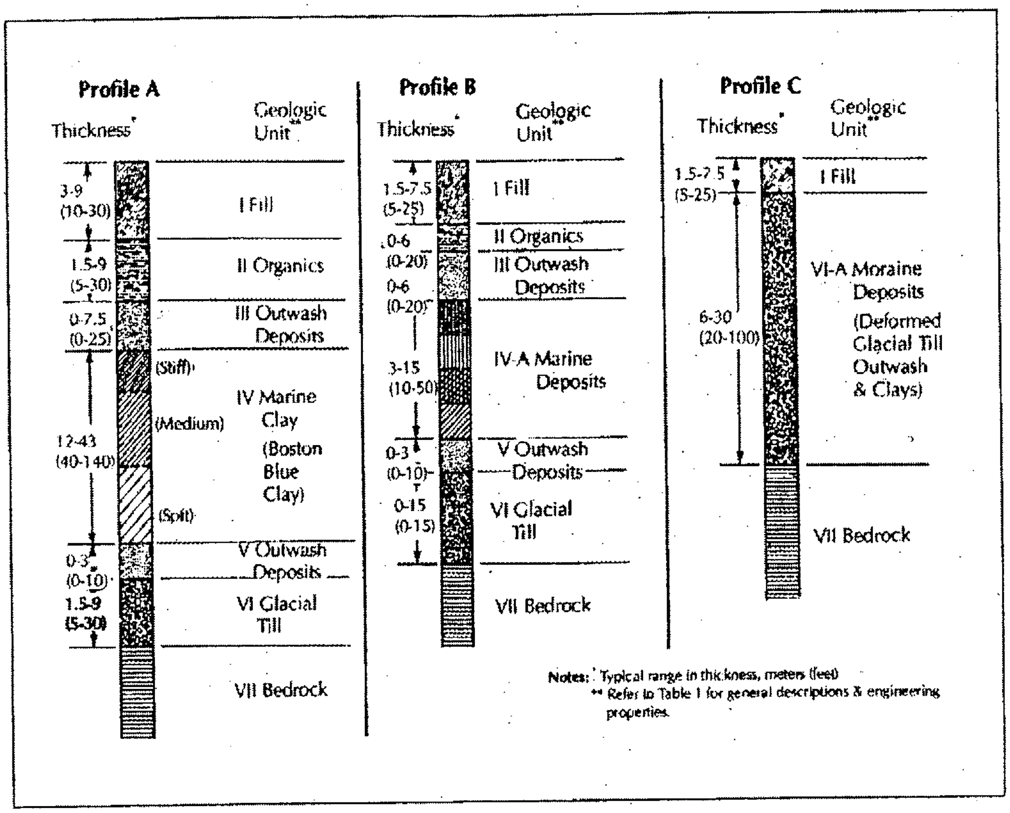


Figure 4.3: Geologic units encountered in typical major foundations in Boston (Johnson, 1989).

Much of the present city of Boston has been constructed on fills that were placed gradually since the colonial times. Woodhouse [1989] gives more information about the sequence of fill placement in the Boston. The fill layers are loose and variable in content, with brick fragments and occasional building remnants.

Unit III consists of well-stratified outwash sands and gravels that were deposited over the surface of weathered clay in some areas, following another advance of glacial ice (Johnson, 1989). These deposits are present in some areas while in others they are absent. They are medium to compact and are relatively highly permeable, with the layer thickness ranging from 10 to 25 ft.

Organic silt and clay deposits were formed throughout the lower lying areas surrounding Boston following the ice age (Johnson, 1989). The thickness and the content of this layer varies greatly. When present, the organic layer thickness ranges from 5ft to 25ft. In infilled areas like the Back Bay this layer has been compressed due to the weight of overlying fill.

Johnson also summarized engineering properties of foundation material in Boston. Table 4.3 shows basic soil properties as summarized by Johnson [1989]. More recently, Ladd et. al. [1999] researched extensively the engineering properties of Boston Blue Clay as part of a special testing program for the CA/T project in South Boston.

Table 4.3: Typical Engineering Soil Properties in Boston (Johnson, 1989).

Geologic Unit	General Description	Saturated Unit Weight kg/m³ (lb/ft³)	Natural Water Content (percent)	Atterberg Limits (percent) LL	PI	Undrained Shear Strength kg/m² (lb/ft²)	Other	Allowable Bearing Pressure kg/m² (lb/ft²)
I. Miscellaneous Fill	Loose to very dense sand, gravelly sand or sandy gravel, intermixed with varying amounts of silt, cobbles or boulders, & miscellaneous brick, rubble, trash or other foreign materials.	1600-2000 (100-125)	—	—	—	—	—	—
II. Organics	Very soft to medium stiff, grey clayey organic silt or brown fibrous peat with trace amounts of shells, fine sand & wood.	1440-1760 (90-110)	40-100	—	—	1465-3900 (300-800)	Organic Content 5-25%	—
III. Outwash Deposits	Medium dense to dense, brown coarse to fine or medium to fine sand with varying amounts of gravel & silt.	1760-2160 (110-135)	—	—	—	—	—	19500-48800 (4000-10000)
IV. Marine Clay	Stiff, yellow-grey silty clay.	1840-2000 (115-125)	25-35	40-55	15-30	3900-9760 (800-2000)	Compression Ratio =	14650-39000 (3000-8000)
	Medium stiff, grey silty clay, occasional layers of fine sand or silt.	1824-1920 (114-120)	30-40	40-55	15-30	2930-5860 (600-1200)	0.15-0.25 Recompression Ratio =	9760-19500 (2000-4000)
	Soft to very soft, gray silty clay, occasional layers of fine sand or silt. (Note: This unit sometimes becomes stiffer at lower levels.)	1810-1890 (113-118)	30-50	40-55	15-30	1950-3900 (400-800)	0.02-0.04	4880-9760 (1000-2000)
IV-A. Marine Deposits	Interbedded grey silty or sandy clay, silty fine sand & fine sandy silt	Too variable	—	—	—	—	—	Variable
V. Outwash Deposits	Medium to dense, stratified sands & gravels in discontinuous layers.	—	—	—	—	—	—	Variable
VI. Glacial Till	Dense to very dense, heterogenous mixture of sand, gravel, clay & silt with cobbles & rock fragments.	2000-2240 (125-140)	10-20	15-30	10-20	9760-39000 (2000-8000)	—	39000-98000 (8000-20000)
VI-A. Moraine Deposits	Miscellaneous deposits of deformed glacial till, outwash & clays.	Too variable	—	—	—	—	—	Variable
VII. Bedrock	Cambridge Argillite.	—	—	—	—	—	—	78000-195000 (16000-40000)
	Roxbury Conglomerate.	—	—	—	—	—	—	195000-975000 (40000-200000)

Note: Metric units above English units in parentheses.

4.3 Soil Testing Results

Ladd et. al [1999] conducted an extensive program of lab and in-situ tests in BBC in at two special test sites in South Boston (SB) and East Boston (EB) for the Central Artery Project. This program sought to develop correlations and guidelines for BBC characterization that would be useful for geotechnical practice throughout the Boston area. Figure 4.4 shows the general stratigraphy at the test sites together with index properties at the test sites.

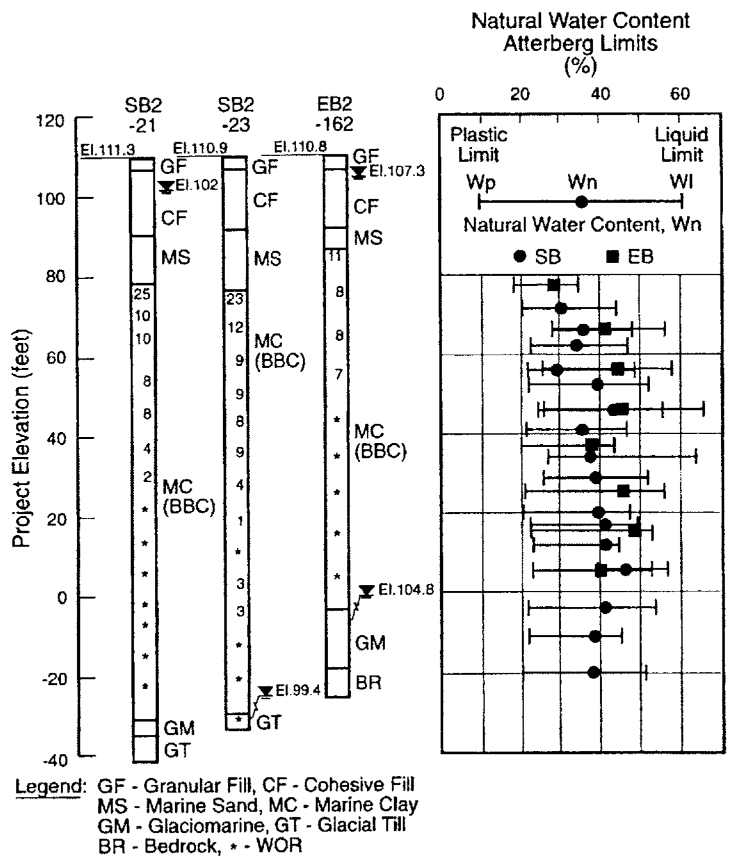


Figure 4.4: Soil Profile, Index Properties and Piezocone Data (Ladd et. al. 1999).

Figure 4.4 shows that natural water contents within the crust range from 29% to 40%, and approximately 40% in the normally consolidated BBC. Liquid limits ranged from 40% to 60% which is typical for BBC and illitic clays in New England.

Figure 4.5 shows that there are major variations in the stress history profile between the South and East Boston sites. For example there is a 20ft difference in the elevation of the base of the crust.

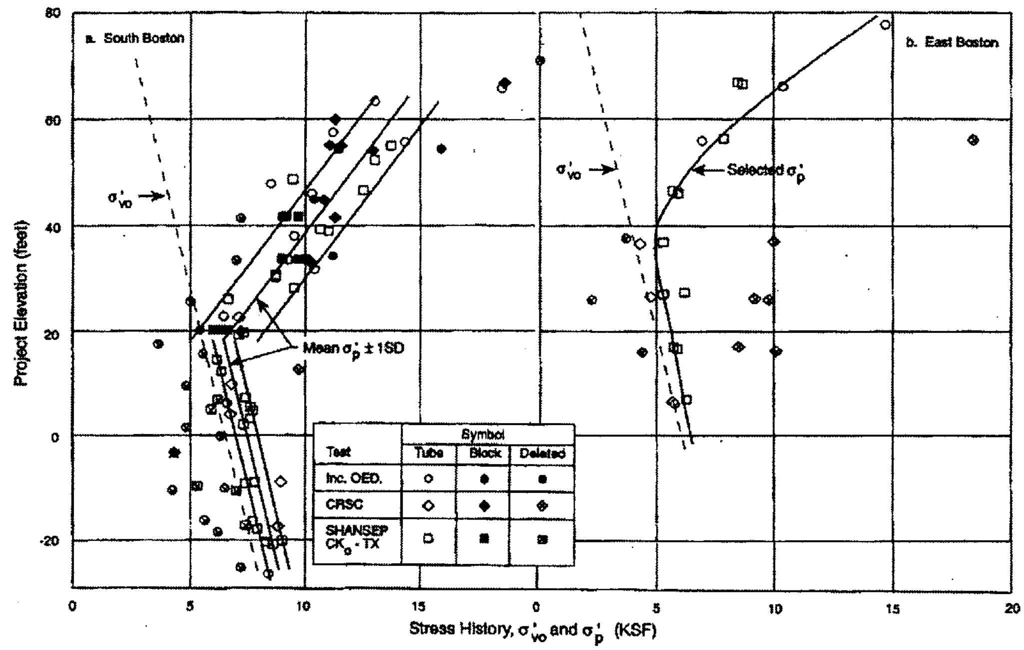


Figure 4.5: Stress History from 1-D Consolidation Tests (Ladd et. al. 1999).

The overconsolidation ratio within the crust ranges from 7 and decreases to 1.1. Using the SHANSEP method (Ladd & Foott 1974), Ladd et. al [1999] report the following undrained shear strength for this clay profile $s_u = 0.2\ \sigma'_{vo}\ (OCR)^{0.8}$ which leads to average s_u= 2.0 ksf in the upper clay crust, which decreases to s_u=0.9 ksf in the lower normally consolidated clay.

At rest coefficients of lateral earth pressure (K_o) were also estimated using lab and in-situ dilatometer tests (Marchetti, 1980). Figure 4.6 shows that K_o decreases from K_o =1.0 to 1.5 at the top of the clay to K_o =0.5 in the normally consolidated clay.

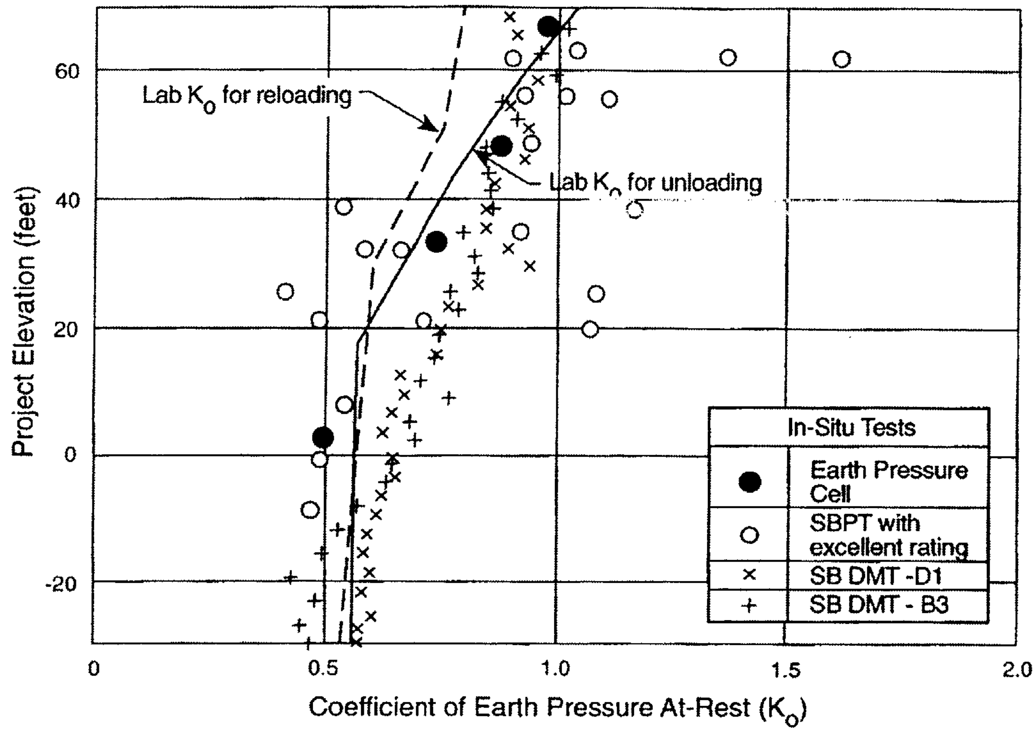


Figure 4.6: K_0 versus Elevation from Laboratory and In Situ Testing (Ladd et al. 1999).

4.4 Summary of Boston Projects

Table 4.4 summarizes the measured performance from the database that has been developed for slurry wall projects in Boston. The first of these projects occurred in 1969 when diaphragm walls were used to mitigate potential damage to the Don Bosco school due to underground construction of the MBTA Orange Line at South Cove (now called the New England Medical Center Subway). This project was extensively documented by researchers at MIT (Lambe et. al., 1972, Jaworski, 1973). Although slurry walls have been used extensively in the CA/T project and are currently being constructed for the MBTA South Boston Transitway, all of the other projects listed in Table 4.4 are related to construction of basements and underground parking facilities of multi-story buildings.

Figure 4.7. shows the project locations listed in Table 4.1. The main cluster are in the downtown and waterfront areas (B2, B4, B5, B10), two in the Longwood Medical area (B11, B12), three projects were within the old peninsula

(B9, and near the Boston Common B3, B13), one in Back Bay (B7), and one in Cambridge in similar conditions to Back Bay (B6). Only one of the projects is located was constructed in profile C conditions (i.e. moraine deposits) (Flagship Wharf, B8).

When ever it was possible and feasible, the diaphragm walls were keyed into the stiff glacial till in order to minimize horizontal movements at the toe of the wall and to transfer vertical superstructure loads. Some walls (B6, B7) were embedded into the clay crust because it was impractical to reach glacial till. These projects were located in Back Bay and in Cambridge near the Charles River where glacial till is found in depths in the order of 130'. The Boston database covers projects with a range of excavation depths from 27' to 90' and with a variety of different bracing systems. Temporary tieback bracing was used in three projects (B3, B6, B7), while top/down construction has been used in six projects. There are only two cases where cross-lot bracing was used (B1, B8).

Table 4.6 shows that most of the excavations performed well with small wall deformations induced by the excavations. Horizontal wall deflections were generally less than 1.5", with settlements in the same order. Most of the walls deflected in a bowing pattern (B2, B3, B4, B5, B8, B10, B11, B13)(Type II, Figure 3.27), although different deflection modes were observed within the same project (B2, B7, B12, B13). Walls anchored with tieback but no toe fixity (B3, B6, B7)(i.e. floating walls), generally wall bending beneath the lowest support level (Type I), and very little bending between bracing levels. Walls with toe fixity measured smaller deflections on average than walls with no toe fixity.

Table 4.4: Summary of measured performance of slurry walls in Boston.

ID	Year	Project Name	Soil Type *	Excav. Bracing/ Excav. Method	Depth (ft) H	Depth (ft) D	Wall Thick (inch)	δ_{Wmax} (Inches)	δ_{Vmax} (inch)	Toe Fix.	Defl. Shape Type
B1	1969	MBTA South Cove	A	3-Lev CLB	50	30	36	1.35	0.5		I
B2	1975	60 State Street	B	2-3 Lev. TB	32	27	30	1.3 0.92	1.2	√	IV II
B3	1982	State Transportation Building	A	2 Lev. TB R, CBL	27	19	24	1.25	1.2		I II
B4	1984	75 State Street	A	6-Lev. TD	65	35	30	1.85	4.0	√	II
B5	1984	Rowes Wharf	B	5 Lev. TD	55	15	30	0.41	___	√	II
B6	1985	One Memorial Drive Building	A	2 Lev. TB	30	11	24	1.3	1.2		I
B7	1987	500 Boylston Building	A	4 Lev. TB, R	42	14	24	3.3	4.5		IV
B8	1989	Flagship Wharf	C	3 Lev CLB	47	13	30	1.81	1.7	√	II
B9	1990	125 Summer St	B	6-Lev TD	60	--	30	0.6	0.38	--	--
B10	1989	Post Office Square Garage	A	7-Lev TD	75	12	36	2.15	2.75	√	II
B11	1994	Beth Israel Deaconess	B	5-Lev TD	55	24	36	0.85	0.7	√	II
B12	1995	Dana Farber Tower	B	6-Lev Rock TB	90	2	36	0.72 0.4	0.64 2.8	√	I, V
B13	1998	Millennium Place	B	5-Lev. TD	55	41	36	0.8	0.7	√	I, II

Notes: * According to Johnson see section 4.2

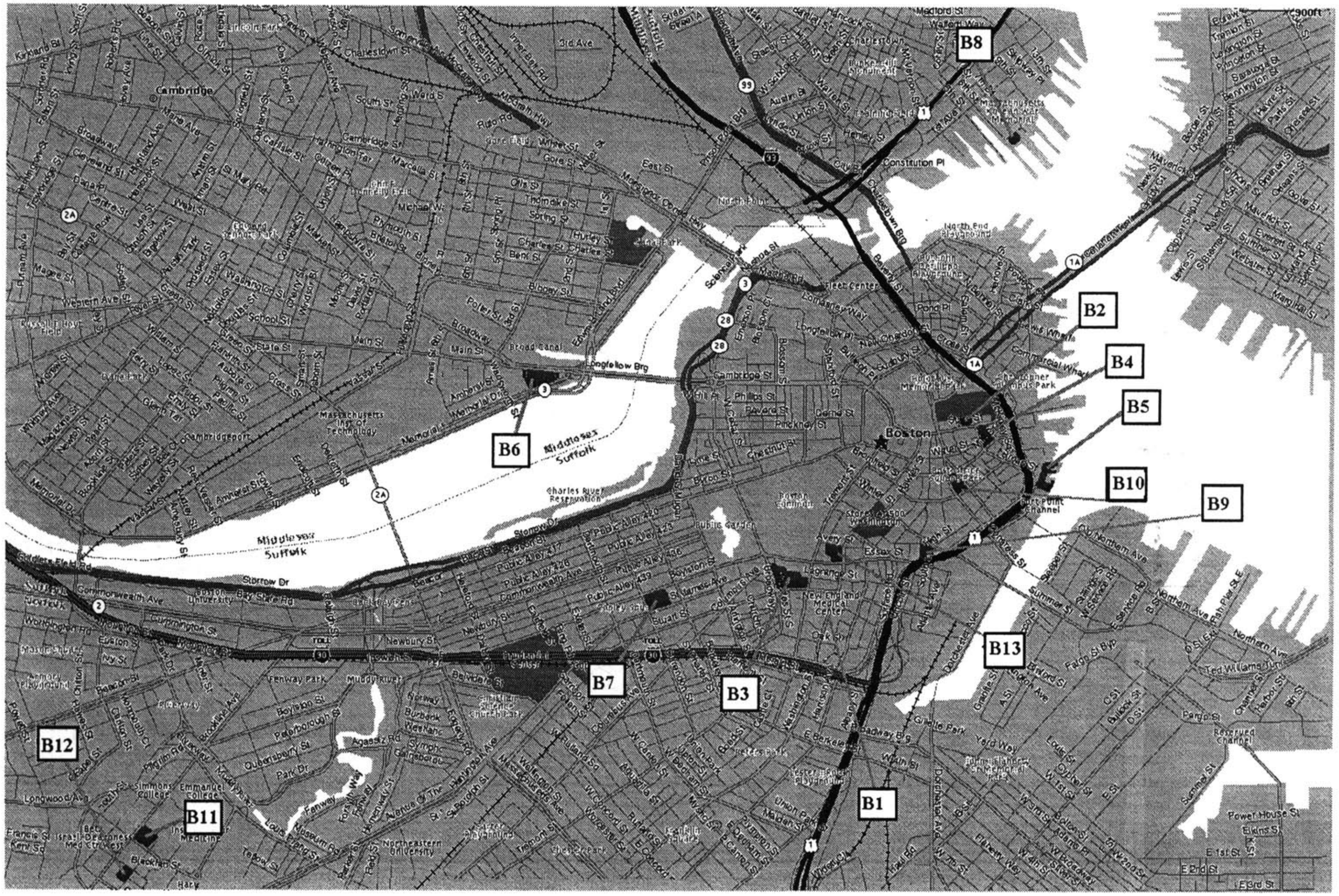


Figure 4.7: Studied slurry wall excavations in Boston

4.5.Individual Case Studies

4.5.1 Case Study B1, South Cove

The MBTA South Cove Project comprised a 50'-deep excavation supported by 3 levels of cross-lot bracing. A sheetpile supported the excavation for most of its length except for a 200' long section adjacent to the Don Bosco School, where a 3'-thick diaphragm wall was used to minimize ground deformations. The toe of the wall was embedded 30' beneath the final excavation grade into the medium BBC. Figure 4.8 shows the soil profile at the site and average undrained strength properties. The bottom of the clay is located at approximately 100' depth and the groundwater table 25' below the ground surface (within the upper clay layer).

The measured pore pressure behind the diaphragm wall at the end of excavation was 10% to 15% less than the hydrostatic condition (Lambe, 1970). Trenching for the slurry wall caused an inward movement of the trench face by 1". When the base of the excavation was reached the wall had moved inward by δ_W =0.5" and the soil by δ_H =1.5". By comparison, the lateral movements of the sheet pile wall were much larger and ranged from δ_H =4.5" to δ_H =7.0". However, the cross-lot bracing was only preloaded at points along the diaphragm wall, and this was likely the major factor affecting the sheetpile deformations. Figure 4.8 shows miscellaneous data and comparison of wall movements for the diaphragm wall and sheeting (D'Appolonia, 1973).

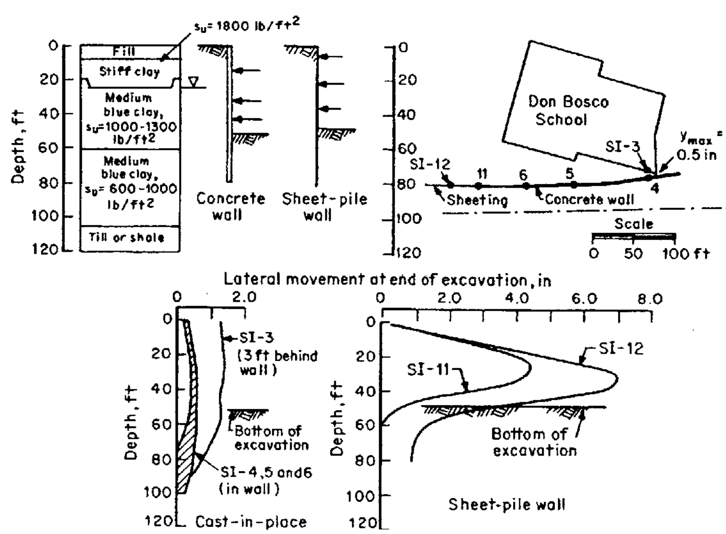


Figure 4.8 Miscellaneous data and comparison of wall movement for slurry wall and sheeting, South Cove Excavation (B1), (From D'Appolonia, 1973)

4.5.2 Case Study B2, 60 State Street

The 60 State Street building is a 38-Story office tower located at the northeastern corner of Congress and State Streets in Boston, Massachusetts. Foundation construction for this project took place in 1975. Many important structures and historic sites surround the building such as the Boston City Hall, the Government Center Complex, the Old State House, Faneuil Hall and the Quincy Markets area. To the east of the project is an 11-story office building at 84-State Street and the 5-story historical Sanborn Building. The MBTA subway passes beneath State St. on the south side (Fig. 4.9).

A three-level deep basement extends to the property limits on all sides and includes an area to the north under the plaza level. The tower, located in the center of the project, is supported on a mat foundation on glacial till, 35' to 50' below street level.

A tied-back perimeter 2.0'-thick diaphragm wall was selected to provide temporary and permanent earth support and water retention. The slurry walls extended to intact bedrock and thus provided a seepage barrier. The tiebacks were prestressed at 100% of their design load. No underpinning of adjacent structures was needed and the construction was completed with a minimum of noise and disruption to this congested neighborhood.

A typical section through east and south sides of the project is shown in Figure 4.10. The marine clay layer lies directly below the fill and has thickness varying from 4' to 35' generally increasing towards the northeast. The upper portion of the clay is a yellow very stiff clay crust containing random lenses or layers of sand. A very compact glacial till varying from clayey to sandy till with boulders or cobbles underlies these deposits. Rock encountered beneath the glacial till is variable but typically is a highly jointed and fractured soft to medium-hard Argillite (Fig. 4.10).

Two or three levels of temporary soil tiebacks were used to hold the slurry walls as the excavation progressed to a final depth of roughly 30' to 37' depending on location. Permanent bracing was provided by the two interior floors and the basement floor slab.

Small lateral soil movements occurred during panel construction. Inclinometer I-1 showed a δ_H=0.3" movement towards the open trench when the adjacent slurry wall panel was excavated and infilled with bentonite slurry. When panels were concreted the soil was pushed back to its original position and beyond (I-4 net apparent movement δ_H= -0.5" back into the soil).

Larger movements occurred in the initial stages of the excavation despite the fact that the slurry walls were embedded in dense glacial till. Including the effect of the wall rigidity, the cantilever movements reached up to δ_W=1.3" at the top of the slurry wall (initial stages). This was surprising since the excavation had only been carried to a depth of 10 to 12'. Wall bending developed as highly prestressed tiebacks were installed, in some cases the wall moved further into the soil than its initial profile. Subsequent building settlements were roughly the same as maximum wall movements at respective locations. Building settlements near the excavation typically ranged from 0.5" to 1" and gradually diminished with increasing distance from the excavation. However, some of the settlement could have been caused by the tieback drilling procedures and may not have been due to wall movements.

Tiebacks were drilled using water and air through a 4"-steel casing and generally performed well. The upper level of tiebacks showed an initial load loss of up to 28 Tons which was more than the theoretical 8 Tons loss resulting from elastic tieback shortening. It was assumed that yielding in the grouting zone and seating losses were the causes of these load losses. The lower level of tiebacks showed slow progressive load decay that matched the theoretical load loss resulting from elastic shortening.

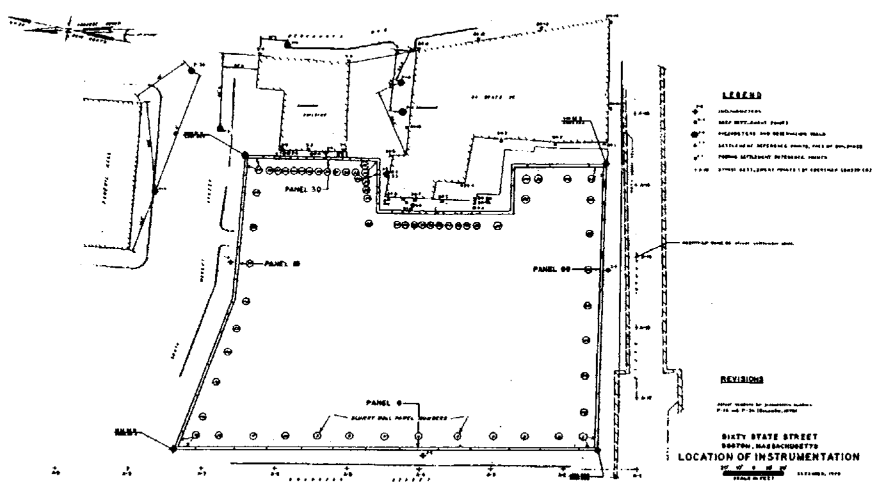


Figure 4.9: 60 State Street site plan, (after Johnson, 1976).

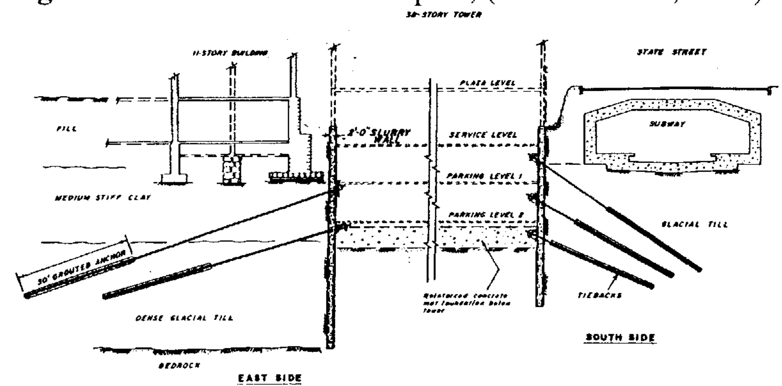


Figure 4.10: Typical excavation section , 60 State Street, (Johnson, 1976).

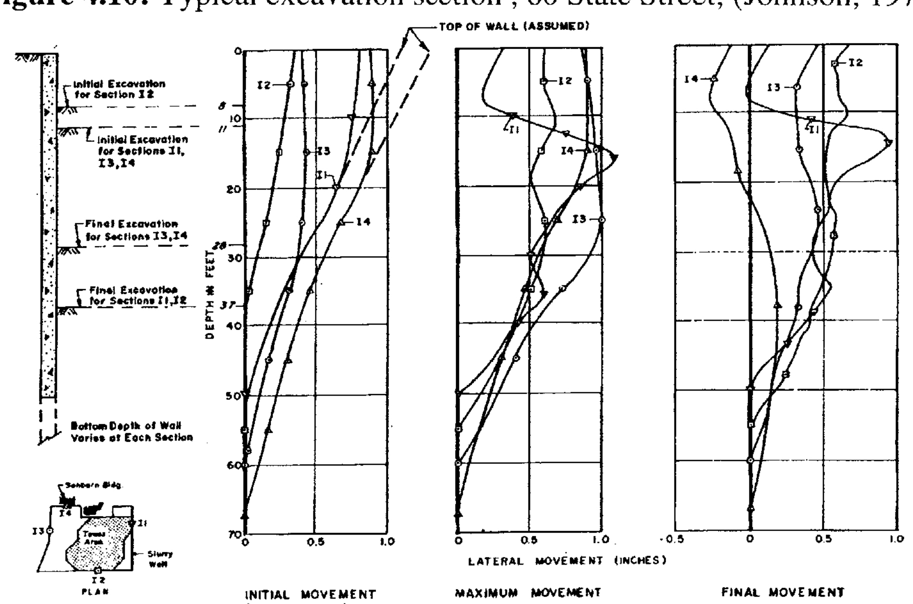


Figure 4.11: Initial, maximum, and final horizontal movements for the 60 State Street Excavation (Johnson, 1976).

4.5.3 Case Study B3, State Transportation Building (Park Plaza)

The Transportation Building is a nine-floor structure located at the corner of Tremont and Stuart Streets in Boston MA. Perimeter slurry walls 2.0ft thick formed the basement walls of the two levels underground parking. Tremont and Stuart Streets bound the eastern and southern sides of the project. Buildings adjacent to the north slurry wall are on shallow foundations and thus settlement control was important due to their proximity to the excavation. A plan of the site can be seen in Figure 4.12.

Soil profiles in the site are variable as Figure 4.14 shows. The soil profile near I-83 can be described by Profile A, while near I-24 it resembles mostly Profile B (Johnson, 1989).

The diaphragm wall was for this project had panels 20' long, 2' wide, and 45' deep. The panels were trenched starting about 8' below the street elevation (El. 25ft), because of pre-existing foundation removal (Fig. 4.14). In addition to earth retention, the slurry walls served as a water seepage cut-off. Eight deep sumps within the excavation perimeter were used to dewater the site during construction.

Two levels of temporary tiebacks were used to support the slurry walls. Typical tieback lock-off loads varied from 50 kips to 90 kips as indicated by load cells (compared to design load of 72 kips). Figure 4.15 shows that measured tieback loads remained fairly constant as the excavation progressed. Corner braces were used for additional support in the eastern and northwestern project corners. The superstructure walls were constructed on top of the slurry walls.

Figure 4.13 shows the contours of settlements at the end of excavation. Building settlements were generally small to insignificant on the northern excavation side. Points in the northwestern corner settled a little less than 0.4" whereas most other points in that area settled less than 0.2". Reference points in the southern side showed the largest settlements during the excavation progression. Near inclinometer I-70 a Borros anchor reference point settled a little more than 1.0". The increase in settlement in that area coincided with the maximum movements (towards the excavation 0.5") recorded by I-70.

Wall movements were generally small throughout the project site. Figure 4.14 compares records from inclinometers I-83 (south wall) and I-24 (north wall). I-24 bulged towards the excavation by δ_W=0.9", while the toe of the wall translated by 0.4" towards the excavation. Most panels on the northern side of the project deflected less than 0.6".

Corner braces provided additional support at the southeastern project corner . In the southern slurry wall, inclinometer I-83, was embedded in the clay. The wall at I-83 moved inwards by approximately δ_W=1.2" with δ_W=0.5" at the toe at the end of excavation (after mat installation). The final deflections were much smaller than then maximum deflections (at the end of excavation). The measured deflections at I-83 decreased when the deflections at the east wall increased and vise versa. Thus the corner braces transferred movements across the walls they were bracing. Apparently, stiffening of the local corner bracing was effective in reducing wall movements in I-83. Measured settlements in the vicinity of panel I-83 were smaller than 0.2", despite the fact that panel 83 had the largest wall deflections in the project.

A gas main located near panel 75 (Fig. 4.13: P75) showed settlement up to 1.3" (0.2" of which occurred during the slurry wall installation). However, it is not clear if these settlements should be considered as representative of surface settlements because steel hangers supported the gas main.

During slurry wall trenching problems arose with the guide walls since in many instances guide walls pieces cracked and fell within the excavated trench. A slope failure took place during trenching of a panel (panel 75) without slurry (Fig. 4.16. The slurry wall contractor had some difficulty maintaining the required slurry levels during that period in time. Slurry pockets were removed from completed panels in two instances. Minor leaks through the panel jointing were observed and they were easily patched with concrete. The slurry wall finish required occasionally that the slurry wall contractor return and cut excess concrete pieces. Leaks were also recorded in some tieback sleeve holes. At the final stages of the project (near total completion) there were some cracks recorded in the upper garage floor, probably caused by stress transfer from the slurry walls.

Water levels outside the excavation were not significantly affected either by the excavation or by the dewatering process. Overall, the slurry wall earth support system performance was satisfactory. Settlement control was successful in crucial areas and wall movements were typically small to moderate.

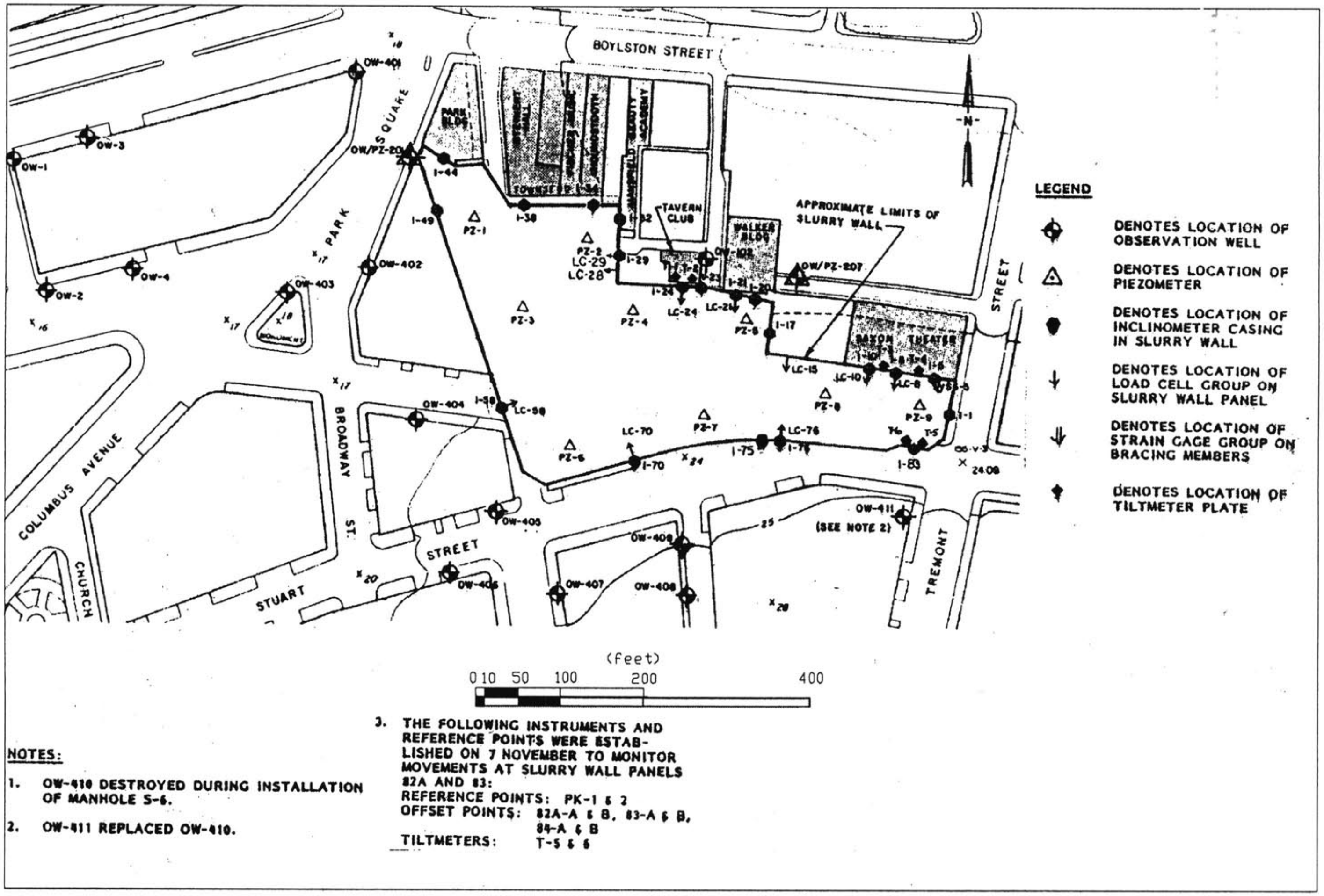


Figure 4.12: State Transportation (B3) site plan.

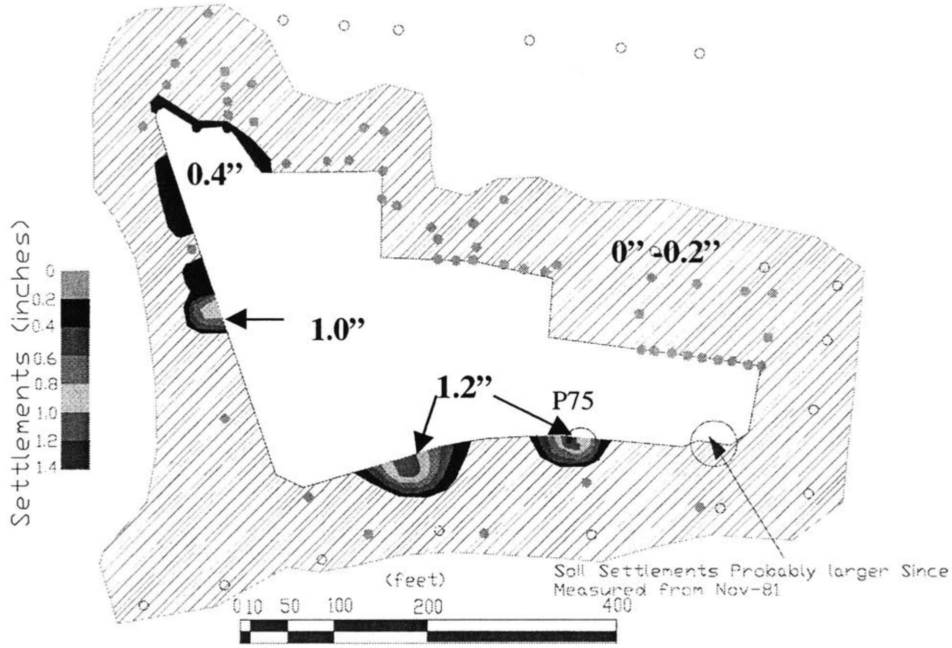


Figure 4.13: Settlement contours, State Transportation Building (B3)

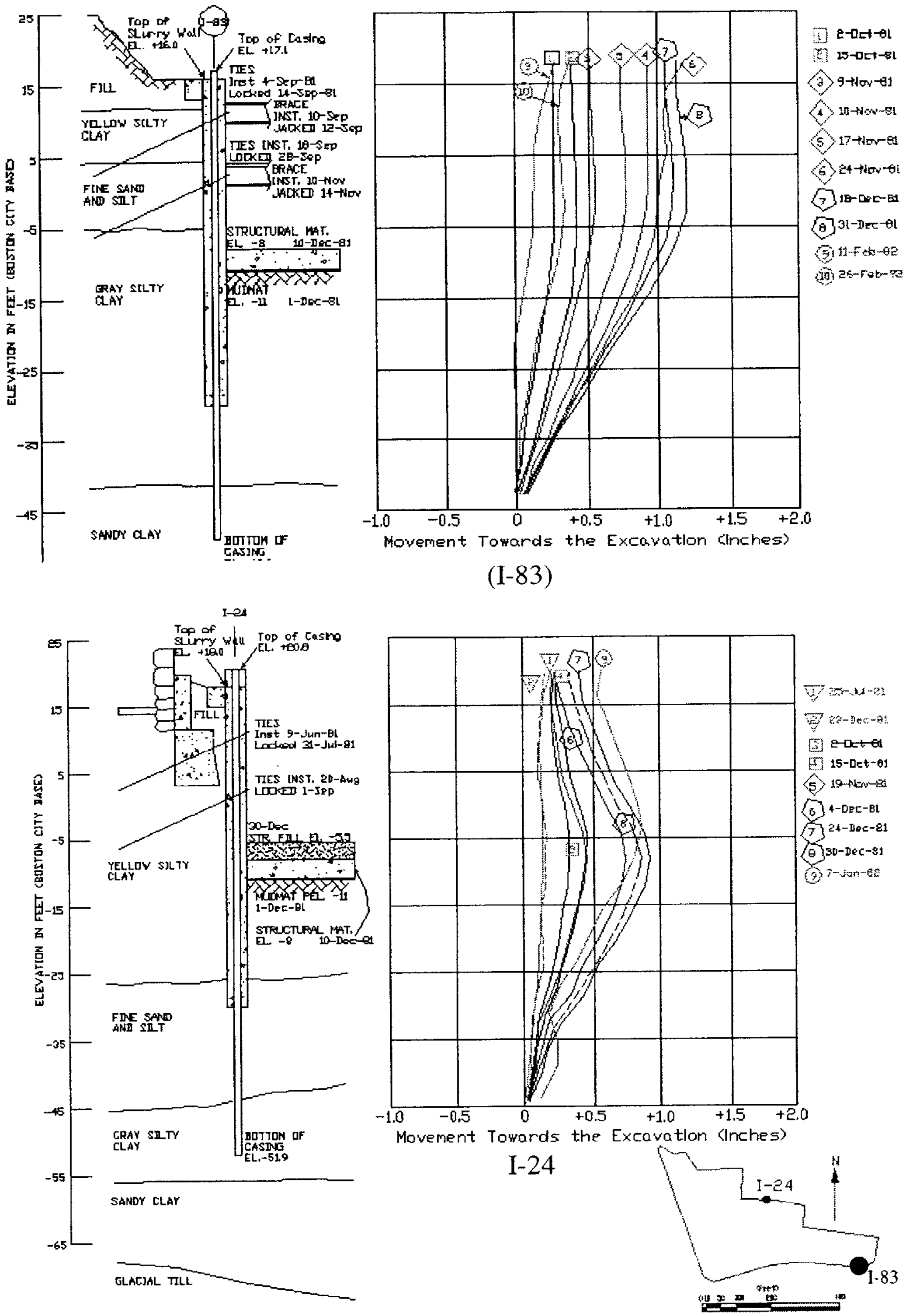


Figure 4.14: Deflections at inclinometers I-83 (Upper) and I-24 (Lower), State Transportation (B3).

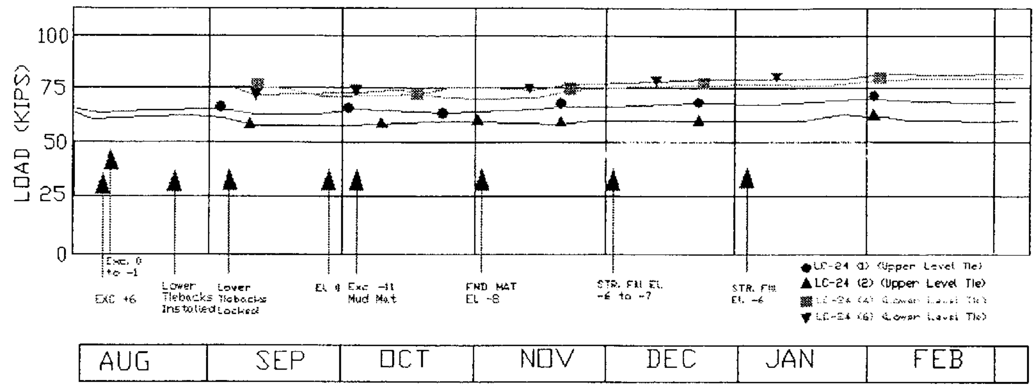


Figure 4.15: Tieback loads at panel 24 as determined by load cells (B3).

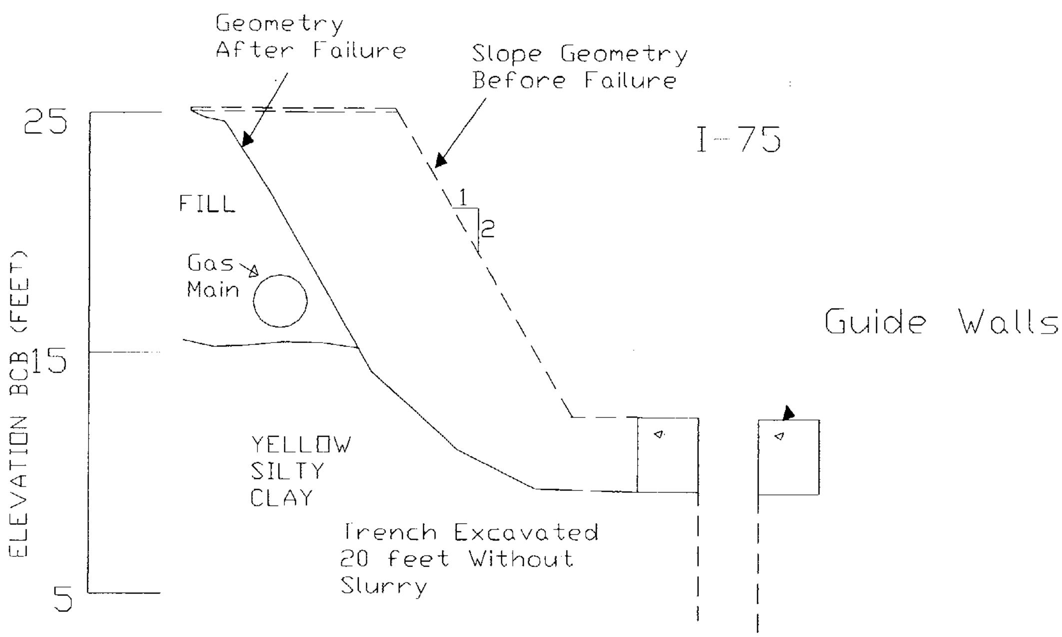


Figure 4.16: Slope failure at a panel trenched 20' without slurry.

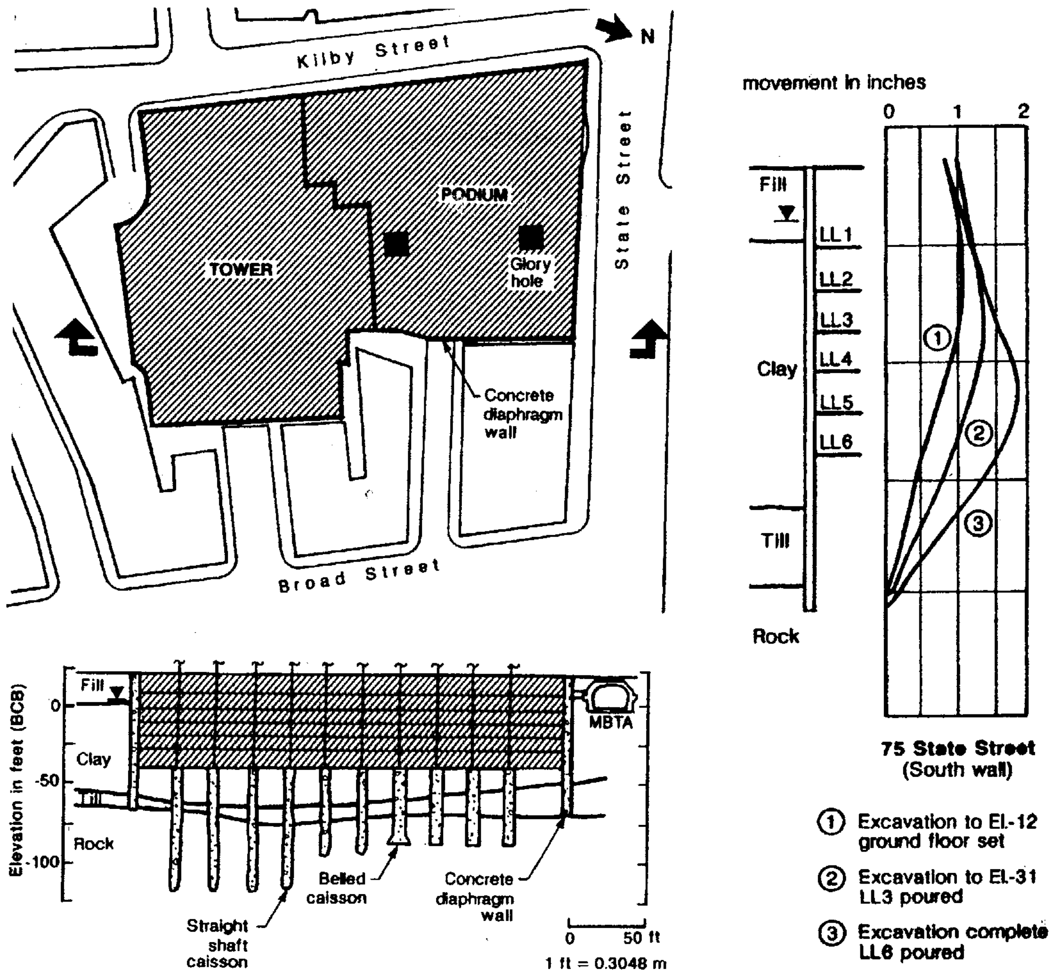
4.5.4. Case Study B4, 75 State Street

The 75 State Street building is a 31-story office tower with six levels of below-grade space utilized primarily for parking. Permanent and lateral earth support was provided by 2.5'-thick, 85'-deep diaphragm wall constructed with the slurry trench method. The 65-ft deep excavation was constructed by the top/down method, braced by 5 floor levels. The diaphragm wall extends 20' below the final excavation grade into the underlying till. The plan and a typical cross section of the excavation can be seen in Figure 4.17.

Soil conditions at the structure site are typical of the Boston area located outside the old colonial shoreline. The overburden soils consist primarily of fill, Boston Blue Clay, glacial till, and highly altered bedrock 70' to 100' below the surface. The upper 10ft to 20 ft top of the clay is an overconsolidated crust, (OCR >2). The overconsolidation ratio decreases rapidly with increasing depth as the clay becomes almost normally consolidated (OCR= 1 to 1.1).

The bedrock at the site belongs to the Cambridge Argillite formation and it is highly altered, containing zones of clay-like kaolinized material. Thickness of these clay-like "soil" zones may vary from a few inches to hundreds of feet (Humphrey & Soydemir, 1991).

The largest movements took place where the clay was the thickest and a two level high mechanical room existed (Fig. 4.17). Adjacent building within 5' to 20' from the excavation settled from 1" to 4". However, 50% of this movement occurred prior to the start of excavation and can be attributed to earlier pile extraction and caisson installation. Horizontal movements were within the predicted range for an excavation of this size. However, settlements were significantly larger than predicted (Fig. 4.18).



(a) (b)

Figure 4.17: 75-State Street project (B4), a) site and cross-section, b) wall deflections (after Becker & Haley, 1990).

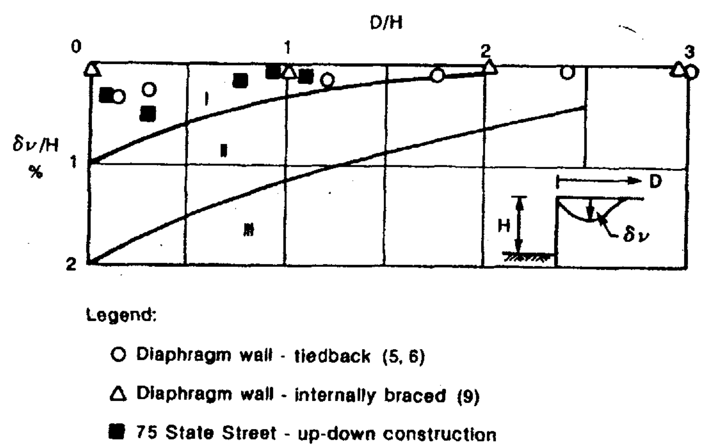


Figure 4.18: Settlement next to the excavation as a function of distance from the edge of the excavation, (Becker & Haley, 1990).

4.5.5 Case Study B5, Rowes Wharf

The Rowes Wharf building is located on the easterly flank of Fort Hill, one of the three original drumlins in Boston. The project consists of a main 15-story building along with three wharf buildings extending out over the water, and a 700-car five level underground parking. A 2.5' thick, 65' deep, with 10' embedment perimeter slurry wall. Rowes Wharf saw the introduction of Up/Down construction in the northeastern U.S. (Becker & Haley, 1990). Figure 4.19 shows the plan, a typical cross section, and measured wall deflections of the project.

In addition to providing excavation support the slurry walls were designed to reduce leakage and avoid excavation dewatering. A major issue that had to be addressed was that the uplift pressure was greater than the building weight. This problem was addressed by providing seepage cutoff with the diaphragm wall extending below the excavation level into the glacial till. This allowed for the lowest level floor to be designed as a fully relieved slab-on grade. Slurry wall panels were typically constructed in alternate 20' sections

According to Haley (1986), wall movements were small (Fig. 4.19). This project faced serious problems with seepage through panel construction joints and through the slurry wall itself. The slurry wall panel bottoms wall were not cleaned adequately before concreting, and as a result waste material had accumulated at the panel bottom. Concrete tremieing pushed part of this waste material to the top of the slurry wall but a lot of that material got entrapped between panel joints, and at the bottom corners of the slurry wall panels. These zones of soft material leaked excessively when they where exposed by the excavation activities. Major sealing efforts including grouting behind the wall and filling with hydraulic cement were undertaken.

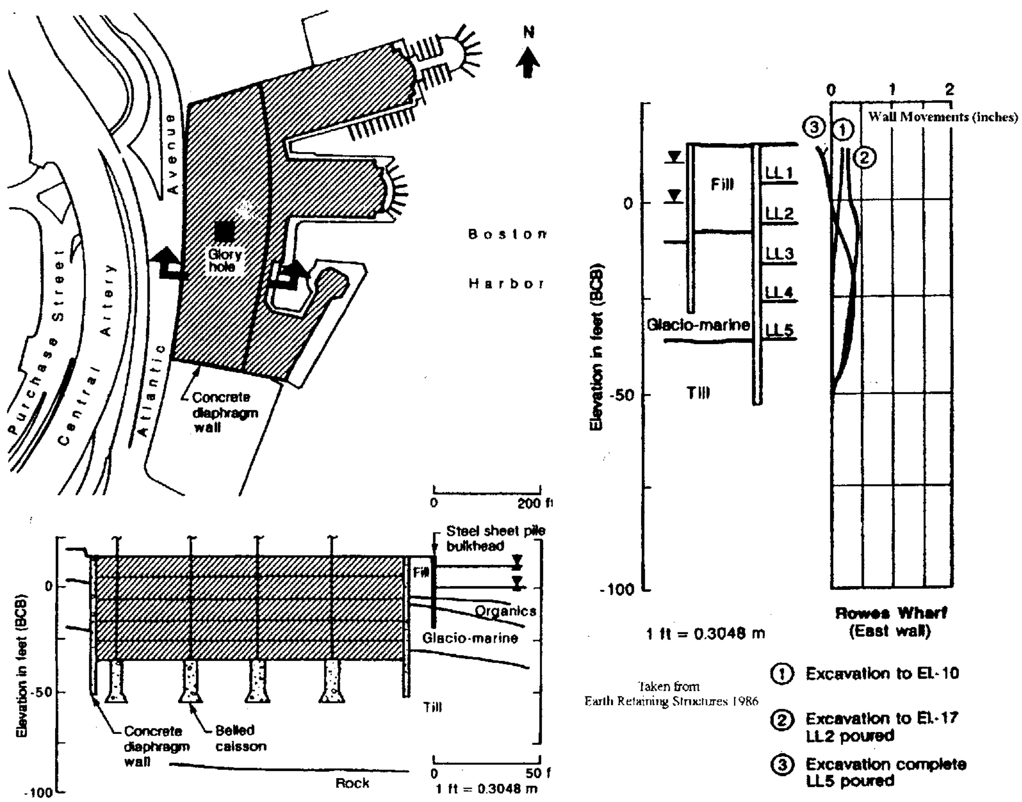


Figure 4.19: Rowes Wharf excavation (B5), plan, cross section and wall deflections (after Becker & Haley, 1990).

4.5.6 Case Study B6, One Memorial Drive

The project site is triangular in shape and is located between Memorial Drive and Main Street near Kendall Square in Cambridge. An asphalt lot separates the project from adjacent buildings on the west side. Approximately 35' from the north side is the MBTA Red Line subway tunnel. The project plan and a typical cross section can be seen in Figure 4.20. The ground surface is at approximately El. 21' CCD (City of Cambridge Datum). Three basement/parking levels (P-1B, P-2B, P-3B) extend beneath the entire building footprint

A 2'-thick perimeter slurry wall provided lateral earth support for the excavation of this project. Most of the slurry wall was constructed in 20'-long panels, and was supported by one level of tiebacks. The slurry walls were typically 36' deep with the toe embedment depth below the excavation base ranging from 11' to 16'.

The subsurface exploration for this project indicated the presence of six major strata namely: fill, organic silt, sand, BBC, glacial till and bedrock. Soil conditions are similar to those in Back Bay corresponding to a soil profile A according to Johnson [1989].

Two levels of tiebacks supported central panels on long faces. Tieback design loads ranged from 85 to 130 kips with average of 124 kips. The design line loads ranged from 10.90 kips/ft to 16.74 kips/ft with the latter being most common. Loads of around 50 kips were used for second level tiebacks. Estimated total tieback lengths averaged 40' to 45' with some tiebacks having total lengths of 35'. All tiebacks picked up load as the excavation progressed and wall movements increased, until equilibrium was reached (Fig. 4.24).

The overall performance of the lateral earth support system used in this project was satisfactory. Lateral movements of the slurry wall were moderate to insignificant (maximum 1.3", Fig. 4.21). Most of the wall movement occurred prior to tieback installation, when the wall cantilevered. After the excavation was completed an additional movement of up to 0.3" occurred when loads were transferred from the tiebacks to the base mat. Such movements should be expected since stress transfer from the tiebacks to the floor bracing does take place (Fig. 4.21).

Wall movements were noticeable despite the fact that the excavation was relatively shallow. The floating slurry wall of this project translated up to 0.15" near its base as offset surveys indicated (Fig. 4.21). Such translation at the toe of the slurry wall is to be expected when the wall is floating. One of the interesting aspects that offset surveys indicated was that wall movements can vary significantly even between adjacent panels (Fig. 4.22). For example, at one stage one panel moved back into the soil by as much as an adjacent panel moved towards the excavation.

Excavation for this project did not affect any adjacent structures. Figure 4.23 shows that settlements increased as the excavation progressed deeper and reached δ_V=1.05" near the excavation, while building settlements further away were practically insignificant.

One panel did cave-in during trenching, due to fill "sliding" into the trench under the guide walls. This was probably caused by the difficulties in stabilizing the loose granular fill. Slurry wall water leakage was very small and temporary (i.e. detensioning and removal of tiebacks, covering of the sleeve hole).

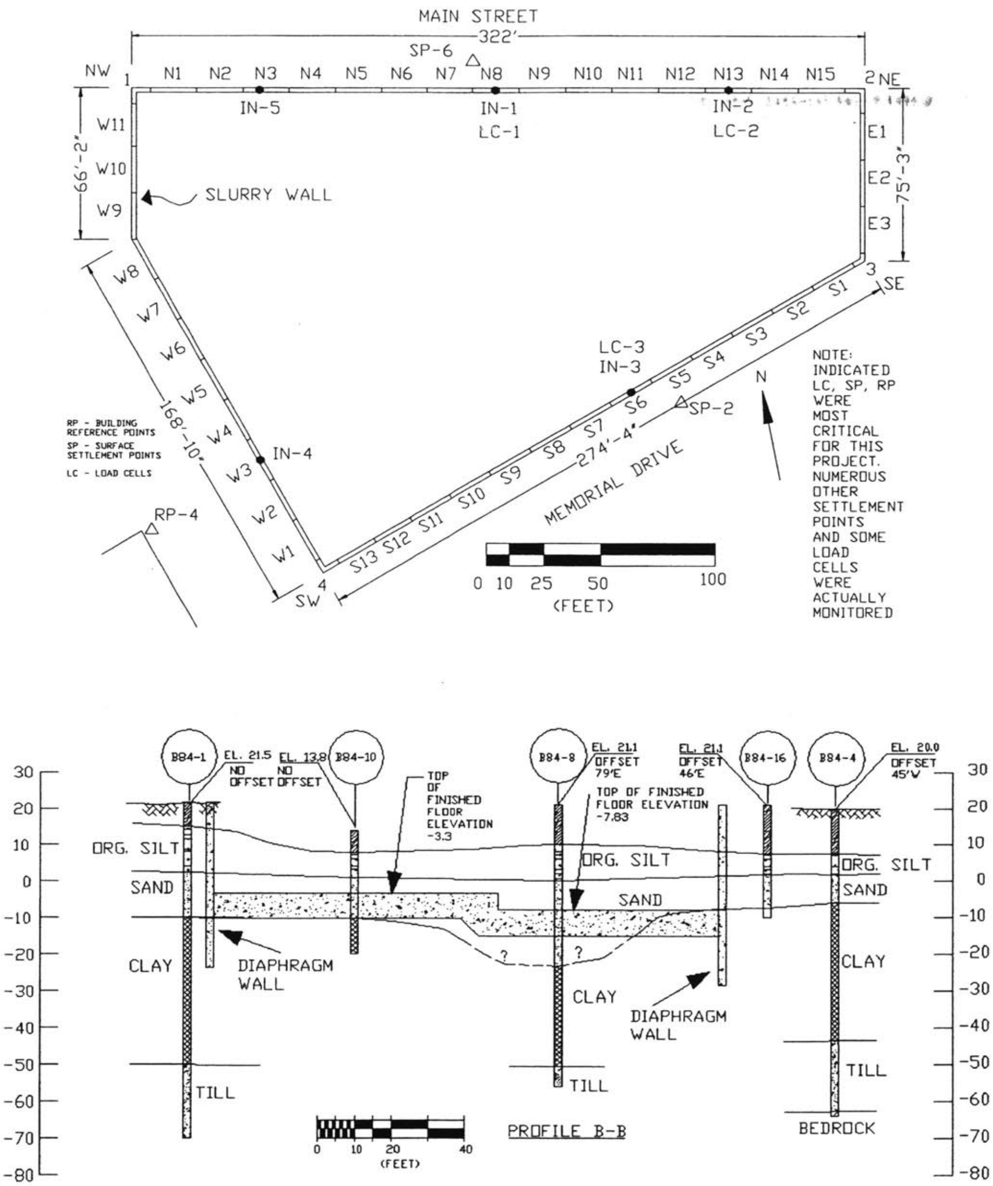


Figure 4.20: One Memorial Drive (B6) site plan & cross section.

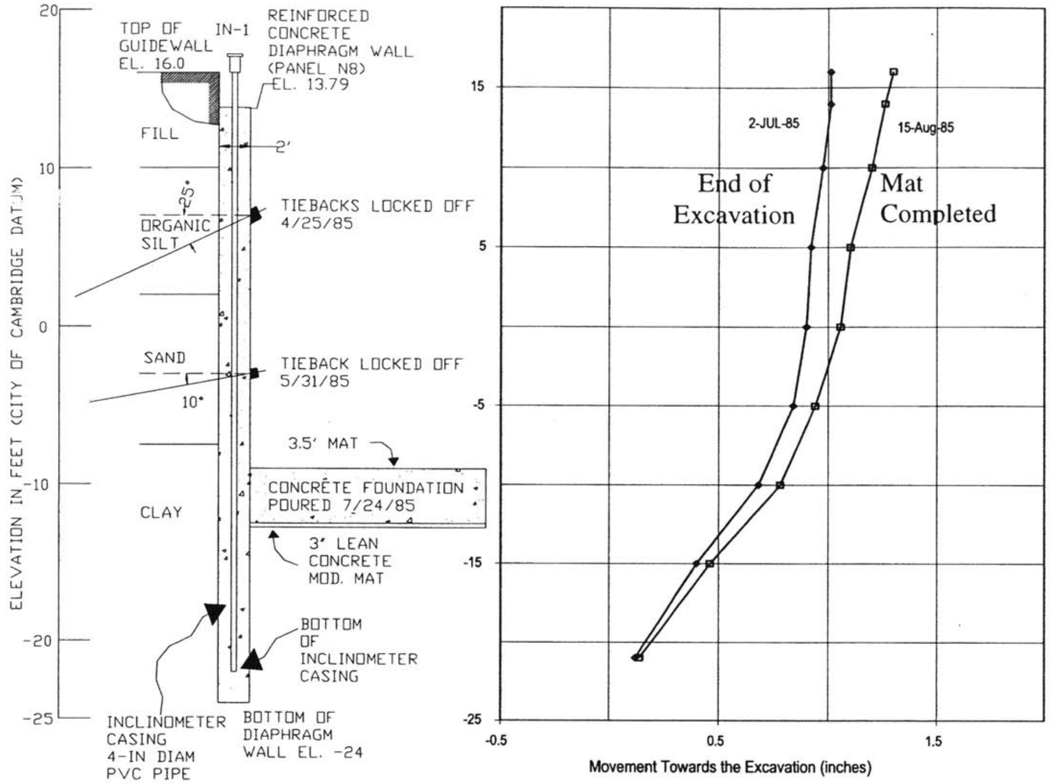


Figure 4.21: Deflections at In-1, One Memorial Drive (B6).

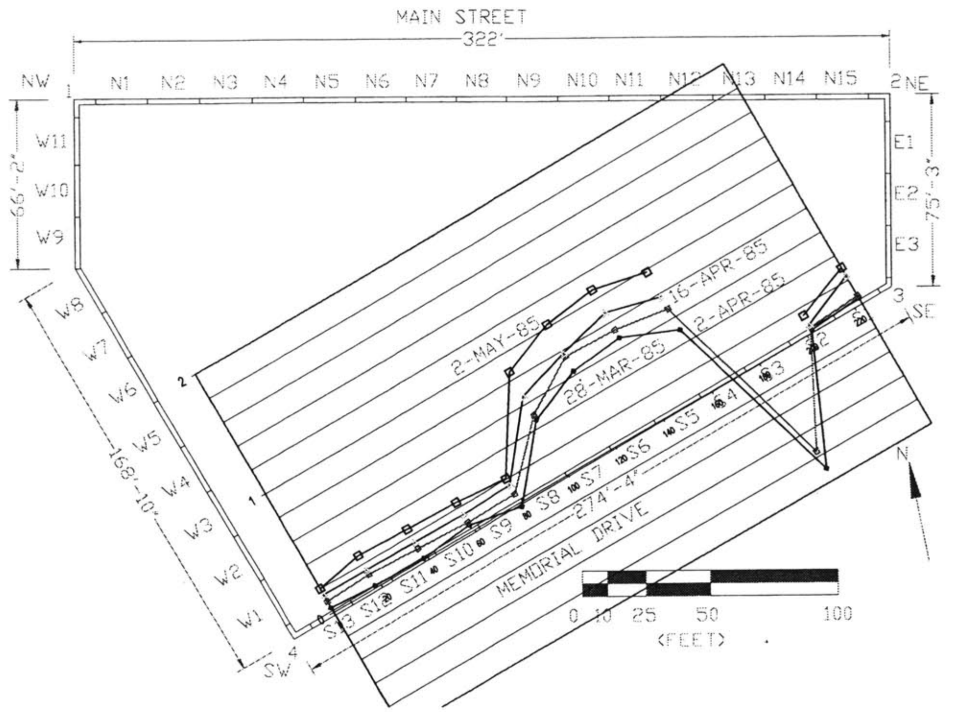


Figure 4.22: Wall deflections as determined by offset surveys, One Memorial Drive (B6).

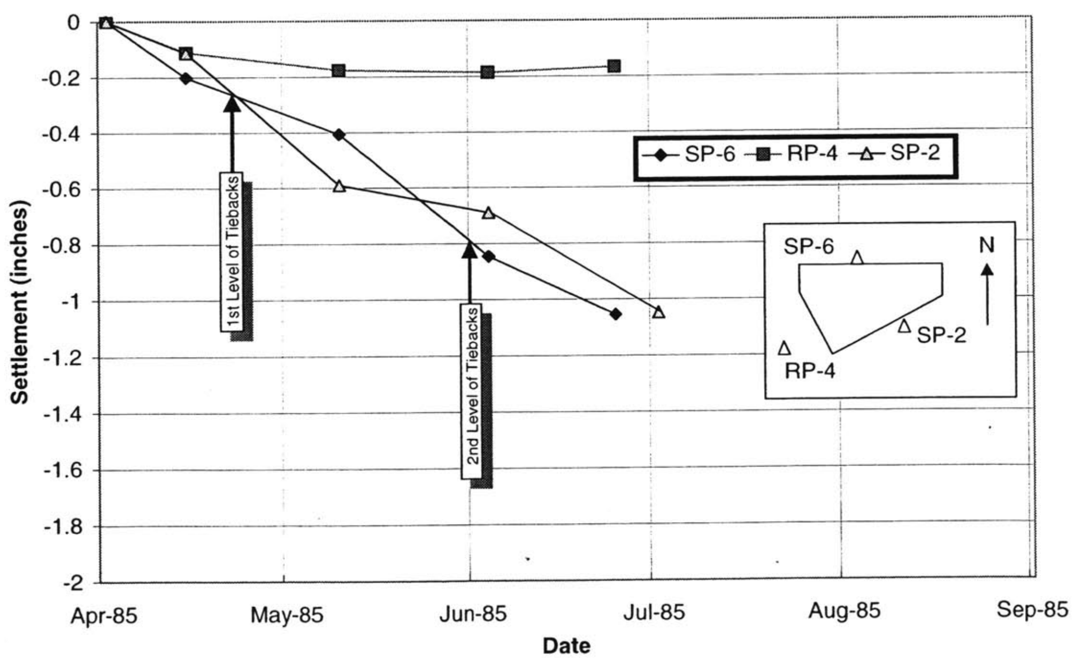


Figure 4.23: Surface & building Settlements, One Memorial Drive (B6)

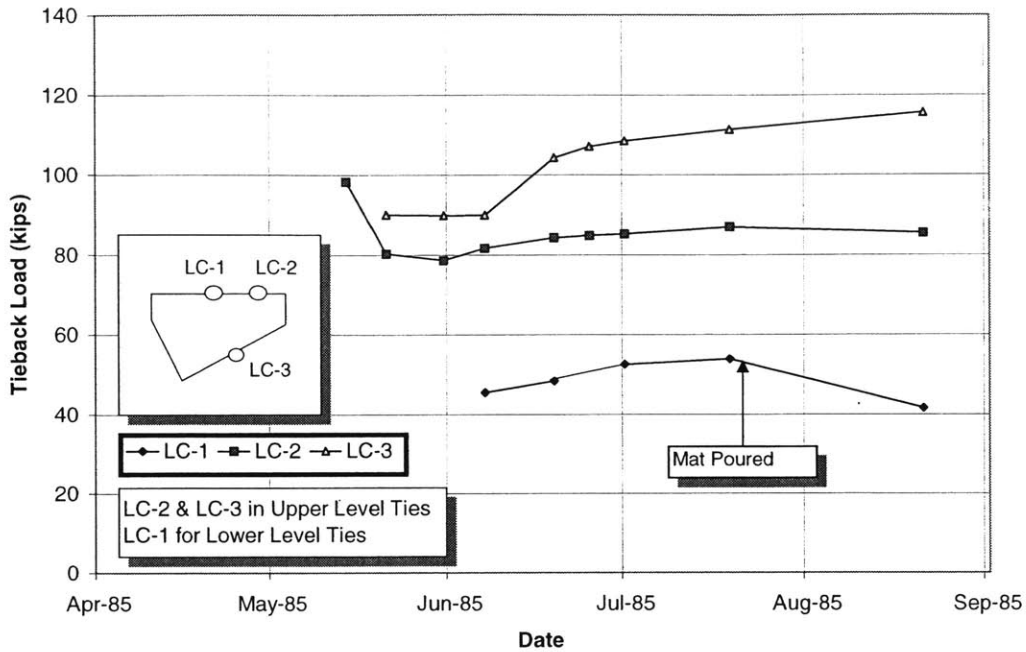


Figure 4.24: Tieback loads from load cells, One Memorial Drive (B6).

4.5.7 Case Study B7, 500 Boylston Street

The 500 Boylston Street building is located in the Back-Bay Boston area and includes two medium-rise towers, numerous low-rise structures and plaza areas. Boylston Street bounds the site to the north, with the MBTA Green Line subway running underneath. Clarendon Street bounds the site to the west, with historic Trinity Church opposite of the site. St. James Street bounds the southern side of the project. Subsurface conditions at the site are typical of the Back Bay area in Boston (Johnson, 1989).

The 2.0'-thick slurry walls were typically 40' to 45' deep with a final toe embedment, 15' below the base of excavation (BOE). Four levels of tiebacks provided temporary lateral support along the western wall. Tieback inclinations of 12°, 5°, 30°, and 30° were used for the 1^{st}, 2^{nd}, 3^{rd}, and 4^{th} levels respectively. Floor bracing provided permanent lateral support. Tiebacks were spaced at 5 ft intervals horizontally, and a minimum of 10 ft vertically. Numerous additional tiebacks were used due to the poor performance of the original tiebacks so that the final tieback spacing was even smaller. Rakers were used together with tiebacks along St. James Street for the same reasons. Figure 4.25 shows the project site and a typical north-south excavation profile.

In this project, settlement control was very important along Clarendon and Boylston Streets. Lateral wall movements were the greatest along Clarendon Street (Fig. 4.26: I-10) where there was significant lateral soil straining as well near the Church (Fig. 4.27: I-2). The maximum horizontal slurry wall deflection was δ_W=3.3" along Clarendon (I-10) whereas other walls deflected to a much smaller extend. Inclinometer I-2 in front of Trinity Church and opposite of I-10 across Clarendon had a maximum deflection of close to δ_H=2". Ties were not successful in reducing wall movements since their tensioning was not able to reduce inward wall deflections. Wall movements were smaller along St. James Street, where rakers were used in place of the three lower levels of tiebacks. Walls translated up to 1" at their bases even though they all extended into stiff overconsolidated clay. Such translation movements should be expected under such conditions where the wall is floating and soft base material is present.

Excessive surface settlements along Clarendon and St. James Streets were observed during the excavation. Maximum settlement was δ_V = 5.2" along Clarendon Street and 3" along St. James whereas the Boylston Street settlements were insignificant. Settlements along St. James are puzzling since rakers were used in place of the three lower levels of tiebacks. Possibly the groundwater lowering below minimum levels that went on for three or more weeks might be responsible for these settlements.

Figure 4.28 shows that wall deflections were smaller than surface settlements along Clarendon Street (same was true for other project sides). This indicates that there were possible soil losses through the tiebacks or soil disturbance caused by the tieback installation procedure. Tieback load deficiencies are also to blame for the excessive movements. Major parts of the effective length of tiebacks were in the organic silt, resulting in load shedding and excessive creep. Furthermore when additional tiebacks were installed adjacent previously locked tiebacks could not sustain their initial lock-off loads. Wooden pile extraction in the preliminary excavation construction phase may be responsible for part of the induced movements. Building settlements were small, with Trinity Church settling up to δ_V=0.62". The excavation did not affect other buildings, supported by pile foundations.

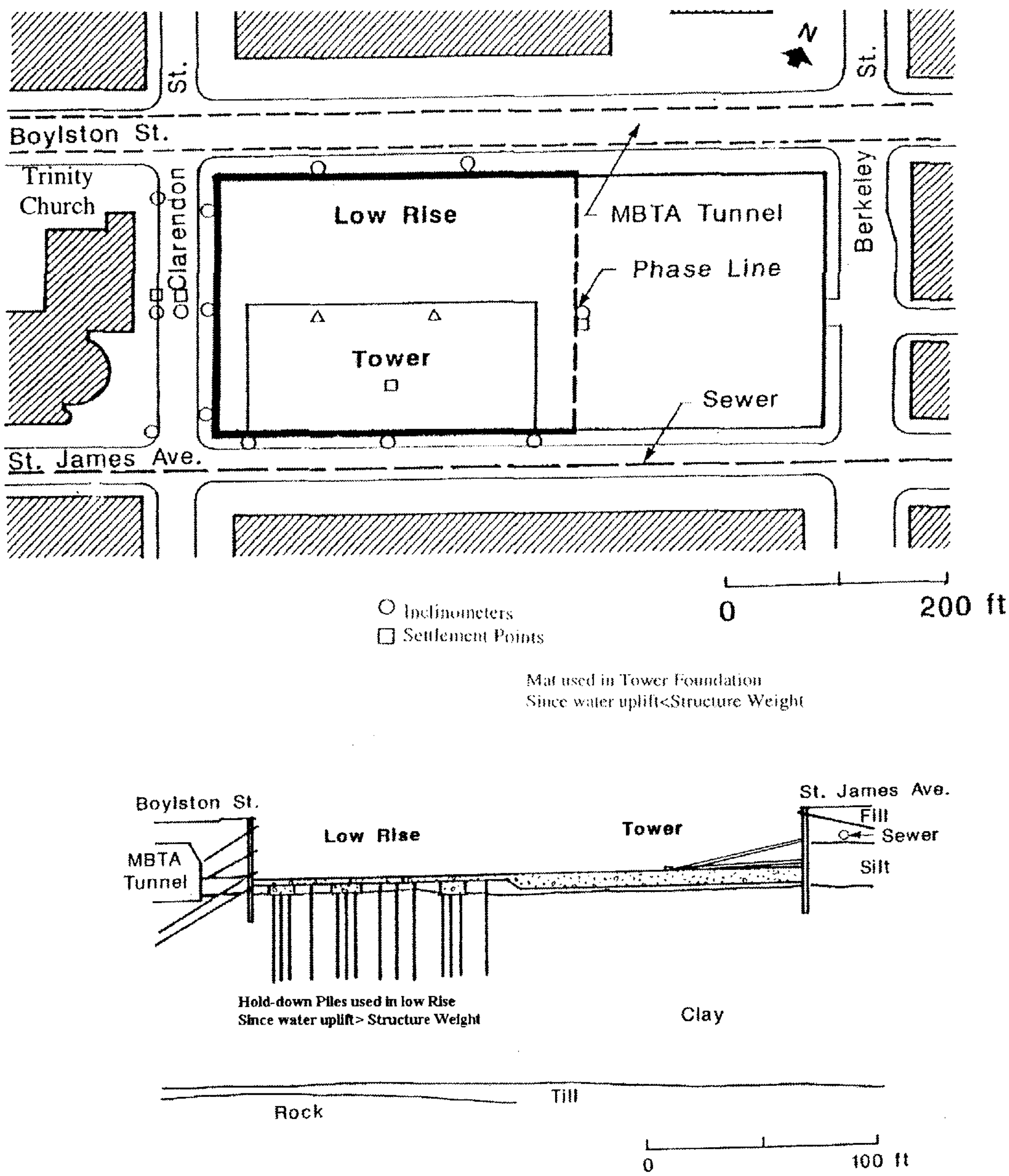


Figure 4.25: 500 Boylston Street project (B7), site and cross-section.

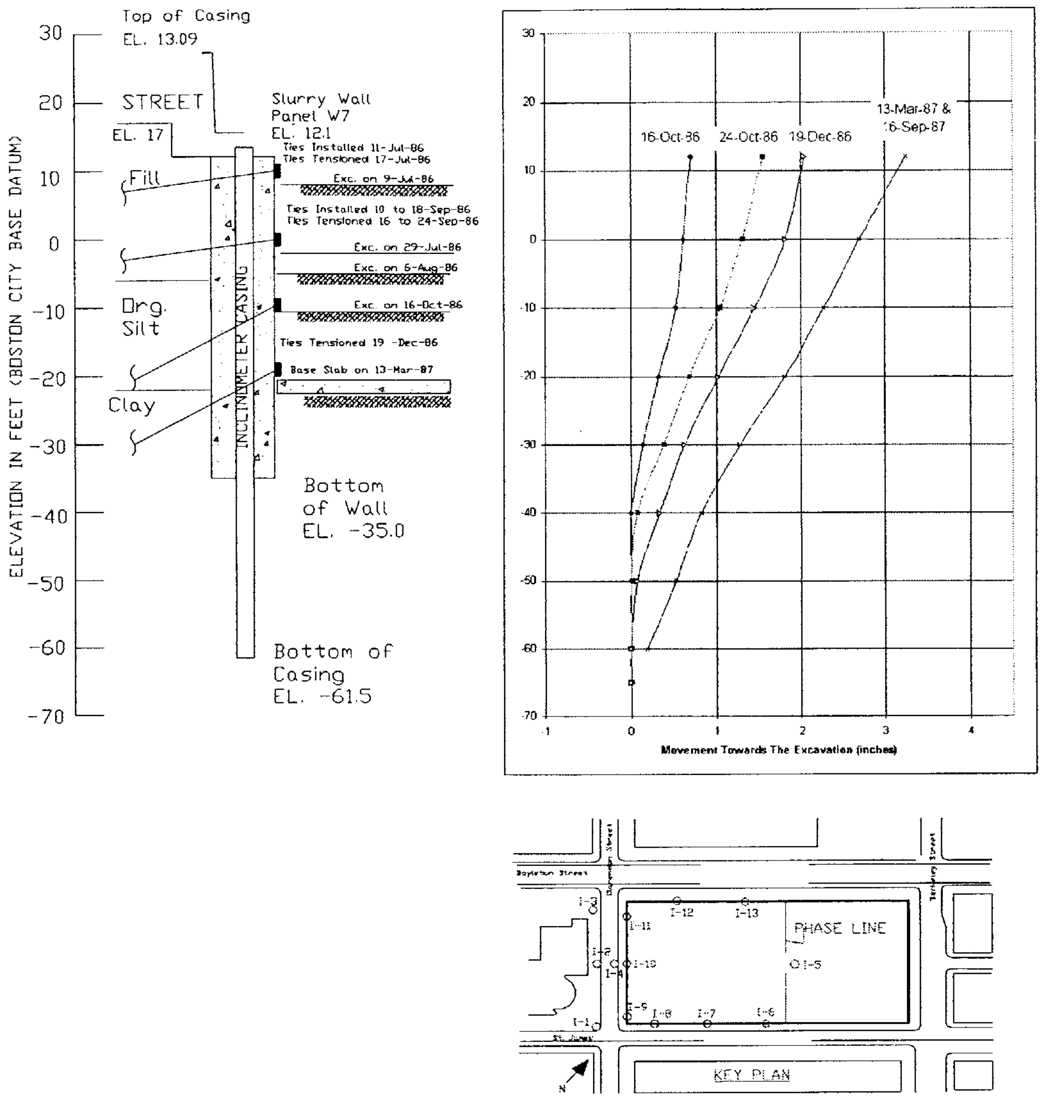


Figure 4.26: Wall deflections along western slurry wall (I-10), 500 Boylston Street (B7).

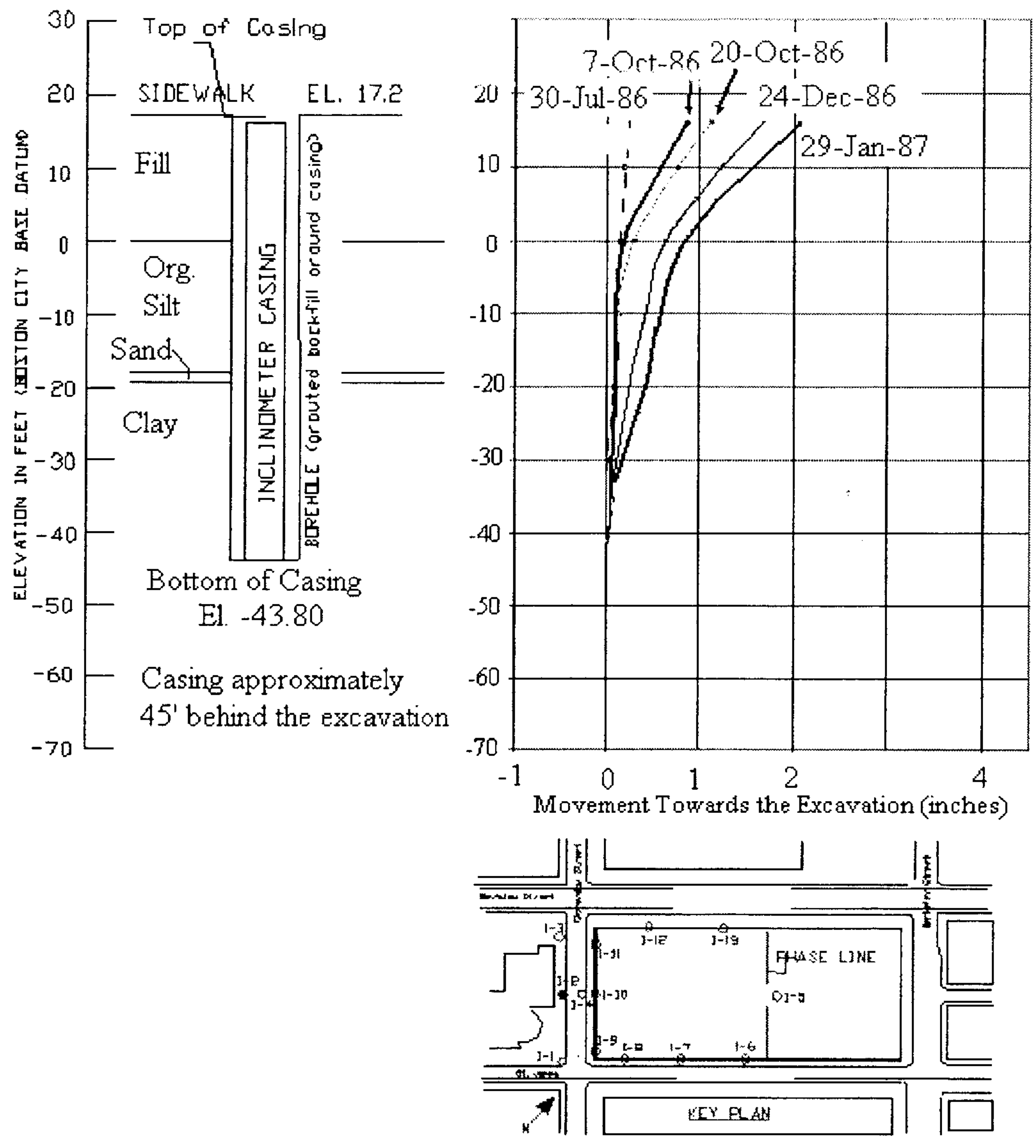


Figure 4.27: Horizontal soil deformations in front of Trinity Church, I-2, 500 Boylston Street (B7).

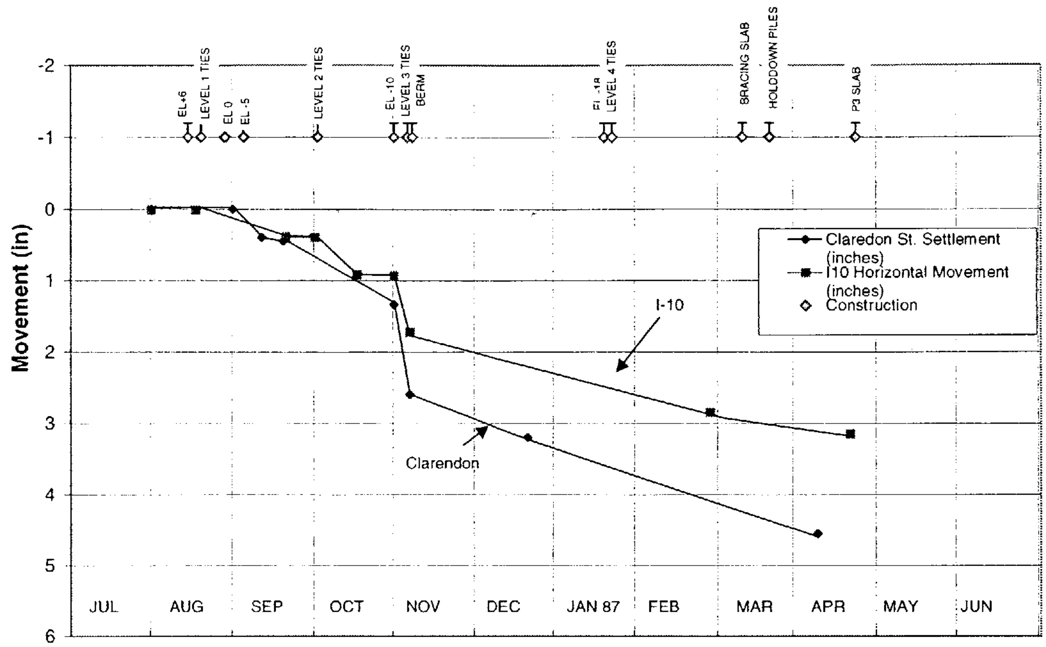


Figure 4.28: Maximum settlements and wall deflections with time and construction events along Clarendon Street, 500 Boylston Street (B7)

4.5.8 Case Study B8, Flagship Wharf

The Flagship Wharf project was an addition to a pre-existing building in Charlestown, and was constructed in land reclaimed from the sea. It consists of two eleven story residential/retail towers and a low-rise (3 story) plaza/retail terrace structure built above a four level below-grade parking. The plan area of the excavation for this project is 255 ft. (E-W) by 107 ft. (N-S), (Fig. 4.29).

A 60-ft deep, 2.5-ft thick slurry wall provided temporary and permanent lateral earth support for the excavation. The excavation is approximately 50-ft deep, and thus the slurry wall was embedded 10ft into glacial till (Fig. 4.29). The slurry wall was constructed using the patented "ICOS-FLEX" wall system. This wall differs from traditional slurry walls in that the wall is reinforced with post-tensioned, high strength strands, similar to those used for tieback anchors.

The soils found in the site and the surrounding area comprised of miscellaneous fill, organic deposits, glaciomarine, and glacial till deposits (Fig.

4.29). The stratigraphy can be classified as profile C according to Johnson [1989] (Fig. 4.3).

The concrete diaphragm wall serves as both the temporary excavation support wall and permanent foundation wall. The slurry wall was designed to resist temporary and permanent static lateral pressures from soil, water, and lateral surcharge loads from building #197 to the north, and other adjacent surface loadings.

Cross-lot bracing was used for the lateral support of the foundation walls of this project. Three levels of temporary prestressed (jacked) steel struts and corner braces supported the slurry wall. The struts were hollow cylinder steel pipes with a diameter ranging from 24" to 34" and 3/8" to 0.5" wall thickness. Steel HP sections were in the connections between struts supporting the north and south wall (Fig. 4.29). The horizontal forces jacked on the struts were transferred on the wall by means of walers. The arrangement of the struts can be seen in Figure 4.29. The schematic of the wale-HP steel section-strut arrangement can be seen in Figure 4.30.

Most wall movements at the Flagship Wharf project were moderate but along the northern slurry they were large reaching δ_W=2". The caissons of building #197 added significantly more surcharge on the northern slurry wall, and as a result, the southern wall was pushed back into the retained soil by as much as 1" at the top. Wall movements at opposite panels, braced by common struts, showed opposing trends (0). That is when a panel moved towards the excavation the opposite panel moved back into the soil. The concave bending seen in the slurry walls of this project is indicative of the cross-lot bracing construction sequence.

Settlement control was important in the north side of the project due to the proximity of building #197 that was supported by a series of caissons. Settlements up to δ_V=1.8" occurred along the section containing the caissons but the settlements were not transmitted to the exterior walls or the floors of the building (1). Wall movements and settlements tied very well together.

A cave-in occurred during trenching of a panel, at a small depth in the fill layer during excavation without slurry. Clearly if trenching was carried out under slurry this cave-in might have been avoided. The cave-in caused additional problems with the vertical construction of the trench. Construction of this panel took a longer time since numerous underground obstructions were encountered.

Figure 4.32 shows strut loads as measured from strain gages. A quick 0.5" movement at I-2 occurred as a result of a jacking box failure that resulted in the sharp load loss of the L3 strut during April 1989 (2). The structural factor of safety of the struts was only $F_s=1.2$ and as a result 50% of the total wall movement occurred after the excavation base was reached (indicating some creep movement in the bracing). The upper and lower level struts picked up only 70% of the design loads, that were in the order of 500 tons for the 3^{rd} level (2). The load at second level struts picked up as the excavation progressed to the third level (El –38ft).

More frequent readings indicated that daily variations in temperature caused expansion and contraction of the struts that showed up as variations in the strain gage readings.

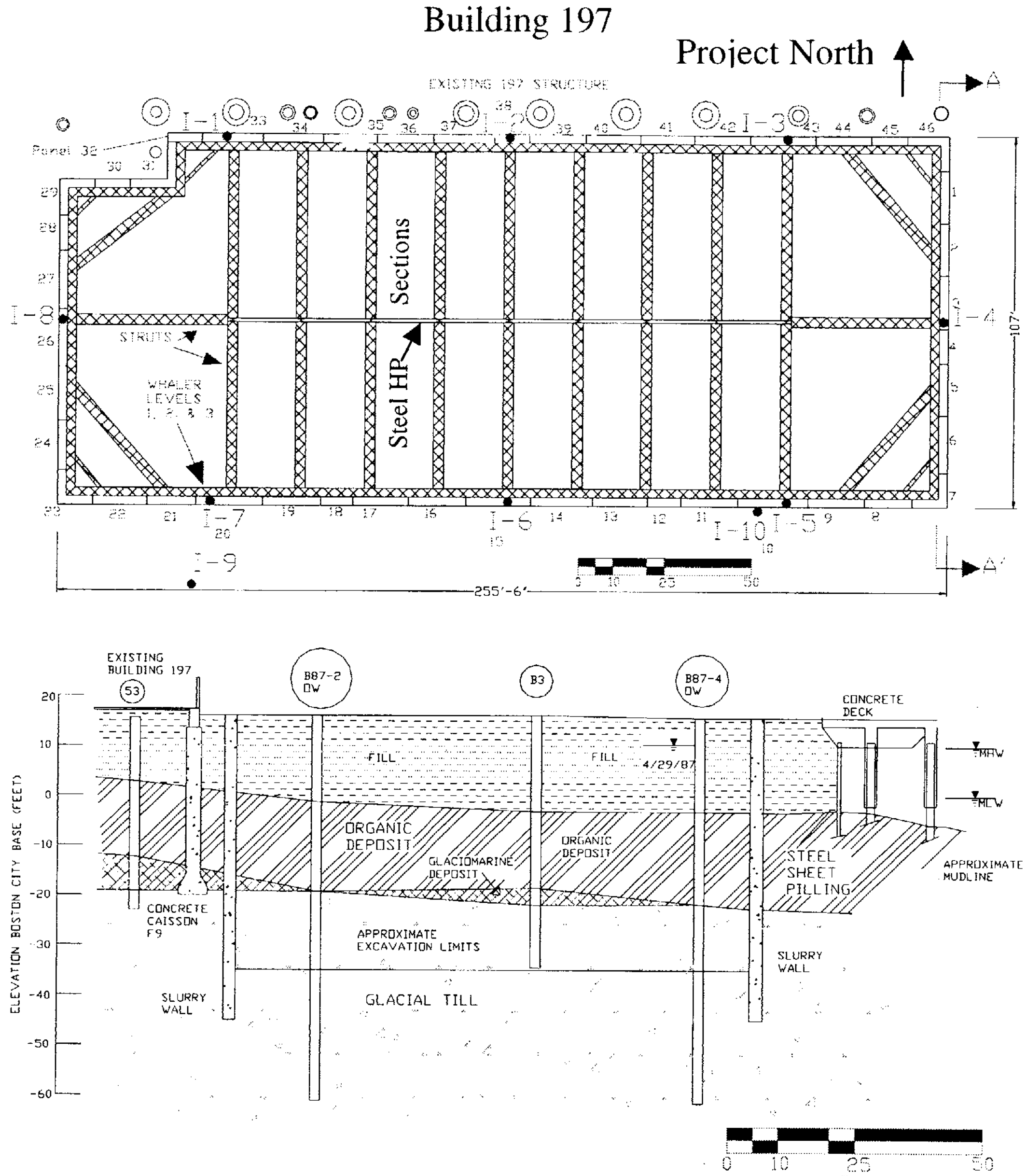


Figure 4.29: Flagship Wharf (B8), site plan and Cross Section A-A

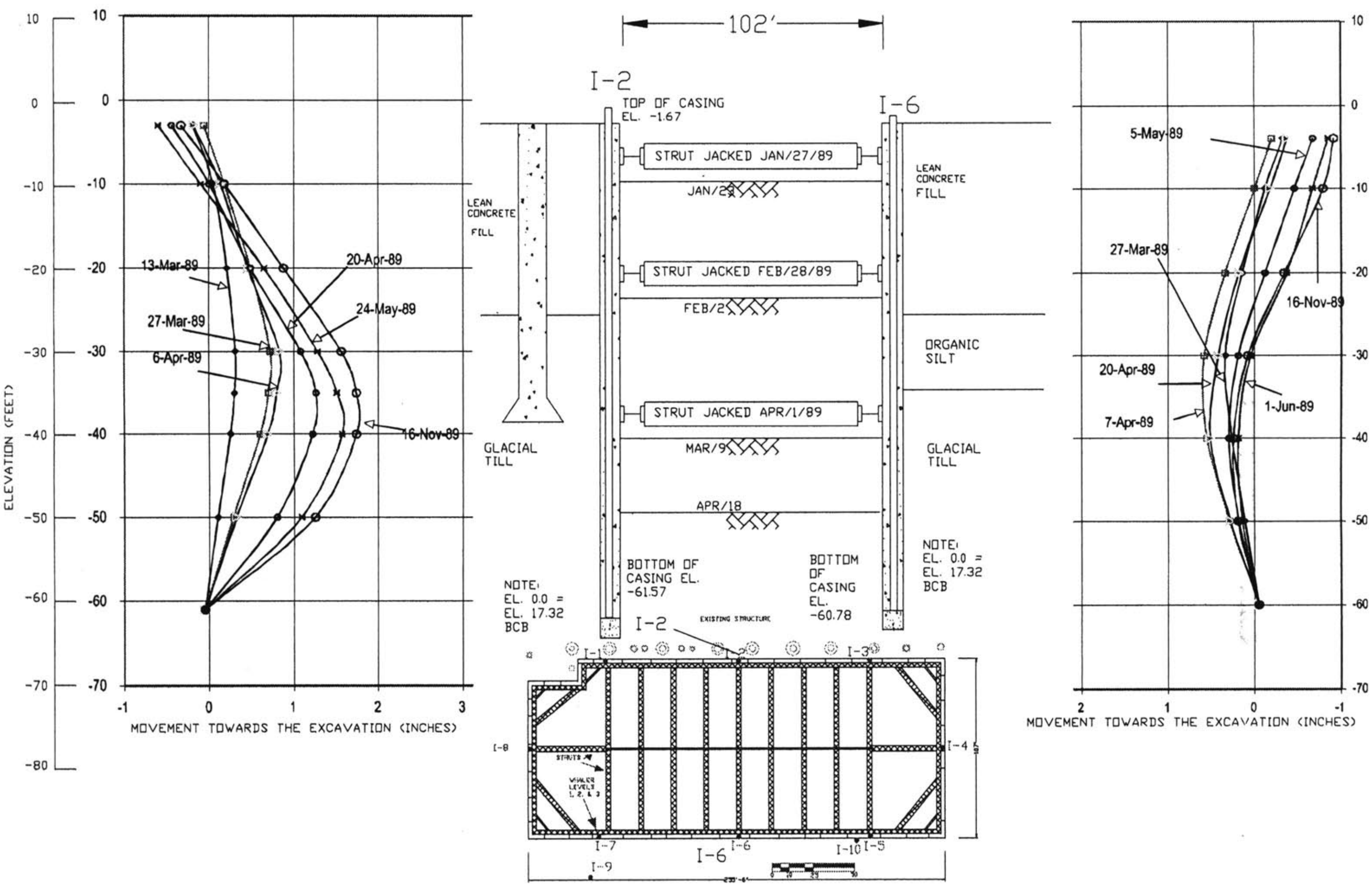


Figure 4.30: Inclinometer I-2 & I-6 Wall Movements, Flagship Wharf (B8)

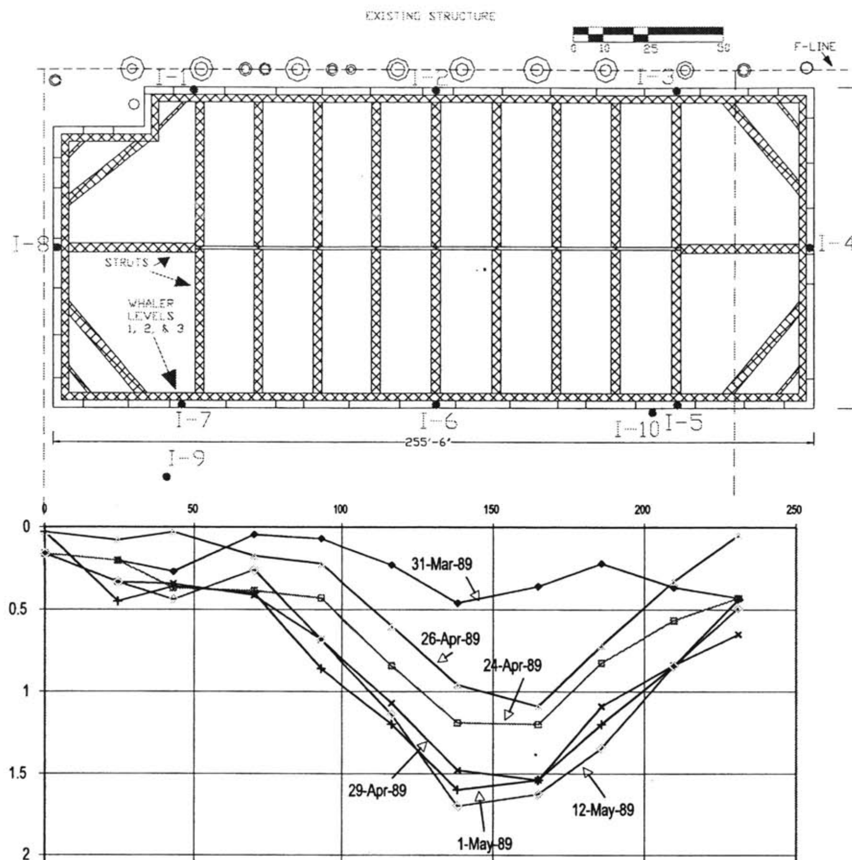


Figure 4.31: Settlements along F-Line (inches), Flagship Wharf (B8)

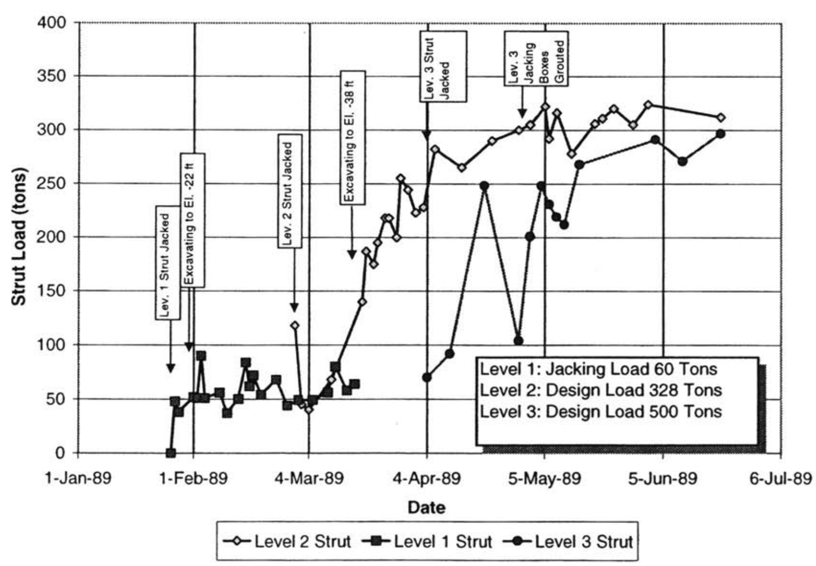


Figure 4.32: Measured strut loads at Flagship Wharf project (B8).

4.5.9 Project B9, 125 Summer Street

Performance data for this project was obtained for Becker & Haley [1989], which are shown in Table 4.5. No further was recovered for this project and thus extensive conclusions can not made. Non the less, the reported deformations induced by the 6't-deep top/down excavation were very small.

Table 4.5: Performance data for the 125 Summer Street (B9), after Becker & Haley [1989]

ID	Year	Project Name	Soil Type*	Excavation Bracing/ Excavation Method	Exc. Depth (ft)	Slurry Wall Thick. (inches)	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Defl. Shape Type
B9	1990	125 Summer St	A	6-Lev Top/Down	60	30	0.6	0.38	

4.5.10 Case Study B10, Post Office Square Garage

The Post Office Square Garage includes a 7-level, 75'-deep underground garage, constructed with the top/down method, bounded by Franklin, Milk, Pearl, and Congress Streets. Existing buildings up to 40 stories tall are located adjacent to the site. The plan site can be seen in Figure 4.33.

A perimeter slurry wall, 3.0' thick, extended approximately 10ft into bedrock, provided both the temporary and permanent earth pressure support for the excavation. The diaphragm wall is internally braced by the garage floor slabs (LL1-LL7), which are supported by interior steel H-section columns founded on bedrock at depths 85' to 96' below the ground surface. The interior columns, installed using slurry trenching methods, were concreted into till and rock to form load bearing elements (LBE).

Most of the site is located within the old Boston peninsula; however, the northwestern section is located in an area reclaimed from the sea. Figure 4.34 shows two selected cross sections through the site reflecting the initial conditions (Internal Report H&A, 1987). Soils in the two sections (3: E-E, D-D) comprise 15' of fill, local pockets of organic silt, 30' to 40' thick clay (Boston Blue Clay, BBC), 0' to 16' of sand, 3' to 20' of glacial till, weathered bedrock, and sound bedrock. Boring logs in the Boston show significant variations in the thickness of the individual strata across the site, reflecting the complex glacial geology in Boston (Johnson, 1989).

Figure 4.35 plots the actual tremied concrete volumes for each constructed panel against the theoretical panel volume (theoretical panel volume = panel width x length x depth). Concrete overpours were small and averaged 9.2% of the theoretical panel volume. There was only one panel cave-in during trenching that was backfilled with concrete and re-excavated later. This panel collapse can easily be distinguished in Figure 4.35 since the required panel volume was much higher than the theoretical panel volume. Otherwise, cobbles and boulders in the glacial till and in the clay caused some difficulties in slurry wall construction.

Figure 4.36 shows the maximum measured wall deflections at various inclinometers as wells as the floor levels were the maximum wall deflections occurred. All the walls bulged towards the excavation by up to δ_W=2.2" at elevations in the clay (LL3 to LL4).

Figure 4.37 shows inclinometer deflections inclinometers IN-13 and IN-14 installed in the eastern wall and 34' behind the excavation along Pearl Street respectively. The wall at IN-14 deflected the most near the current excavation level until the LL3 or LL4 slab was completed (4.37). Thereafter, maximum wall deflections did not change significantly in magnitude and in position. Horizontal soil movements at IN-13 were smaller than wall movements at IN-14, reaching up to δ_H=0.75" at the top of the inclinometer. Deflections at IN-3 increased the most from the construction of the LL2 slab until the construction of the LL4 slab.

As the excavation progressed beneath LL3 and LL4 the upper part of the wall was slightly pushed back into the retained soil. At the same time, wall deflections beneath the LL3 level steadily increased. One possible mechanism could be that as the excavation progressed deeper the lower slabs pick up more of the total lateral earth load and thus the load that the upper slabs shared decreased. As a result, the upper slabs slightly expand and the top of the slurry wall is pushed back. Thermal expansion and creep deformations of the upper slabs could also have contributed to the deflection decrease at the top of the wall. When the base of excavation was reached, there was very little wall bending above the LL3 and beneath the LL5 slabs. The rock where the slurry walls were keyed appeared to act as a pin support.

Figure 4.38 shows building and surface settlements caused by the garage excavation. Building settlements were much smaller than surface settlements. The Meridien Hotel settled up to 0.65". Buildings supported by deep foundations were essentially unaffected by the excavation. Surface settlements on the other hand reached up to 2.75" within 20' from the excavation along Pearl Street, where wall movements were the largest. Again, a large settlement increase occurred by the time the LL1 slab was installed. Surface settlements further away from the excavation started increasing when the excavation progressed beneath the LL3 slab, as a result of deep soil movements. Figure 4.39 plots shows subsurface settlements measured within the BBC by multiple point extensometers at two construction stages (LL3 slab completed, and LL7 level reached). Subsurface settlements were much smaller than surface settlements (0.65" maximum, typically less than 0.4"). Figure 4.39 also indicates that subsurface settlements within the Boston Blue Clay decrease with increasing depth.

Figure 4.40 plots piezometric levels at two locations near the southeastern project corner. The excavation did not affect water levels in the fill, while slightly lowered the piezometric head in the clay by 10'. However, dewatering within the site and excavation beneath LL3, decreased the piezometric head in the rock at one location by 45'(Fig. 4.40).

The excavation did not affect adversely any adjacent structures. However, deformations were larger than expected because the backfill material that was used for the LBEs was softer than the surrounding soil. Thus wall deflections were larger along the eastern wall (2.0"), where the LBEs formed a parallel to the wall zone of softer material (Figs. 4.5.10.3, & 5). A major portion of the deformations occurred in the initial stages of the excavation, before the roof or the first level slab was installed. During these first stages, the lateral earth loads were transferred to the softer LBE backfill material, through out the wall depth, thus causing large cantilever movements. Movements were smaller in locations where the LBEs were further away from wall.

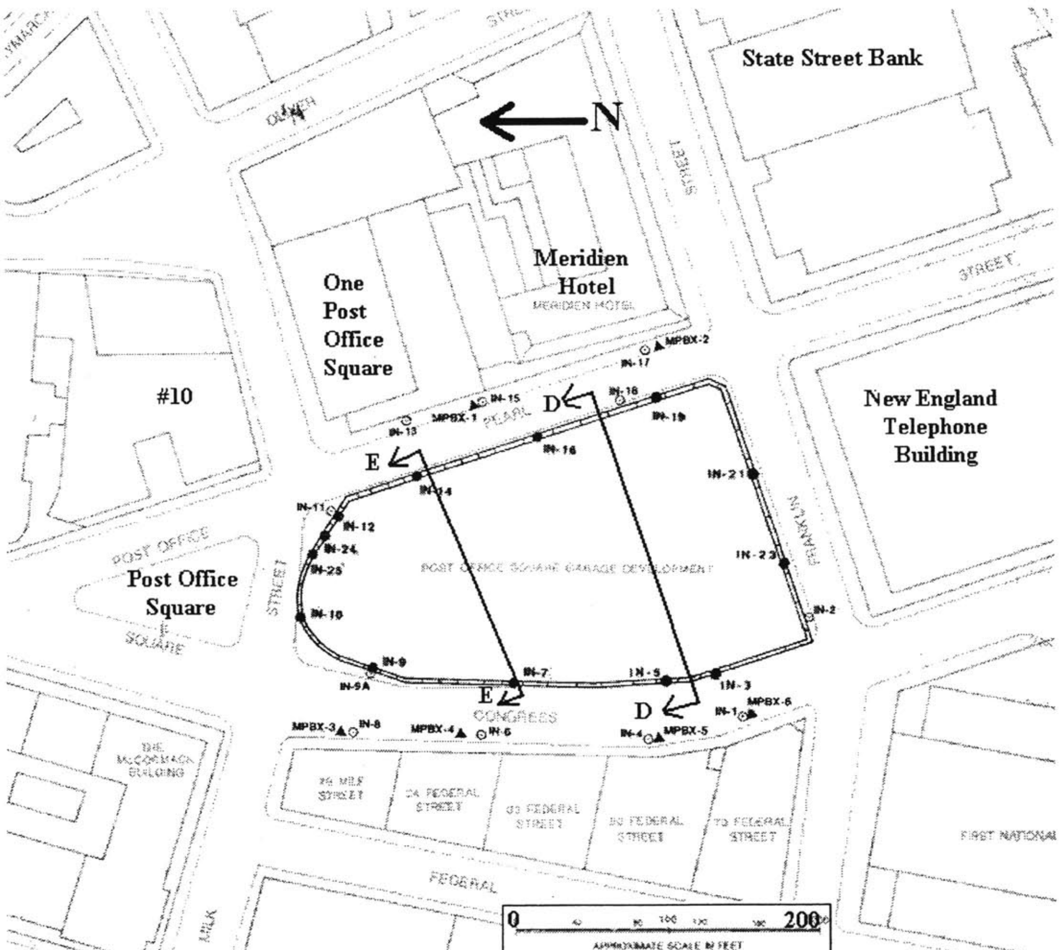
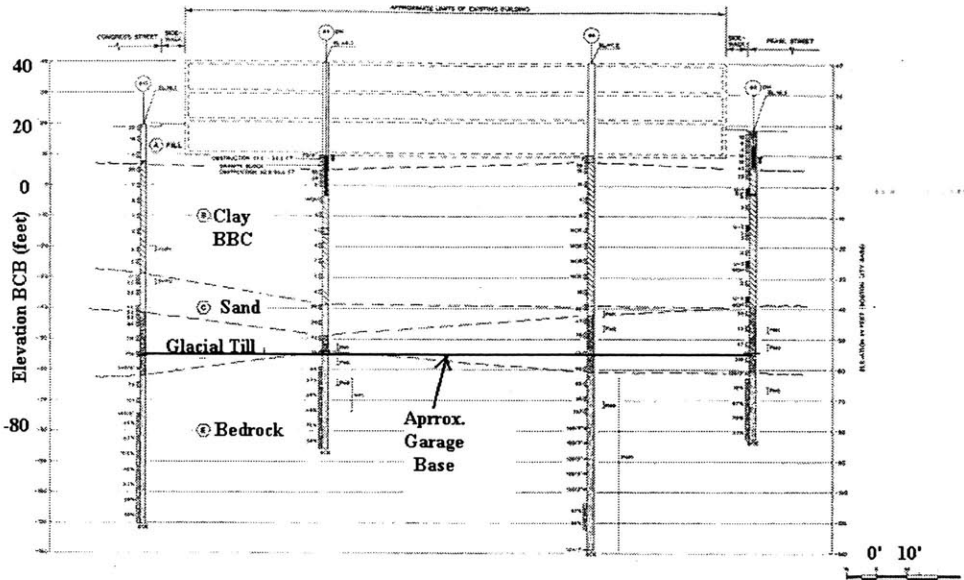
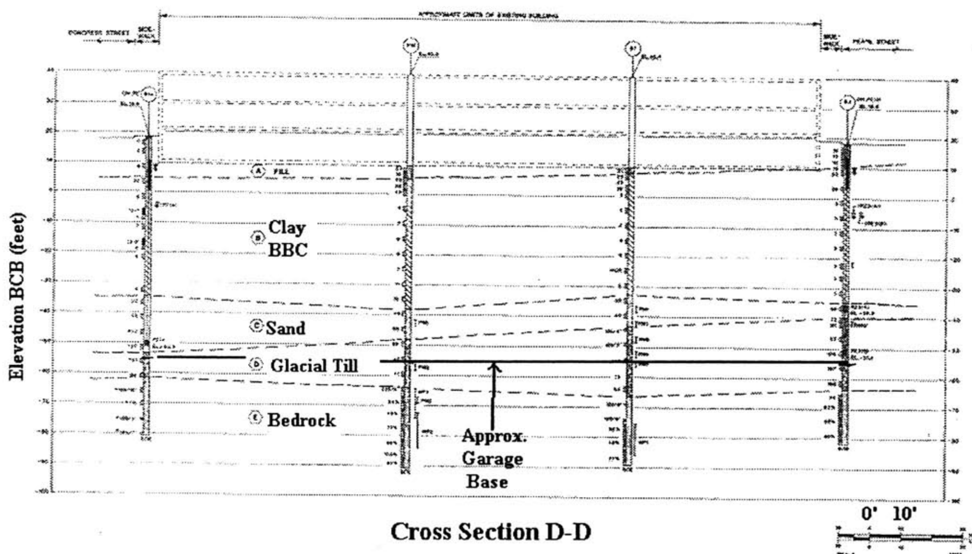


Figure 4.33: Post Office Square Garage (B10), Site and instrumentation



Cross Section E-E



Cross Section D-D

Figure 4.34: Selected soil profiles reflecting initial conditions, Post Office Square Garage (B10).

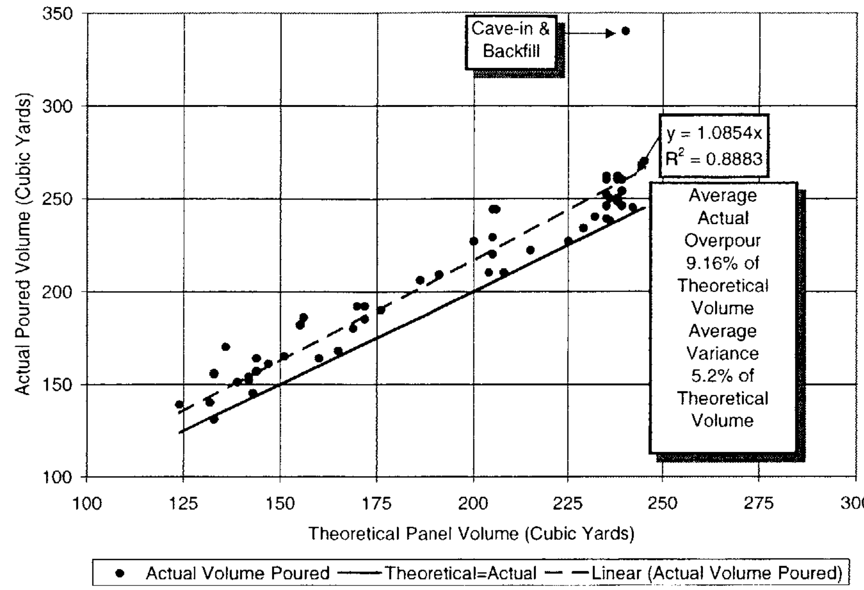


Figure 4.35: Actual and Theoretical pouring volumes of concrete for slurry wall construction, Post Office Square Garage (B10)

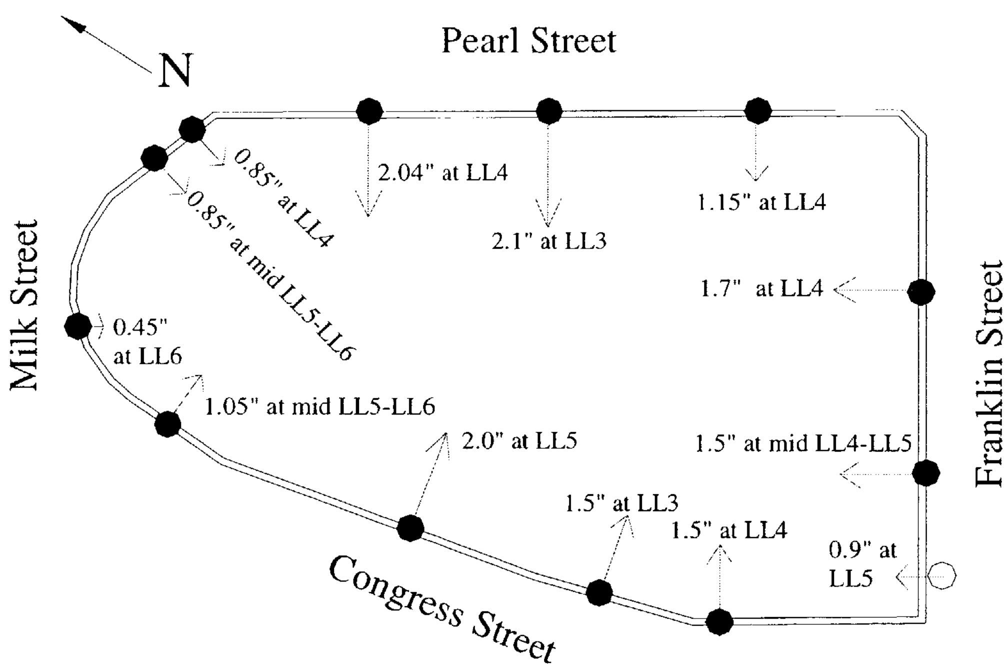


Figure 4.36: Maximum measured horizontal wall and soil deflections for Post Office Square Garage (B10)

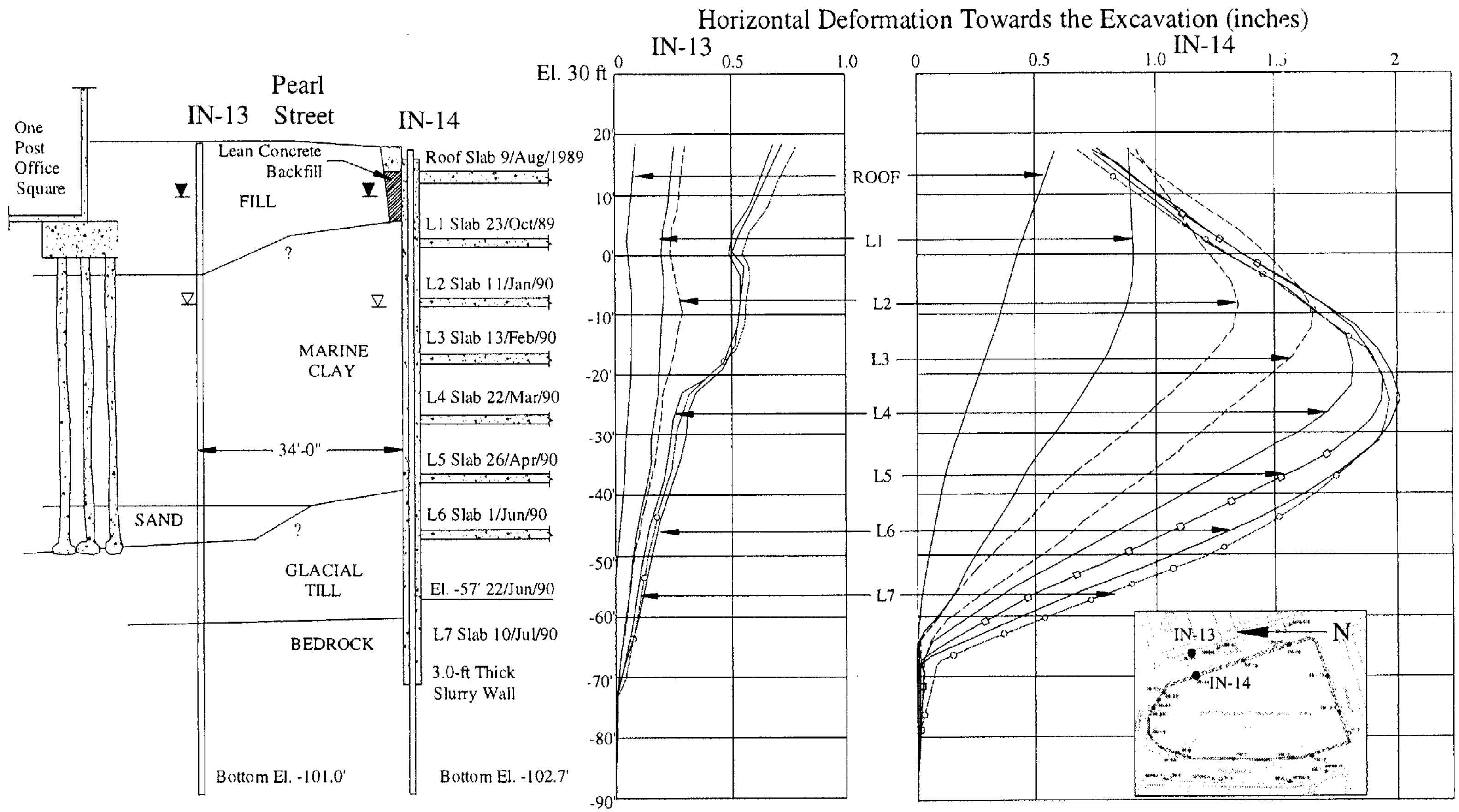


Figure 4.37: Inclinometers IN-13 & IN-14, Pearl Street, Eastern Wall (B10).

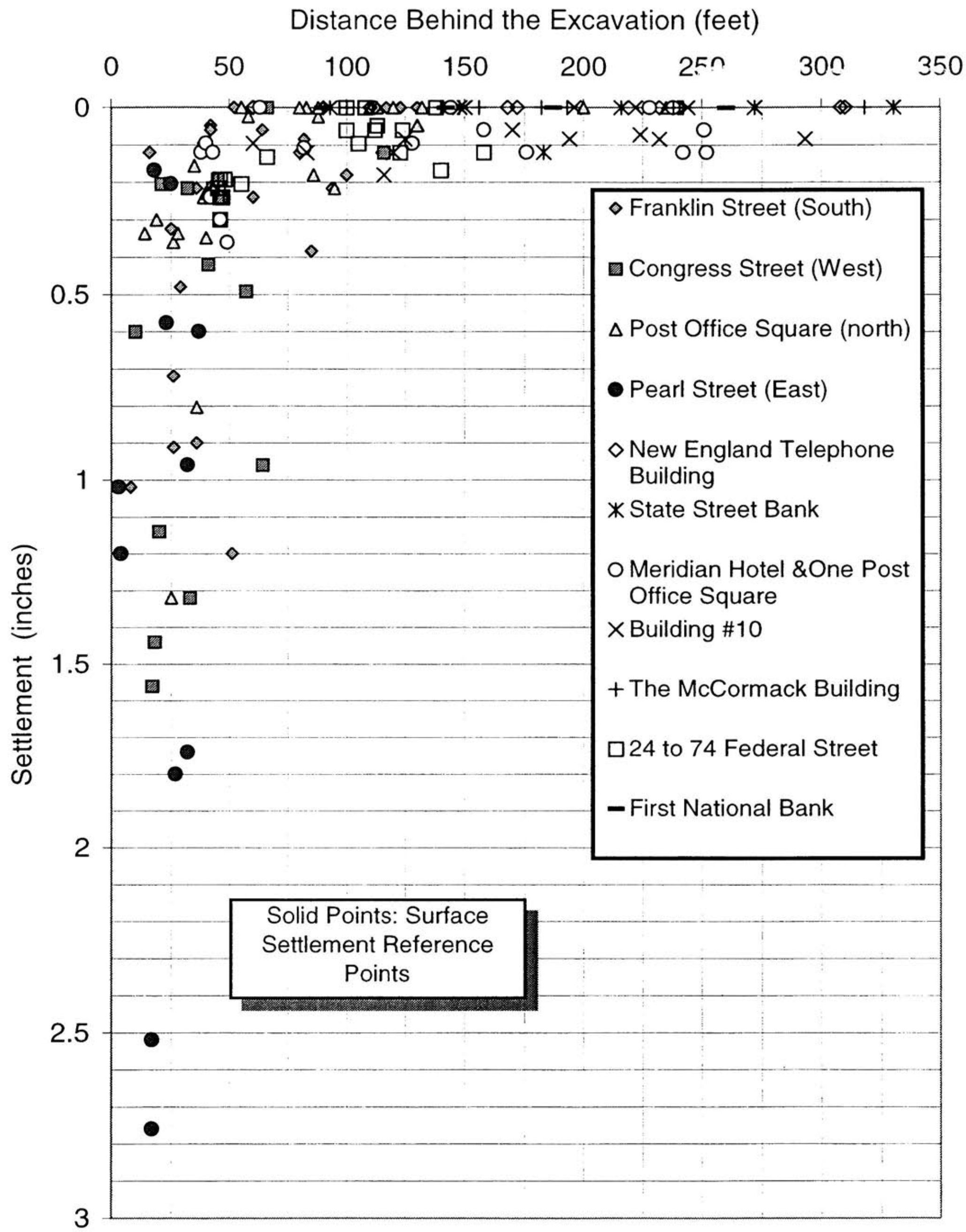


Figure 4.38: Final settlements versus distance behind excavation (B10).

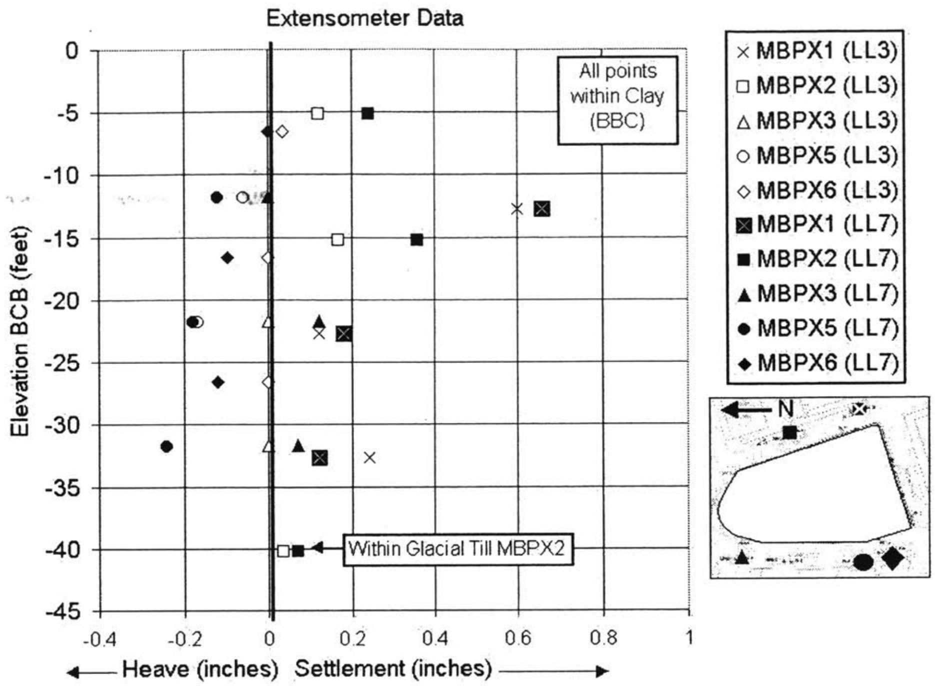


Figure 4.39: Subsurface settlements (measured by extensometers)

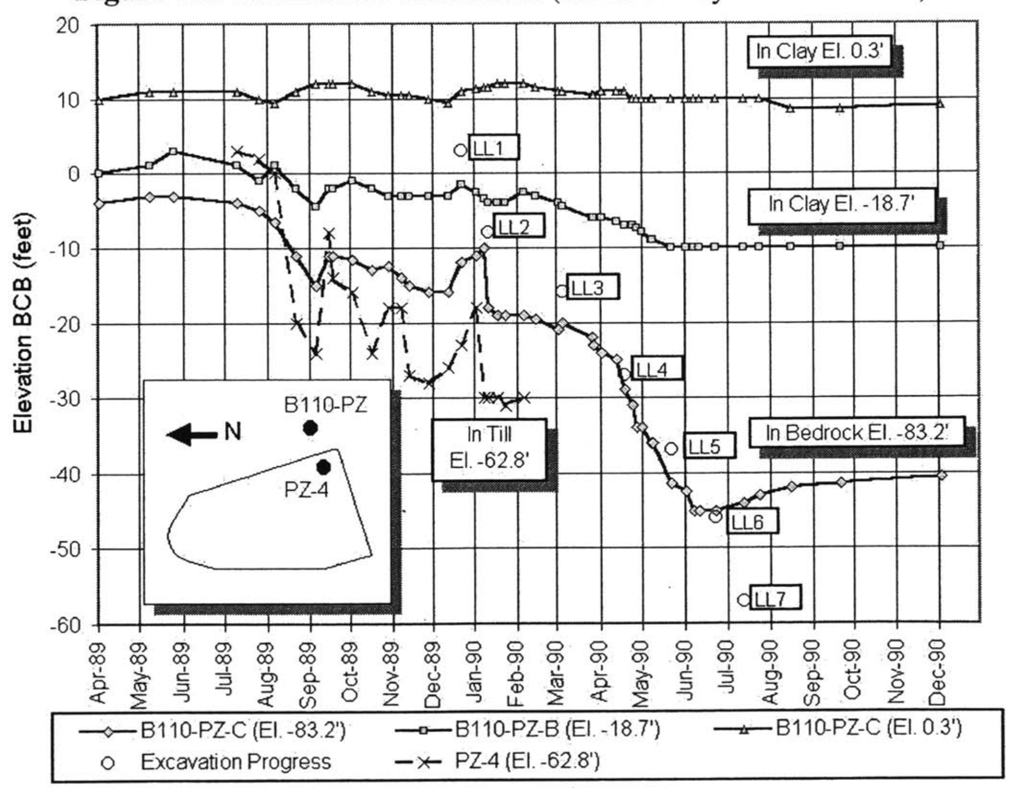


Figure 4.40: Piezometric measurements at B110-PZ and PZ-4 and construction (B10).

4.5.11 Case Study B11, Beth Israel Deaconess Hospital

The Beth Israel Deaconess (BID) Hospital is located in the corner of Longwood and Brookline Avenues in Boston Massachusetts. The building consists of a 4-storey low-rise area and a 18-storie high-rise section. A 5-level underground parking garage was constructed under the building using the up/down construction method. The Massachusetts College of Arts building (MCA) previously occupied a major portion of the site. This added to the complexity of the project since the facade of the MCA building facing Brookline Avenue had to be incorporated to the new BID building. The foundation perimeter walls were constructed using the slurry trenching method (slurry walls) with a saw-toothed configuration.

Settlement control was important since damage to adjacent buildings had to be minimized. The type of foundations that respective buildings have affects their settlement during an adjacent excavation (including the up/down construction method). Buildings with deep foundations like end-bearing piles typically experience smaller settlements during adjacent excavations from buildings founded on shallow foundations. A plan of the site is shown in Figure 4.41.

Five major soil strata and two major rock units were encountered in the test boring program conducted for the BID project namely: 6'-15' miscellaneous fill (SPT<15 bpf), over 10'-36.5' of loose to very dense medium to fine sand (30<N<50), 25' to 64' marine clay (BBC), 3'-10' glaciomarine deposits, and either Argillite or Conglomerate bedrock. The soil conditions at the site are usual of the glacial past of the Boston area and the soil profile can be classified as profile B according to Johnson [1989] (Fig. 4.3). Typical cross sections reflecting the initial and final conditions and can be seen in Figure 4.42.

The top/down construction method required that deep foundation units were installed prior to the general excavation. Load Bearing Elements (LBE) were used to support the basement columns. LBE's were designed as to utilize both skin friction and end bearing. The reinforced concrete diaphragm (slurry) wall served both for temporary and permanent earth support of the basement area. The below grade garage floors provided temporary and permanent bracing of the slurry walls. The walls were embedded a minimum of 3ft into glacial till in locations where the walls carried vertical loading from the superstructure. An allowable pressure of 30ksf was used for slurry wall bearing design. Slurry wall panels were typically 20' long with most panels at the northern, eastern, and southern sides of the project 2.5' thick, while some other panels were 3.0ft thick

Horizontal wall movements in this project were moderate to small. Maximum wall deflections ranged up to 0.88" at LL4 (Fig. 4.43). Most of the wall deflections occurred after the excavation reached below the third or the fourth garage level. At the final stages in the Brookline Avenue slurry wall the point of maximum wall movement occurred between the fourth and the base slab levels.

Soil horizontal movements that were monitored next to slurry wall inclinometers were smaller than wall movements. Typically soil movements matched slurry wall deflections with the exception of some erroneous inclinometer data. Horizontal soil movements were observed at considerable depths below the ground surface despite the large slurry wall embedments (15 to 30 feet). Maximum horizontal soil movements up to 0.64" occurred at the Brookline Avenue side inclinometer where wall deflections were the largest throughout the project.

Horizontal soil movements along Longwood Avenue were slightly larger reaching up to 0.6" ten feet below the ground surface. Slurry wall deflections at the Longwood panels were also moderate reaching up to 0.84" between the fourth and the base garage slab. At the base slab elevation the maximum horizontal soil movements was 0.25" and the corresponding wall deflection was close to 0.75".

Figure 4.44 shows final measured settlement contours. Typical building settlements were small to moderate with the exception of the MCA facade. The MCA facade settled as much as 1.4" but close to 0.4" of this settlement occurred before any foundation construction probably due to the demolition of most of the MCA building. Buildings supported on shallow foundations closer to the east face of the excavation had moderate settlements. The southwestern corner of the

Kirstein Building settled close to 0.8" (closer to the excavation) and the Beth Israel garage settled up to 0.5". The Kirstein building was founded on spread footings while the Beth Israel garage has pressure injected footings at 20' below ground surface, while other buildings settled less than 0.2".

Overall, performance of the BID foundation system was satisfactory since most slurry wall deflections and adjacent building settlements ranged from small to moderate values. There were no reports regarding cave-ins during slurry wall construction or other notices regarding slurry wall leakage.

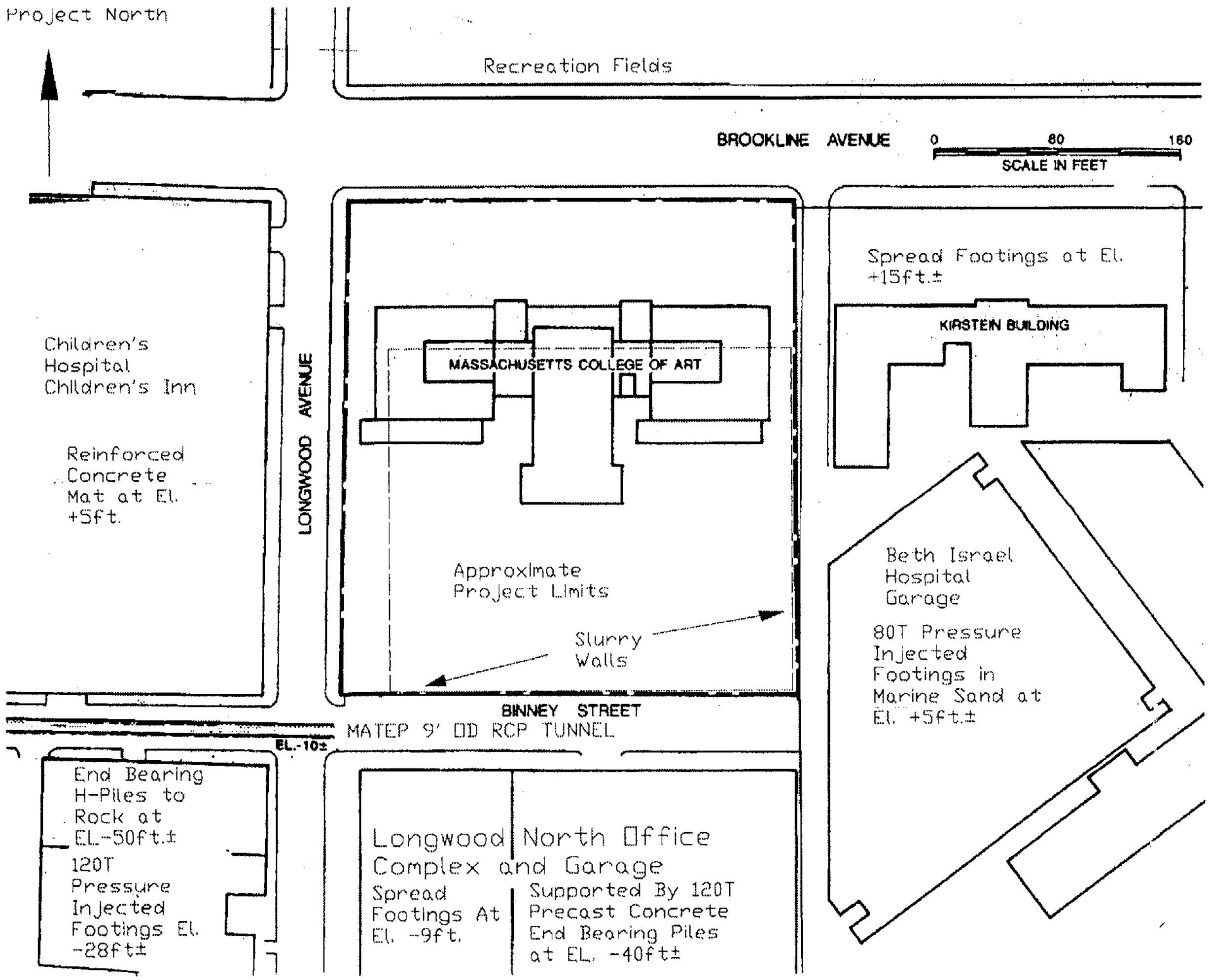


Figure 4.41: Beth Israel Deaconess site and adjacent buildings (B11)

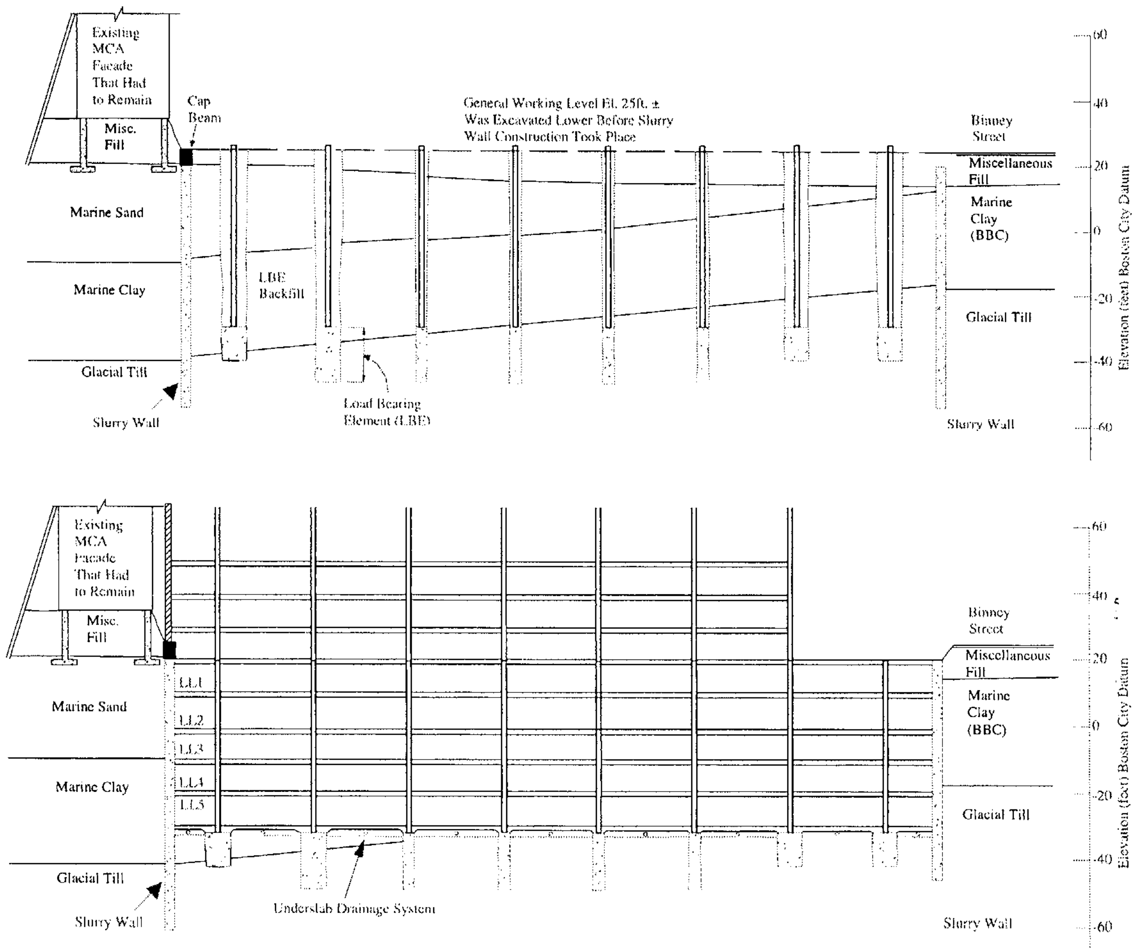


Figure 4.42: Initial and final cross sections of the excavation (B11)

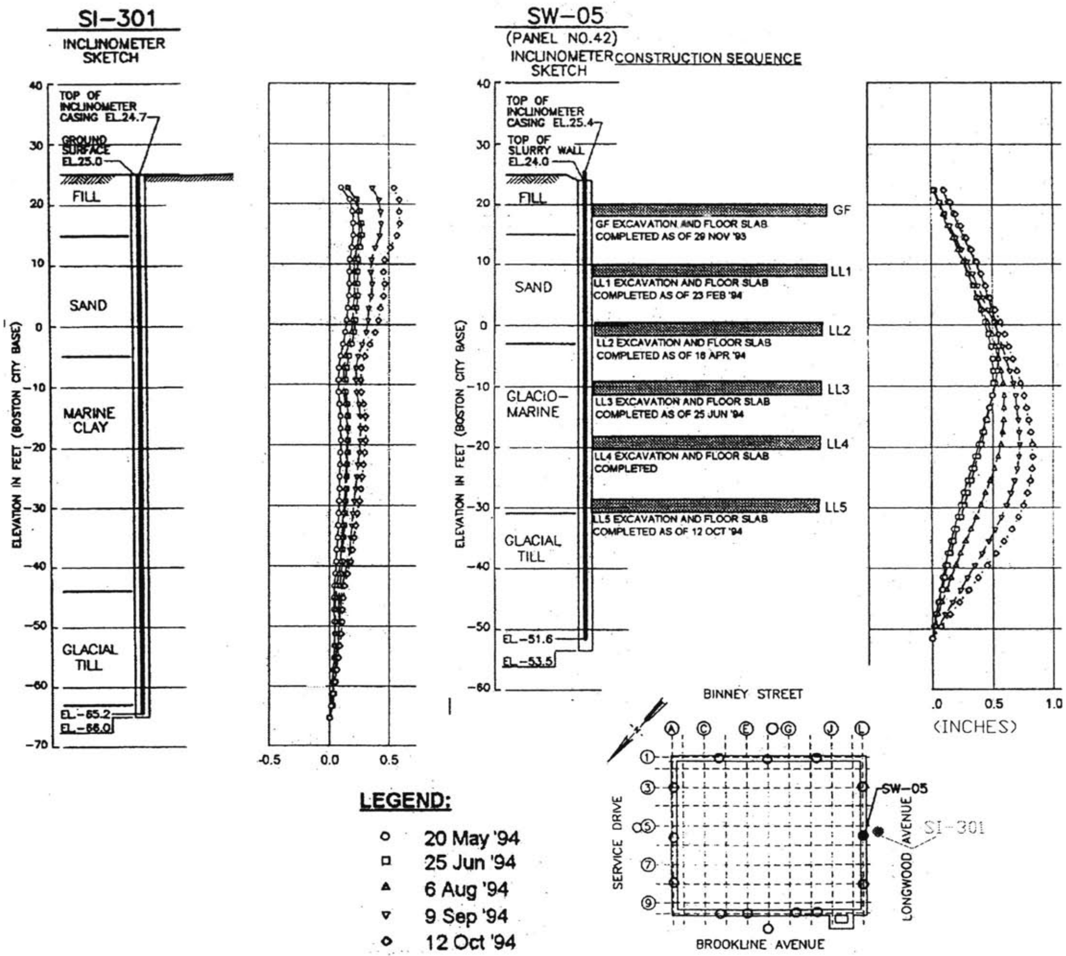


Figure 4.43: Longwood Avenue slurry wall deflections & horizontal soil movements, Beth Israel Deaconess Hospital (B11).

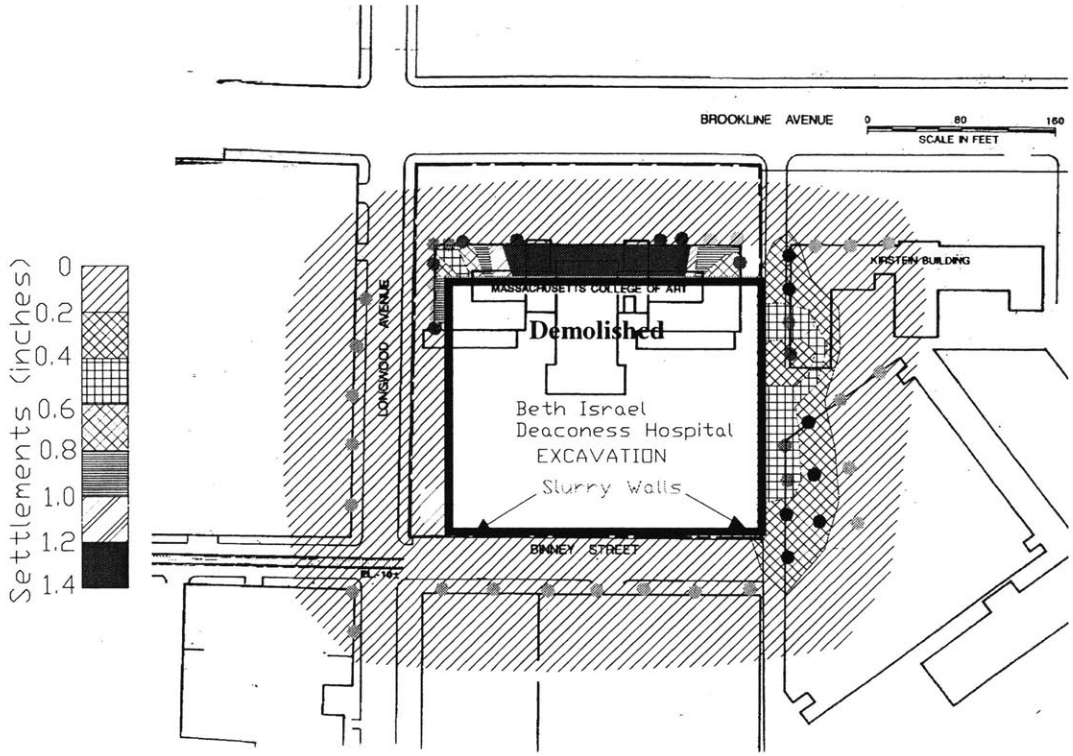


Figure 4.44: Total building and surface settlements (B11).

4.5.12 Case Study B12, Dana Farber Tower

The Dana-Farber Research Tower is located in the Longwood Medical Area at the northwest corner of Binney Street and Deaconess road. The tower has 14 above-ground stories devoted to office and research laboratory uses and five underground parking levels. The structure occupies the entire site, with the building perimeter as close to the property line as practical. The bottom floor of the garage is at about EL. –9 feet or about 52 feet below the original grade (El. 43 ft). The research tower is abutted by existing structures on three sides of the site and by Binney Street along the southern edge (Fig. 4.45).

Two orthogonal cross-sections (A-A, B-B) through the center of the site are shown in Figure 4.46. The stratigraphy of the studied sections is typical of the subsurface of the site. The soil profiles d include the following layers: 1'-14' of miscellaneous fill, over 13' to 22' of sand, 31' to 57'-thick BBC with a 8'-10' yellow crust at the top of the layer, glacial till, and conglomerate bedrock. The bedrock was medium to hard, slightly weathered gray to purple, coarse-grained conglomerate with closely spaced dipping joints. The Rock Quality Designation (RQD) ranged from 28 to 40%. The depth to bedrock varied between approximately 66' and 90'. Based on the information form borings T1-T6 and from boring EC-17 drilled within the limits of the proposed tower and the slurry wall installation, the bedrock surface drops from east to west and from north to south. As discovered by the slurry wall installation the actual bedrock surface between borings was fairly irregular.

The tied back slurry wall provided temporary support during the excavation of the basement, but is not part of the permanent structure of the research tower. Instead it acts as a permanent lateral earth support system and a barrier that isolates the tower from vibrations (mainly from the adjacent MATEP power plant) in order to protect delicate (and expensive) medical experiments. The research tower itself is founded on a series of caissons that bear on the underlying bedrock (Roxbury Conglomerate).

The slurry walls are supported by six- (6) level of tiebacks to bedrock at an average inclination of 45° to the horizontal. All the tiebacks extend to bedrock and have a bonded length of about 20'. The actual bonded length is unknown. Typical design values for the lock-off loads of the tiebacks are shown in Table 4.6.

On the east wall the first two levels of tiebacks do not exist because of the existence of the MATEP utility tunnel. A prestressed concrete edge was used to provide the necessary lateral support for the east wall, supported at the north and south walls. Figure 4.47 shows the final profiles of the excavation along sections A-A and B-B.

The design thickness of the slurry wall is 3' feet, and the wall extends a minimum of 2' feet in the underlying bedrock. The slurry wall/bedrock friction was designed to be enough to counteract the lateral forces without the need for the 2' key that provides added safety.

Slurry wall deflections were very small, with a maximum value reached close to δ_H=0.7" in the south wall (Fig. 4.53). Most slurry walls were slightly pulled back for major portions of their length although at some construction stages small inward movements were recorded (Figs. 4.51 & 4.52).

Despite the small wall movements or even the pulled back walls, the settlements were excessive in the eastern and western sides. Air drilling caused ground softening and ground losses can explain the occurrence of small wall movements and large settlements (Fig. 4.48). As indicated by Figures 4.49 and 4.50 the settlement troughs were typical of excavation projects. Maximum settlements were recorded near the wall and diminished with increased distance. Settlements up to δ_V=2.8" were measured in the eastern side within the MATEP utility tunnel. Inward east wall movements in combination with the soil losses through the tiebacks contributed to the increased settlements in the MATEP tunnel. Clearly excessive settlements were induced by soil and water losses through the anchor heads. On the slurry walls that the losses through the tiebacks were kept to a minimum the settlements were acceptable and the damage to the adjacent structures was kept to a minimum. Adjacent buildings (Jimmy Fund, and Brigham and Women's Hospital buildings) settled far less than the tunnels and with no recorded damage.

The strain gages placed at the bottom of the slurry wall reinforcement showed that all of the vertical forces induced on the slurry wall by the tiebacks were transferred to the base of the wall (Fig. 4.55). This could be expected because the slurry wall was not allowed to settle and thus no side friction developed between the slurry wall and the retained soil. Induced moments at the bottom of the slurry wall became significant only after the excavation had reached the 5^{th} level (out of 6 levels total) (Fig. 4.49).

At the slurry wall construction phase two cave-ins with sinkholes developing at the ground surface occurred (Case Study B12: south wall: Section 5.1). One of the cave-ins occurred when a panel excavation was resumed after a halt of 4 days for logistical reasons (panel SWP-4). The depth reached when the cave-in occurred was into the deep silty sand stratum. A 3-foot diameter and 5 foot deep sinkhole developed on the ground surface. It was presumed that a channel had developed between the face of the panel excavation and the sinkhole at the ground surface. Another cave-in and a soil depression occurred at panel SWP-2 on the day of the concrete pour. The reasons for this cave in are not known.

Table 4.6: Typical tieback design loads and data, (B12).

	Design Values				
LEVEL	Vertical Angle (Degrees)	Lock-Off Load (Kips)	Test Load (kips)	No. of Strands	Approximate Stressing length (feet)
Level P1	---	---	---	---	---
Level P2	---	---	---	---	---
Level P3	45	354	531	12	80
Level P4	45	484	726	16	64
Level P5	45	438	663	15	50
Level P6	45	421	632	16	36

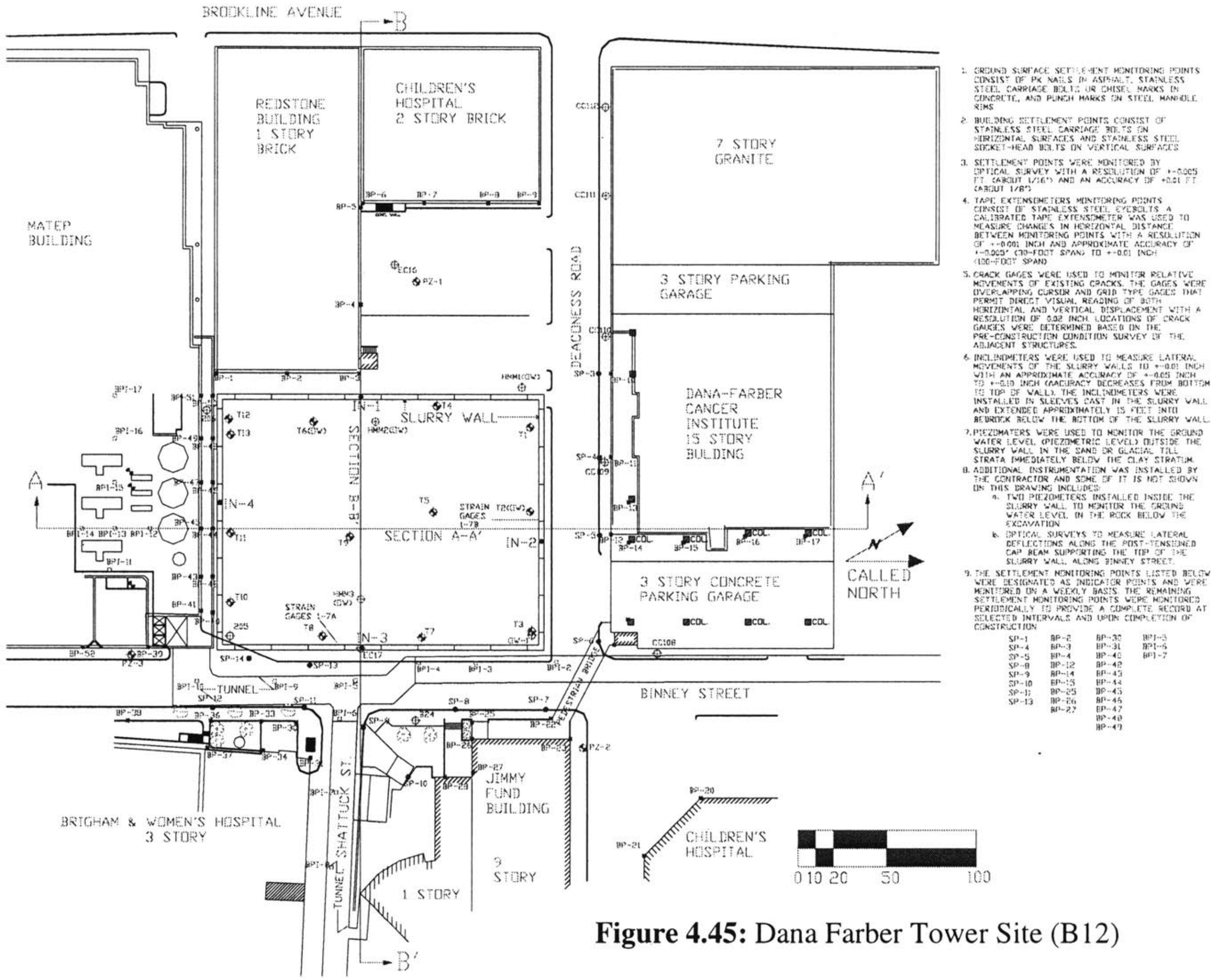
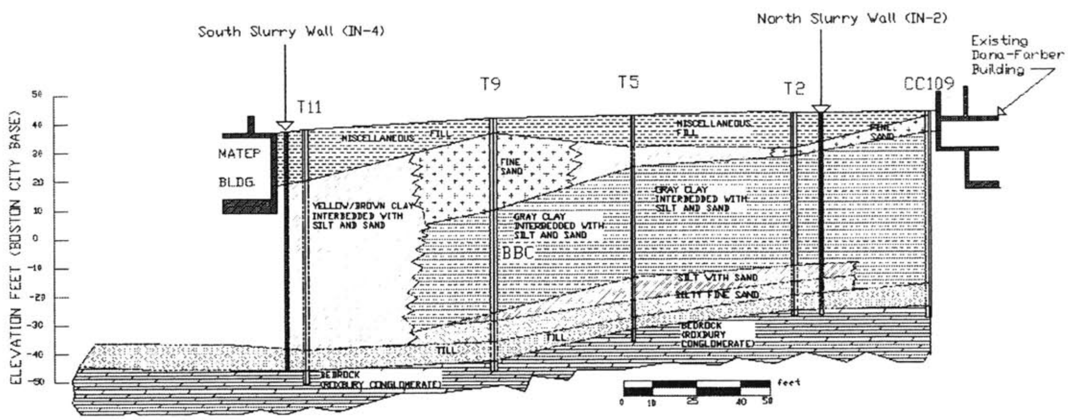
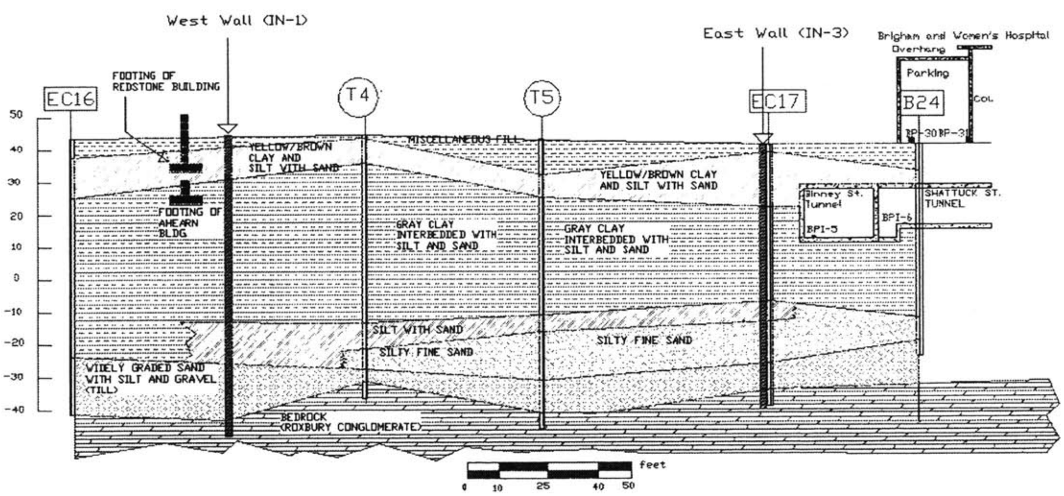


Figure 4.45: Dana Farber Tower Site (B12)

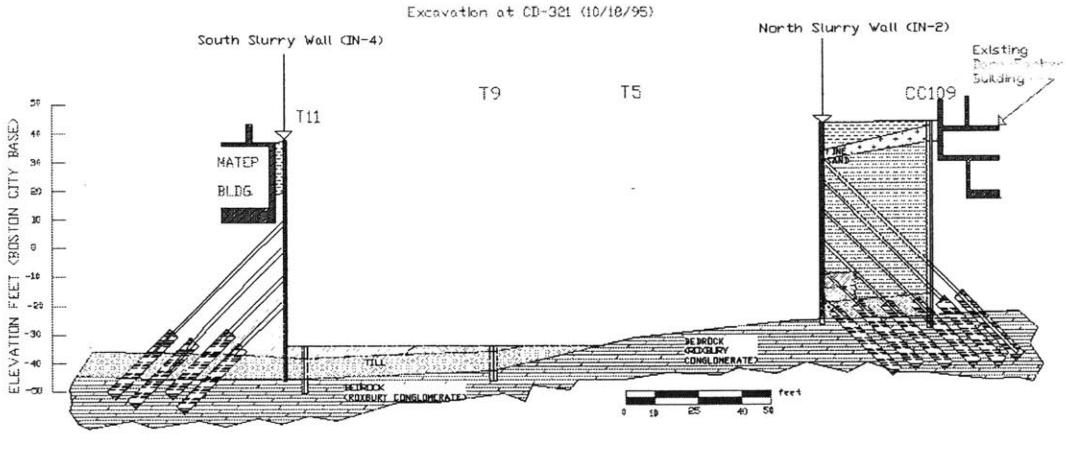


a) Section (A-A).

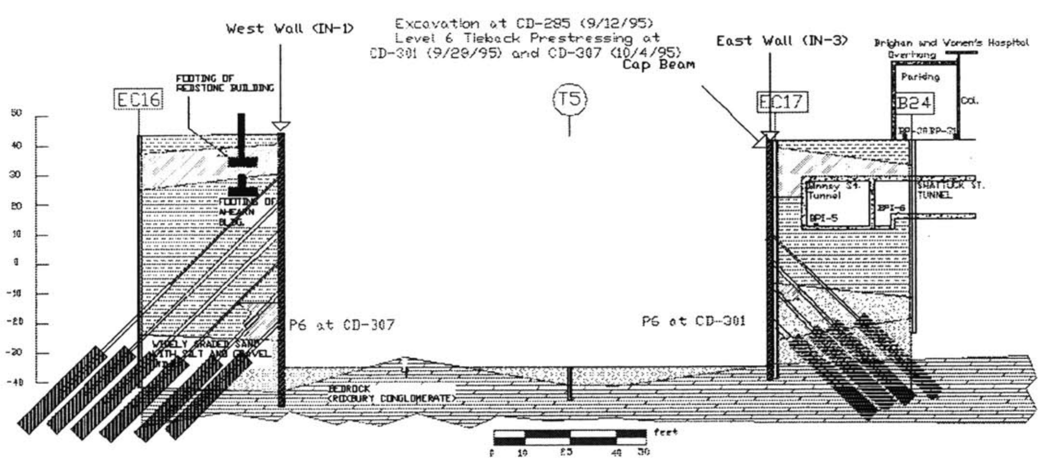


b) Section (B-B).

Figure 4.46: Initial soil profiles at the Dana Farber project site (B12).



a) Section (A-A).



b) Section (B-B).

Figure 4.47: Excavation cross-sections at the final stage, Dana Farber (B12).

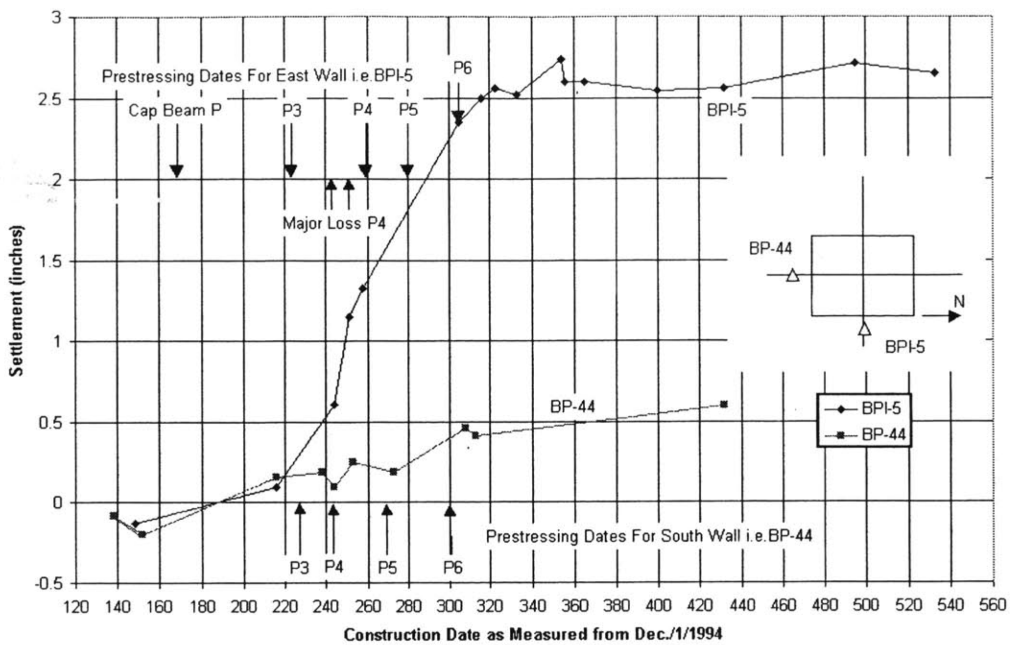


Figure 4.48: Settlements with time and construction events (B12).

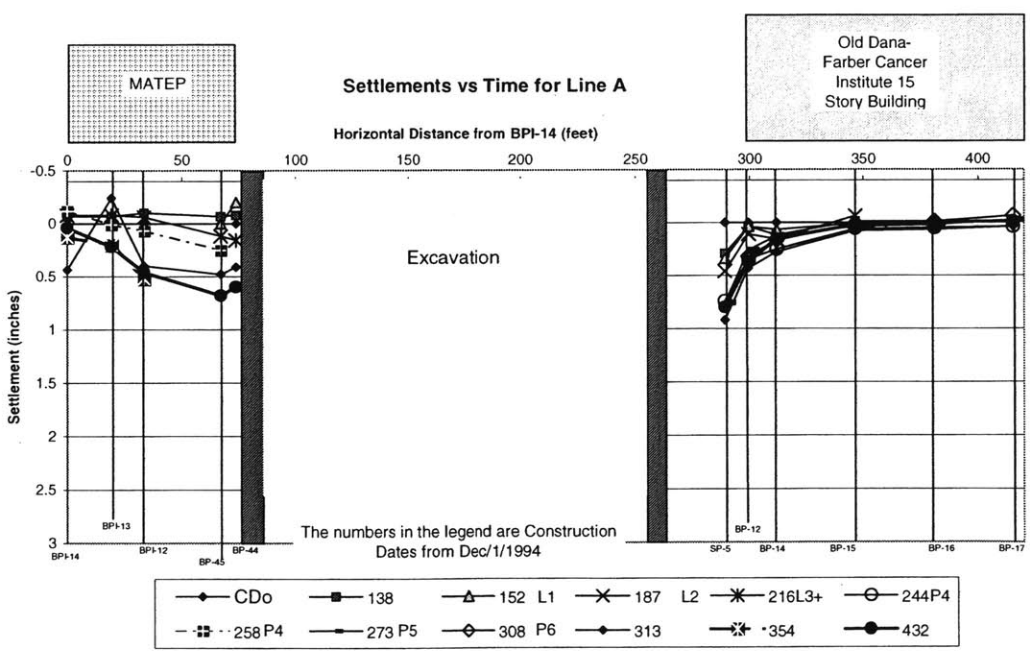


Figure 4.49: Settlement troughs at line A, (B12).

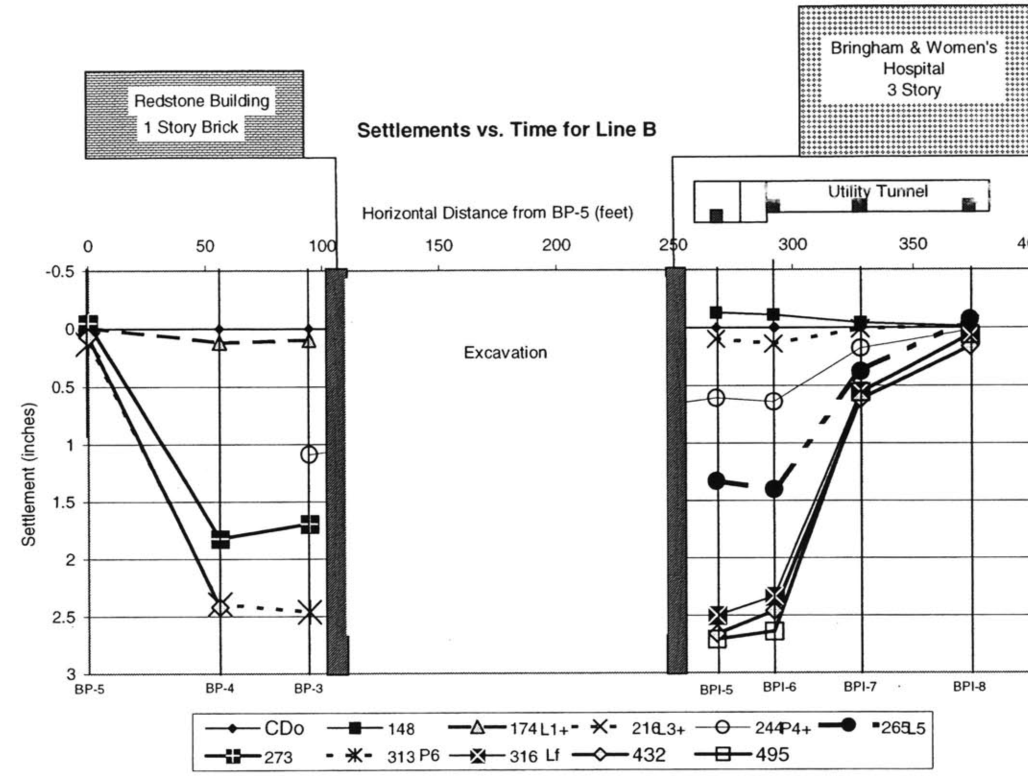


Figure 4.50: Settlement troughs at line B, (B12).

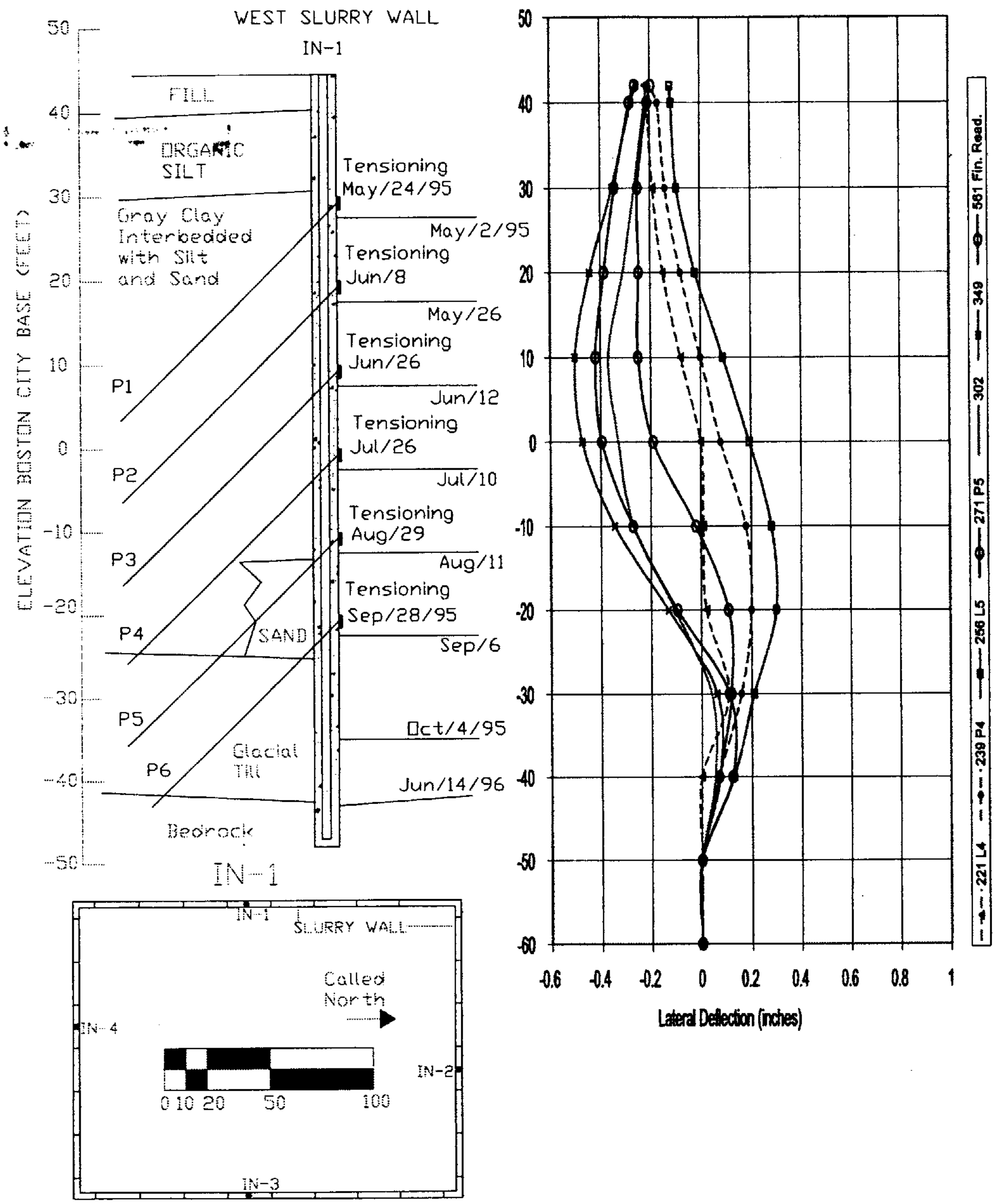


Figure 4.51: Wall deflections at inclinometer In-1, Dana Farber Tower (B12).

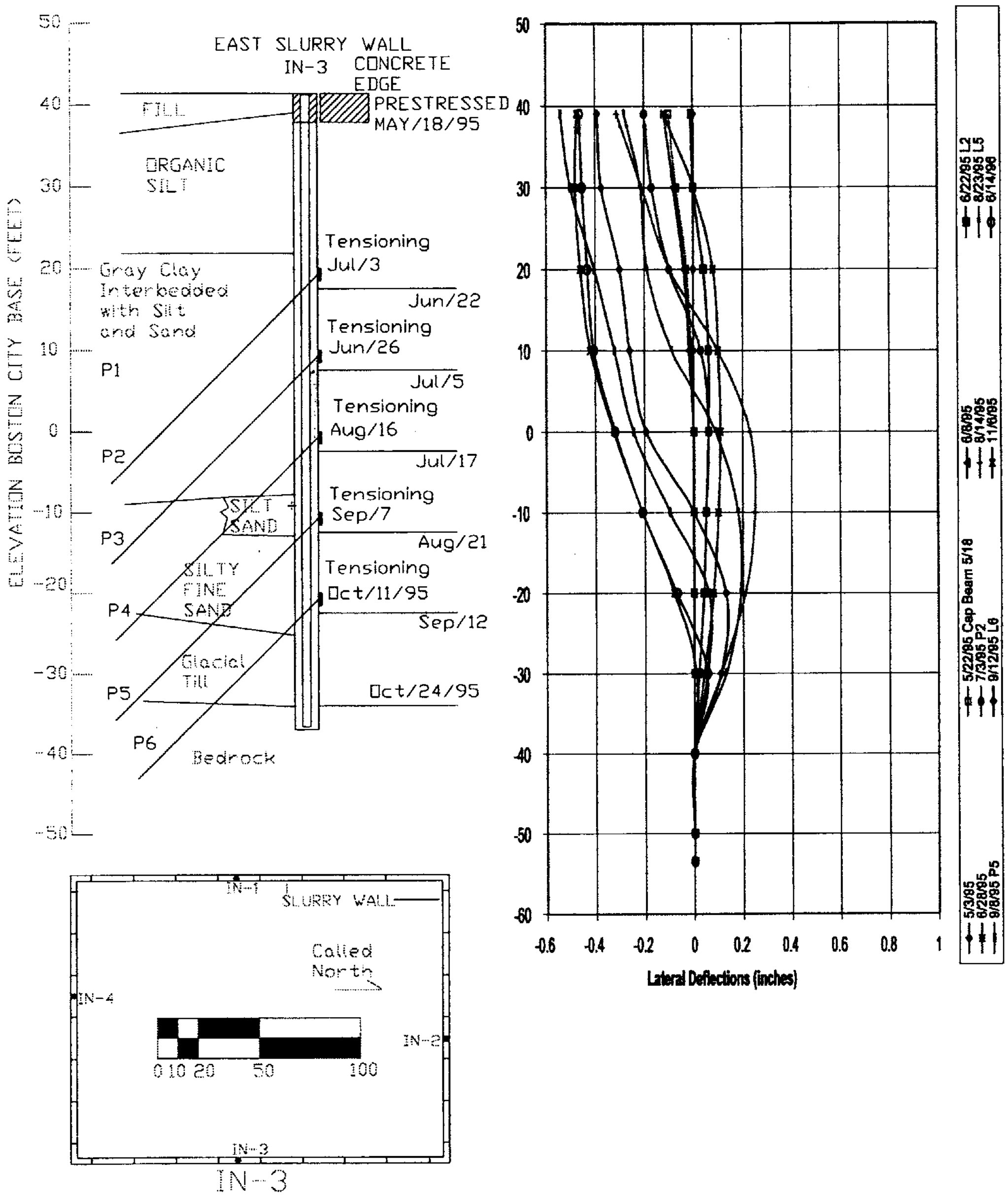


Figure 4.52: Wall deflections at Inclinometer In-3, Dana Farber Tower (B12).

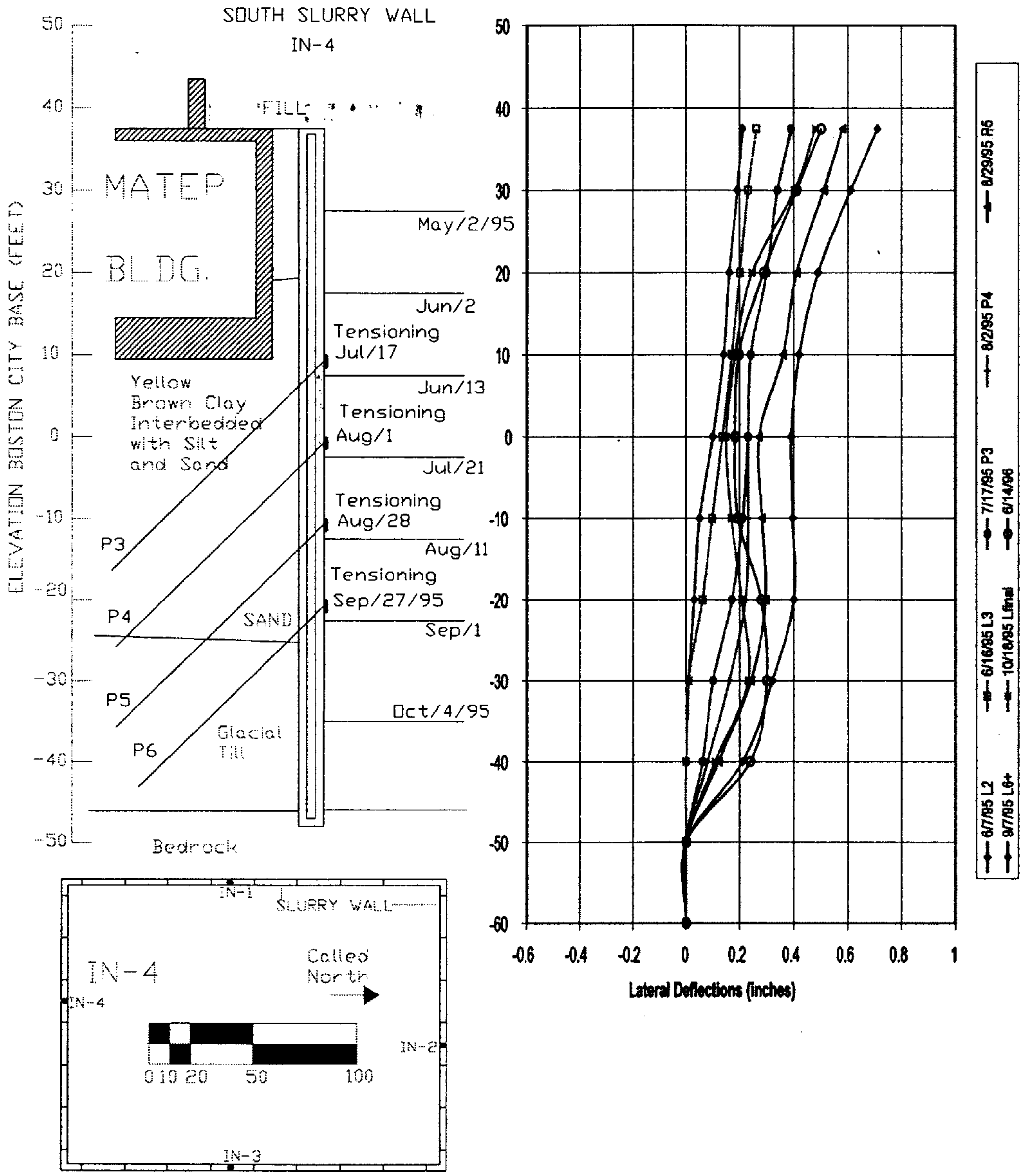


Figure 4.53: Wall deflection at inclinometer In-4, Dana Farber Tower (B12).

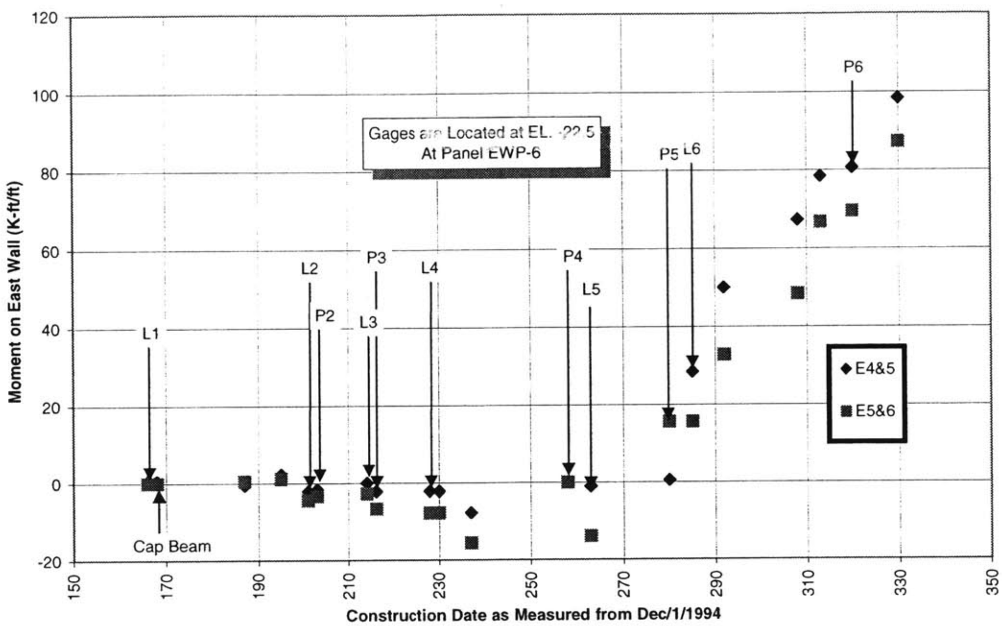


Figure 4.54: Moments on eastern slurry wall, deduced from strain gages attached to the diaphragm wall reinforcement, Dana Farber Tower (B12).

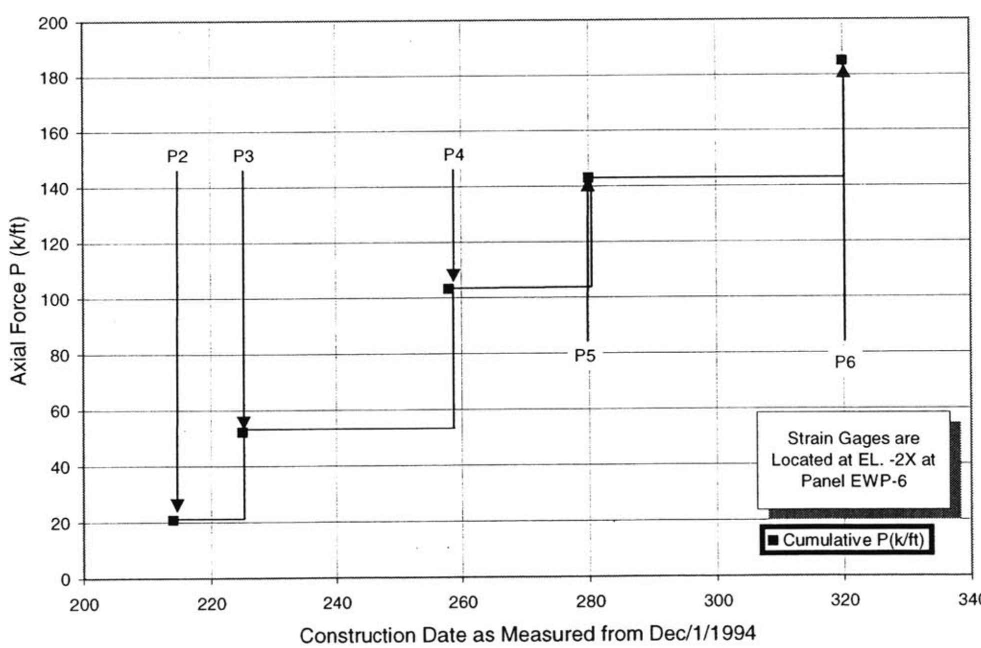


Figure 4.55: Axial force on the eastern slurry wall as determined by embedment gages, Dana Farber Tower (B12).

4.5.13 Case Study B13, Millennium Place

The Millennium Place project includes a 55'-deep excavation with five underground parking levels constructed by the up/down method. The excavation has just been recently completed. Numerous buildings are right next to the excavation along the northern, western, and southern sides of the project. The MBTA Orange Line runs under Washington Street to the east of the excavation. Thus, slurry walls were selected to protect adjacent structures from excavation induced damage. Figure 4.56 shows the site plan and Figure 4.57 displays a typical wall section.

Soils at the site are representative of profile B as classified by Johnson 1989. The soil profile consists of fill, organics, outwash deposits, marine deposits, outwash deposits, glacial till, and bedrock.

The diaphragm walls are 3.0'-thick and have their toe embedded 40' beneath the base of the excavation, into the marine and glacial till deposits. Load bearing elements and caissons were constructed to carry the vertical loads from the basement floors and the superstructure.

Measured wall deflections are in the order of 0.3" to 0.8" towards the excavation, with some bending evident. Measured settlements are in the same order as wall deflections but at some localized points settlements were larger probably because of other construction activities.

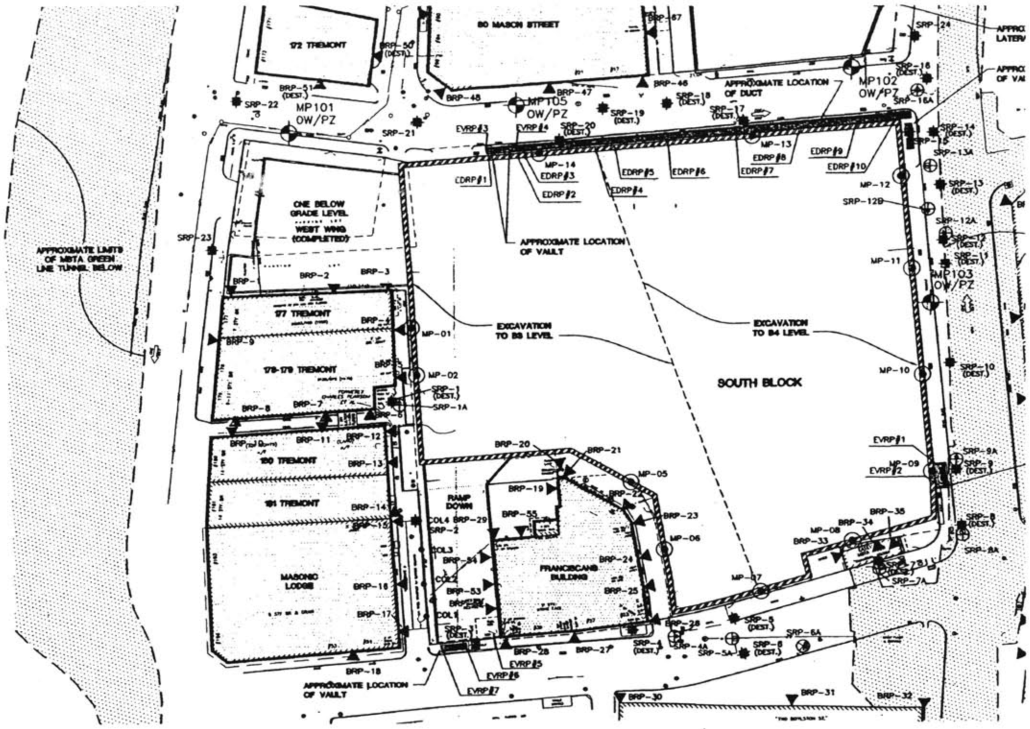


Figure 4.56: Millennium Place (B13) project site.

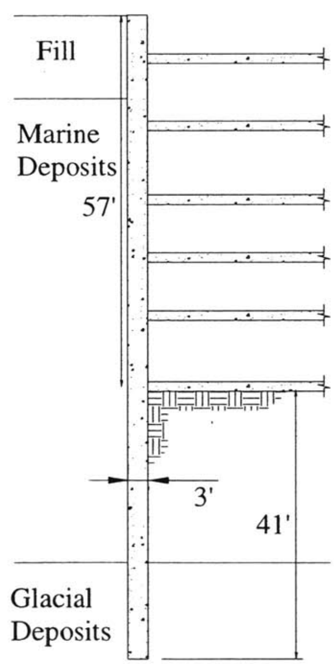


Figure 4.57: Typical wall section at Millennium Place (B13).

4.6 Summary of Wall Deflections

Figure 4.58 shows the maximum wall deflections for Boston projects plotted as a function of the system stiffness approach as proposed by Clough et. al. [1989]. Table 4.7 summarizes the input parameters used to plot Figure 4.58. The majority of the projects plots have $\delta_W/H<0.4\%$, with a notable exception of 500-Boylston St. in which movements were up to 0.66%. Top down projects generated movements $\delta_W/H<0.15\%$ corresponding to the higher system stiffness. Floating tieback projects generated larger wall deflection ratios than top/down projects .

If we exclude the 500-Boylston project (B7) then there is correlation of decreasing wall movements with increasing system stiffness. Top/down projects do not show any significant correlation with system stiffness.

The two tieback projects that were keyed into glacial till or bedrock (B2 & B12) showed smaller wall deflections δ_W/H than those with no toe fixity (B3, B6, B7).

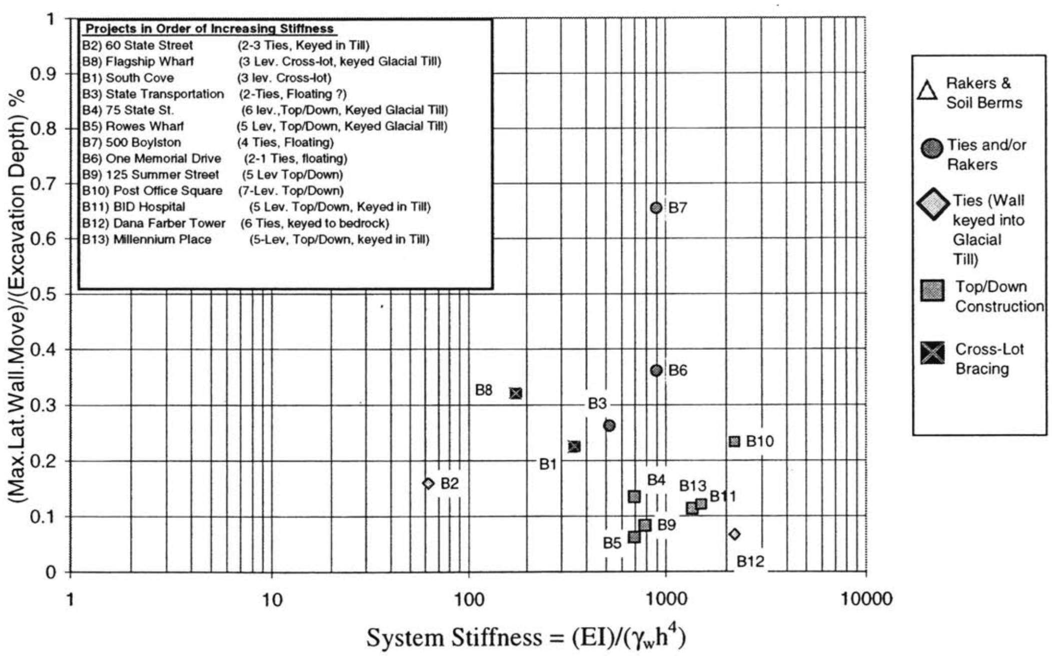


Figure 4.58: Maximum wall deflections for Boston projects plotted according to Clough 1989 method

Table 4.7: Summary of system stiffness input parameters and horizontal deflections for diaphragm wall excavations in Boston.

	Project	Wall Thick (ft) t	Exc. Depth (ft) H	Vertical Support Spacing (ft) h	f_c' (ksi)	Horizontal Deflections (inches) Caisson+ Other δ_{other}	Total δ_{total}	Exc. Only δ_{Exc}	System Stiffness $EI/\gamma_w h^4$	$\delta_{Exc/H}$ %
B1	South Cove	3	50	15	3.5	0	1.35	1.35	345.9	0.225
B2	60-State Street	2	35	17	3.5	0.25	0.92	0.67	62.1	0.160
B3	State Transportation	2	27	10	3.5	0	0.85	0.85	518.8	0.262
B4	75 State Street	2.5	65	11	3.5	0.8	1.85	1.05	692.1	0.135
B5	Rowes Wharf	2.5	55	11	4.5	0	0.41	0.41	784.7	0.062
B6	One Memorial Drive	2	30	9	4.5	0	1.3	1.3	896.6	0.361
B7	500 Boylston Street	2	42	9	4.5	0	3.3	3.3	896.6	0.655
B8	Flagship Wharf	2.5	47	16	4.5	0	1.81	1.81	175.3	0.321
B9	125 Summer Street	2.5	60	10	4.5	0	0.6	0.6	1148.9	0.083
B10	Post Office Square	3	75	10	5.5	0	0.85	0.85	2194.9	0.094
B11	Beth Israel Hospital	3	55	11	5.5	0	2.1	2.1	1499.2	0.318
B12	Dana Farber	3	88	10	5.5	0	0.72	0.72	2194.9	0.068
B13	Millennium Place	3	55	11	5.5	0	0.8	0.8	1499.2	0.121

Note:

Modulus of Elasticity: E (psi)=$57000\sqrt{f_c' \text{ (psi)}}$ => E(psf)= $395.83\sqrt{f_c' \text{ (psi)}}$ (ACI-8.5.1)

f_c' : 28 Day strength of concrete

Moment of Inertia of Uncracked Section = I

$$I(ft^4) = \frac{(1ft)\, t^3}{12}$$

t (ft) = Diaphragm wall thickness

γ_w = Unit weight of water = 62.4 pcf

Table 4.8: Inclinometers per project (Boston) used to derive statistics in Fig. 4.59 & Fig. 4.60.

ID	Project	Inclinometers
B1	60 State Street	4
B2	MBTA South Cove	3
B3	State Transportation Building	19
B4	75 State Street	1
B5	Rowes Wharf	1
B6	One Memorial Drive	5
B7	500 Boylston St.	8
B8	Flagship Wharf	11
B9	125 Summer Street	1
B10	Post Office Square	13
B11	Beth Israel Deaconess Hospital	15
B12	Dana Farber Tower	4
B13	Millennium Place	
	Total	85

Figure 4.59 summarizes a frequency plot of the maximum and final deflections of the 85 inclinometers used in the 13 study projects in Boston. The same data are replotted in Figure 4.60 in terms of the dimensionless ratio δ_W/H where H is the final excavation depth. Deflections were subdivided into 0.5" intervals , like -1.0" to -0.5", and 0.0" to 0.5" (negative sign indicates movement back into the retained soil). For the δ_W/H the selected interval was 0.1% of the final excavation depth. Table 4.8 gives the number of inclinometers per project used to derive the frequency plots in Figures 4.59 & 4.60.

From Figure 4.59 we can clearly see that the overwhelming majority of the inclinometers deflected less than 1.0", while only 16% of the inclinometers had a final deflection greater than 1.0". The average maximum deflections is δ_H =0.70" ± 0.68", while the final deflections δ_H = 0.60" ± 0.59". However, the movements within a project varied more than this figure implies. The effect of actions to reduce wall movements can be seen in this figure. In particular the percentage of deflections in the range of 1.0" to 1.5" decreased by about 5% in favor of smaller range of movements. It is interesting to note that almost 50% of the inclinometers deflected from 0" to 0.5".

Figure 4.60 seems to suggest that the frequency of δ_H/H follows a lognormal skewed to the left distribution, for both the maximum and final wall deflections. The mean deflection ratio for the maximum cases are δ_H/H = 0.171%± 0.132%, while for the final deflections δ_H/H = 0.158± 0.130%. Almost 70% of all the inclinometers had a ratio of δ_H/H smaller than 0.2% while the most prominent range of movements fell in the range from 0.1% to 0.2% of the final excavation depth. For greater ranges of movements the percentages steadily decreased. Only 17% of the inclinometers had a δ_H/H greater than 0.3%. The effect of actions to reduce wall movements shows up since the percentage of inclinometers in the range $\delta_H/H > 0.2\%$ decreased by 5% in the final stage compared to the maximum measured.

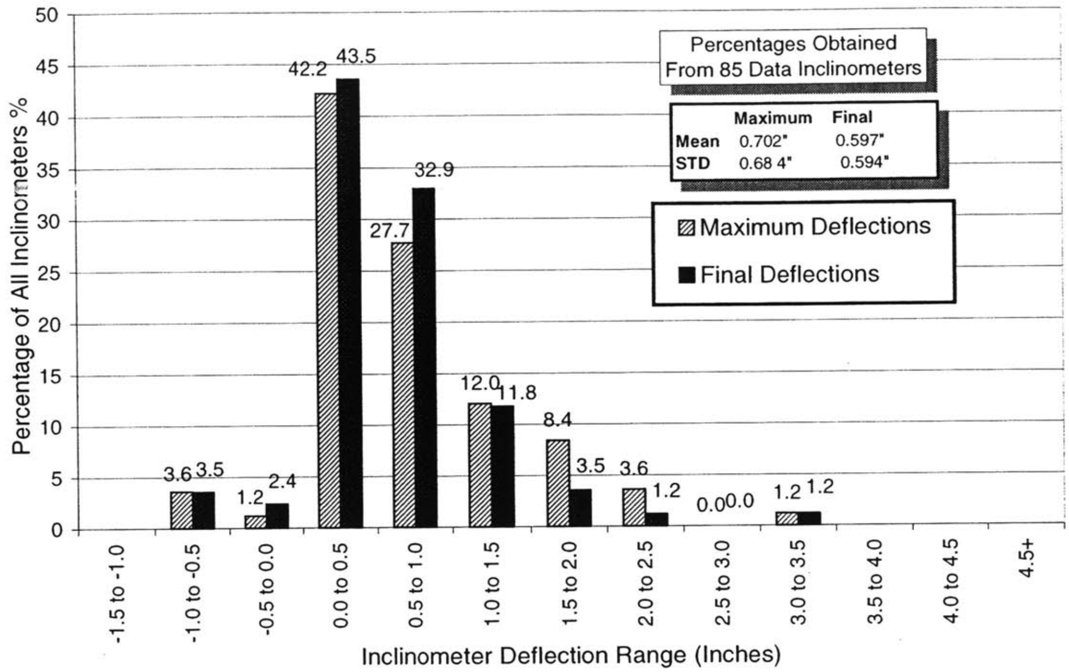


Figure 4.59: Statistics of maximum and final inclinometer deflections for all inclinometers in slurry wall projects in Boston

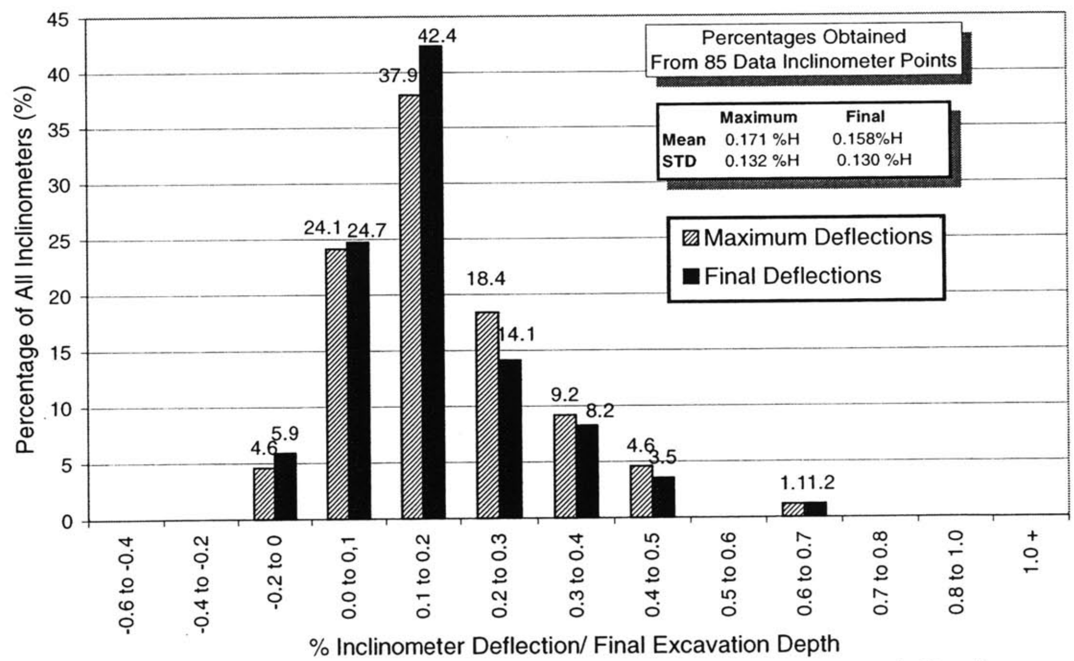


Figure 4.60: Statistics of maximum and final inclinometer deflections as percentages of the excavation depth for all inclinometers in Boston

4.7 Summary of Measured Settlements

Figure 4.61 shows selected settlement data versus distance from the excavation for 11 out of the 13 Boston slurry wall projects. Both the settlement and the distance from the excavation data have been standardized by the excavation depth. As we can see the majority of the data points fall in zone I according to Peck 1969. As expected only points from the 500 Boylston project fall beyond zone I due to the poor tieback performance. We can clearly see that settlements decrease the further away from the excavation.

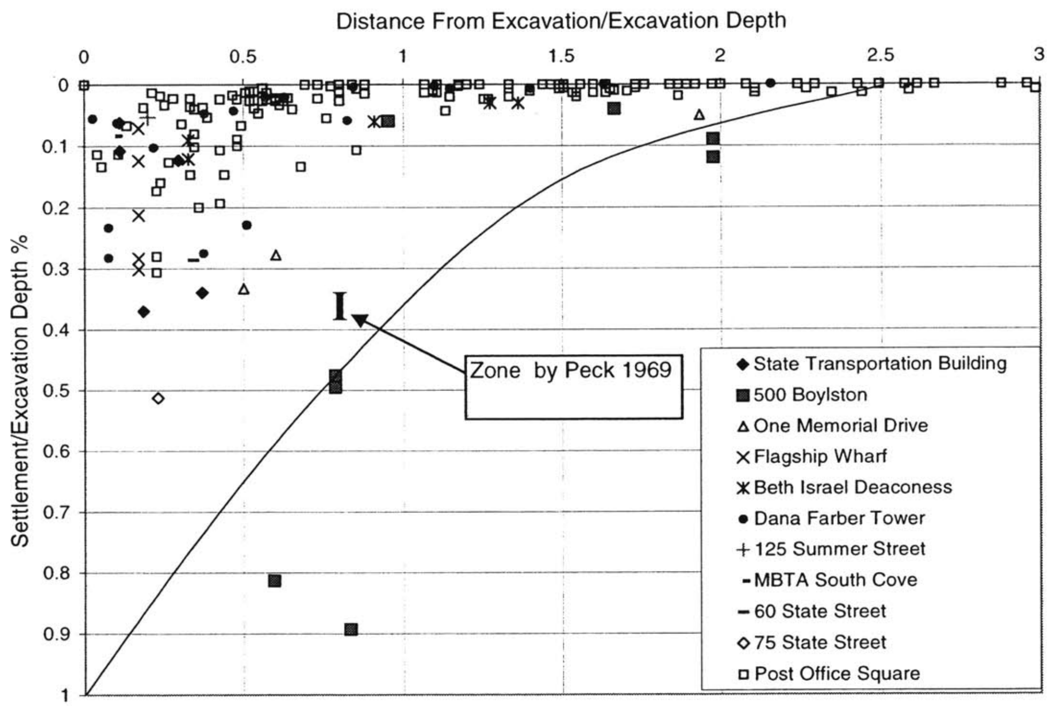


Figure 4.61: Settlements at distances from the excavation, standardized by the excavation depth, for slurry wall excavations in Boston

Figure 4.62 compares the maximum settlements and wall deflections normalized by the excavation depth, H. The data show that $\delta_V/\delta_W \simeq 0.7 - 2.0$ in most projects. The higher ratios of δ_V/δ_W are associated with projects where substantial ground loss occurred (e.g. due to tieback installation or internal losses through the tieback holes).

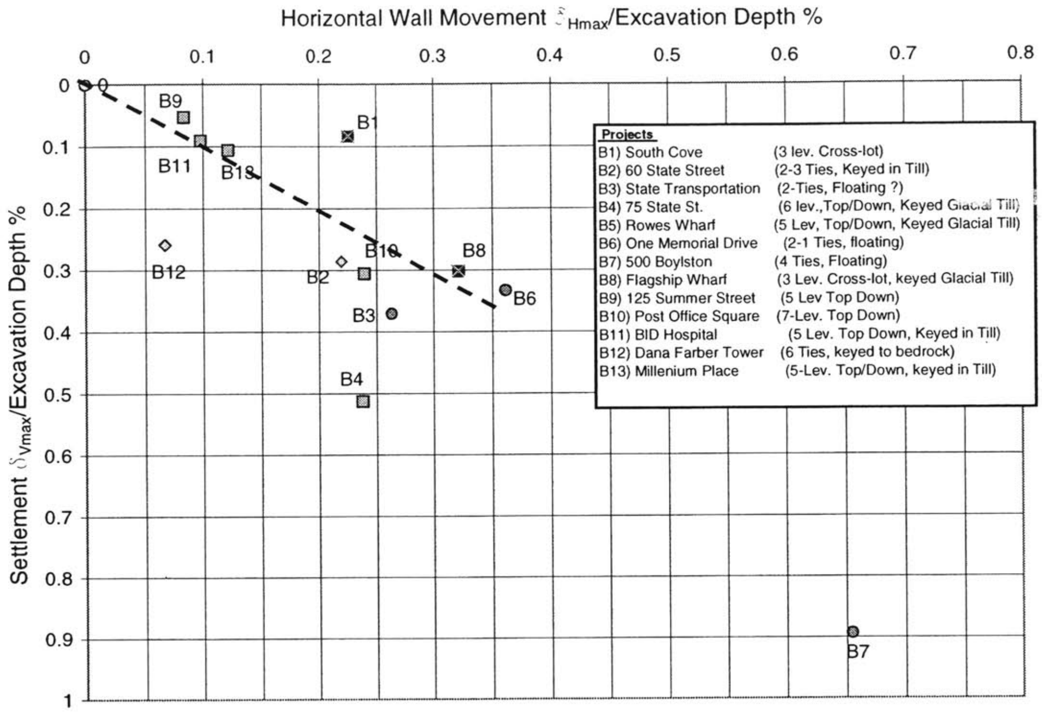


Figure 4.62: Maximum settlement/excavation depth against maximum wall deflection/excavation depth.

4.8 Special Observations

Table 4.9 summarizes special observations during construction of slurry walls in Boston. Many of these projects report cave-ins (usually not more than 1 panel). Leaving panels open during rains or over long periods of several caused some of these cave-ins. In two projects (B3, B8) the contractor trenched panels without slurry to a 15' depth before the panels collapsed. Otherwise, obstructions during trenching caused some delays and difficulties. Also in the B3 project, the guide walls were unstable and tended to crack and collapse.

Horizontal soil movements during slurry wall were not measured in the majority of projects. When they were measured (B2, B11) they were in the order of 0.2" towards the slurry filled trench. These movements were largely reduced or reversed after concreting (B2, B11).

The use of tiebacks in combination with slurry walls has been problematic in several projects (B3, B7, B12). For example, in the Dana Farber Tower (B12) and the 500 Boylston Street, the water stop detailing of tiebacks was not adequate and water leakage through the tiebacks was quite common. In the 500-Boylston Street (B7), there reports that tiebacks at one side of the excavation were leaking for an extended period of three weeks. Drilling tiebacks with air tended to soften the retained ground and cause soil losses as well (B2, B9). Pile extraction in two cases softened the ground and caused large soil movements (B7, B4).

Water leakage through panel joints was observed in nearly all the projects and was repaired with cement or chemical grouting. Only in the Rowes Wharf (B5) project was leakage through joints a major issue. In this project the bottom of the panels was not cleaned properly before concreting. During concreting the "waste" material that had accumulated at the panel bottom was displaced by the concrete. Some of this "waste" soil material eventually got entrapped between the panel joints and the tremie concrete. These zones of soft material leaked considerably when the excavation exposed them.

Table 4.9: Special observations for slurry wall excavations in Boston

Job	ID	Year	Soil Profile (A,B or C)[1]	Thickness (ft)	Typical Panel Depth (ft)	Typical Panel Length (ft)	Gs Bentonite mix	Cave-ins Reported	Movements During Slurry Wall Construction	Other Special Observations
60 State Street	B2	1975	A & C, 62' to bedrock	2	60	22		No reports	0.25" Towards open trench during trenching. Reversed during concreting	Non mentioned
State Transportation Building	B3	1982	A	2	48	18	1.08 - 1.20	A slope failure when a panel was dug 20' without slurry	Not measured	Many guidewalls cracked & collapsed, Bentonite pockets located at some panels, Excess concrete Periodically chipped, Small water leaks-Patching at 7 locations, some at Tieback locations , Small Cracks at two panels
75 State Street	B4	1983	A, with no organics 70-100feet	2.5	85	-		No data	No data	1" by pile extraction[2] (Johnson 1985)
Rowes Wharf	B5	1984	B, Glaciomarine waterfront sloping profile	2.5	65	20		No data available	No data available	Serious leakage observed, poor wall finish, cleaning of bottom of panels very poor, slurry "contaminated" concrete got entrapped in joints
One Memorial Drive	B6	1985	A, typical	2	44	20	1.08 - 1.20	One cave-in, loose granular fill could not be supported by slurry head	Not measured	Not mentioned
500 Boylston St.	B7	1987	A, typical of Back Bay soils	2	48	20	1.08 to 1.15	One trench collapsed, Left open during weekend with heavy rains	Not measured	Large wall deflections caused partially by pile extractions, Water leakage through tieback holes for 3 weeks
Flagship Wharf	B8	1989	B, Charlestown	2.5	57	20		One cave-in Corner panel within fill	Not measured	Numerous obstructions encountered during trenching slowed slurry wall construction, Some water leakage at panel joints easily sealed
Post Office Square Garage	B10	1989	A, Downtown	3	85	24		One cave-in backfilled with lean concrete	Not measured	Occasionally boulders were encountered during trenching
Beth Israel Deaconess Hospital	B11	1994	A, no organics, occasionally small glaciomarine layer	3	89	22	1.09 - 1.20	None	Overall very small, back into the soil typical	Obstructions had to be removed before trenching, Slurry contaminated concrete at top of walls chipped
Dana Farber Tower	B12	1995	Similar to A, but major sand outwash deposits below BBC	3	92	20.5		Two cave-ins, One after trenching was suspended for 4 days for logistical reasons, shortly after a 3' deep sinkhole depeloped	Not measured	Major problems of tieback sealing

1): General soil profiles as mentioned by Johnson 1989

4.9 Summary of Boston Slurry Wall Excavation Experience

Most slurry wall excavations in Boston have performed well with moderate measured wall movements and settlements (B1, B2, B3, B5, B6, B9, B11, B13). Walls deflected excessively when the tieback bracing was inadequate (B7) or when pile extraction caused ground softening (B4, B7). Inadequate tieback installation procedures, like drilling with air, caused ground losses in one case (B12) and while the wall did not deflect much or was pushed back into the soil the ground settled much more.

It is interesting to note that wall deflections within the same project showed a considerable range of values. The maximum deflections were measured in localized sections and reflect only about 17% of the total instrumented panels. Some of these maximum deformations were caused by local deficiencies in the bracing (B7) or variations in local conditions. Slightly less than 70% of the instrumented wall panels deflected less than 0.2% of the final excavation depth.

Floating slurry walls embedded in BBC (B3, B6, B7) translated at their bases by up to 0.5" to 1.0". These translative wall movements were not observed when the walls were keyed into the stiff glacial till or into bedrock. The diaphragm walls in floating excavations showed very little bending above the excavation base, with most of the bending observed beneath the mud mat or base slab. The basic wall deflection shape in floating walls was that of type I (bending only beneath the lower bracing level), but a lot of panels deflected in the shape mode Type II. The wall deflected with no signs of bending (Deflection shape: Type IV) when the tiebacks did not provide sufficient restraining (B7). Tieback creep is an issue and has to be considered if the anchored tieback zone is installed within clay (B6, B7). Maximum settlements in floating tieback walls were typically slightly greater than maximum wall deflections as a result of soil losses and soil disturbance caused by the tiebacks (B2, B3, B6, B7, B12).

Top/down construction in Boston has given the best results in the minimizing excavation induced movements. It is interesting to note the bowing deflection shape (Type II) that has been the most common deflection pattern in top/down excavations in Boston. The elevation where the maximum wall deflection is observed in top/down excavations is close or slightly above the current excavation level, within the Boston Blue Clay (B4, B10, B11). Keying wall in a stiff stratum such as glacial till or bedrock caused the wall to rotate as if it is resting on a pin support at its base (B4, B10, B11).

Wall deflections and settlements in one top/down project were large but only because of ground softening by pile extraction (B4). In the Post Office Square Garage (B10), movements during the initial cantilever excavation were large because the backfill for the Load Bearing Elements formed a zone of soft material behind the wall. Settlements in top/down projects were in the same order or smaller than the maximum wall movements. Of course, this was the case when there were no problems with pile extraction.

Cross-lot braced excavations displayed similar deflection shapes with the top/down excavations (B1, B8). Excessive strut forces tend to push the walls back into the soil in many cases (B8). As expected, struts picked up load as the excavation progressed deeper. Thermal expansion and contraction seem to have a more profound affect on strut loads than expected (B8).

Water leakage through panel joints was easily repairable with the exception of one project where many pockets of soft slurry contaminated material were entrapped in the concrete (B5). Leakage through the tiebacks and the tieback holes has been a quite difficult issue, since the water stopping details in some cases were not adequate (B7, B12).

Cave-ins or soil collapses during trenching have been encountered in most projects. One to two cave-ins were reported in several cases. Leaving panels open for a long time, or over rains caused most of these cave-ins. In some cases the guidewalls were not deep enough to prevent fill from sliding under them during trenching. The stabilizing effects of slurry during trenching are clear since when one panel was trenched without slurry it collapsed causing a slope failure. Soil movements during trenching have been in the range of 0.2" to 0.3", with most of them reversing during concreting. Occasionally, obstructions have caused difficulties and in slurry wall construction.

Chapter 5
Chicago Slurry Wall Excavations

Chapter 5
Chicago Slurry Wall Excavations

5.1. Introduction

Chicago was among the first cities in the USA to use slurry walls for excavation support in the 1970 construction of the CNA building. Data have been compiled for 11 projects in Chicago, 4 of which were archived from the 1970's (Table 5.1). In all of these projects the excavation depth ranges from 25 ft to 44 ft (i.e. much shallower than projects described in Boston Chapter 4).

In Chicago the upper 35 ft to 45 ft of the ground profile comprises soft clay layers with low undrained shear strengths, below which are much stiffer deposits (more details about soil properties in Chicago can be found in the following section). As a result, in all of these projects the slurry walls are embedded into the stiffer underlying clays as to increase the stability of the excavation.

In most cases, the slurry walls were not intended to act as major load carrying elements but typically only carry their own weight and their proportional share of the connecting basement slabs with the upper floors carried by independent columns and column foundations extending through the wall or immediately adjacent to the wall (Baker, Pfingsten, & Gnaedinger, 1998). Vertical loads from the superstructure are typically all carried by caissons bearing either on "hardpan" or bedrock.

Slurry walls were basically selected to: a) minimize excavation-induced deformations, and b) reduce water seepage (high water table due to Lake Michigan).

Figure 5.1 shows the location of these projects in Downtown Chicago, while Figure 5.2 shows the location of projects in southern Chicago. Ten out of the eleven studied excavations are located within downtown Chicago. Only the Museum of Science and Industry (C10) is located in south Chicago. Other known slurry wall excavations in Chicago are: 1) Olympia Center, 2) Provident Hospital, 3) Northwestern University Memorial Hospital, and 4) 311 River East (Figures 5.1 & 5.2: Projects 1 to 3 are 80, 81, 37 respectively, and 4) not shown).

Table 5.1: List of studied slurry wall excavations in Chicago, IL.

ID	Project Name	Year	Depth(ft) H	Depth(ft) D	Thick (inches)	Soil Type	Bracing	Wall Type	Toe Fixity
C1	CNA	1970	31	33	30	Typ.	1-Lev CLB	RCDW	
C2	Sears Tower	1970	32	24	30	Typ.	3-Lev R, SB	RCDW	
C3	Amoco Standard Oil	1971	23 44 *	15	30	Typ.	1-Lev. TB SB	RCDW Bearing SP	
C4	Water Tower	1973	44	22	24	Typ.	1-Lev. TB, 1-Lev, R	RCDW, Caissons	
C5	Loyola University Business School	1993	20 Est.	--	--	Typ.	1-Lev. CLB IB, Estimated	RCDW	
C6	Prudential Two	1986	25	28	27	Typ.	1-Lev. TB & 1-Lev. R	RCDW SP	
C7	AT&T Corporate Center	1987	27	32	30	Typ.	3-Lev. R, & CB	RCDW	
C8	Guest Quarters Hotel	1989	35	20 60	24	Typ.	3-Lev. TD	RCDW	– √
C9	Northwestern University Memorial Parking Garage	1990	23	25	24	Typ.	1-Lev. TB	RCDW	
C10	Museum of Science & Industry	1997	34	10	30	Out-side Down Town	3-Lev. TB	RCDW	
C11	311 South Wacker Drive	1987	35	31	24		TD, TB, R, IB	RCDW	

Note: **H** – Excavation Depth, **D**- Embedment depth, **TB** -Tiebacks, **CLB** – Cross-Lot Bracing, **TD** – Top/Down, **R** – Rakers, **SB**- Soil Berms, **CB** – Corner Bracing, **IB**-Internal Bracing, **PC** – Precast, **PT** – Post Tensioned, **SP** – Soldier Piles, RCDW – Reinforced Concrete Diaphragm Wall, **SPTC** – Soldier Piles & Tremie Concrete.

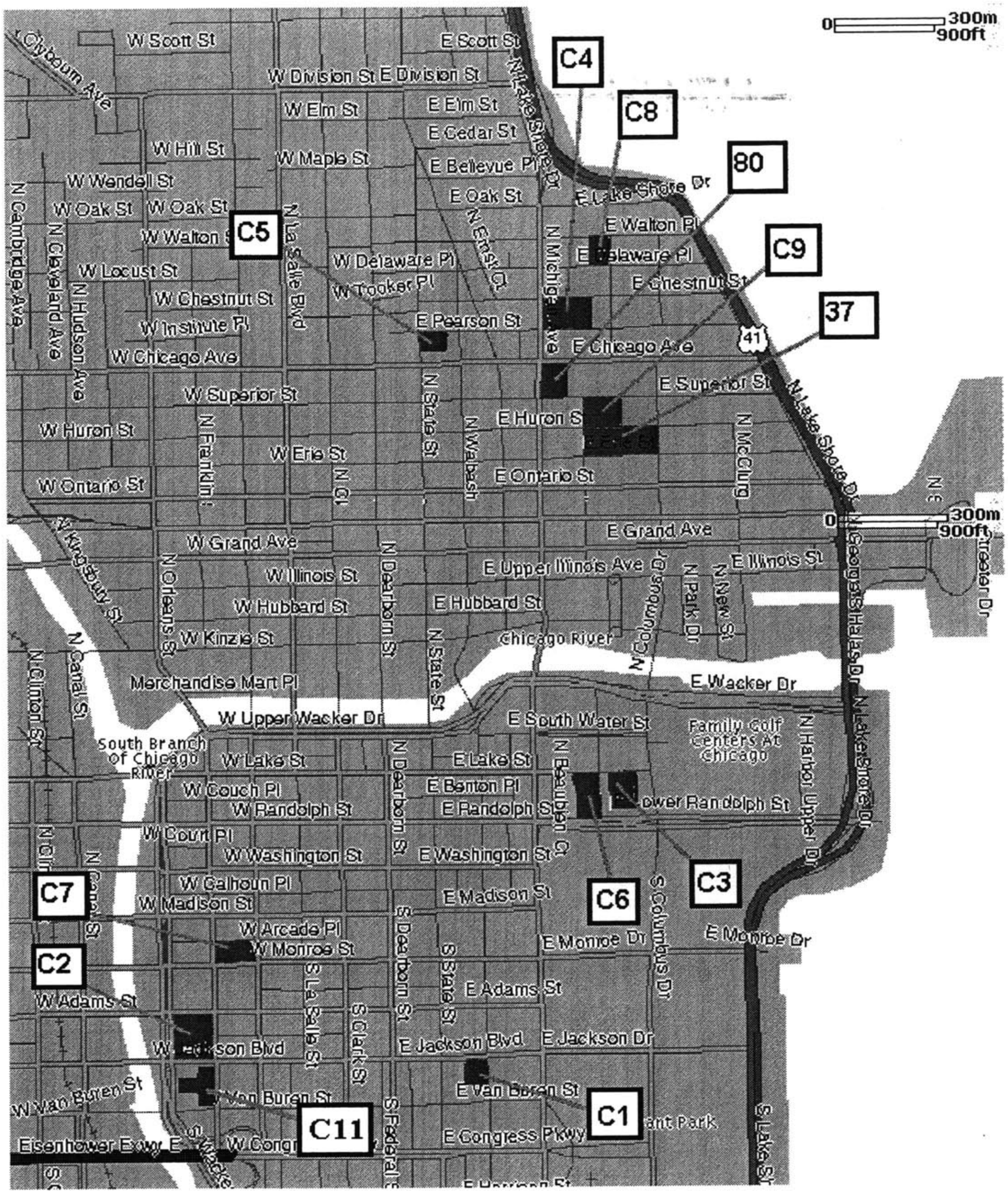


Figure 5.1: Locations of downtown Chicago slurry wall excavations

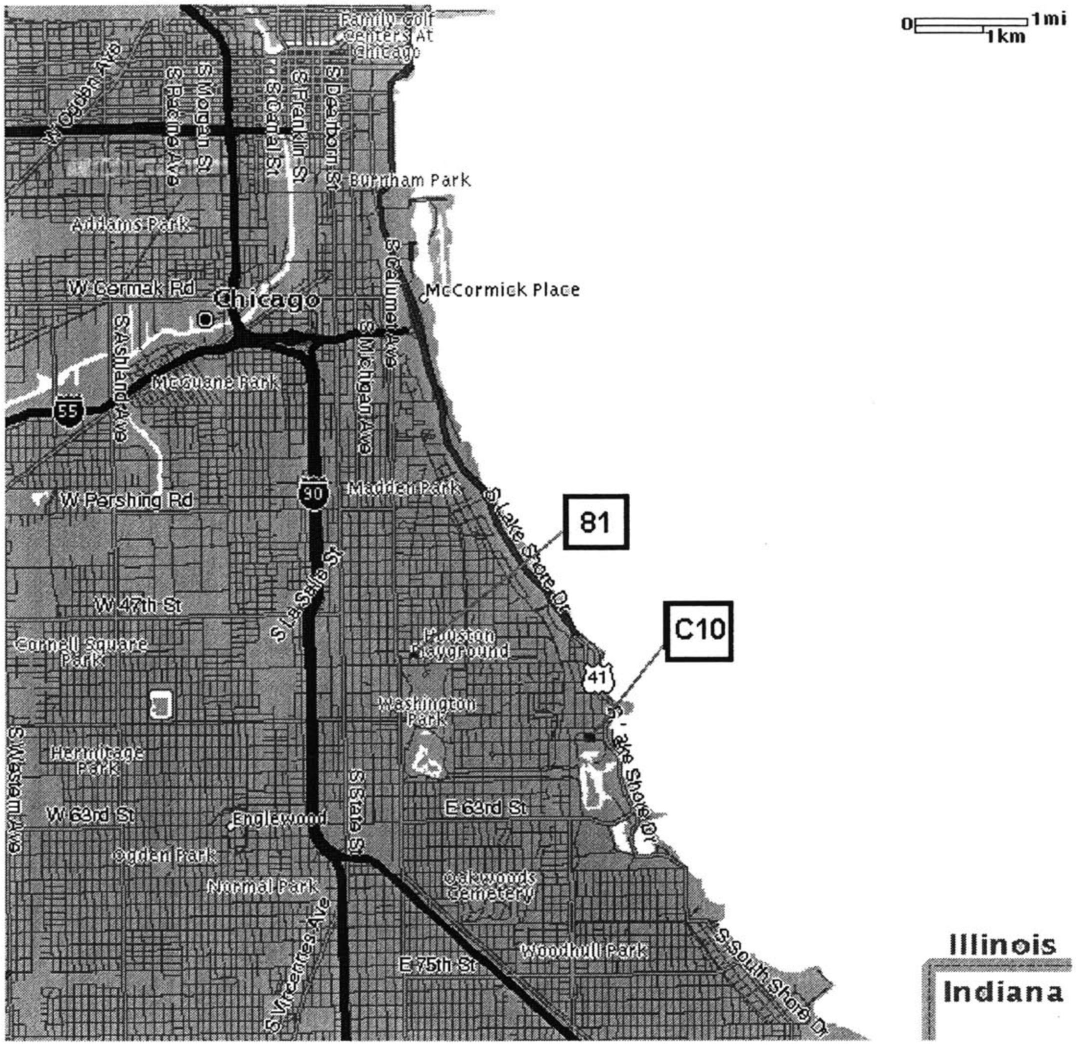


Figure 5.2: Locations of Chicago slurry wall excavations outside the downtown area

5.2 Chicago Soil Conditions

Soils in the Chicago area are primarily glacial and post-glacial sediments from the most recent advance of the continental glaciers (Finno, 1992). This section summarizes the undrained shear strengths of soft glacial clays in the Chicago area slurry wall projects that were studied in this research. Figure 5.3 shows the general soil profile in downtown Chicago (Peck 1948).

Bretz [1939] and Otto [1942] suggested that soil was deposited in six fairly distinct till sheets. These sheets are presented in order of deposition: Valparaiso, Tinley, Park Ridge, Deerfield, Blodgett, and Highland Park tills. The Deerfield and Blodgett tills have higher water contents and are characterized as compressible by Peck and Reed (1954).

Finno characterizes these compressible clays as supraglacial or subglacial tills. Subglacial tills are deposited beneath the glacier, have a uniform texture, and are overconsolidated as result of ice weight. The degree of overconsolidation of subglacial tills depends on the glacier thickness as the stratum was deposited, drainage conditions, permeability, and the duration that the ice remained in place.

Supraglacial tills are deposited near the front of the glacier as result of three main processes: melting of ice, mudflows and slumps, and sedimenting from meltwater (Finno, 1992). Therefore these deposits are rather erratic and are deposited in normally consolidated state. According to Dreimanis [1976] waterlaid soils tend to be uniform clays with little or no coarse-grained particles. The erratic water content and composition together with low OCR (overconsolidation ratio) indicate that the Blodgett till is supraglacial in origin. Otto [1942] attributed these low OCR values as the result of poor drainage at the lower boundary during deposition in combination with a relatively short duration of ice loading.

Finno was able to define relationships using the SHANSEP characterization of undrained shear strength (Ladd & Foott, 1974) for the Deerfield tills. Local practice in the Chicago area has developed such that undrained shear strengths s_u of these compressible clays are defined in terms of unconfined compression (U), unconsolidated undrained (UU), triaxial compression test results, or occasionally using a field vane. The U and UU tests are relatively unreliable methods of estimating s_U values (sample disturbance, variability of natural deposits. The normalized relationships for the Deerfield and Blodgett clays reported by Finno [1992] are based on CK_OUC (K_O Consolidated, Undrained triaxial compresion shear tests):

For triaxial compression:

$$\frac{s_{uTC}}{\sigma'_{vo}} = 0.46(0.90 - w_n)\, OCR^{0.9} \qquad (1)$$

For triaxial extension:

$$\frac{s_{uTE}}{\sigma'_{vo}} = 0.31(0.90 - w_n)\, OCR \qquad (2)$$

In the absence of extensive lab tests, these equations were used for estimating the reference values of undrained shear strengths for the slurry wall projects studied in this report (assuming OCR=1.1 for both Blodgett and Deerfield tills). Effective stresses were estimated by using hydrostatic conditions and soil unit weights as indicated or estimated from boring logs.

Figure 5.4 compares profiles of undrained clay strengths from unconfined compression tests with s_u values computed using the Finno equations (1) and (2). Figure 5.5 shows further comparison as a function of the effective vertical stress. Equations (1) and (2) generate a much smaller scatter in s_u compared to the UU test data.

In both Figures 5.4 and 5.5 there is a distinct breakdown of equations (1) and (2) in the stiff to very stiff clay and the hardpan layers existing below 50 feet depth (or σ'_{vo} > 3.0 ksf). The dashed lines below the depth of 50 feet (or 3000psf) contain the majority of the UU tests in these strata. Thus the equations (1) and (2) can only b e applied in the supraglacial and Deerfield clays. The Museum of Science and Industry project (C10: MSI) can clearly be distinguished in these two figures compared to the downtown Chicago jobs. Clay water contents for the MSI project were smaller than clay water contents in the downtown areas at the same depth or effective vertical stress.

Figures 5.6 compares water contents and unit weights for the Deerfield and Blodgett clays versus depth. These data show an increase of the total unit weight of the clay below 50ft depth with increasing depth the water content decreases.

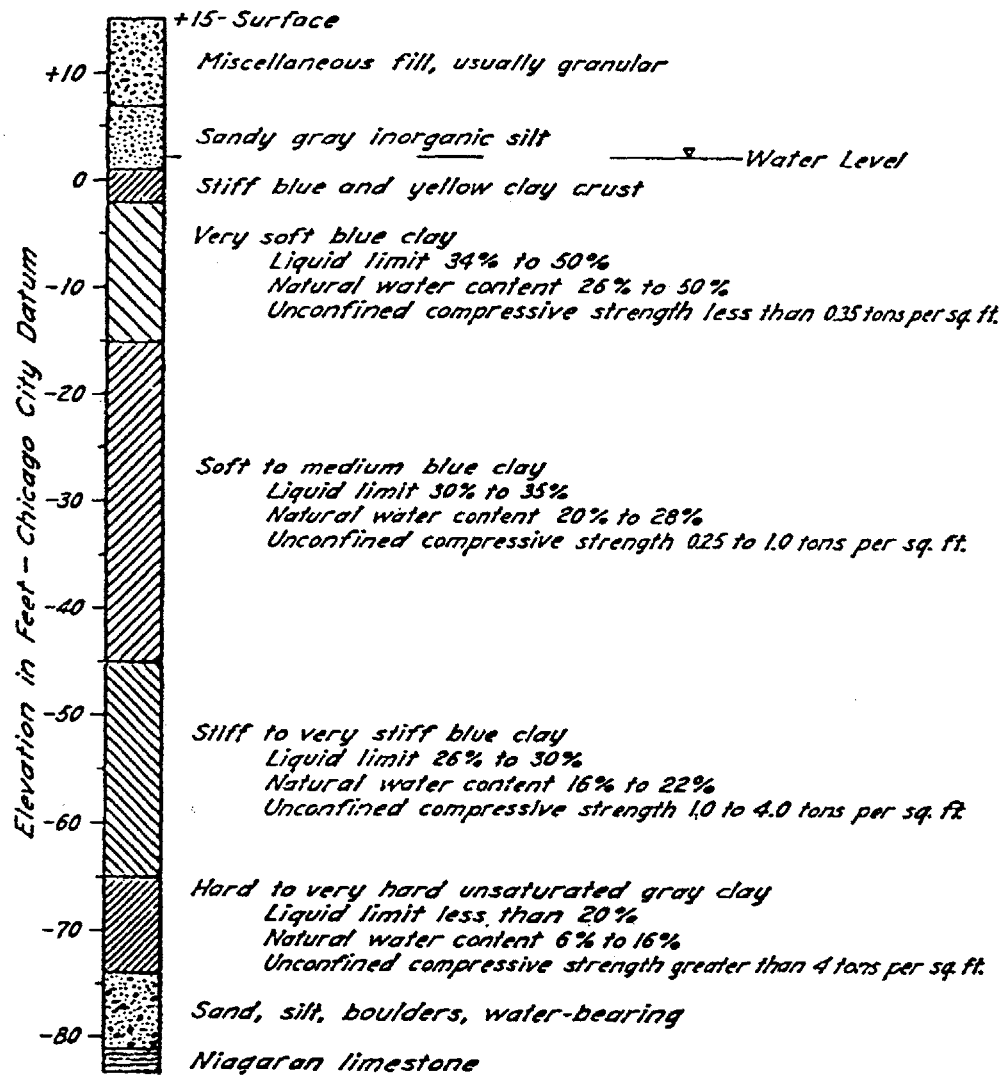


Figure 5.3: Typical soil profile in Downtown Chicago (Loop), (Peck, 1948).

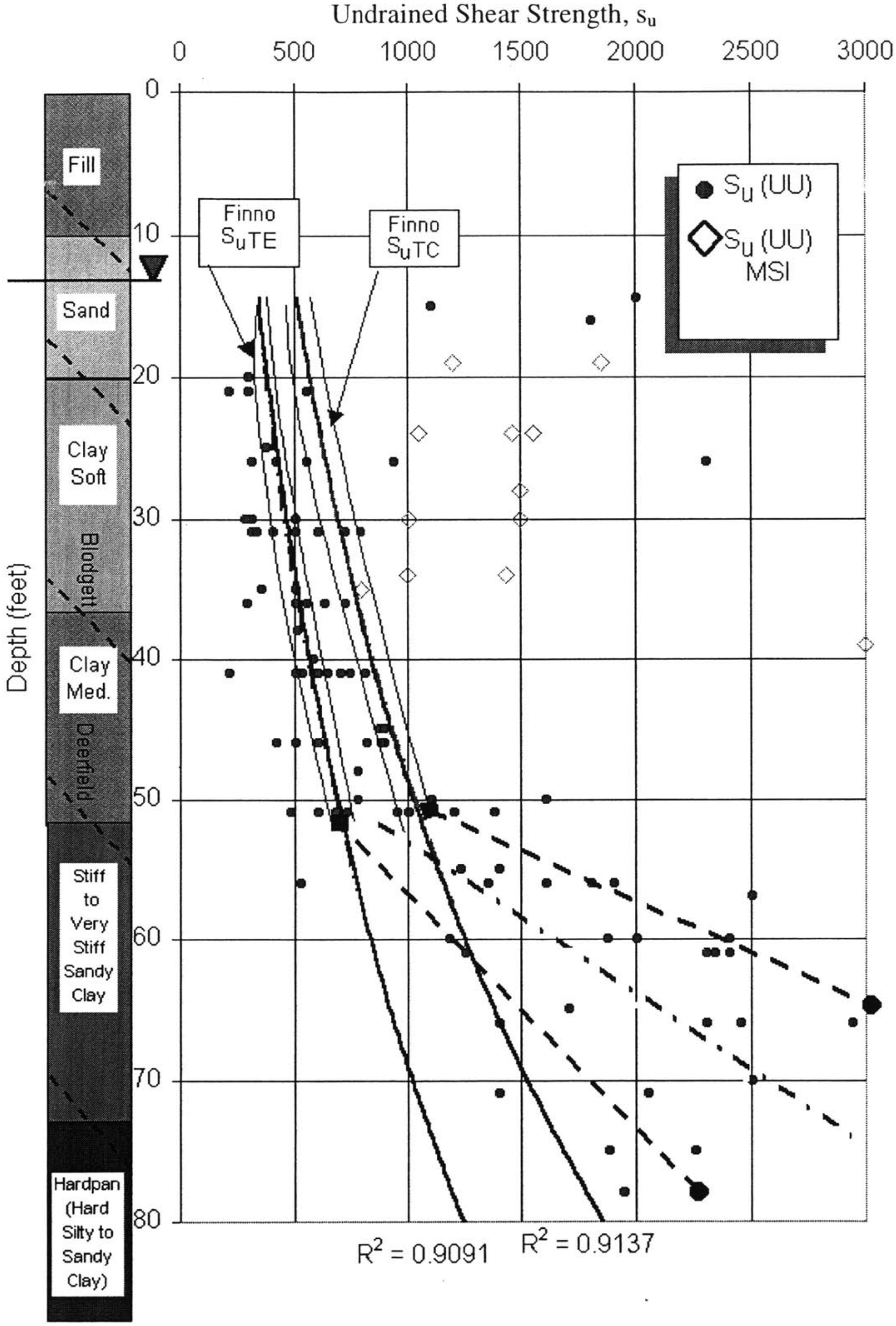


Figure 5.4: Comparison of s_u (UU) and equations proposed by Finno [1992], plotted against depth for Chicago projects

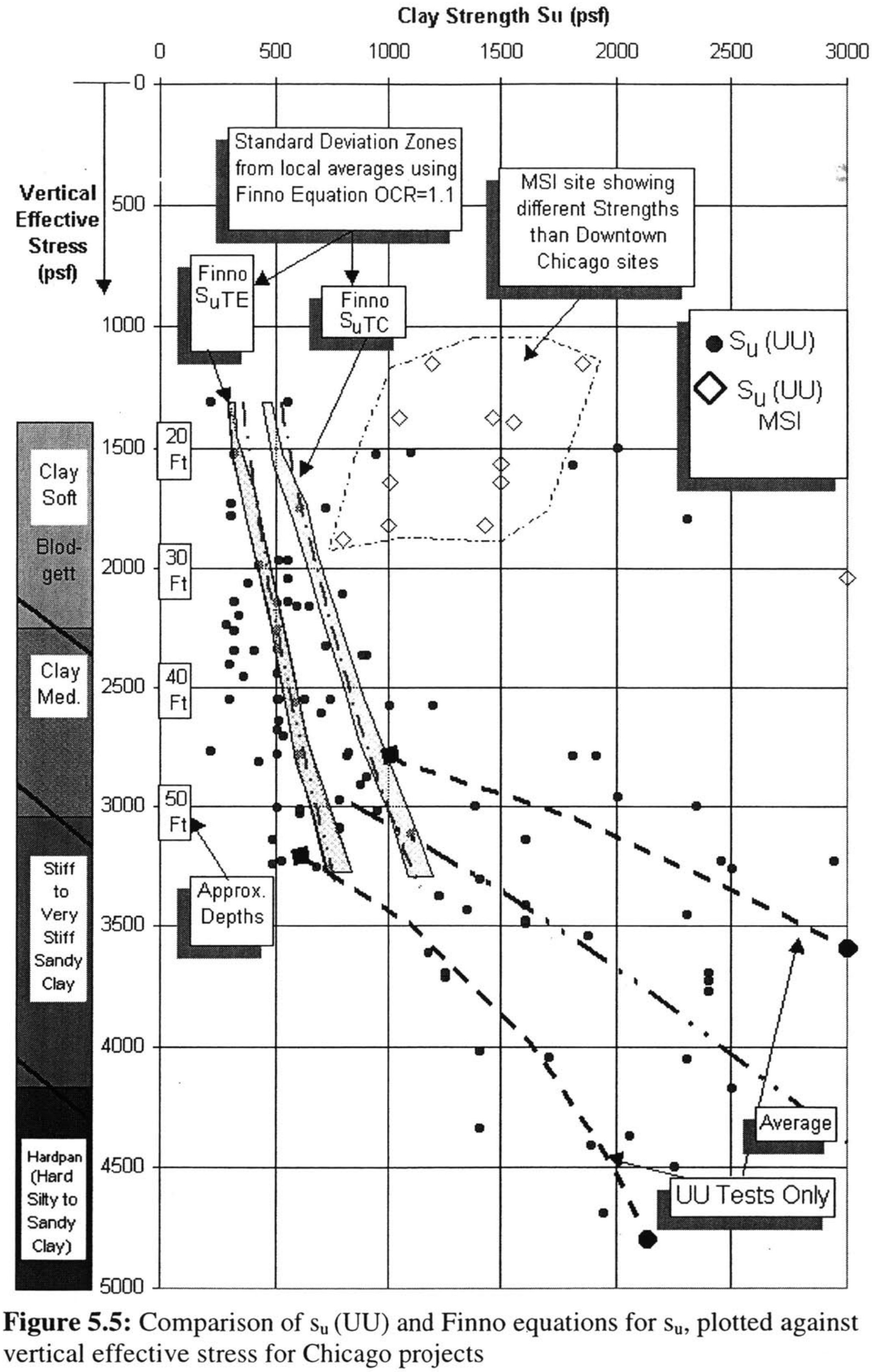


Figure 5.5: Comparison of s_u (UU) and Finno equations for s_u, plotted against vertical effective stress for Chicago projects

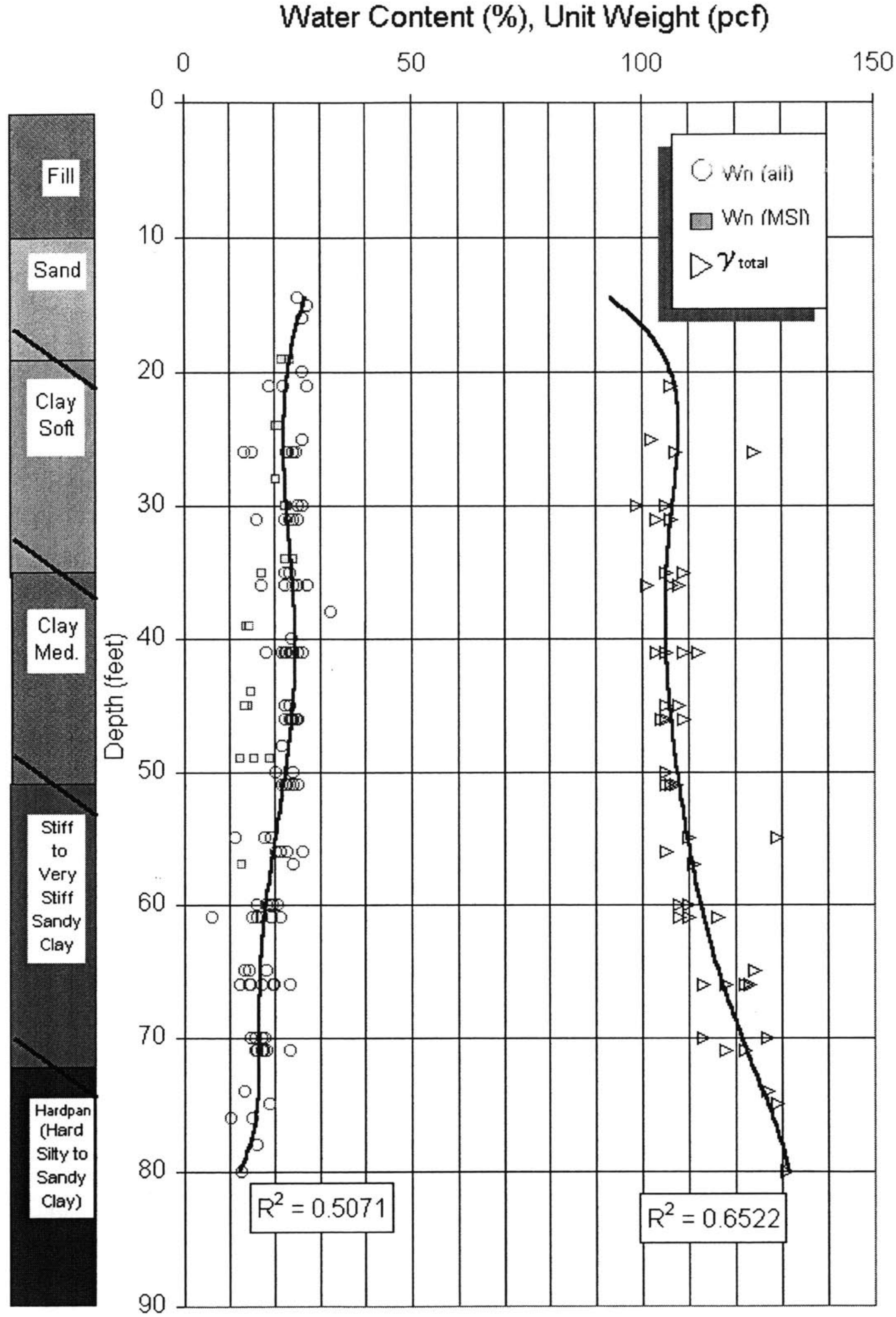


Figure 5.6: Water contents and unit weights plotted against depth for Chicago projects

5.3. Summary of Chicago Slurry Wall Excavations

Table 5.2 summarizes the measured performance from the database that has been developed for slurry wall projects in Chicago. The first of these projects was the CNA building in 1970. In an article from a local newspaper the slurry wall construction was mentioned as the method to overcome "mushy" soil (Chicago Tribune). The Sears Tower (C2) is currently the 2nd tallest building in the world. The main cluster of jobs is located in downtown Chicago, within the Loop and north of the loop near N. Michigan Avenue. Only the Museum of Science and Industry (C10) out of the eleven studied projects is located in south Chicago.

Rakers were extensively used to brace early slurry wall excavations in Chicago. In most of these jobs, the rakers were not prestressed as more reliance was placed on soil berms for wall support (C1, C2, C3). Caisson construction was a major issue in the majority of these early jobs since caissons were constructed without casings or in oversized holes (C1, C2, C3). Lateral soil movements caused by caisson construction totaled more than 3.0" in the Sears Tower (C2), 2.2" for the CNA building (C1) and up to 1.1" for the AMOCO building (C3). In the CNA building the slurry wall was constructed before the caissons whereas caisson construction was done prior to wall installation for AMOCO (C3). Caisson construction only caused up to 0.4" of horizontal movements for the Water Tower (C4), which was the latest of the early 1970's projects. In projects after 1974 caisson construction methods were changed and movements caused were decreased significantly.

The AMOCO building (C3) introduced a new concept in slurry walls at that time. The slurry walls for this project were supported by steel H-piles bearing on stiffer strata in order to decrease wall settlements and meet bearing capacity requirements on the slurry wall.

Slurry wall panels at the Water Tower project spanned across caissons that were designed to resist the entire soil pressures alone. Additional support was provided by one level of ties

Table 5.2: Summary of measured performance of slurry walls in Chicago.

ID	Year	Project Name	Soil Type	Excav. Bracing/ Excav. Method	Depth (ft) H	Depth (ft) D	Wall Thick (inch)	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Toe Fix.	Defl. Shape Type
C1	1970	CNA	Typ.	1-Lev CLB	31	33	30	3.3	5		---
C2	1970	Sears Tower	Typ.	3-Lev R, SB	32	24	30	6+	6+		---
C3	1971	Amoco Standard Oil	Typ.	1-Lev. TB SB	23 44*	15	24	4.6	--		IV
C4	1973	Water Tower	Typ.	1-Lev. TB, 1-Lev, R	44	22	24	2.5	1.5		I, II
C5	1993	Loyola University Business School	Typ.	1-Lev. CLB IB, Estimated	20 Est.	--	--	0.8	--		IV
C6	1986	Prudential Two	Typ.	1-Lev. TB 1-Lev. R	25	28	27	0.45	--		I
C7	1987	AT&T Corporate Center	Typ.	3-Lev. R, & CB	27	32	30	1.55	1.55		II
C8	1989	Guest Quarters Hotel	Typ.	3-Lev. TD	35	20 60	30	0.65	--	— √	I, VI
C9	1990	Northwestern University Memorial Parking Garage	Typ.	1-Lev. TB	23	25	24	0.45	--		I
C10	1997	Museum of Science & Industry	Out side Down Town	3-Lev. TB	34	10	30	0.8	--		I IV
C11	1987	311 South Wacker Drive	Typ.	TD, TB, R, IB	35	31	24	2.95 0.65	--		IV II

* At the center of the site

Maximum horizontal soil or wall movements caused only by the excavation ranged up to 3.5", but typically the range was up to 2.2" maximum. However, total horizontal deflections were much larger, especially for the earlier jobs (C1, C2, C3) as they reached $\delta_H \geq 6$" for the Sears Tower (C2). Most walls did not bend significantly, as many walls cantilevered and translated about their bases (C3, C5, C10, C11) (Type IV). The majority of the tieback and raker-supported walls showed some wall bending beneath the lowest bracing level (Type I), (C4, C6, C8, C9, C10). A few walls did bulge towards the excavation (Type II) (C4, C7, C11), but they represent the minority of the observed movements. In the Guest Quarters Hotel (C8), the wall at one location deflected the most beneath the base of the excavation. In projects were data was available, horizontal soil deformations were measured to a considerable depth beneath the toe of the wall (C3, C4, C5, C6, C7, C8, C9, C11). These deformations were in the order of 0.1" to 0.2" towards the excavation in projects with no major caisson construction problems (C5, C6, C8, C9). This suggests that in soft clays, small ground deformations occur well beneath the toe of diaphragm wall even when the excavation is relatively shallow.

Settlements for the early jobs were large, is some cases exceeding 6" (C1, C2, C3). Most of the total settlement was caused by caisson construction and not by excavation activities. For the CNA project (C1) settlements at an adjacent elevated train rail reached up to 5", with 2.8" inches occurring prior to any excavation. In the Sears Tower street settlements exceeded 6" with 3" induced by caisson construction. Maximum settlements were much smaller for the Water Tower (C4) since steel casing was provided for the caissons. For this later job maximum settlements reached up to 1.5" within 40 feet of the excavation and decreased at larger distances. Unfortunately, there were no settlement data archived for the rest of the projects.

In these early slurry wall excavations the most major issue was improper caisson construction and bracing design. Caisson construction caused most of the observed soil and wall movements. Over-reliance on soil berms for wall support and non-prestressed rakers increased deflections in these earliest projects. These kinds of difficulties could be expected since these jobs were amongst the first slurry wall projects in the United States.

Bracing in most projects during the 80's and 90's included tiebacks and/or rakers (C6, C7, C9, C10) with one excavation constructed by the top/down method (C8). The major issue in these slurry walls was to achieve adequate toe embedment into the stiffer clays at depths below 45 ft, in order to minimize translative displacement of the toe of the wall and increase the stability of the excavation. Soil movements were much smaller for these projects than projects for the 1970 to 1980 period. Caisson construction induced movements were limited to less than δ_H=0.5" and in most cases were less than δ_H = 0.3". In the AT&T project (C7) caisson and slurry wall induced soil movements combined to a maximum of δ_H = 0.5" with δ_H = 0.15" being caused by slurry wall installation. For the Guest Quarters Hotel (C8) caisson construction caused up to δ_H = 0.33" of wall movements. The 311 South Wacker project (C11) is special in that it used all the known types of bracing namely rakers, tiebacks or deadman, top/down, and cross-lot struts or corner braces. Not a lot of information exists about the C5 project but it is most likely that it was braced by one level of cross-lot struts.

Most of the observed soil movements occurred before or immediately after the first level of bracing was installed. For the AT&T center (C7) up to $\delta_V \leq 1.1$" of the total 1.5" settlement and $\delta_H \leq$ 1.0" out of 1.5" of horizontal soil movement occurred before the 1st level of rakers was installed. For Guest Quarters Hotel (C8) movements where the largest before the first level garage slab was installed.

In the Prudential Two job (C6) very small movements up to δ_H = 0.45" due to excavation were observed. In this job soil movements increased very slowly as the excavation progressed. Soil movements caused by slurry wall construction reached up to δ_H = 0.2 inches towards the open trench.

Top/down construction was very effective in minimizing horizontal soil movements. The Guest Quarters Hotel basement (C8) was constructed by the top/down method. The Museum of Science and Industry (MSI) (C10) can not be directly compared to the downtown Chicago excavations since clay strengths at the MSI site are higher than shear strengths of downtown clays (see Figs. 5.3, 5.4, 5.5, 5.6). In the NU project (C9) caissons were constructed in between slurry wall panels to support superstructure loads. Tiebacks braced the slurry walls in both projects, with the excavation depths were relatively shallow compared to other contemporary excavations in Boston (Chapter 4). In the MSI site tiebacks were permanent whereas temporary tiebacks were used for the NU job. Minimizing damage to adjacent structures as well as water seepage was key to both projects. The MSI garage was constructed next to the northern side of the existing museum, thus damage control was very important Slurry walls were embedded into stiff clays in both projects as to minimize movements.

5.4 Individual Case Studies

5.4.1 Case Study C1, CNA Center

The CNA Center is a 45-story steel frame building located at the northeast corner of Wabash Avenue and Van Buren Street The structure occupies the entire site with the dimensions at basement level being approximately 248 ft. in the north-south and 176 ft. in the east-west direction (Fig. 5.7). The building is supported in on 32 caissons extending to rock and partially on the slurry wall. The CNA Center was the first project in Chicago to use slurry walls for temporary and permanent earth retention for a two-level deep basement. The lowest top of slab elevation is El. –11.37ft CCD (Chicago City Datum), with the maximum cut being El. –17ft CCD for grade beams, and the street level at El. +14.5ft CCD (Fig. 5.8). Soil conditions at the site are typical of those within the Loop (Downtown Chicago).

A 30"-thick slurry wall was used along three sides of the site, the east, the south, and the west. The slurry wall along the eastern side of the site was designed to carry a vertical load of 25klf, or a bearing pressure of 10 ksf at the base of the wall. Along the southern and western walls the required bearing pressure was only 2 ksf. Therefore, the wall was designed to extend to El. –50ft CCD along the east side and El. –45ft CCD along the south and west sides.

Because of the favorable size of the lot and the desire to minimize ground movements, the wall was cross-lot braced during construction in the east-west direction, using permanent building steel at both floor levels (Cunningham & Fernandez, 1972). Along the south wall, a cantilever excavation was carried to the first level and braced at this level to carry the reaction of the sub-basement level slab, since the adjacent wall could not support the loads.

Lateral movements were measured by offset surveys at the top of the slurry wall cap beams. Final movements at the western wall were the greatest reaching up to δ_H=3.5". The slurry wall at that location also settled by as much as δ_V=3.6" at the final reading. Along the south slurry wall deflections reached δ_H =2" while wall settlement reached $\delta_V = 0.9$". In the eastern slurry wall, deflections and wall settlements were very small.

Settlement data were also taken at Elevated train columns on a survey line parallel to the west wall at distances of 35 and 51 feet (Fig. 5.7). The nearest column to the excavation settled by as much as $\delta_V = 5$" before it was jacked up by 3" in order to reduce differential settlement. In this column most of the settlement occurred during caisson construction ($\delta_V = 2.7$"). The vertical and horizontal movements of the west slurry wall increased considerably during slurry wall construction. The west wall moved in by 2.5" as the result of caisson installation while it settled by as much as 2.9" in the same period.

Cunningham and Fernandez [1972] report that the maximum ground settlement occurred not at the wall face, but at some distance away. At a distance of 2 to 3 times the excavation depth no movement was recorded. Maximum surface settlement was 0.45% and 1.60% of excavation depth, at the south and west walls respectively, and occurred at 0.50 to 0.70 times the excavation depth from the walls.

"Squeezing" occurred at two caisson locations along the west wall while drilling below the upper casing to such an extend that the ground surface around the caisson depressed in a dish-shaped pattern 2' or 3' deep, causing the rig to tip. These two caissons were the first to be installed and were probably the cause of the excessive movements observed along the western side of the excavation.

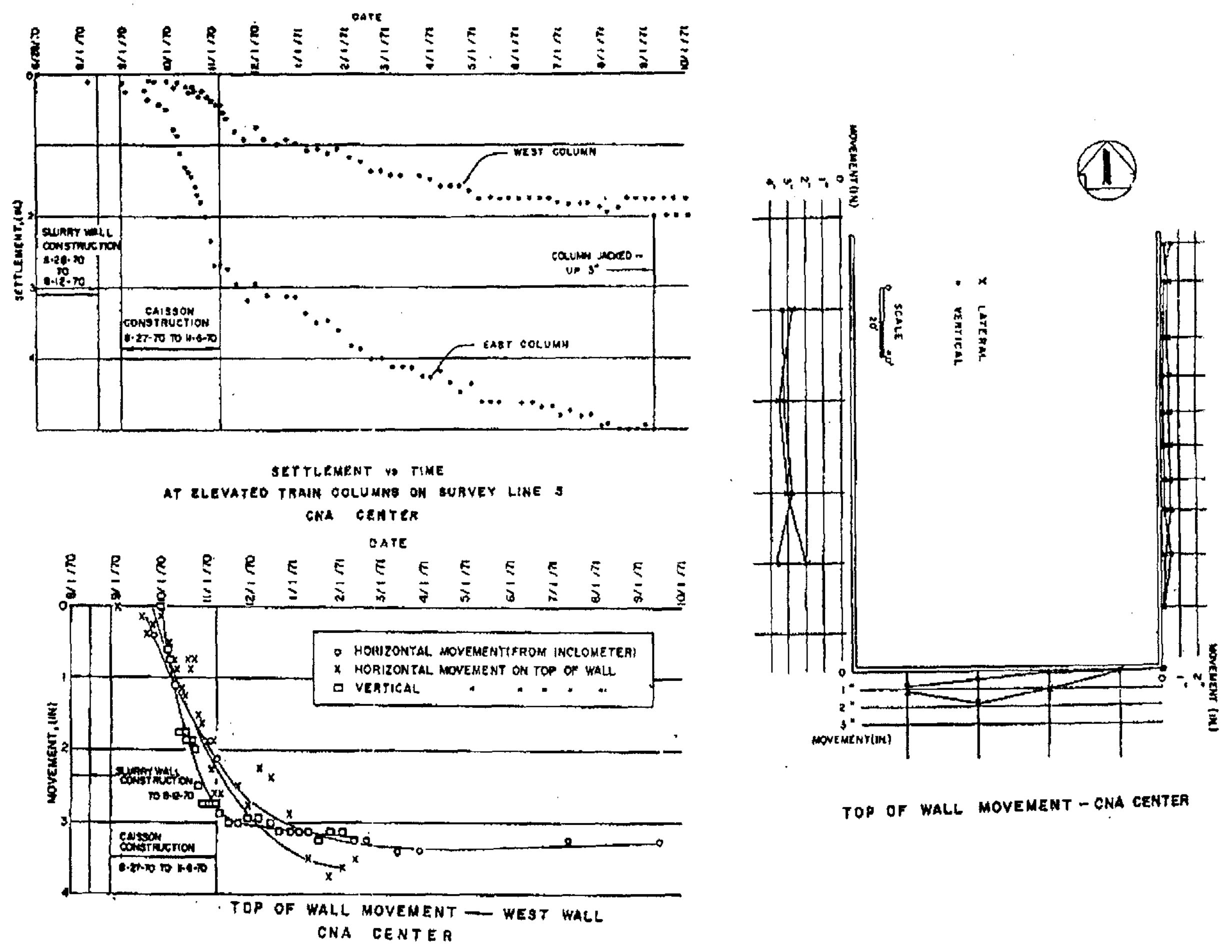


Figure 5.7: Excavation movements at CNA Center (C9) (Cunningham & Fernandez, 1972)

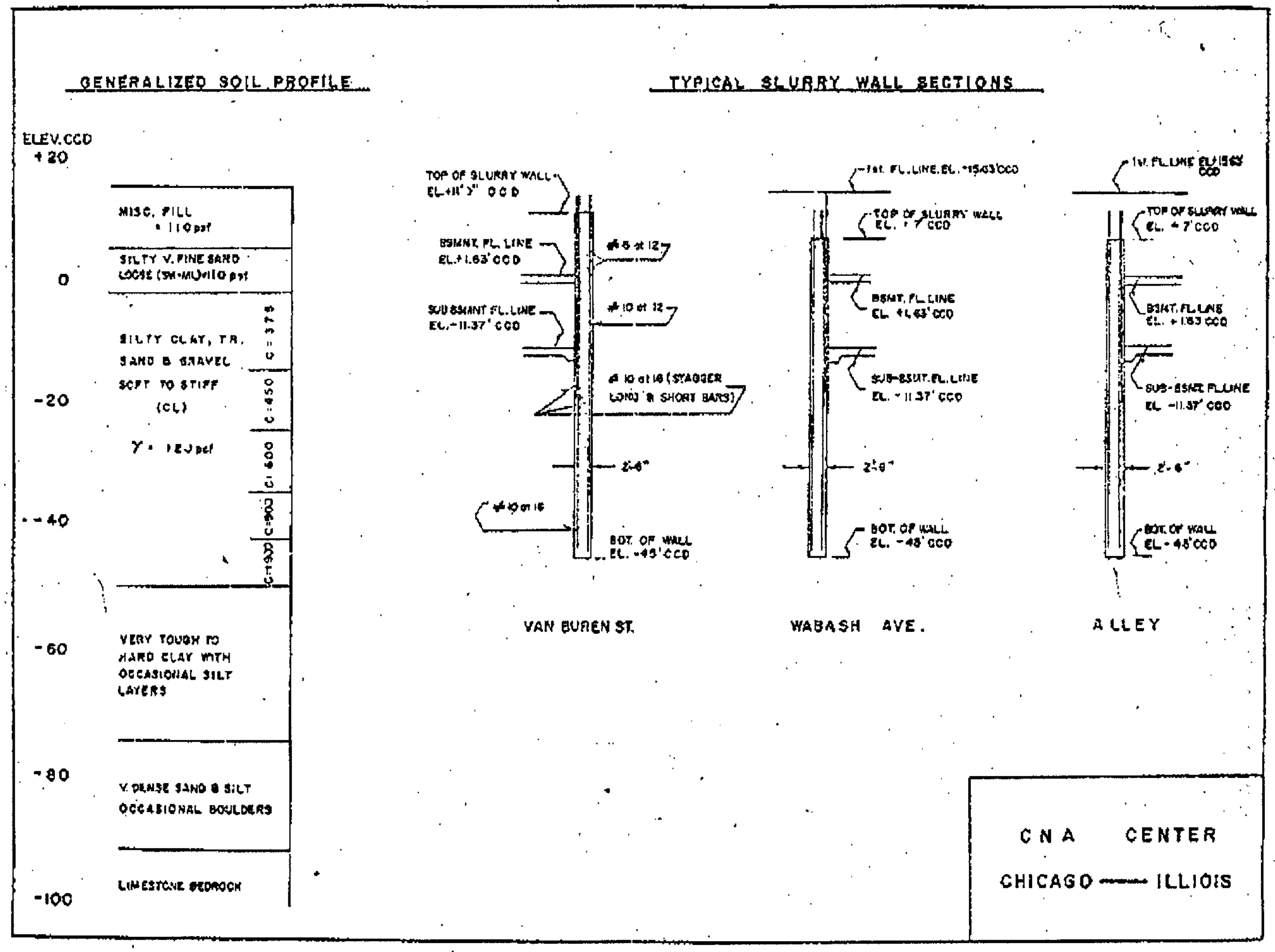


Figure 5.8: Typical slurry wall sections, CNA project C1.

5.4.2 Case Study C2, Sears Tower

The Sears Tower was among the first high-rises to use slurry walls to form the basement. There was little data available in this project since most of the recorded data was not archived. The site plan and a general excavation profile can be seen in Figure 5.9.

Cunningham and Carpenter [1975] reported measured raker loads as deduced from strain gages. Raker loads were generally smaller than design loads as determined by Terzaghi and Peck. The lowest level of rakers did not pick up as much load as initially designed. Design loads for most rakers were in the order of 485 kips, while the lowest level of rakers did not pick up loads more than 115 kips (Fig. 5.9). Maximum raker loads occurred at the end of the excavation.

No direct inclinometer or settlement data was left to retrieve in this project. However, indirect horizontal soil movements and settlement data could be retrieved from internal references and memos.

Caisson construction caused large soil movements along the northern side of the project. Horizontal soil movements at inclometers (similar to inclinometers) reached up to 6" in the north while Adams Street settled by as much as 3" during caisson construction. There is no direct mention of wall movements during the excavation. However, movements must have been large since Cunningham and Carpenter (1975) mention that the rakers displaced the base piles for a considerable distance out of plumb. Cracks along the northern slurry wall were observed with water running through fairly rapidly until it froze.

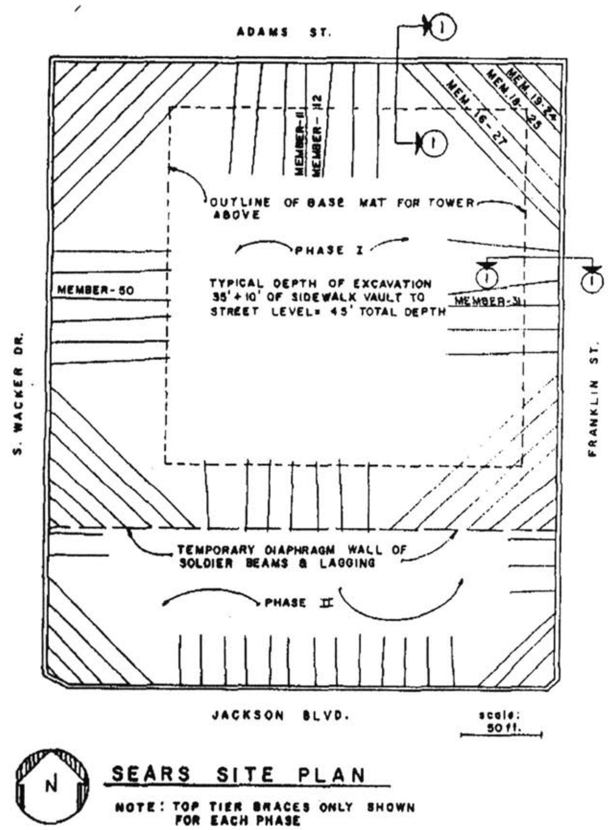
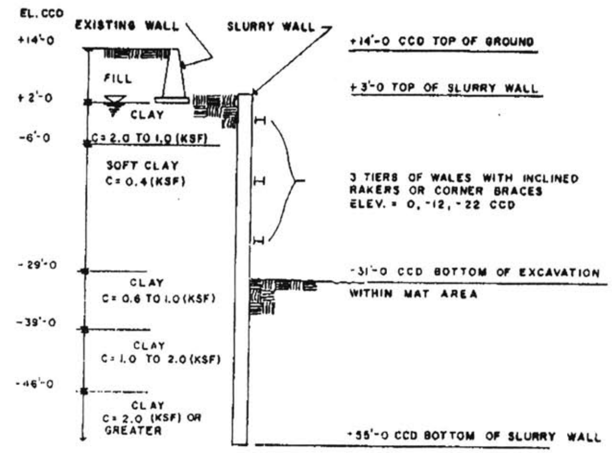


FIG. 1—*Site plan and brace locations for Sears Tower excavation.*



SEARS WALL SECTION I-I

FIG. 2—*Wall and soil profile for Sears Tower excavation.*

TABLE 1—*Sears Tower brace loads.*

Mark	Location	Measured, kips	Design, kips
Adams	No. 12 top tier	406	405
Adams	No. 12 mid tier	250	390
Adams	No. 12 bottom tier	73	362
Franklin	No. 31 top tier	295	425
Franklin	No. 31 mid tier	206	461
Franklin	No. 31 bottom tier	47	405
Adams	No. 11 top tier	461	405
Adams	No. 11 mid tier	334	390
Adams	No. 11 bottom tier	126	362
Wacker	No. 50 top tier	117	250
Wacker	No. 50 mid tier	221	400
Wacker	No. 50 bottom tier	31	381
16–27	N.E. 1st (top) tier	636	500
18–25	N.E. 2nd tier	480	556
19–24	N.E. 2nd tier	350	486
19–24	N.E. 3rd tier	111	486

Figure 5.9: Sears Tower site, excavation profile, and measured bracing loads (C2), (after Cunningham, and Carpenter, 1975).

5.4.3 Case Study C3, AMOCO (Standard Oil) Building

The Standard Oil Building is an 80-story steel "tube" building placed near the center of a 5-story reinforced concrete plaza structure. There are two separate buildings although they function as a single unit. The project is located at 200 East Randolph Street in Chicago, Illinois, covering the entire city block bounded by Randolph Street, Stenson Avenue, Lake Street, and Columbus Drive on the south, west, north and east (Cunningham & Fernandez, 1972). The project consists of a high-rise and a low-rise portion that are supported separately due to dissimilarity of loading and boundary conditions. Caissons extending to hardpan and rock were used to support these two areas.

Slurry walls were used in conjunction with steel piles were used to construct the basement for the AMOCO building. The piles were used as load bearing elements for the slurry wall and not as wall reinforcing. The slurry wall scheme was adopted after caisson construction had already been built when a restraining order was issued to halt construction. The slurry walls were 21" thick and extended from El. +8.0ft to El. –32ft CCD (Chicago City Datum), and bracing consisted of one level of temporary tiebacks at El. +1.5ft (Fig. 5.10).

Movements at the top of the slurry wall reached up to 4.6" at one station (Sta. #2) while other inclometers measured deflections close to 2" at the top of the casing (Fig. 5.11). Cunningham and Fernandez [1972] reported deflection data 10' below the top of the casing in order to minimize surface effects. Soil movements started as soon as the center of the site was excavated to El –26ft CCD even while a 25' wide berm was left in place. Apparently the berm was not sufficient to restraint deformations since movements increased by as much as 0.8" during this period. Movement continued as the perimeter was excavated to El. –15ft CCD. Movements did not stabilize until some time after removal of the berm was complete. Although tiebacks were restressed on several occasions, actually pushing the wall back somewhat, the average rate of movement did not seem to change. This indicates that re-stressing of the tie-backs provided only temporary restraint to the movement that was initiated by a change in the stress equilibrium of the soil mass, and the movement stopped only after a new state of equilibrium was reached (Cunningham & Fernandez, 1972). Most of the slurry walls translated and rotated about their toe, with very little to no wall bending evident. The horizontal toe translation was in the order of 1.0".

Considerable difficulties were experienced in compacting the backfill between the slurry wall and the deadman sheetpile wall (installation of tieback rods), due to freezing weather and/or water. This was particularly true along the east and south walls, and resulted in relatively poor compaction of the backfill with the exception of a 20-ft strip in front of the deadman wall, where select and relatively dry material was utilized. It is possible that large movement along the east wall may have been partly due to this condition.

Significant lateral movements were observed to a considerable depth. In stations 2, 3, and 4 the slurry wall had clearly rotated about its base since there was no sign of bending at any elevation higher than El. –32ft CCD. This clearly indicates that the embedment of the slurry wall was not enough to restraint movement of the stiff slurry wall. As it is shown in Figure 5.10, the slurry walls were embedded in medium clay; clearly this clay did not possess enough strength to resist the lateral loads imposed on the slurry wall.

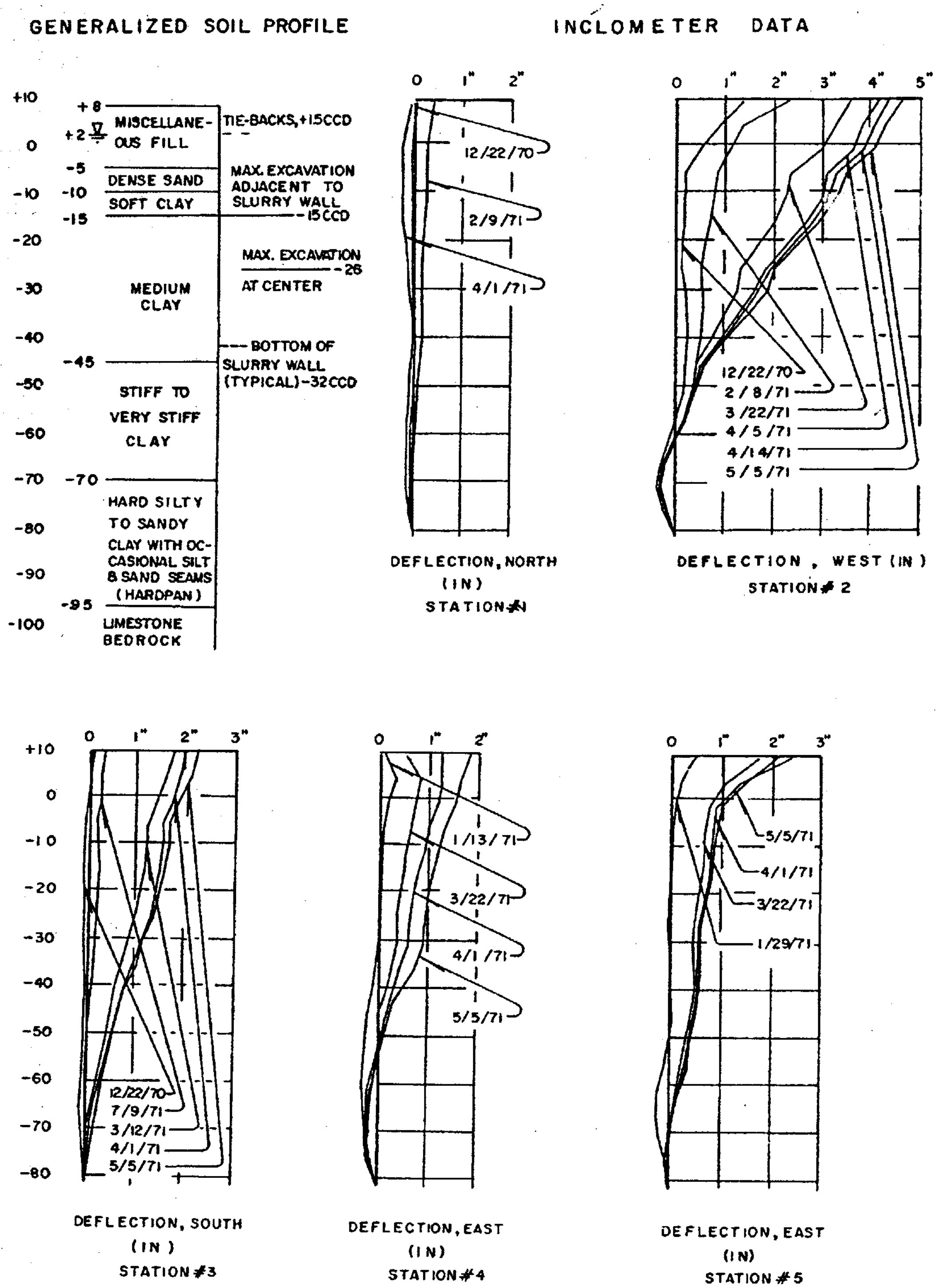


Figure 5.10: Generalized soil profile and inclinometer data for AMOCO (Standard Oil) building (C3), (Cunningham & Fernandez, 1972).

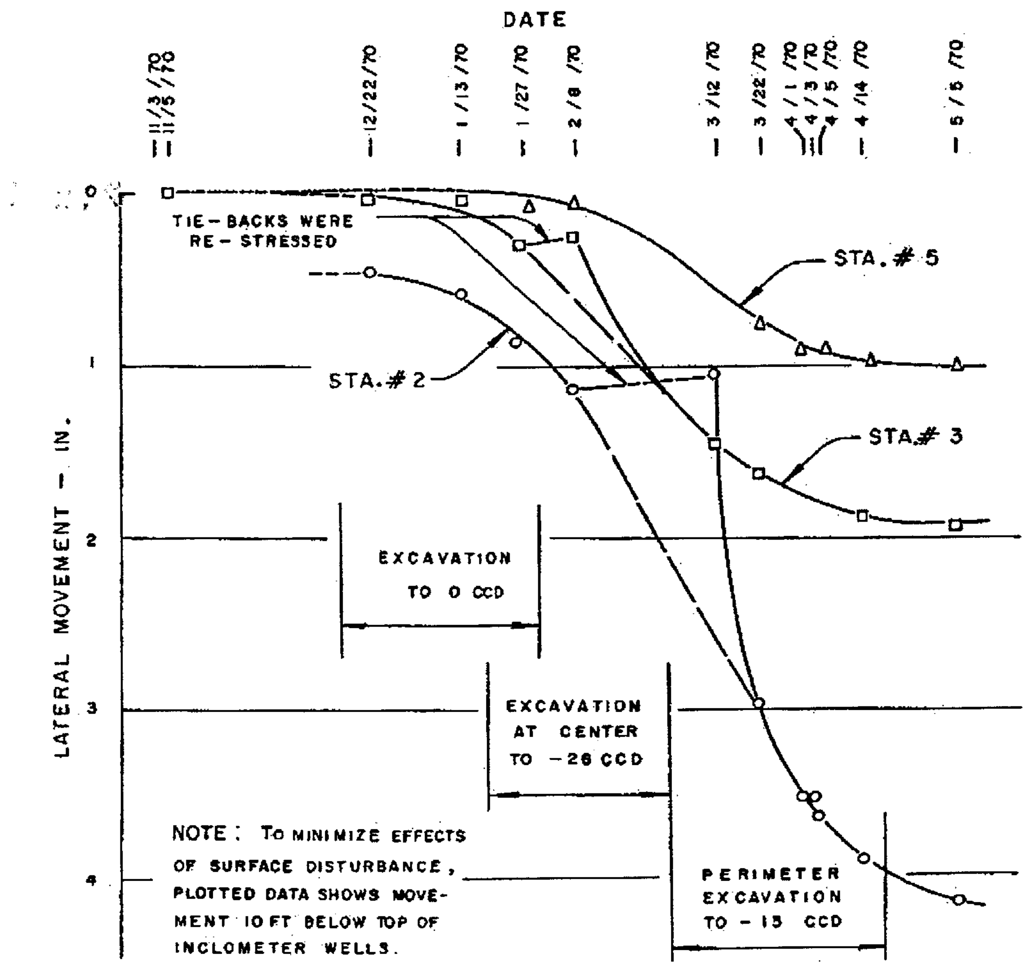


Figure 5.11: Horizontal deflections vs. time, AMOCO (Standard Oil) Building, (Cunningham & Fernandez, 1972).

5.4.4 Case Study C4, Water Tower

The Water Tower Place consists of a 74-story tower with a contiguous 14-story commercial building, both of reinforced concrete and with four levels of underground parking. The site is located at the northeast corner of Michigan Avenue and Pearson Street The excavation for the parking covered an area of 530' by 212' and extended to 44' below the surface.

Slurry walls 30"- thick, 62 ft deep, spanning between the shafts of foundation perimeter caissons were used as the basement walls. The caisson shafts were designed to carry part of the lateral load. Two levels of bracing were provided: one level of tiebacks at 11' from the top of the slurry wall, and a lower level of inclined rakers at 27' below the top. The slurry wall was designed to span between the perimeter drilled caissons. The drilled caissons supporting the slurry wall are spaced 30' to 31' of center, most having a 5'-diameter and an 11' bell at the bottom. The caissons were designed to resist part of the earth pressure and act as cantilevers. Slurry wall panels, 25' to 26' long, were constructed in between the caissons. Details of the slurry wall excavation can be seen in Figure 5.12.

Soil conditions in the site were typical of downtown Chicago: 7' of fill, 18' of fine sand (SP), 30' of soft to tough silty clay (CL), over very tough to hard clay (CL), and hardpan. The soft clay has undrained shear strength $s_u = 0.4 - 1.0$ ksf, increasing with depth.

The tiebacks were inclined from 10.3° to 17° in order to obtain a grouted zone of 10' to 20' within the sand stratum. All ties were regroutable with a design capacity of 60 Tons each. Five tiebacks and two rakers were installed in each slurry wall panel. The rakers had a design capacity of 225 Tons each, with one raker placed at a drilled caisson and another placed midway between caissons.

Construction operations involved some overlapping but generally followed the below sequence:

I. Site excavated to –5'.

II. Inclinometer installation outside of wall.

III. Foundation caisson installation.

IV. Slurry wall installation and wall inclinometer installation immediately.

V. Site excavated to –12'.

VI. Tieback installation, five ties per panel. Each tieback was preloaded to 80 Tons, which was maintained until no movement occurred. Then the load was reduced to 60 Tons and locked off.

VII. Site was excavated to the final level (-44') leaving a berm having a minimum dimension of 40' from the face of the wall. The berm top was at –28'and it was at least 23' wide at the top.

VIII. Base concrete slab was placed in the central excavated area with a deadman beam on its edge.

IX. Rakers installed maintaining a symmetrical bracing configuration for balanced load transfer across the slab. Each raker was prestressed to 50 Tons.

X. Excavation of the berm in segments and placement of the base slab up to the slurry wall

XI. Alternate rakers supporting drilled caissons were removed

XII. Third level basement construction and removal of remaining rakers.

Out of the 174 ties, 14 did not obtain the design capacity and had to be regrouted to give acceptable results. All ties were locked off at 60 Tons but load picked up from 63 Tons to 75 Tons. Tieback loads remained at prestress levels while the berm was not removed. All tiebacks were cut-off once the ground-floor slab was in place. The average maximum raker load was 200 Tons, slightly less than the 225-Tons design capacity. Readings taken at the longest diagonal strut at the northeast corner showed an axial load varying from 31 Tons to 181 Tons, with a design load of 320 tons. It was believed that soil arching partly caused the smaller observed load.

Offset surveys indicated that maximum wall movements at the top of the wall were in the range of 1" to 3". Lateral movements as measured from the surveys were smaller than inclinometer measurements at the top of the wall. Inclinometer deflection data is displayed in Figures 5.13 & 5.14 (Gnaedinger et al., 1975). Maximum soil movement as determined by inclinometers was slightly more than 6", but most were less than 2" throughout the depth of the inclinometer casing. These large movements at the top the casing most likely reflect fill sliding under the guidewalls or other surface construction activities. The bottom of the slurry wall translated by as much as 1.2" towards the excavation, which emphasizes the importance of adequate embedment into a stiff stratum for controlling such deformations.

Wall movements were more or less observed throughout the wall perimeter. This indicates that the wall actually spanned across the drilled caissons as designed. Lateral movements were increasing as the excavation progressed. The effect of tieback prestressing can be seen in the sharply bent deflection shapes at the tie elevations. Raker prestress did not seem enough to prevent the bulging deflections that occurred in most locations. Apparently the raker prestress level was not high enough to prevent such a condition.

Under 200 tons with a 50-ton prestress the calculated elastic yielding is close to 0.21" for each raker. This means that the average maximum stress on each steel pipe was 14 ksi. This calculation was made assuming that the rakers were under compression only. Apparently the walls deflected much more than the 0.21" at the raker elevation, which suggests that either the base slab shortened under the raker load or that the rakers actually slipped to accommodate for the typical 2" observed movements.

Settlement observations were only made at distances greater than 20' to 40' from the wall. At points within 40' of the wall settlements were in the order of 1.5" on Pearson, Seneca, and Chestnut Streets, and 0.5" on Michigan Avenue. On Chestnut and Pearson Streets maximum settlements at 60' from the excavation were about 1inch. At the same distance the maximum settlement at Michigan Ave. and Seneca was close to 0.5". At this project there were little to no movement during caisson construction since the perimeter caissons were provided with steel liners, and cement grouting was used to seal the space between the liner and the excavated hole.

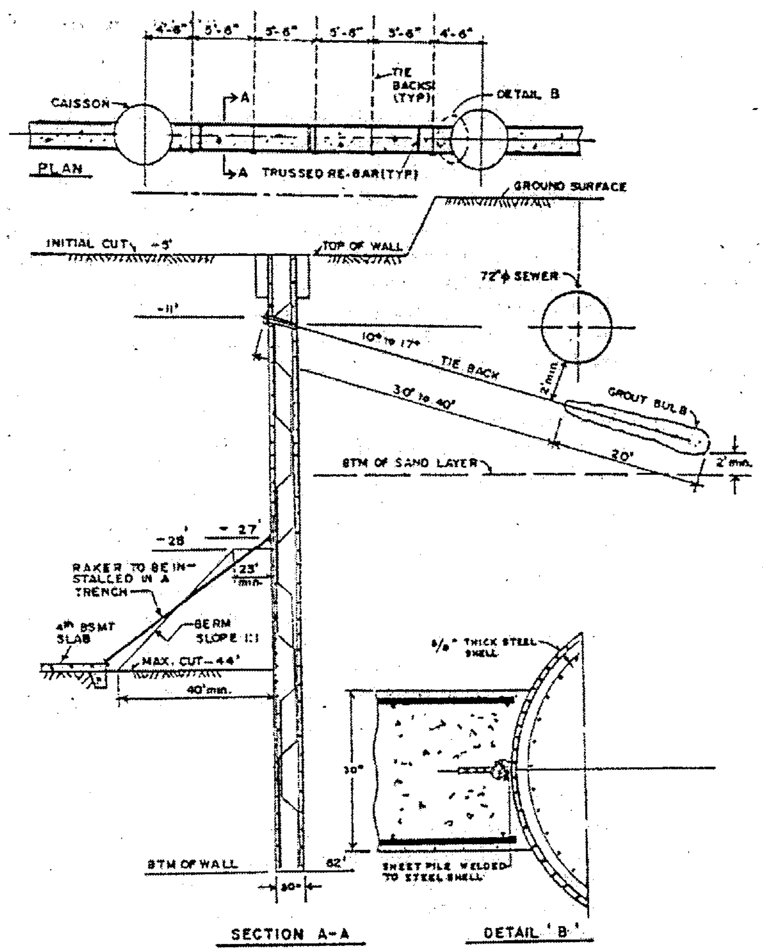


Figure 5.12: Slurry wall and excavation details after Gnaedinger et al. [1975]

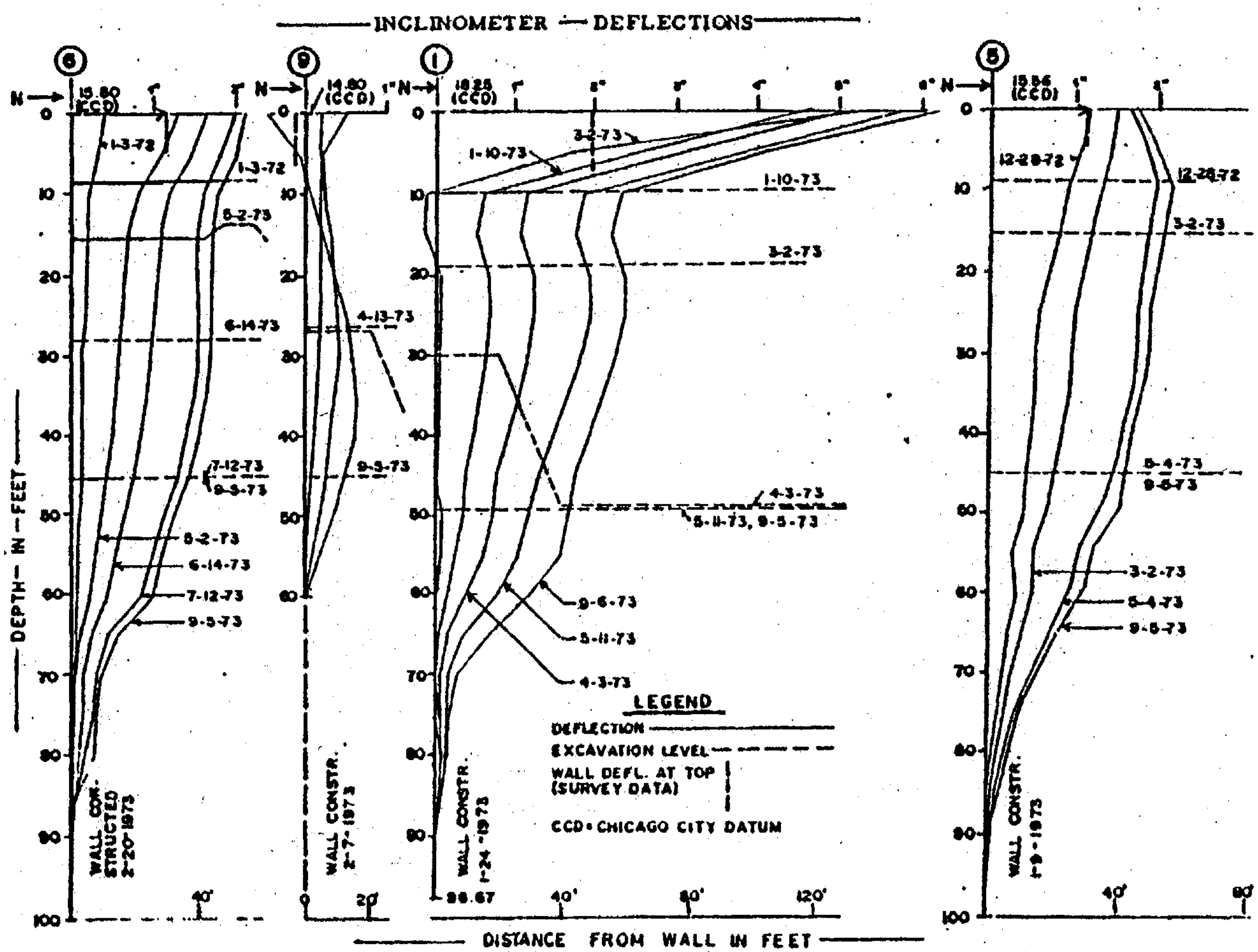


Figure 5.13: Inclinometer data, after Gnaedinger et al. (1975)

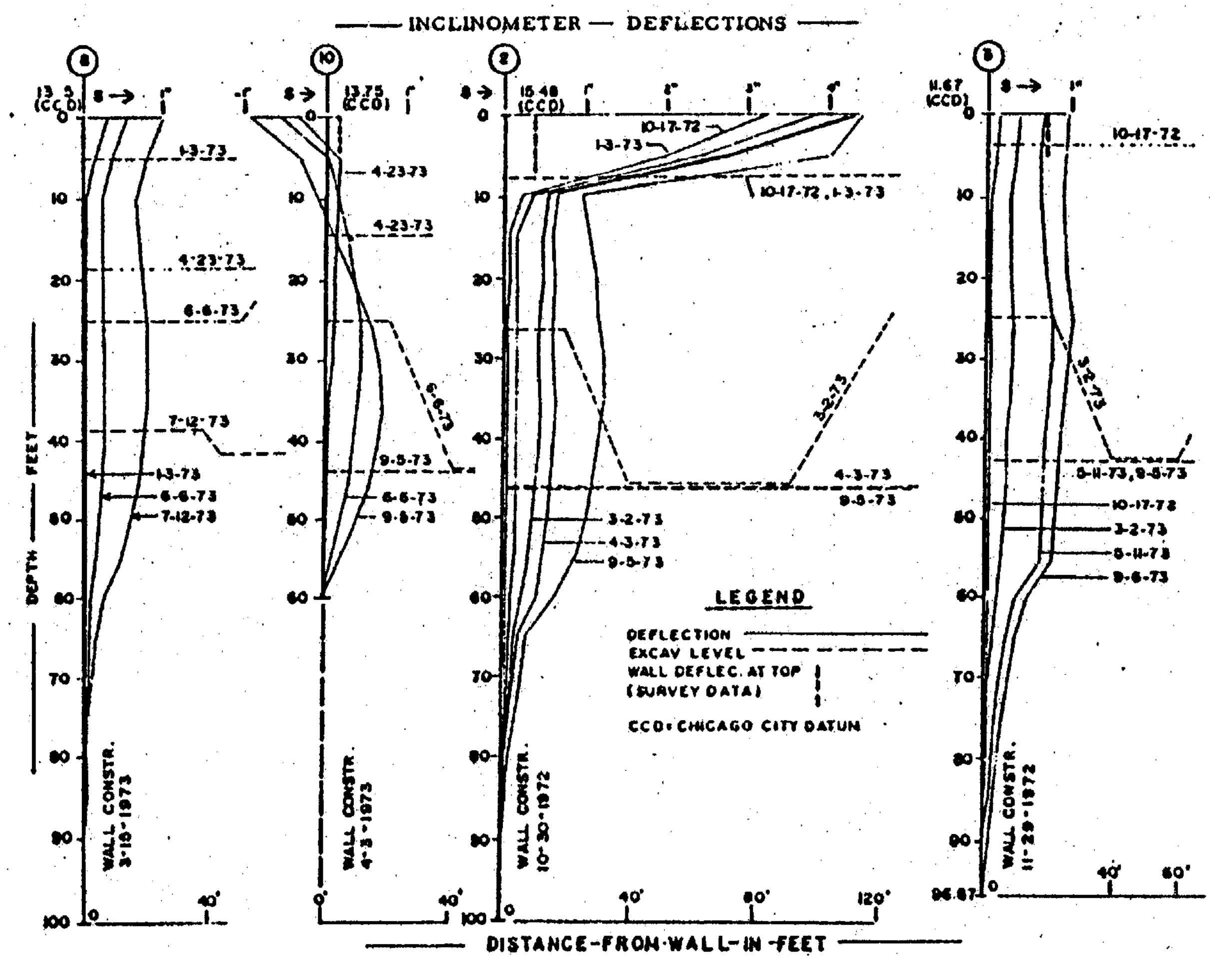


Figure 5.14: Inclinometer data, after Gnaedinger et al. (1975)

5.4.5 Case Study C5, Loyola University Business School

Very little data archived data exists regarding this project. Field reports mentioned that one level of struts was used but the excavation profile could not be reconstructed. The wall thickness in not known either, nor have any settlement data been preserved. Inclinometer data showed cantilever deflections reaching up to 0.8" at the top of the wall, and translational movements in the order of 0.2" at near the toe of the wall. When permission was obtained to publish data for this project, the owners mentioned that they had long term leakage problems with this type of construction, which they had to repair.

5.4.6 Case Study C6, Prudential Two

The Prudential Two is a 60-story high-rise located in the southwestern corner of Stenson Avenue & Lake Street. The new building was constructed next to the pre-existing 41-story Prudential building at Randolph & Stenson Streets. Five levels of parking were constructed under the tower portion of the site. Three out of these parking levels are below the lower street level at Stenson Street (eastern side). Stenson Street has three levels with the lower being at El. +6'2"CCD, the mid-level at El. +26'CCD, and the upper at varying elevation close to El. +46'ft. To the north of the project there is the 80-story AMOCO (Standard Oil) (C3) building constructed during the 70s with its basement constructed using slurry walls as well. A plaza area occupies the northwestern portion of the site.

Slurry walls 26"-thick, 56'-deep, formed the walls for the 3-level basement (base of the excavation at El. –21'3"ft CCD). Tiebacks, rakers, and corner braces were used to brace the slurry walls during excavation (Fig. 5.15). On the north wall a tie-rod and deadman sheetpile system was used in place of the simple tieback bracing because of space restrictions. Tiebacks at 6' depth, inclined at10° from the horizontal and prestressed rakers to 100 tons placed at the second level provided additional support for the east and north slurry walls. The south slurry wall was braced by tiebacks inclined 45° at El. –10ft CCD at the slurry wall face, with fixed length within the hardpan stratum.

Steel H-Beams placed at regular intervals reinforced the slurry wall together with reinforcing steel cages that provided additional tensile strength. Drilled caissons extending to the limestone bedrock were used to support the superstructure loads from the 60-story tower.

Geologically the site lies in a portion of the city that was once below the lake level. The soil profile generally consists of fill, underlain by fine sand, stiff clay, soft to medium clay and stiff to very stiff clay extending to a sandy clay "hardpan" at El. –70ft approximately. Extremely dense silt with variable amounts of cobbles and boulders was found above the surface of bedrock to elevations in the range of El. –90 to El. –100 ft. A deep groundwater table (El. –60'CCD) exists at the site and is associated with the water level in the rock and the dense silt below the hardpan. A shallower perched water level within the sand soils ranged from El. 0' to El. –3'.

During slurry wall and caisson construction soil moved towards the slurry filled trenches by as much as 0.25" inches. During excavation construction, soil moved towards the excavation by as much as 0.5". There was very little wall bending observed in this project, with the wall also slightly translating towards the excavation by 0.15" at its base (Fig. 5.16).

In one inclinometer 25' away from the excavation (In-1A), the upper few feet of soil moved up to 0.75" towards the excavation while 5' from the excavation they reached 0.3". This effect was caused by the deadman sheetpile and tie configuration that was used along the northern project side.

Horizontal soil movements slowly increased while the excavation progressed. Generally most of the movement happened after the soil berm was left in place and the first level ties were tensioned. The final inclinometer reading was taken when most of the rakers were jacked.

Slurry wall construction faced only one major problem regarding the tilt of the slurry wall reinforcing steel H-members during or after panel concreting. Reinforcing cages had to be shortened in many cases because of the large bending of steel H-beams in slurry walls during or after concreting. With the exceptions of some sand pockets in the sand upper sand layer slurry wall construction encountered no other difficulties. There were no records of any slurry wall leaks or any other special observations regarding slurry wall quality found.

Overall, the earth retention system performed very well since the basement excavation caused very small horizontal movements and probably small settlements as well. Actual settlements although not measured but must have been small as well if we consider that maximum lateral soil movements reached up to 0.75".

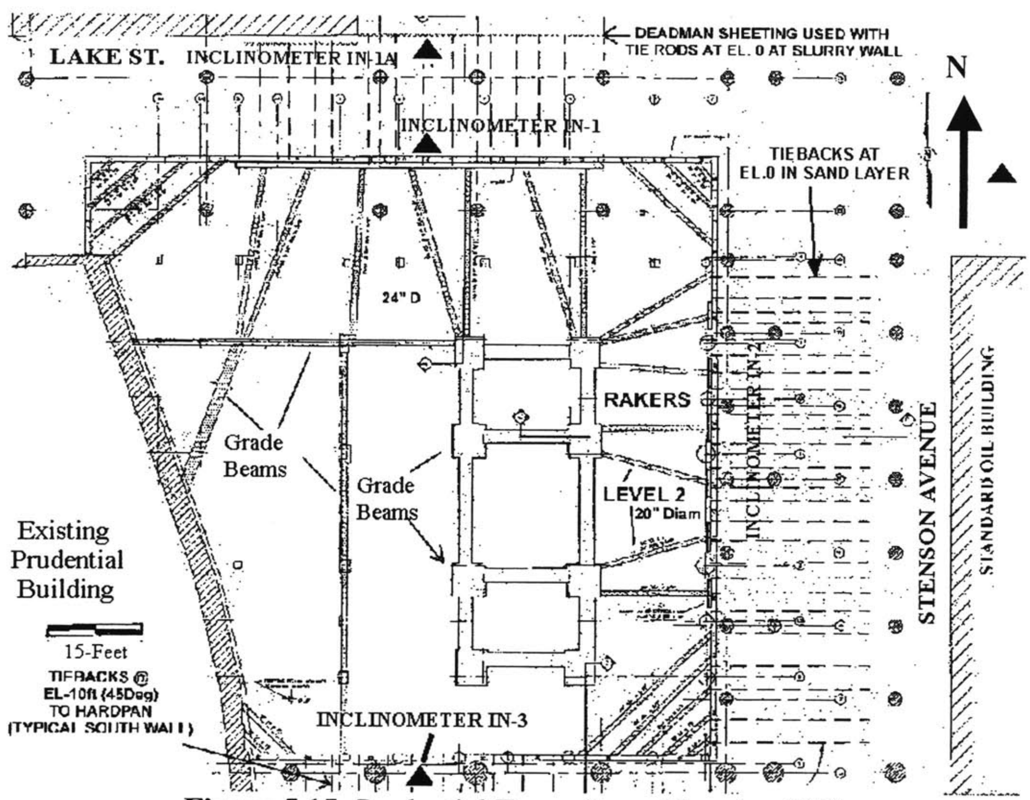


Figure 5.15: Prudential Two, site and bracing (C6).

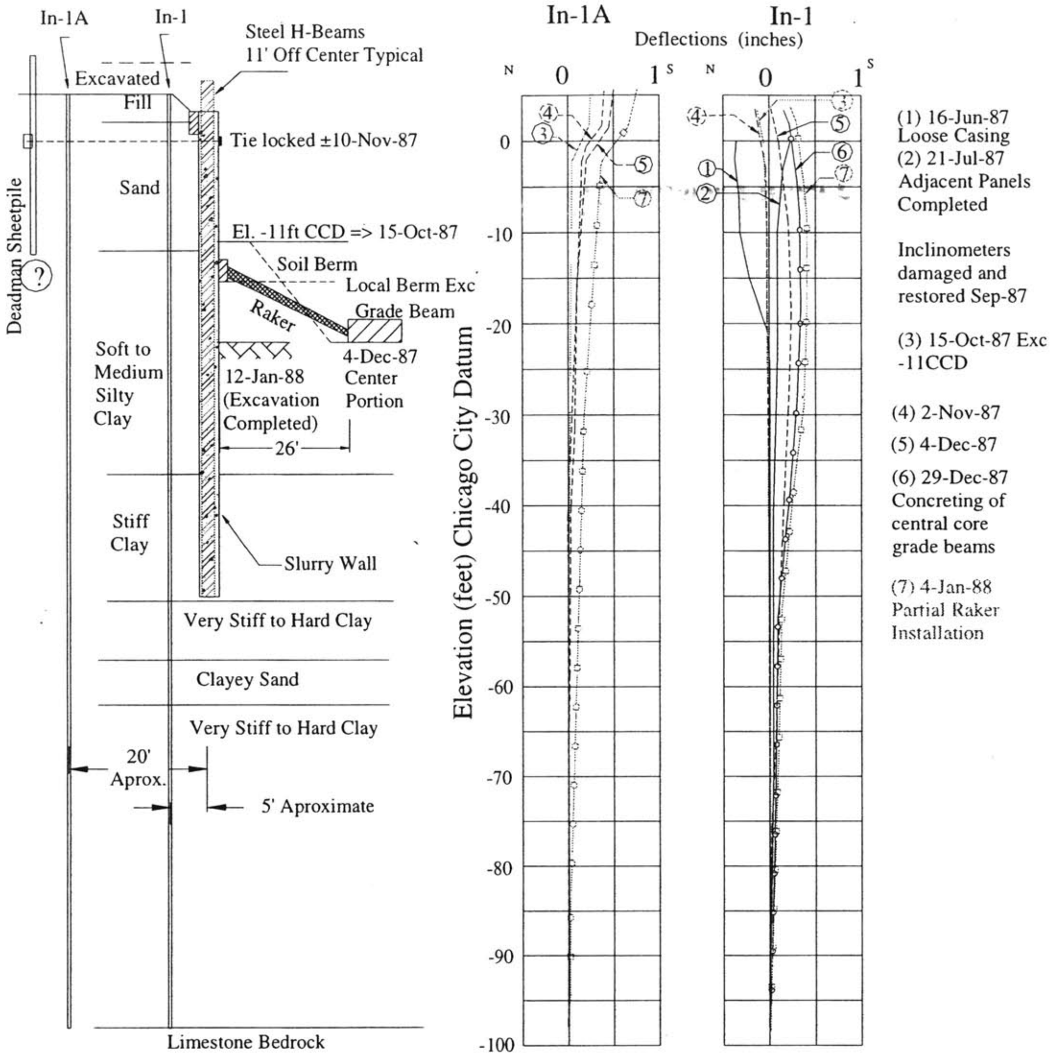


Figure 5.16: Inclinometers In-1 & In-2 (Northern wall), Prudential Two (C6).

5.4.7 Case Study C7, AT&T Corporate Center

The AT&T Corporate Center is a 60-story composite office tower with an attached 16-story low-rise and two basement levels. It is located at the southeast corner of Monroe and Franklin Streets. The presence of an adjacent 10-story structure supported on footings over soft clay required a conservative slurry wall design, construction and bracing. Subsurface conditions consist of fill, thin clay crust, soft to medium clay, stiff to very stiff silty clay, hardpan, and water bearing very dense silt-sand & gravel, which is typical of downtown Chicago.

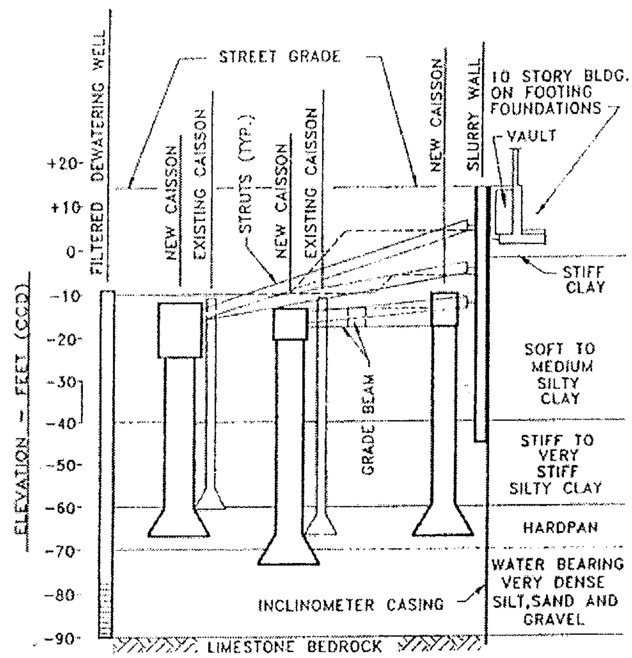
A principal concern in this project was the effects of excavating a 2-level basement on the movements of the 10-story building on the east (Fig. 5.17). It was felt that settlement of this building would be anticipated, regardless of whether the building was underpinned or whether a stiff retention system was adopted (Baker et al., 1987). A 30"-thick slurry wall constructed in 7' to 9' long panels was chosen on the east side of the project. On the other three sides a 2' thick slurry wall was used constructed in normal 20' to 22' panel lengths. The slurry wall extends from street grade at El. +14 ft CCD to elevation El. –45ft CCD.

Three levels of rakers were used to support the east slurry wall. These rakers rested on the caisson caps (Figure 5.17). Raker installation was accomplished with the use of soil berms along the east wall. Two inclinometers were installed to monitor movements along the east wall. One inclinometer monitored movements on the western side along Franklin Street where there was a major sewer line.

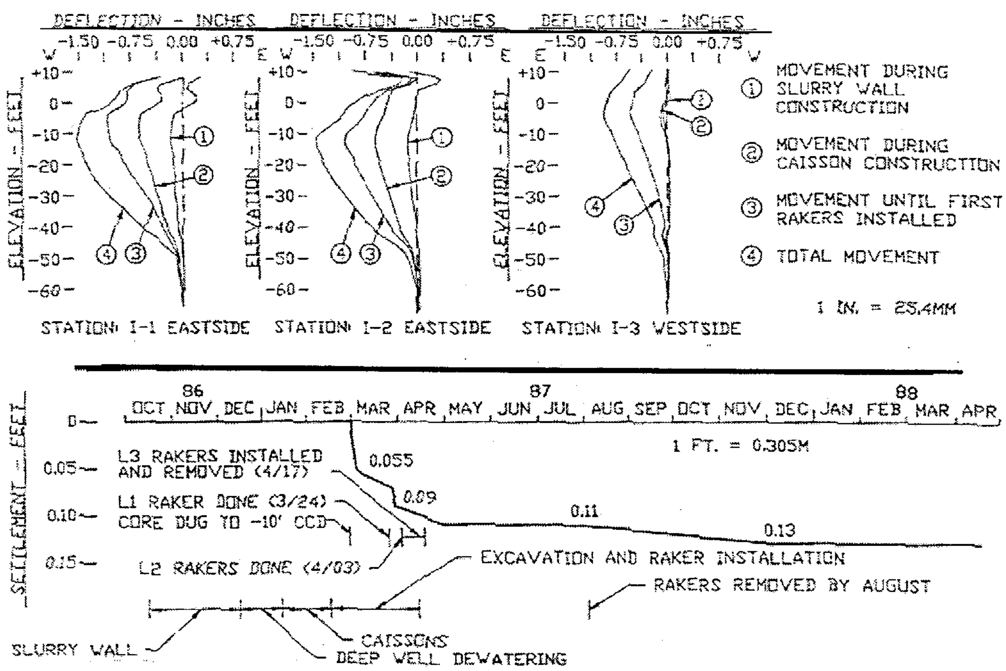
Baker et al. [1989], report that slurry wall construction took place without incident. Small concrete overpours in the order of 10% were typical during slurry wall construction (archived information). Small movements, up to δ_H = 0.15" towards the excavation, occurred during slurry wall construction (mostly observed below El. 0ft CCD along the eastern project side). Above El. 0ft CCD, soil moved slightly back by as much as 0.35" during slurry wall construction. In the western side these movements were practically not existent.

Caisson construction caused additional soil movements up to $\delta_H = 0.5"$ at 25' depth. When the first level of rakers was installed soil movements reached close to $\delta_H = 1.15"$ along the east and $\delta_H = 0.5"$ along the west. Final horizontal movements at the final stage reached $\delta_H = 1.5"$ along the east and $\delta_H = 1.0"$ along the west (Fig. 5.4.7.1). The added building surcharge on the eastern wall was the cause of this difference in eastern and western wall deflections.

Seventy percent (70%) of the total settlement occurred during the first level raker installation. This settlement was close to $\delta_V = 0.09$ft ($\delta_V = 1.08"$). An additional 0.02ft settlement occurred when the 3rd level of rakers was installed and removed. A total of $\delta_V = 0.11$ ft of settlement is directly attributed to the excavation and raker installation. When all the rakers were removed settlements increased to a total of $\delta_V = 0.13$ft ($\delta_V = 1.56"$).



Typical excavation Profile



Inclinometers, and Surface Settlements

Figure 5.17: AT&T Corporate Center (C7), excavation profile, inclinometer deflections, and settlements (Baker et al., 1987).

5.4.8 Case Study C8, Guest Quarters Hotel

The Beacon Guest Quarters Hotel is a 30-story reinforced concrete structure with a three level basement constructed with the top/down method, located at the southwest corner of E. Walton Place and N. Mies Van der Rohe Way. Drilled and underreamed caisson foundations support the vertical loads from the building with a slurry wall earth retention system was used as the basement wall. A general plan of the site and adjacent structures can be seen in Figure 5.18. Soil conditions are typical of downtown Chicago.

Immediately to the west of the site there is a 12-story high-rise Knickerbocker Hotel, with a one level basement. The University of Chicago Business School is located to the west of the site. There is also an abandoned 10-foot diameter water tunnel crossing the site at 100' depth. Until that time, the usual practice was to bulkhead the tunnel at the property lines, dewater the bulkheaded section and fill the tunnel with grout or concrete in the dry (Baker, Pfingsten et al., 1998). However, at the Guest Quarters Hotel dewatering caused the brick work of the tunnel to collapse because there were breaks or weaknesses. It was thus necessary to construct the bulkheads in the wet using low-slump grout and to fill the tunnel by pressure grouting with alternate grout holes serving as vent holes for water displacement (combined with leakage past the bulkheads) (Baker, Pfingsten et al., 1998).

The 2-foot thick slurry wall was theoretically able to carry wall loads in the order of 5 to 110 kips per foot. The slurry wall penetrated from 58' to 90' below grade depending on location (90' being adjacent to the University of Chicago Business School).

The top/down slurry wall excavation at the Guest Quarters Hotel performed well, as final wall deflections were in the order of δ_H=0.6" and only in one point did they reached δ_W=1.0" (Fig. 5.19, 5.20). Caisson construction caused to slurry walls to deflect by as much as 0.35" inwards, with typical induced movements were smaller (Fig. 5.19). Surprisingly, inclinometer In-1 deflected up to δ_W=0.97" in the cantilever excavation phase (about 15' below street), but movements decreased to δ_W=0.5" when the second level slab was completed (24' below street).

Other wall locations deflected up to δ_W=0.62" below the lowest slab level. One of the inclinometers had its maximum deflection 8' feet below LL2 whereas another deflected the most at the LL3 level (34ft-depth). It is clear that the larger wall deflections below the excavation base are associated with the transition from soft to stiff silty clay, where the slurry walls were embedded.

Near a re-entrant corner, movements up to 0.56" in the primary direction at the top of the slurry wall and up to 0.3" in the secondary direction occurred at the final stage of the excavation (Fig. 5.21). The combined primary and secondary movement totaled 0.62" close to 30' below street level. This suggests that deformations near re-entrant corners are affected by construction activities at both excavating faces, and are directed towards the axis of the corner.

There were no settlement data available, thus actual settlements are not known. Settlements could be expected to be in the same magnitude or less than wall movements if we consider that the top-down construction method was used.

The granular nature of the rubble fill caused problems in the guidewall construction. Vibrations from the street caused some guidewall trenches to collapse as they were excavated. This caused a section of the curb along Mies Van der Rohe Way to slough into the excavation. It was decided to use soldier piles and wood lagging on parts of the exterior slurry wall face as to avoid this situation again.

Slurry leaked into a basement when panels were trenched in the northwestern section of the project (Fig. 5.18). The slurry leak into the Knickerbacker hotel was observed 12' below street level, where the retained soil consists of rubble fill. This, clearly demonstrates that running fills (i.e. very permeable) may not be able to form an adequate cake filter. Guidewall construction was a little problematic due to vibrations from street traffic that caused soil collapses of the original guidewalls that had to be re-constructed to a greater depth.

Although no data was available building settlements are not expected to have been significant considering the small wall movements that occurred in this project.

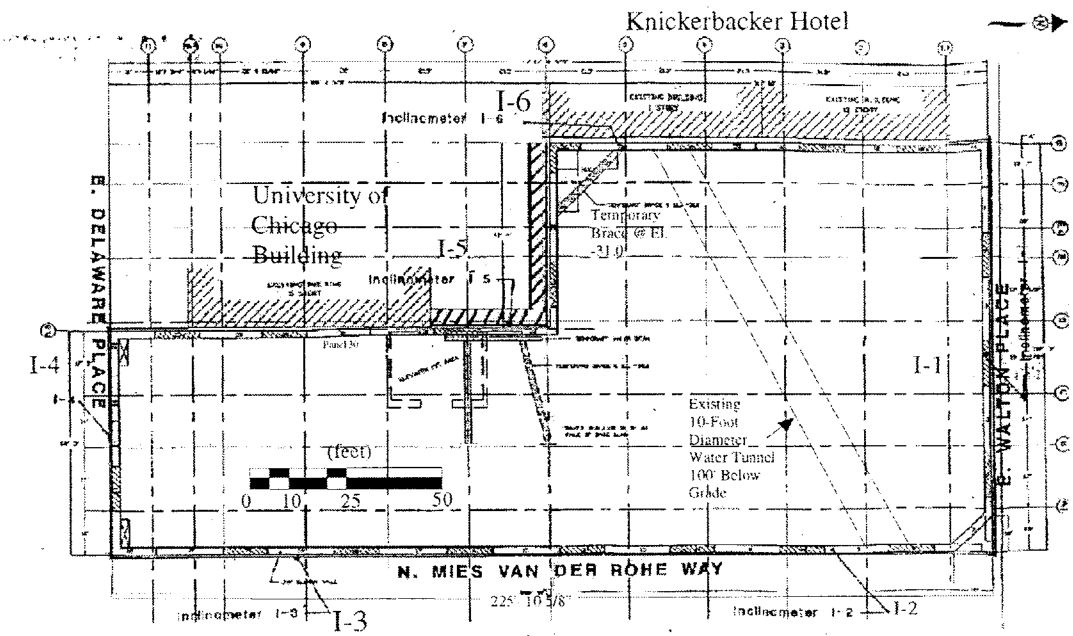


Figure 5.18: Guest Quarters Hotel (C8) site, slurry wall, and inclinometers.

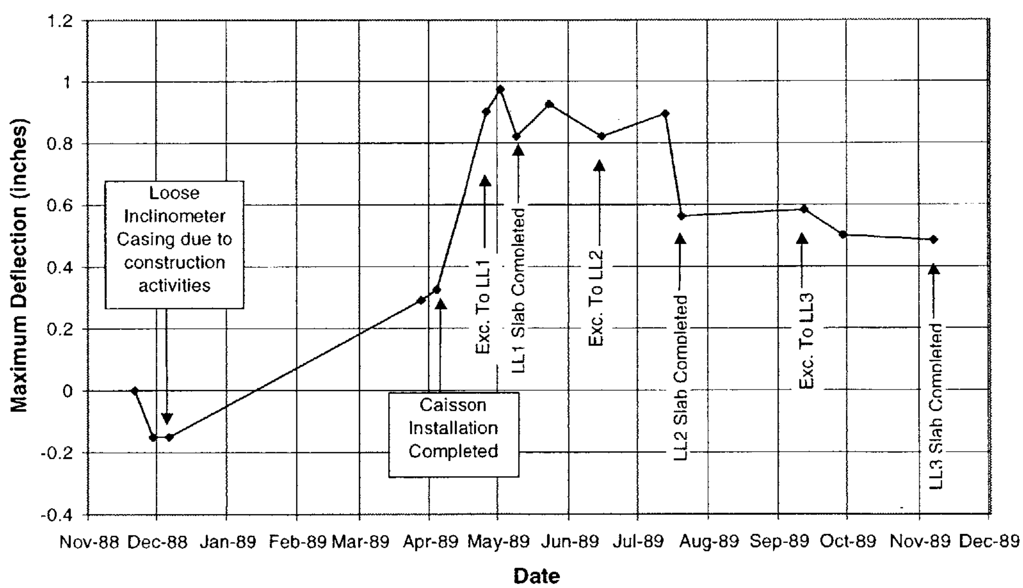


Figure 5.19: Inclinometer I-1 vs. time, Guest Quarters Hotel (C8).

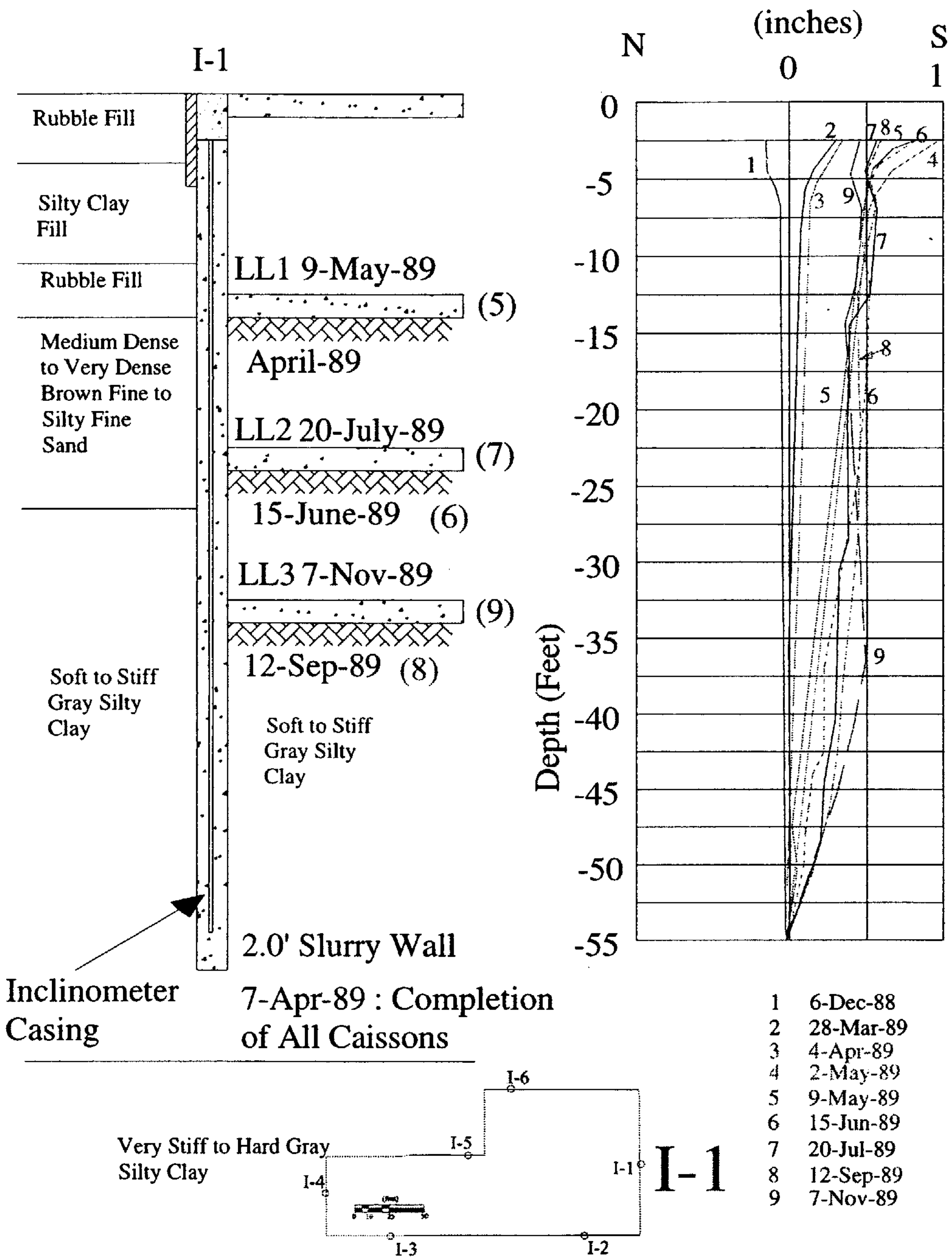


Figure 5.20: Inclinometer I-1, northern slurry wall, Guest Quarters Hotel (C8).

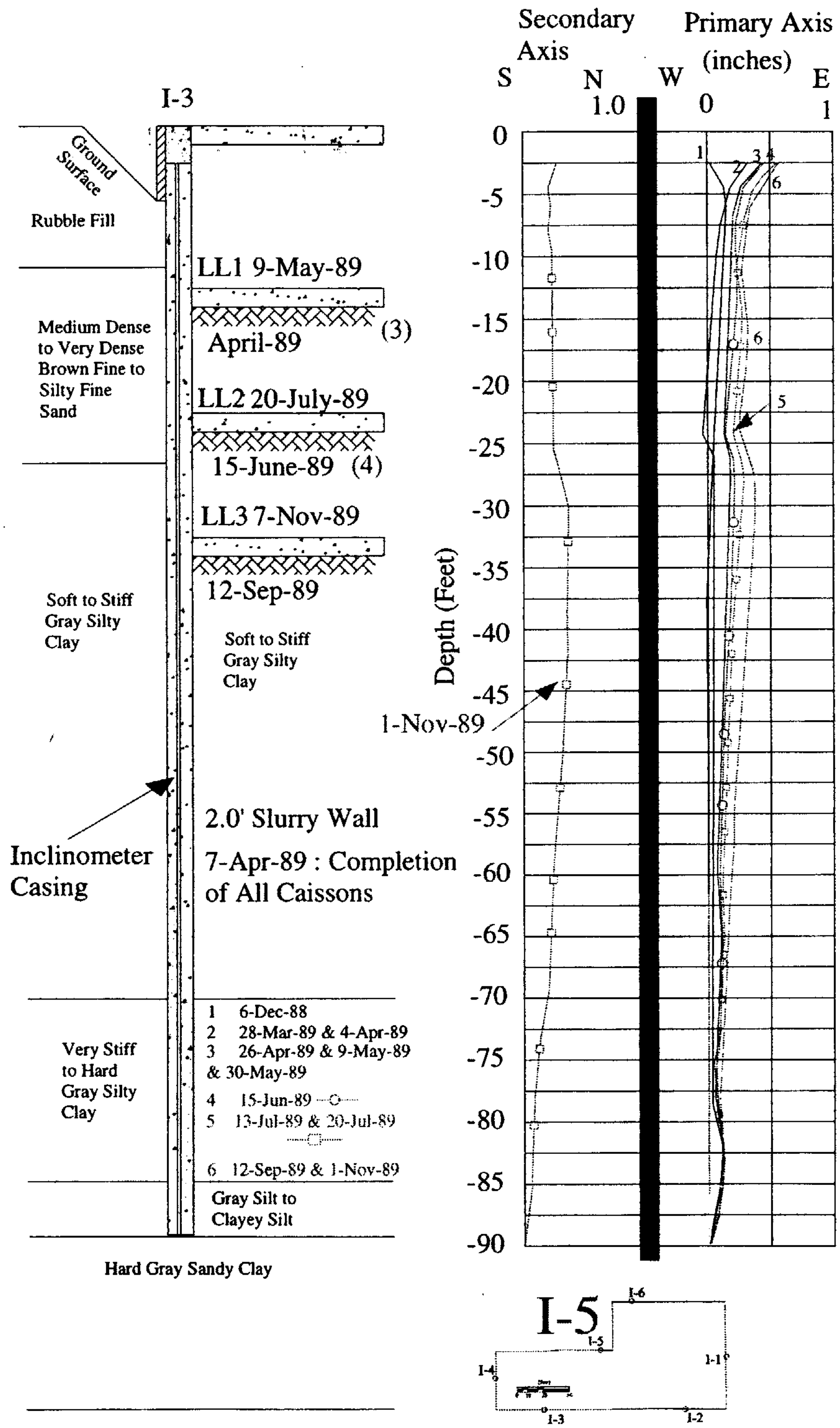


Figure 5.21: Inclinometer I-5, western slurry wall, near re-entrant corner, Guest Quarters Hotel (C8).

5.4.9 Case Study C9, Northwestern University Memorial Parking Garage

The Northwestern University Parking Garage is a twelve level structure bounded by Superior, St. Clair, and Huron Streets. Olsen Pavilion of Northwestern Memorial Hospital occupies the eastern portion of this block. The garage includes a relatively deep (23'), single basement level for loading dock facilities. Slurry walls were used to construct the basement walls of the new garage structure. Caissons were constructed in between slurry wall panels since the slurry wall bearing capacity was would not have been sufficient to carry the superstructure loads (Fig. 5.22). The soil profile at the site is typical of downtown Chicago. Street level at the site is approximately at elevation El. +13 CCD (Chicago City Datum).

In order to construct the slurry wall and the foundation caissons on the same alignment the caissons were constructed first to the top level of the slurry wall and then the slurry wall panels were excavated in between caissons. The caissons were provided with a steel shell with welded stubs of sheetpiles or a pipe for connection with the slurry wall. The stubs were to extend into the slurry wall by 4" to 6" to reduce seepage along the contact of steel shell and the slurry wall concrete.

Most slurry wall panels were close to 18' long and 39' deep, all being 2.0' thick. Bracing was provided by temporary tiebacks installed at El. +3.0 CCD at 14° inclination from the horizontal. Two tiebacks were placed near the caissons with a typical 4'3" horizontal spacing. Measured data was available until the site was thereafter excavated to El. –4.5ft CCD, after which there were no more available data regarding the performance of the retention system. It is known that the final excavation reached El. –7.0ft CCD.

The slurry wall excavation system performed well with small horizontal soil movements occurring during the excavation (Figs. 5.23 & 5.24). Caisson construction caused small near surface soil movements in the order of 0.05", while slurry wall construction caused horizontal soil to movements up to 0.1" towards the site.

Excavation to 11' below street caused soil to move towards the excavation by as much as 0.35", while tieback installation reversed deflections by as much as 0.15". Thereafter soil movements slightly increased as the excavation progressed to 17.5 ft below street level. The upper 2' of all inclinometer casings deflected more than the remainder portion of the casing, whereas 4' below the deflection was much smaller (Fig. 5.24). Most likely this additional deflection was caused by backfilling against the casing.

One inclinometer was placed 20' from the western slurry wall (I-2) while other two others were located 5' from the slurry wall (I-1, I-3). All these inclinometers did not show any significant differences in the magnitude of the soil movements except for the upper portion of the casing but movements for the inclinometer 20' from the excavation were slightly smaller below the tieback elevation (C9). The final deflection shape of the inclinometer 20' away was mostly cantilevering whereas other inclinometers displayed some translative motion at the toe of the wall as well (0.2"). Measurable deep-seated soil movements were observed in all inclinometers down to 40' below the slurry wall base. These movements were related to the clay that is present below the excavation, and clearly show that deep-seated soil movements occur in soft clays even for relatively shallow excavations.

Despite the fact that there was mention of settlement markers there was no available settlement data found for this project. However, settlements must have been small if we consider the small magnitude of soil movements and the fact that the excavation was not very deep.

There were numerous locations where seepage was observed at the joints between caissons and slurry wall panels, despite the care had been taken to construct watertight joints (Fig. 5.25). These leaks were to be repaired with grouting or another sealing method.

In several areas wet bentonite was observed along the wall which was later removed. In addition several voids were observed at the slurry wall face that had to be patched with concrete. Although all of the above needed repair work was probably minor, it does clearly demonstrate that an increasing the number of construction joints increases the chance of water leaks through the slurry wall, and thus should be avoided.

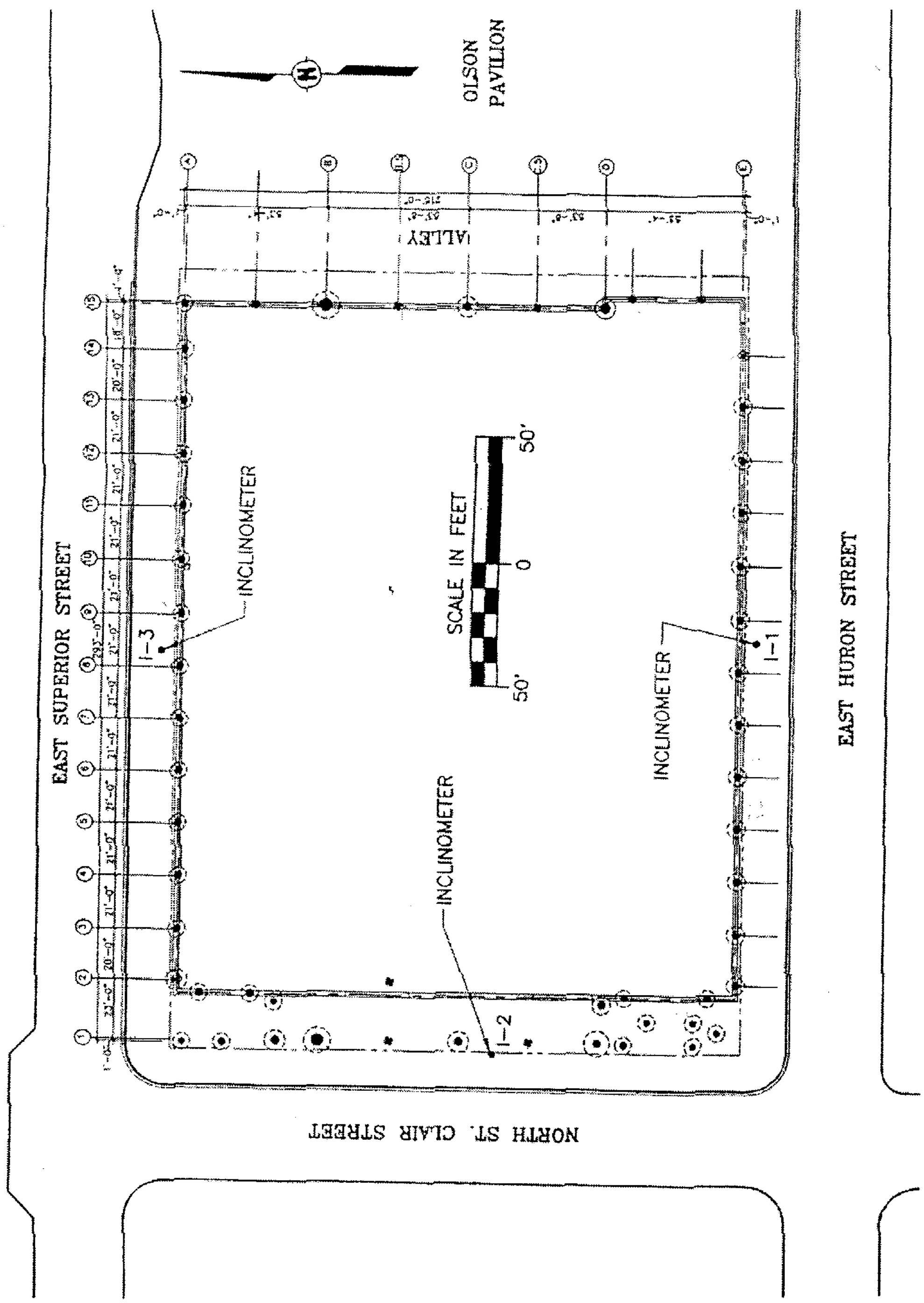


Figure 5.22: Northwestern University Memorial Parking Garage site (C9)

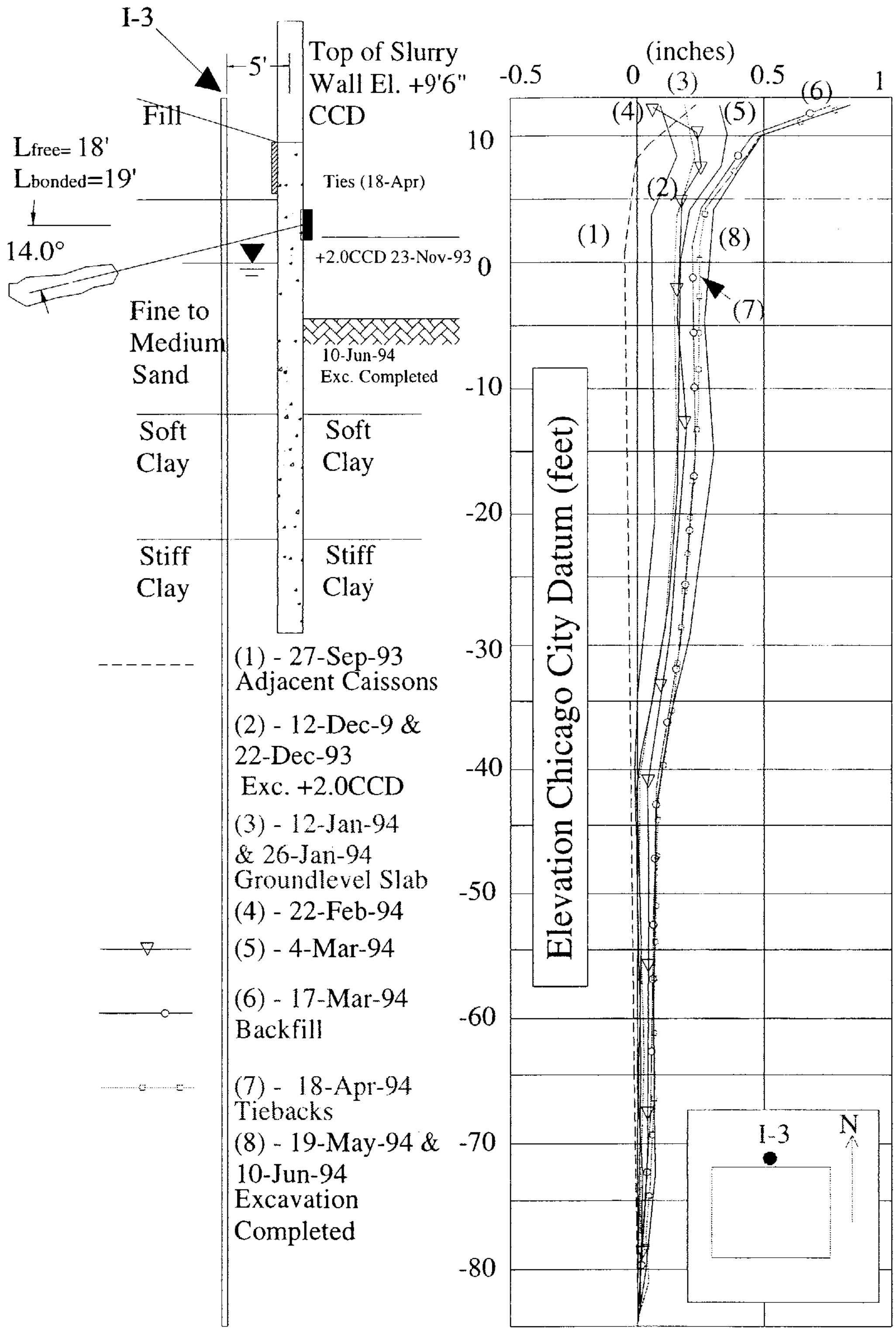


Figure 5.23: Inclinometer I-3, northern slurry wall, NU project (C9).

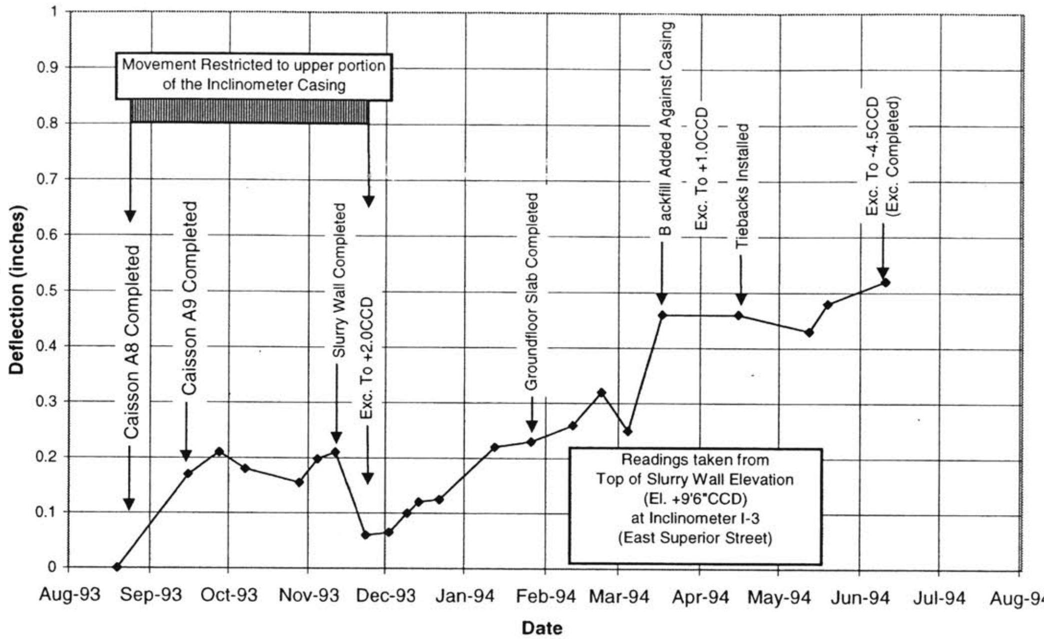


Figure 5.24: Inclinometer I-3 deflections with time, NU project (C9).

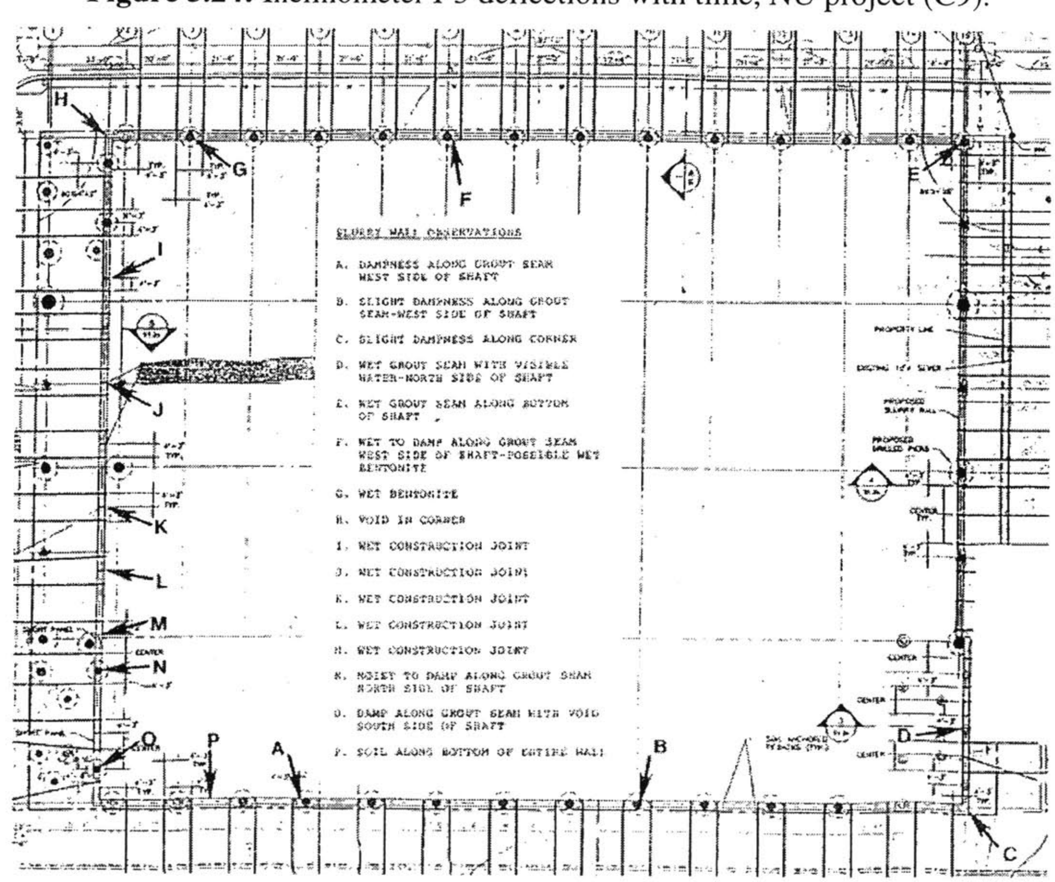


Figure 5.25: Special observations at NU project (C9).

5.4.10 Case Study C10, Museum of Science and Industry Parking Garage

The Museum of Science and Industry is located in 57th and Lake Shore Drive. An underground 3-level parking was constructed adjacent to the northern side of the existing Museum Building as to satisfy parking needs. Slurry walls 2.5'-thick and 45'-deep braced with three levels of permanent tiebacks were used to construct the underground parking. Tiebacks were air-drilled and extend into the hardpan soils with the grouted body being about 60' away from the slurry wall face. The excavation was 34'-deep and covered an area of 633'x 278' for the parking and a smaller lobby section on the south (Fig. 5.26).

The site of the Museum is located on reclaimed land from lake Michigan. Soils in the site consist of 20' of sand fill, over 20'-thick medium to stiff clay, and very stiff clay for the next 20' feet The water table at the site is controlled by the lake water elevation, which is at El. +3 ft CCD. Borings did not penetrate more than 60' of soil but it is known that "hardpan" soils exist below that depth. Clays at the site have larger unconfined strengths than downtown Chicago clays. The medium clay at depths of 20' to 40' has typical unconfined undrained strengths slightly more than 1.0 ksf (s_U UU value) whereas for downtown clays unconfined shear strengths range from 0.3 ksf to 1 ksf at the same depths. Clays at depths of more than 40' typically had unconfined undrained s_U strengths in the range of 2.0 ksf to 5.0 ksf.

Maximum slurry wall deflections were kept to small values throughout the excavation period (Fig. 5.26). Most monitored slurry wall panels deflected up to 0.5" with some panels deflecting up to 0.87" (I-32, Fig. 5.27 & 5.28). Maximum wall movements were observed above or close to the second levels of tiebacks. Slurry walls generally rotated about their base with slight bending observed above the second level of tiebacks (Figs. 5.27 & 5.28). In some locations, approximately half of the total measured deflections might have been caused by the extensive grouting efforts that were undertaken to seal water leaks (i.e. 0.4").

Injection grouting was used to repair water leaks through slurry walls during excavation. It was mentioned that in many locations this grouting caused the wall to move laterally up to 0.25". The exact locations where these grout induced wall movements were observed are not known but inclinometer deflections increased considerably after the 3rd level of ties was installed. It was also reported that the southeastern re-entrant corner from I-17 to I-8 had seriously cracked during the excavation and that repeated grouting had attenuated this cracking.

During the installation of the second level of tiebacks in the southeast corner of the construction site, cracks were observed in the asphalt concrete pavement and the concrete slab supporting the observation tower of the Museum. These cracks were situated approximately 20' to 35' further south from the slurry wall in which the tiebacks were being installed. Tieback installation in this location started on Oct 21st and progressed easterly to panel 18 by Oct 23rd. During this period cracking was observed as the tiebacks were installed. According to present engineers the platform of the tower raised in elevation by approximately 0.5" and then later in the day it settled back about 0.25". Inclinometer I-17 in that location indicated that movement at the top of the wall towards the north was in the order of 0.38". It was believed that drilling with air through the stiff to very stiff clayey soils resulted in incomplete removal of the clay cuttings between the drill pipe and the side of the drill hole thus not allowing air pressure to escape (field report).

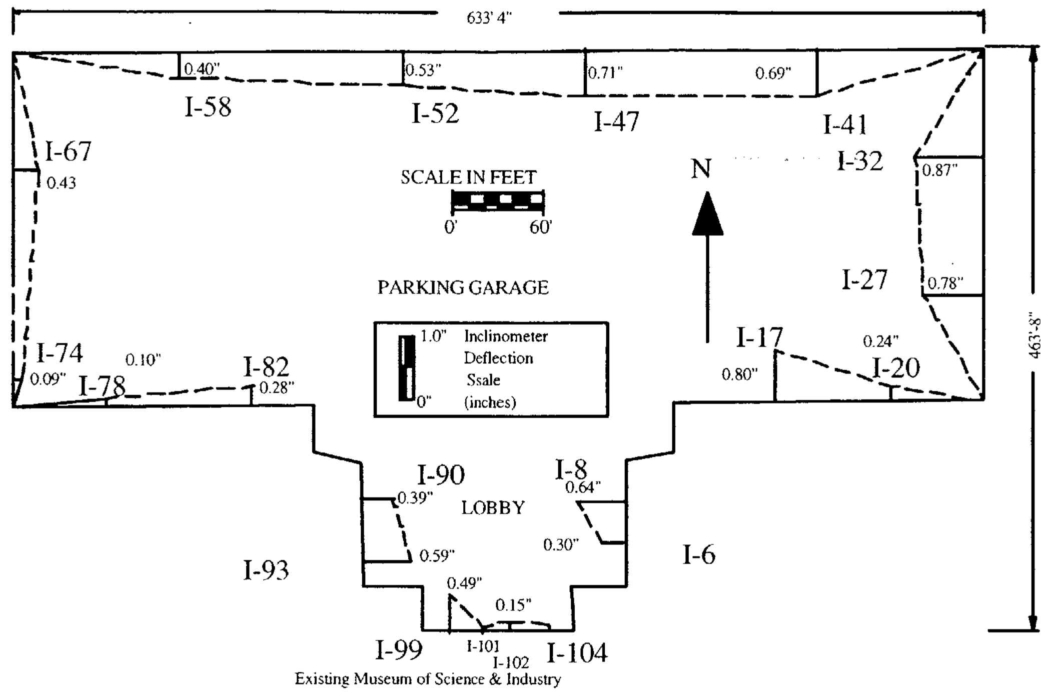


Figure 5.26: MSI (C10) site and maximum final inclinometer deflections.

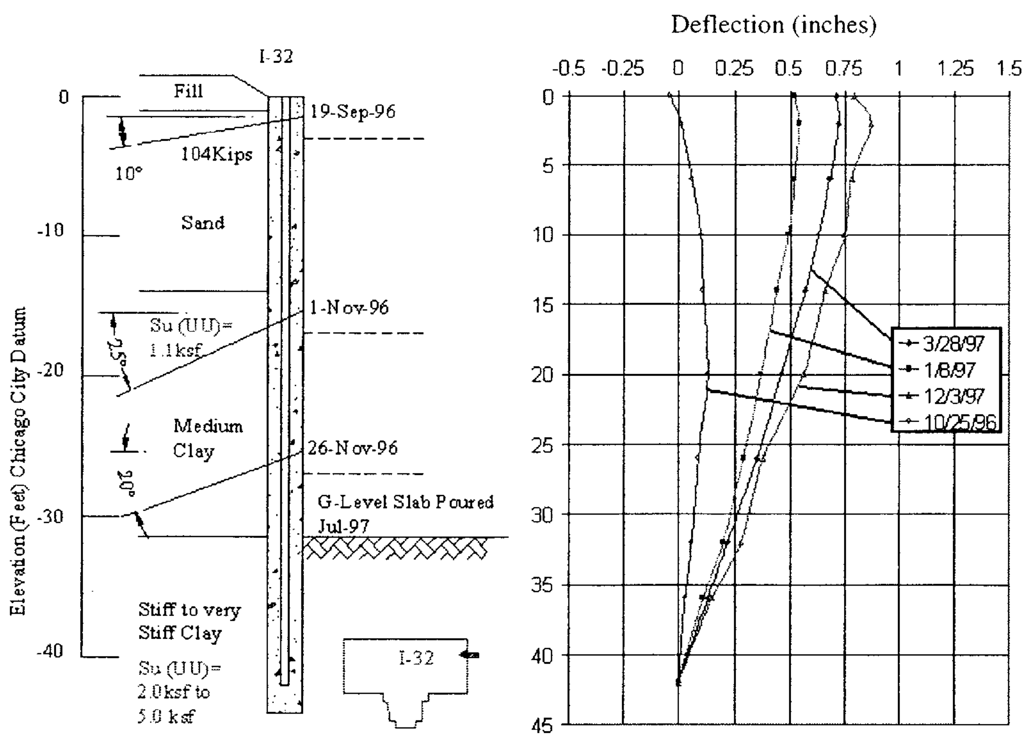


Figure 5.27: Inclinometer I-32, MSI (C10).

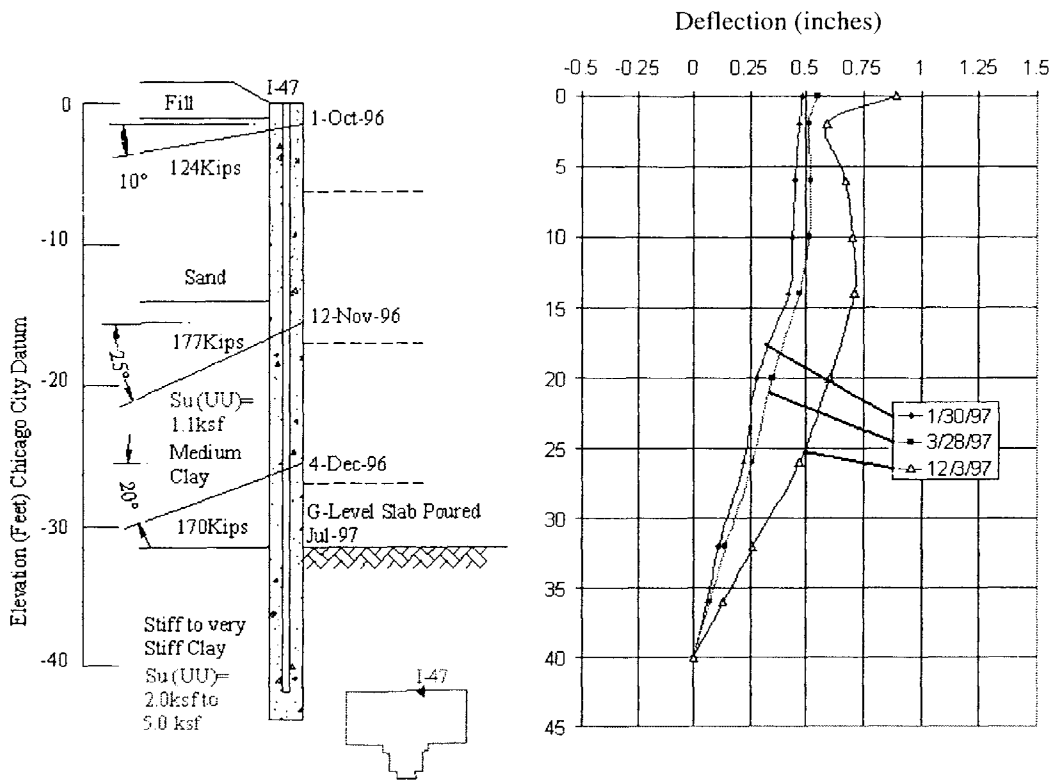


Figure 5.28: Inclinometer I-47, MSI (C10).

5.4.11 Case Study C11, 311 South Wacker Drive

The 311 South Wacker Drive project is located at the southwest corner of Jackson Boulevard and Franklin Street It covers a "T" shaped area approximately 395' by 330' in plan (Fig. 5.29). The structure has a three level, 34.5'-deep basement. A 2.0'-thick diaphragm wall that extends 65' below the surface provided both temporary and permanent lateral earth support for the excavation. The slurry wall was constructed in 18' to 25' long panels. Soil conditions are typical of downtown Chicago.

Four types of bracing were used in this project (Gill et al., 1989) namely cross-lot struts and inclined rakers, corner braces, tied back deadman, and top/down construction. The core area was excavated to the base of the core mat in a rectangular sheeted cofferdam covering an area of 145' by 56', that was braced by three levels of cross-lot pipe struts and corner struts (Gill et al. 1989).

Significant lateral displacements occurred when an open excavation was made in the interior using the sheeted cofferdam for the construction of the tower mat. Although the cofferdam was braced by three levels of cross-lot struts, the effect of excavation was observed at Inclinometer I-3 at a distance of about 60' (18.3m) from the face of the excavation (Gill et al, 1989) (Fig. 5.30). Ground displacements might have also been attributed to open excavation close to the slurry wall for removal of obstructions from previous construction, from excavation of caissons and from oversize excavations at each caisson for constructing columns for the top/down section. Inclinometer I-3 showed mostly cantilevering deflections totaling up to 2.75" near the top of the casing. The horizontal translation measured at I-3 at the elevation of the bottom of the wall was close to 1".

Deflections at inclinometer I-6 were much smaller than those measured at I-3, as they reached a maximum of 0.60" (15mm). However, I-6 did not extend beneath the excavation base and thus any translative movements of the toe of the could not be recorded. If we apply the translation of 1" measured at the base of the wall at I-3, then actual movements at I-6 could have been as large as 1.60".

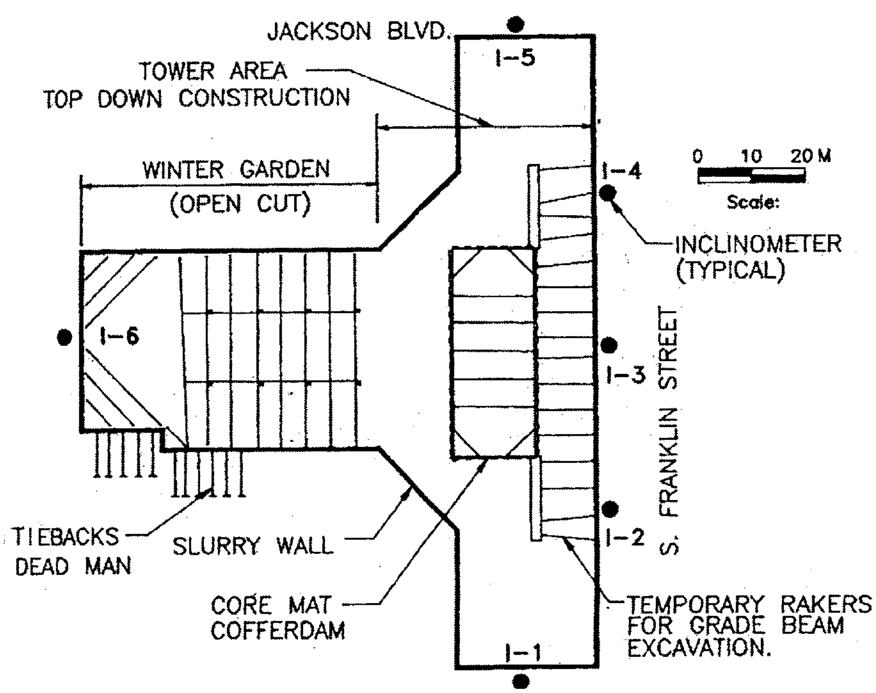
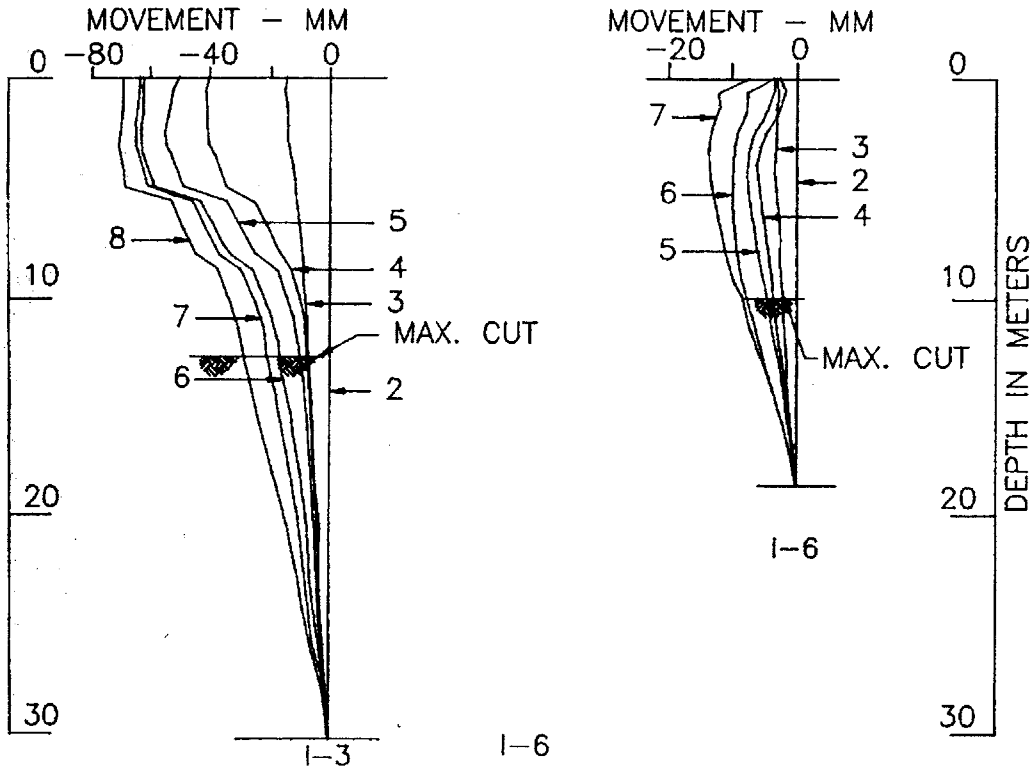


Figure 5.29: 311 South Wacker Drive (C11), (Gill et al, 1989).



I–6

2. INITIAL
3. 12–22–87 CUT TO LEVEL 1 BRACING (–2.0 M)
4. 1–15–88 CUT TO LEVEL 2 BRACING (–5.5 M)
5. 1–26–88 CUT TO LEVEL 3 BRACING (–7.62 M)
6. 6–21–88 FINAL CUT (–9.8 M)
7. 8–9–88 FINAL READING

I–3

2. INITIAL
3. 6–2–87 CAISSON CONSTRUCTION
4. 7–14–87 EXCAVATION FOR CORE AREA
5. 10–13–87 SLURRY WALL EXCAVATION
6. 2–18–88 CUT FOR P–2 LEVEL (–7.0 M)
7. 3–10–88 CUT FOR P–3 LEVEL (–10.7 M)
8. 9–9–88 FINAL CUT (–12.84 M)

Figure 5.30: Inclinometer deflection at 311 South Wacker Drive (C11).

5.5 Summary of Inclinometer Deflections

Figure 5.31 summarizes the maximum wall or horizontal soil deflections measured for Chicago slurry wall excavations plotted as a function of the system stiffness as proposed by Clough et al. [1989]. The effects of improper caisson construction were removed by subtracting the deformation caused by caisson construction from the total deflection at the same elevation. Table 5.3 summarizes the input parameters used to plot Figure 5.31. Projects in Figure 5.31 can easily be distinguished by the bracing that was used.

The majority of the excavations have a lateral deflection ratio $\delta_H/H<0.4\%$ (C4, C6, C7, C8, C9, C10, C11(a)). Two of the projects that plot above $\delta_H/H=0.4\%$, are early excavations and thus movements can be attributed to poor bracing performance (C2) or inadequate toe embedment (C3). Projects braced by tiebacks in combination with rakers suggest a correlation of decreasing movements with increasing system stiffness (C3, C4, C6, C9, C10). Most of these projects (C3, C4, C6, C9) can be relatively easily compared to each other because the site conditions were quite similar. Out of all projects the top/down project (C8) had the smallest ratio of maximum wall movement to excavation depth $\delta_H/H=0.064\%$. Two of the raker supported projects (C2, C11 (b)) with similar system stiffness plotted very close to each other.

Figure 5.32 summarizes a frequency plot of the maximum and final deflections for the 47 of the inclinometers used in 10 out of the 11 projects studied (C5 excluded). The same data are replotted in Figure 5.33 in terms of the dimensionless ratio δ_H/H where H is the final excavation depth. The effect of improper caisson construction was not accounted for in constructing the frequency plots. Table 5.4 gives the number of inclinometers per project used to derive the frequency plots in Figures 5.32 & 5.33. Unfortunately, the inclinometer database for the Chicago projects is not as extensive as the one for the Boston case studies. The database is drawn from only 47 inclinometers.

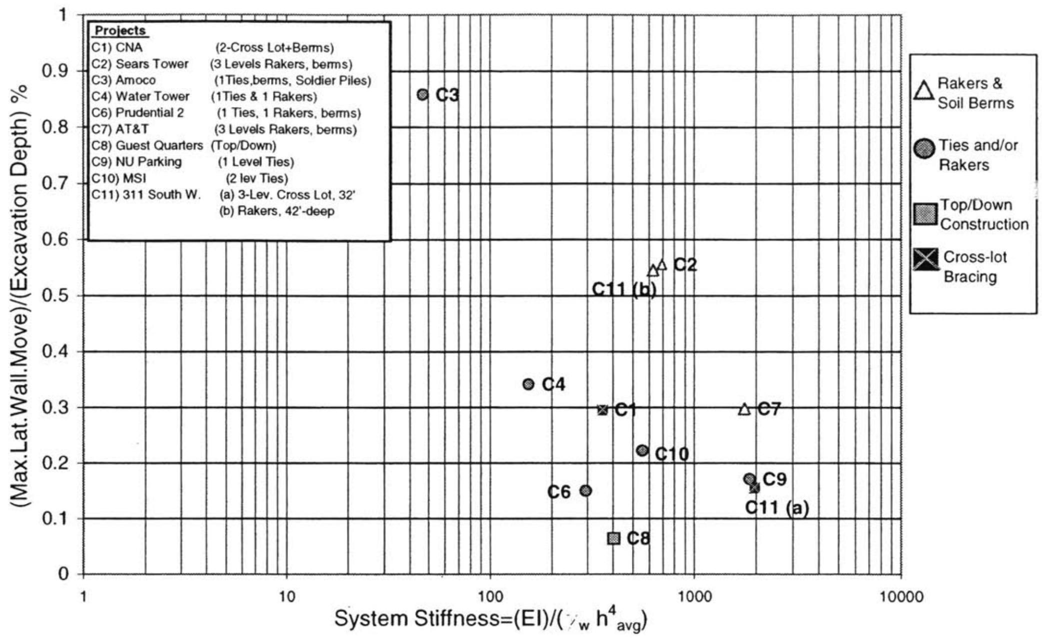


Figure 5.31: Maximum wall deflections versus system stiffness for Chicago slurry wall excavations, (excavation induced movements only, when applicable the effect of caisson caused deflections was eliminated)

Table 5.3: Summary of system stiffness input parameters and horizontal deflections for diaphragm walls in Chicago.

	Project	Wall Thick (ft) t	Exc. Depth (ft) H	Vertical Support Spacing (ft) h	f_c' (ksi)	Horizontal Deflections (inches) Caisson+ Other δ_{other}	Total δ_{total}	Exc. Only δ_{Exc}	System Stiffness $EI/\gamma_w\ h^4$	$\delta_{Exc/H}$ %
C1	CNA	2.5	31	13	3.5	2.2	3.3	1.1	354.8	0.296
C2	Sears Tower	2.5	45	11	3.5	3	6	3	692.1	0.556
C3	AMOCO	1.75	34	16.5	3.5	1.1	4.6	3.5	46.9	0.858
C4	Water Tower	2.5	44	16	3.5	0.4	2.2	1.8	154.6	0.341
C6	Prudential Two	2.25	25	13	4.5	0	0.45	0.45	293.3	0.150
C7	AT&T Corp. Center	2.5	28	9	4.5	0.5	1.5	1	1751.2	0.298
C8	Guest Quarters Hotel	2	35	11	4.5	0.33	0.6	0.27	401.8	0.064
C9	NU Mem Parking Gar.	2	19	7.5	4.5	0.1	0.49	0.39	1859.2	0.171
C10	Museum Scien. Indus.	2.5	30	12	4.5	0	0.8	0.8	554.1	0.222
C11 (a)	311 South Wacker	2	32	7.4	4.5	0	0.6	0.6	1961.7	0.156
C11 (b)	311 South Wacker	2	42	9.8	4.5	0	2.75	2.75	637.8	0.546

Note:

Modulus of Elasticity: $E\ (psi) = 57000\sqrt{f_c'\ (psi)}$ => $E(psf) = 395.83\ \sqrt{f_c'\ (psi)}$ (ACI-8.5.1)

f_c' : 28 Day strength of concrete

Moment of Inertia of Uncracked Section = I

$$I(ft^4) = \frac{(1ft)\ t^3}{12}$$

t (ft) = Diaphragm wall thickness

γ_w = Unit weight of water = 62.4 pcf

From Figure 5.32 we can clearly see that the overwhelming majority of the inclinometers deflected less than 1.0" (approximately 70% of all data points). The largest deflections that are shown here were mostly caused by improper caisson construction and reflect the earlier projects only (C1, C2, C3). About 22% of all the monitoring locations deflected in the range of $1.0" \leq \delta_H \leq 2.5"$, but this range does not reflect all of the maximum measured movements in all projects. The average horizontal deflection was δ_H=0.98" ± 0.88

In Figure 5.33 we can see that the 70% of the maximum inclinometer deflections fall were in the $0.1\% \leq \delta_H/H \leq 0.3\%$ range. The average ratio δ_H/H=0.304% ± 0.172%. It is interesting to note that there was an absence of data plotting in the 0.0-0.1% range. This probably reflects the soft nature of the soils and the fact that all if not the majority of the excavations in Chicago are floating. As expected, caisson construction has caused 11% of the inclinometers plotting in the 0.6-1.0 range (C1, C2, C3). For a 35'-deep excavation (typical depth studied according to database in Chicago) this would have δ_H =1.28" average and range possibly up to δ_H =2.42" if the workmanship is not good. However, the expected movements should be much smaller if the workmanship is good, and caisson construction does not induce significant deformations.

Table 5.4: Inclinometers per project (Chicago), used to derive the frequency plots in Figures 5.32 & 5.33.

ID	Project	Inclinometers
C1	CNA	3
C2	Sears Tower	2
C3	AMOCO	5
C4	Water Tower	3
C6	Prudential Two	2
C7	AT&T Corp. Center	5
C8	Guest Quarters Hotel	7
C9	NU Memorial Parking Garage	3
C10	Museum of Science & Industry.	15
C11	311 South Wacker Drive	2
	Total	47

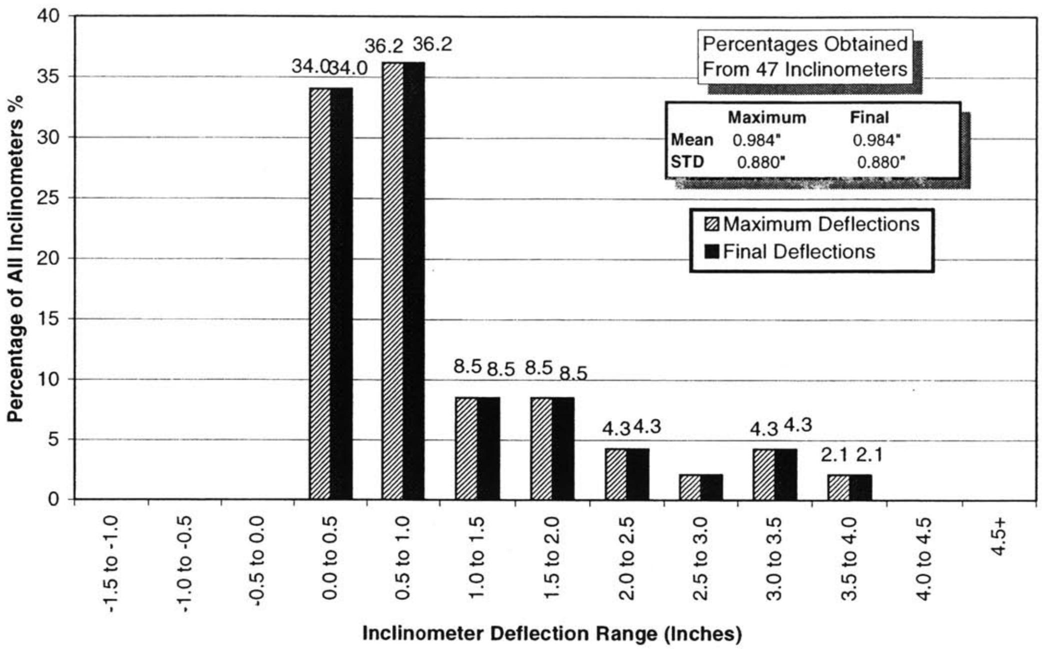


Figure 5.32: Statistics of maximum and final inclinometer deflections for all inclinometers in slurry wall projects in Chicago.

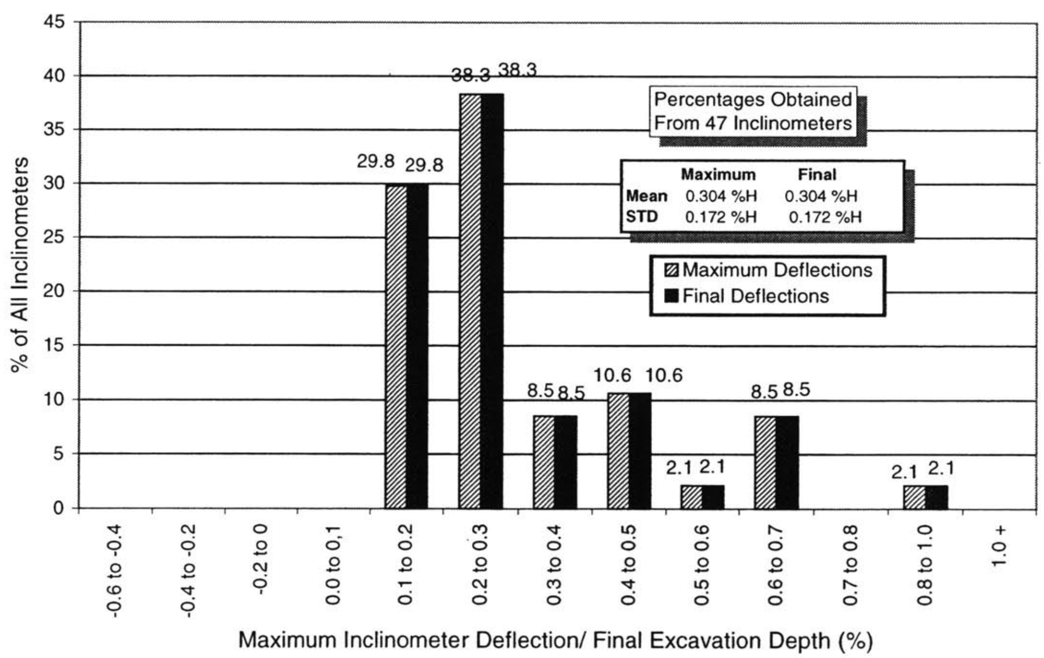


Figure 5.33: Statistics of maximum and final inclinometer deflections as percentages of the final excavation depth for inclinometers in slurry wall projects in Chicago.

5.6 Special Observations

Table 5.5 summarizes various special observations made during slurry wall construction and the excavation for projects in the Chicago. In regards to slurry wall construction special attention must be given to the Prudential Two project (C6). In this job tilting problems were encountered with the steel H-beams that reinforced the slurry wall. In particular the H-beams bent considerably about their weak axis of bending during concreting of the slurry wall panels. It was believed that this condition was caused by the unbalanced soil and tremie concrete pressures that were acting on the H-beams during concreting.

Attention must also be given to the Museum of Science and Industry (C10), where the southeastern re-entrant corner (Fig. 5.34) cracked considerably, and major leaks were observed. It was reported that large volumes of grout were used to seal these leaks, and that they caused some of the slurry wall movement. Additional grout sometimes caused new leakage because of the new differential movements that were induced. The exact volumes of grouting and the volumes of water inflow are not known.

Water leakage problems were considerable when caissons were incorporated into the slurry walls (C9), because the water stopping details between the slurry wall and caissons was inadequate. In earlier projects, the large wall movements caused serious cracks in the slurry walls resulting in water leakage (Sears Tower, C2). Other problems during slurry wall construction included instability of guide walls due to street traffic vibrations. In the Guest Quarters Hotel (C8), the guide walls and small sections of the sidewalk collapsed due to such vibrations. These guide walls were installed in open trenches and without the use of any slurry as it is commonly used. Slurry wall construction caused small horizontal soil movements towards the trench in the order of 0.1" to 0.2" (C6, C7, C8, C9). Small concrete overpours, typically 10% of the theoretical panel volume, were measured during construction of these slurry wall projects. Occasionally, small sand pockets were found within slurry walls that were subsequently filled with expansive concrete (C6, C9).

Table 5.5: Special observations for slurry wall excavations in Chicago.

Job	ID	Year	Soil Profile	Wall Type	Thickness (ft)	Typical Panel Depth (ft)	Typical Panel Length (ft)	Gs Bentonite mix	Cave-ins Reported	Movements During Slurry Wall Construction	Other Special Observations
CNA Building	C1	1970	Typ. Downtown Chicago	RC	2.5	54	29		No data/ non mentioned in referenced papers	Practically none	
Sears Tower	C2	1970	Typ. Downtown Chicago	RC	2.5	60	-		non mentioned	no data left	Cracks along one wall/ water running rapidly
Amoco Standard Oil	C3	1971	Typ. Downtown Chicago	RC & BP	1.75	40	10 & 20		No data/ not mentioned in referenced papers	Caisson and Slurry wall construction combined for up to 1.0 at one location near the surface, otherwise in the order of 0.3" for both	One 6" crack with leak - later patched,Some slurry wall bearing Steel H-piles encountered obstructions
Water Tower Place	C4	1973	Typ. Downtown Chicago	RC & CS	2.5	62	12		No data/ not mentioned in referenced papers	Small but can not be distinguished from published paper	
Loyola University Bussiness School	C5	1993							No data available	No data available	Water Leakage problems mentioned several years after construction
Prudential Two	C6	1987	Typ. Downtown Chicago	RC & CS	2.25	53	12 to 20	1.03 to 1.12	Not mentioned	up to 0.25" towards the trench near the top of the slurry wall	Excess Tilt of steel H-members in slurry wall during concreting, Sand pockets occasionally encounetered in the upper sand, Obstructions encountered during trenching
AT&T Corporate Center	C7	1987	Typ. Downtown Chicago	RC	2.5	60	-		Not mentioned in referenced paper	0.15" towards the excavated trench/ excavation	Small concrete overpours 10%
Guest Quarters Hotel	C8	1989	Typ. Downtown Chicago	RC	2	55	13 & 18		Not mentioned	In the order of 0.1 to 0.2 inches	Slurry leak into an adjacent basement, Pile foundations encountered during trenching, Street traffic vibrations caused 60' of guidewall to collapse
NU Memorial Parking Garage	C9	1994	Typ. Downtown Chicago	RC	2	39	19	1.06	Not mentioned	In the order of 0.1 to 0.2 inches towards the excavation	Numerous problems with seepage between caissons and slurry wall, Wet bentonite observed along the wall, Several voids in the slurry wall had to be patched
Museum of Science & Industry	C10	1997	Outside soft clays but stiffer than downtown	RC	2.5	40			Not mentioned	Not measured/ Inclinometeres within the slurry walls	Major cracking of a re-entrant corner, Excessive leakage required major grouting efforts

Note: RC- Reinforced Concrete, SP- Soldier Piles as reinforcing,BP-Bearing Piles incorporated into wall, CS- Caissons incorporated into the wall

5.7 Summary of Chicago Slurry Wall Excavation Experience

The slurry walls in all of the studied projects in Chicago have been floating walls in soft to medium clays. Depths of excavation were smaller than other cities in the U.S. since no excavation was deeper than 44'. The first slurry wall in Chicago was installed in the CNA building in Chicago (C1).

Earlier projects had major problems with inadequate caisson construction that caused large movements before any excavation took place. In these early projects the bracing was not adequate since the rakers that were used were not preloaded. Thus the walls showed very little bending between supports and mostly rotated and translated about their bases (C3, C4). The large differential movements between panels caused major cracks in the slurry wall concrete that were resulting in considerable seepage (C2).

Projects from 1980 after performed much better with the wall movements being kept to small values (C5, C6, C7, C8, C9, C10). Caisson construction in these projects did not cause any significant movements since experience with previous projects had accumulated, and caissons were constructed with steel lining and not in oversized holes. In most of the recent projects the walls showed pure translation with little bending above the base of the excavation (Type I, deflection shape) (C6, C7, C9, C10). These translative movements at the toe of the walls were expected since the all the slurry walls in Chicago are embedded into medium clays that can not provide a lot of resistance as glacial till or bedrock. Bending in these walls was mostly observed below the lowest bracing level or the excavation base.

Close to 70% of the wall movements fell in the range of $0.0\% \leq \delta_H/H \leq 0.3\%$, which is the same as the percentage of inclinometer deflections in the $0.0" \leq \delta_H \leq 1.0"$ range. Earlier projects contributed to the larger movements in the range of $0.6\% \leq \delta_H/H \leq 1.0\%$ (C1, C2, C3, C4). However, the range of wall movements within each project was not as large as that of Boston (Chapter 4). Unfortunately, very little to no settlement data was found in archived in the records of slurry wall excavations in Chicago.

Chapter 6
Washington, DC, Projects

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6.1 Introduction

Slurry wall excavations have been quite popular in Washington DC, with the first projects constructed in the 1970s. Sections of WMATA subway tunnels constructed with the cut and cover method were amongst the first slurry wall projects in the city. Today the ongoing construction of the Washington Convention Center (W3) is the largest single slurry wall basement excavation in the United States, covering five and one-half city blocks. The reasons for using slurry walls for deep excavation support in Washington are as follows:

I. Control ground deformations caused, and mitigate potential effects on adjacent structures (notably WMATA tunnels).

II. Minimize or avoid underpinning of adjacent structures.

III. Minimize groundwater leakage

IV. Economic reasons when used as the permanent basement wall.

In all projects, toe fixity was provided by embedding the walls into bedrock, decomposed rock, or extremely stiff layers. Table 6.1 lists the five slurry wall projects studied and referenced in this research, while Figure 6.1 shows their locations. The author is aware of at least 10 other projects in the Washington DC area (Table 6.2), but unfortunately was not able to locate any performance data from these projects.

Table 6.1: List of studied slurry wall excavations in Washington, DC.

ID	Project Name	Year	Depth(ft)		Thick (inches)	Soil Type	Bracing	Wall Type	Toe Fixity
			H	D					
W1	World Bank	1991	60	7	30	Cretaceous	5-Lev. TB	RCDW	√
W2	Petworth Subway Station	1995	60 – 100	12 16	36	Cretaceous	5-Lev CLB 6-Lev CLB	RCDW	√
W3	Washington Convention Center	1999 2000	30 - 55	18 23	36, 48	Cretaceous Pleistocene	1,2 Lev TB & 1 Lev R 3-Lev.TB	RCDW	√
W4	Metro Center II	1990	31	22	24	SandClay	2-Lev. Ties	RCDW	√
W5 *	Federal Center Station (6)	1973	60		36	Sand, Clay	3 Cross Lot	RCDW	√

Note: * Performance data only, Tamaro & Gould 1993.
H – Excavation Depth, **D**- Embedment depth, **TB** –Tiebacks, **CLB** – Cross-Lot Bracing, **TD** – Top/Down, **R** – Rakers, **SB**- Soil Berms, **CB** – Corner Bracing, **IB**-Internal Bracing, **PC** – Precast, **PT** – Post Tensioned, **SP** – Soldier Piles, RCDW – Reinforced Concrete Diaphragm Wall, **SPTC** – Soldier Piles & Tremie Concrete.

Table 6.2: Other known slurry wall excavations in Washington DC

Project	Comments
Federal Triangle	Supported by tiebacks
Marriott Hotel	Permanent basement wall to protect adjacent National Press Club & National Theatre
Smithsonian South Quadrangle	Permanent basement wall and protection to adjacent buildings
National Geographic Society	Alternative to sheeting, bracing and underpinning
1411 New York Avenue	Permanent basement wall / protect adjacent building
Corcoran Office Building	Permanent basement wall
Techworld	Barrette slurry wall used to protect adjacent subway
Metro Center 5A	Protect adjacent subway on two sides
Hub Building	Protect adjacent subway
Washington Subway, Federal Center, Southwest Station, Sect. D004	In lieu of underpinning and sheeting

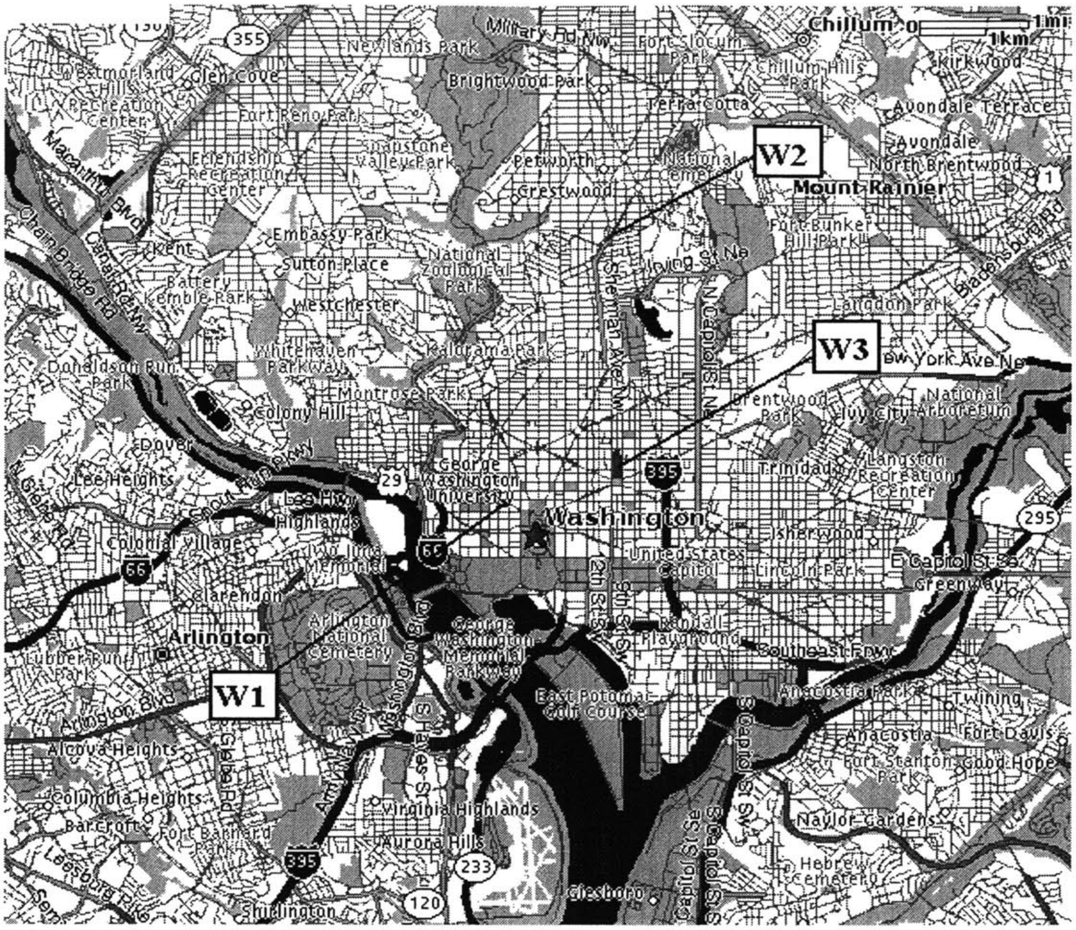


Figure 6.1: Location of studied slurry wall excavations, Washington DC

6.2 Washington DC geology and subsurface conditions

Washington DC is characteristic of the "fall line cities" along the Atlantic seacoast where deep water access is terminated abruptly by former falls and rapids (Gould et. al 1998). The southeastern portion of the District lies in the "coastal plain" which consists of a broad belt of deep flat-lying sediments over deep bedrock (WMATA, 1969), while the northwestern District lies within the "Piedmont" province with a relatively thin soil cover of crystalline bedrock.

A "fall line", running southwest from the Montgomery County boundary through Farragut Square, passing south of the Pentagon, forms the boundary of these two units (Fig. 6.2). This causes the Cretaceous plain sediments to dip southeast in wedge shaped lenses. These coastal plain sediments comprise continental and near-shore deposits, alternating layers of arkosic sands with clays, all heavily pre-consolidated (mainly by erosion cycles). Table 6.3 provides a more detailed description and subdivision of the Cretaceous deposits, which are locally referred to as the Potomac Group of the Cretaceous period or just P-group (WMATA 1989).

Pleistocene sands and gravels, deposited as terraces during sea level fluctuations of glacial times overlie the Cretaceous soils (directly on bedrock). The terrace soils deposited during the Pleistocene era are also locally referred to by the WMATA as the T-group (Table 6.4). The deposits formed as glacial meltwater carried runoff sediments from the western upland and they primarily comprise of crudely bedded gravel, and sand with interstitial silt and clay. In the central district these deposits form amphitheater like steps. Boulders and coarse soils are often found at the base of the Pleistocene terrace. The different zones of T-group have been subjected to varying degrees of weathering.

Slightly organic Holocene clays deposited during the rise of the sea level after the last glacial recession are found near the existing river borders. These clays have a stiff desiccated upper crust over lightly overconsolidated material (similar to the Boston Blue Clay).

Bedrock in the District is primarily Wissahickon schistose gneiss, with small amounts of intrusive rock and alteration products. Typically a layer of weathered in-situ (residual soil or saprolite) decomposed rock lies over the competent bedrock. The decomposed bedrock is very hard with Standard Penetration Resistance ranging close to 100 blows per foot.

Table 6.5 design soil properties for design for the T & P groups as proposed by WMATA [1989]. The T1 clay stratum has shear strengths from 0.7 ksf [(T1)C] to 3.5 ksf [(T1)D]. The Cretaceous clays are highly overconsolidated (24 to 40 ksf) and have higher shear strengths (2 ksf to 6ksf) compared to the T1 clays.

The T and P notations used by WMATA [1989] are contradictory with the USGS notations. USGS [1994] refers to the Pleistocene deposits as Q3, Q4, Q5 (late, middle, and middle Pleistocene respectively), while the T-group is classified as Tertiary deposits of the: late Pliocene (T1, T2), late Miocene (T3), middle Miocene (T4), and middle and early Miocene (Tc). Lower cretaceous soils are noted as Kps, and Kpc by the USGS 1994 survey, and Q1 soils are classified as Holocene clays. However, only the WMATA [1989] notation was used in the five projects studied, and this notation is used throughout this chapter. Part of the USGS [1994] geologic map is shown in Figure 6.2. The WMATA manual does not include a map with the locations of the respective strata using the WMATA notation.

The general groundwater conditions in Washington DC are very complex. Near the Potomac and Anacostia Rivers, groundwater levels are probably controlled by the water levels at the rivers. Studies by Mueser et. al. [1969] report that a separation appears between an upper normal water table and a lower depressed water table with differences from 8' near Connecticut Ave to 25' near E. Street. It appeared at the time that pumping from the gravely sand T5 layer was causing this separation in the water tables. However, water levels currently are higher than in 1969 and will continue to rise because pumping has been restricted (unpublished report, Clark Foundation Company). For excavations in Pleistocene soils, Mueser et. al. [1969] indicate that trickling flow from utilities may be observed at any level from top to bottom of an excavated face because the Pleistocene soils are very lenticular.

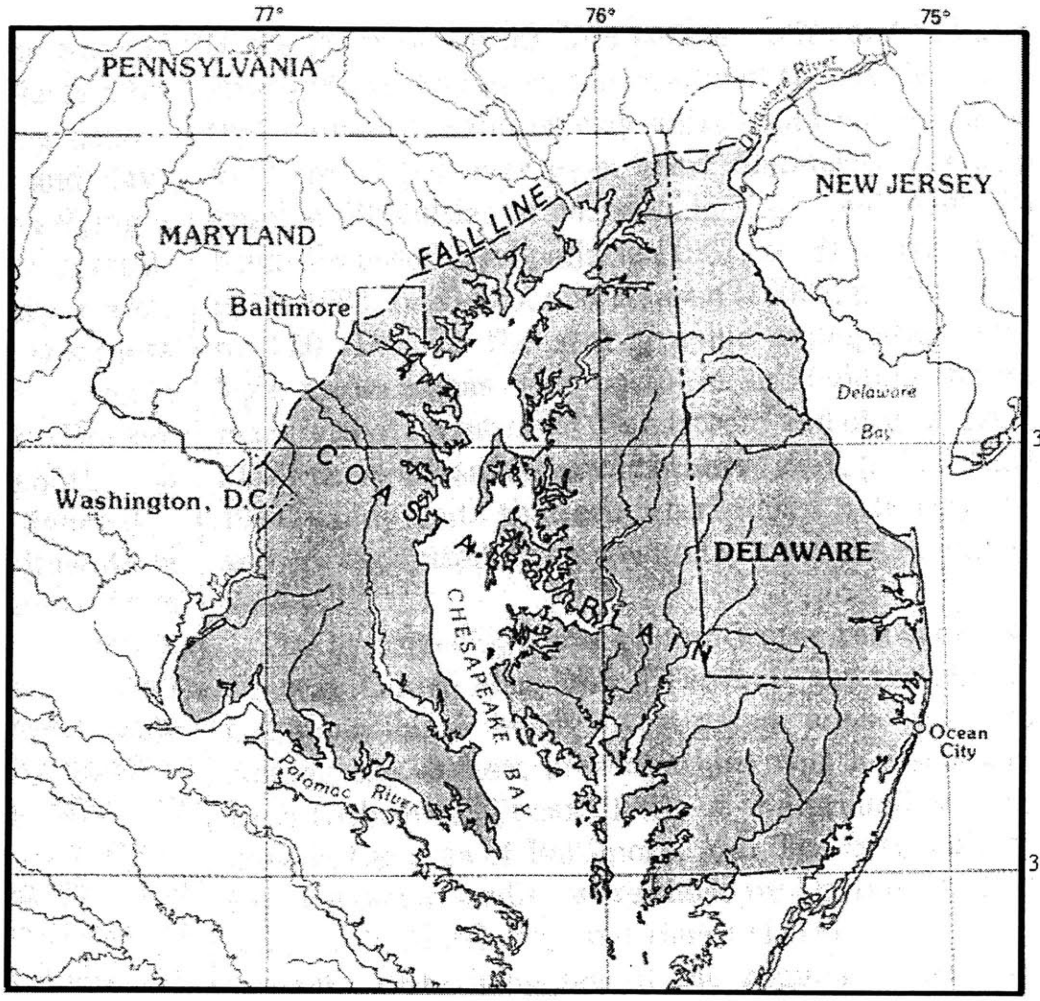


Figure 6.2: "Fall line" and the Coastal Plain (Vroblesky & Fleck, 1991)

Table 6.3: Cretaceous strata, WMATA notation,(USGS notation in parentheses)

Stratum	Description	Unified Soil Class	
		Prim	Sec
P1	Plastic clay, generally CH or MH or CL with moisture content near the plastic limit containing lignites	CH	CL
P2	Clay or silty sand with some fairly clean sand, containing lignite and cemented sand layers, classified SC, SM, SP-SM or SP	SM SP	SP
P3 (Kps)	Extremely hard clayey sand and sandy clay. This is a distinctive material, gray-green in color, varying from CL to MH or SC. In may cases it is difficult to distinguish from the decomposed bedrock from which it is derived. Blow counts are >60 bpf.	CL SC	ML SM
P4 (Kpc)	Clayey sand, some gravel, scattered cobbles and boulders. This is the lowermost Cretaceous material, apparently regularly above the bedrock surface and exhibiting a relatively high permeability and numerous and erratic oversized rock fragments. It is the very earliest Cretaceous deposit laid down by rapidly flowing streams and old erosion surface in the underlying bedrock.	SM SW	SP GM

Table 6.4: Pleistocene Terrace deposits, WMATA 1989, (USGS in parentheses)

Stratum	Description	Unified Soil Class	
		Prim	Sec
T1	Stiff to medium stiff light brown or gray or mottled brown-gray silty clay or clayey silt with lenses of brown silty fine sand. In some areas, several separate layers of Pleistocene clays have been encountered, which are distinguished by a letter suffix: T1A, T1C, etc.	CL ML CH	Len-ses of SM SC
TO	Medium stiff to stiff dark gray organic clay with numerous wood fragments, usually found interlensed with stratum T4.		
T2	Medium compact to compact brown and red-brown silty clayey fine to medium sand with trace of gravel and occasional boulders	SM SC	SP SW
T3 (Q3)	Medium compact to compact gray and gray-brown fine to coarse sand with some silt and gravel and variable amounts of cobbles and boulders.	SW SM	SP GM
T4 (Q4)	Medium compact to compact gray and gray-brown fine to medium sand with some silt and small gravel. Containing lenses of dark gray clay, occasionally slightly organic	SM SP	SW
T5 (Q5)	Compact to very compact gray and gray-brown fine to coarse sand with some silt and small gravel. Some to trace of silt and variable amounts of cobbles and boulders, often concentrated at the base of the layer.	SW SM	SP GM

Table 6.5: Soil properties in Washington DC, for design, (adapted from WMATA 1989).

	STRATUM	Overconsolidation stress for Cohesive Strata (ksf)	Shear Strength (ksf)	Effective friction angle Φ' degrees	Total Unit Weight (pcf)
(F)	Fill			28 - 30	120-130
(T)	Pleistocene				
(T1)A & (T1)G	Silty Clay	3 to 5	1.5 to 2.5	25 to 28	130
(T2)B	Organic Clay	3.0 to 5.0	2 to 3, higher near surface	25	130
(T1)C & (T1)F	Silty Clay	1.0 to 2.0	0.7 to 0.9 higher near ground surface	25	130
(T1)D	Plastic Clay	5.0 to 6.0	2.5 to 3.5	25	130
(T1)E	Medium Plastic Clay	6.0	2 to 3	25	130
(T1)H	Plastic Clay	3.0 to 5.0	1.3 to 1.5	25	130
(T2)	Silty Sand			34	130
(T3)	Gravely Sand			34 to 38	130
(T4)	Silty Sand			30 to 34	130
(T5)	Gravely Sand			32 to 34	130
(P)	Cretaceous				
(P1)	Plastic Clay	North & West of New Jersey Ave 30 to 40	4.0 to 5.0	25	130
	Plastic Clay	East of New Jersey Ave 24 to 28	2.0 to 5.0	25	130
(P2)	Clayey Sand			33 to 36	130
(P3)	Sandy Clay	30 to 40	4.0 to 6.0	34	130
(P4)	Gravely Sand			34 to 38	130

Q5: (Middle Pliostocene) gravel, sand, silt, and clay, gray to gray brown crudely to well bedded. Found mainly beneath irregular surface between 40' and 105' in Elevation, **Kps:** Lower cretaceous sand, **Kpc:** Lower Cretaceous Clay, **T2:** Late Pliocene, gravel, sand, silt, and clay.

Figure 6.3: Geologic map of Washington DC, (USGS 1994)

6.3 Measured Performance

Only a limited assessment of slurry wall performance is possible given that data were available only from five projects, representing a small fraction of the slurry wall work carried out in Washington DC. Tiebacks were used for bracing in three of the five projects (W1, W3, W4), and cross-lot bracing in the other two (both subway stations, W2, W5).

Table 6.6 summarizes the performance of the five excavations. According to the data, the slurry walls have performed very well, with deformations less than δ_H=1.0" induced by the excavations (though deflections varied widely within the same project). Individual panel behavior was affected by local variations in construction and by differing surcharge conditions for adjacent structures.

Table 6.6

Summary of slurry wall excavation performance, Washington DC

ID	Year	Project Name	Soil Type	Excav. Bracing/ Excav. Method	Depth (ft)		Wall Thick (inch)	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Toe Fix.	Defl. Shape Type
					H	D					
W1	1991	World Bank	Cretaceous	5-L. TB	60	7	30	0.45 -0.45	0.4	√	II IV
W2	1995	Petworth Subway Station	Cretaceous	5-6 CLB	60 100	12 16	36	0.75	--	√	II
W3	1999 2000	Washington Convention Center	Cretaceous Pleistocene	1,2 L TB & 1 R 3-L. TB	30 – 55	18 23	36, 48 36	0.75 0.70	0.3	√	I I
W4	1990	Metro Center II	SandClay	2-L. TB	31	22	24	0.38	--	√	II
W5	1973	Federal Center Station	Sand, Clay	3-Lev CLB	60		36	0.62	--	√	II

Walls braced by tiebacks either slightly bowed towards the excavation (Deflection mode: Type II) or showed very little bending above the lowest bracing level (Type I). The walls in the W2 & W5 projects, where cross lot bracing was used, deflected in the same bowing Type II mode. Building and surface settlements were generally smaller than horizontal wall movements, typically less than 0.5". The scatter in the surface settlement data was some times larger than the actual settlements. In the World Bank Project, slurry wall settlements had large variations even within adjacent panels. The presence of soft zones at the base of some panels in combination with large vertical tieback forces may account for high local settlements in the order of 2".

6.4 Individual Case Studies

6.4.1 Case Study W-1, World Bank

This project involved the construction of a 12-story tower with a 5-level deep basement, in the World Bank complex at the southwestern corner of 18th St. N.W and H St. N.W. Excavation support was provided by a permanent 30"-thick perimeter slurry wall, keyed into decomposed bedrock, and braced with 4 to 5 levels of permanent tiebacks. Slurry walls were selected in this project in order to minimize settlements and water seepage. Settlement control was crucial along the western and southern sides of the site due to the protection of other existing World Bank buildings (A, D: Fig. 6.4).

The soil profile at the site comprises 28' of sandy clay/ clayey sand (Stratum I) overlying 13' to 40' of sand and gravel with cobbles and boulders (Stratum II), 9' of sandy clay (Stratum III), 12' of decomposed rock (Stratum IV), and Gneiss bedrock (Stratum V) (Fig. 6.5). The water table is located approximately 35' to 42' below the ground surface at approx. El. 60'MSL (Mean Sea Level) in stratum II. Piezometric heads measured in the decomposed rock layer were about 8' to 10' lower than the water table at the same locations.

The lateral earth support system of this project provided very good control of wall movements and surface settlements (Figs. 6.4, 6.6-6.9). However, the variation of the measured deflections of slurry wall panels was very large (twice the actual maximum deflections) (Figs. 6.4 6.6, 6.7). Maximum slurry wall deflections towards the excavation reached up to 0.45" at the final readings (Fig. 6.5.1.3). Most of the slurry wall deflections occurred after the excavation progressed below the 3rd level of tiebacks (25 to 30ft deep) as shown in Figures 6.6 and 6.7.

The wall bulged in locations were the sandy clay (Stratum III) and the decomposed bedrock was present and moved back in a location where these layers were absent (30-May-91: Figs 6.6, & 6.7). In addition, the base of the wall appears to have translated by as much as 0.2" when the panels were embedded in decomposed rock. Walls adjacent to buildings moved towards the excavation.

Most slurry wall panels did not show significant settlement or heave but a few panels settled more than 1.0" (Figs. 6.4, 6.8). It is very interesting that almost adjacent panels displayed large differences in the magnitudes of settlement. One panel that was embedded in decomposed rock and moved towards the excavation settled by as much as 1.9" whereas a panel that was embedded in rock and moved back into soil moved upwards by 0.6" (Fig. 6.8). Surprisingly there were no surface settlements observed in locations (δ_V=-0.2" to 0.1") where the slurry wall settled measurably along the 18th Street (δ_V=1.2" to 1.9") (southeastern section: Fig. 6.4).

Otherwise, surface settlements were very small at the final stages of excavation. Although not expected, surface settlement points in the northern project corners (H Street, 18th and H Street intersection) moved up to 0.6" of heave at the final excavation grade (Fig. 6.4). This result could be related to the prestress at these locations.

According to data from tiltmeters building rotations were kept very small, with the most of the rotations occurring after installation of the 4th level of tiebacks (9-Apr, 2-May, 91). Building rotations typically ranged from 0.05° and to 0.07° at monitoring points along the western project side.

Water levels in observation wells during the excavation remained steady throughout the excavation. However, one piezometer screened in the decomposed rock showed a 15' local drop (Fig. 6.9). Tieback drilling probably dewatered the decomposed bedrock at that location, which was isolated from water recharge by the overlying clay.

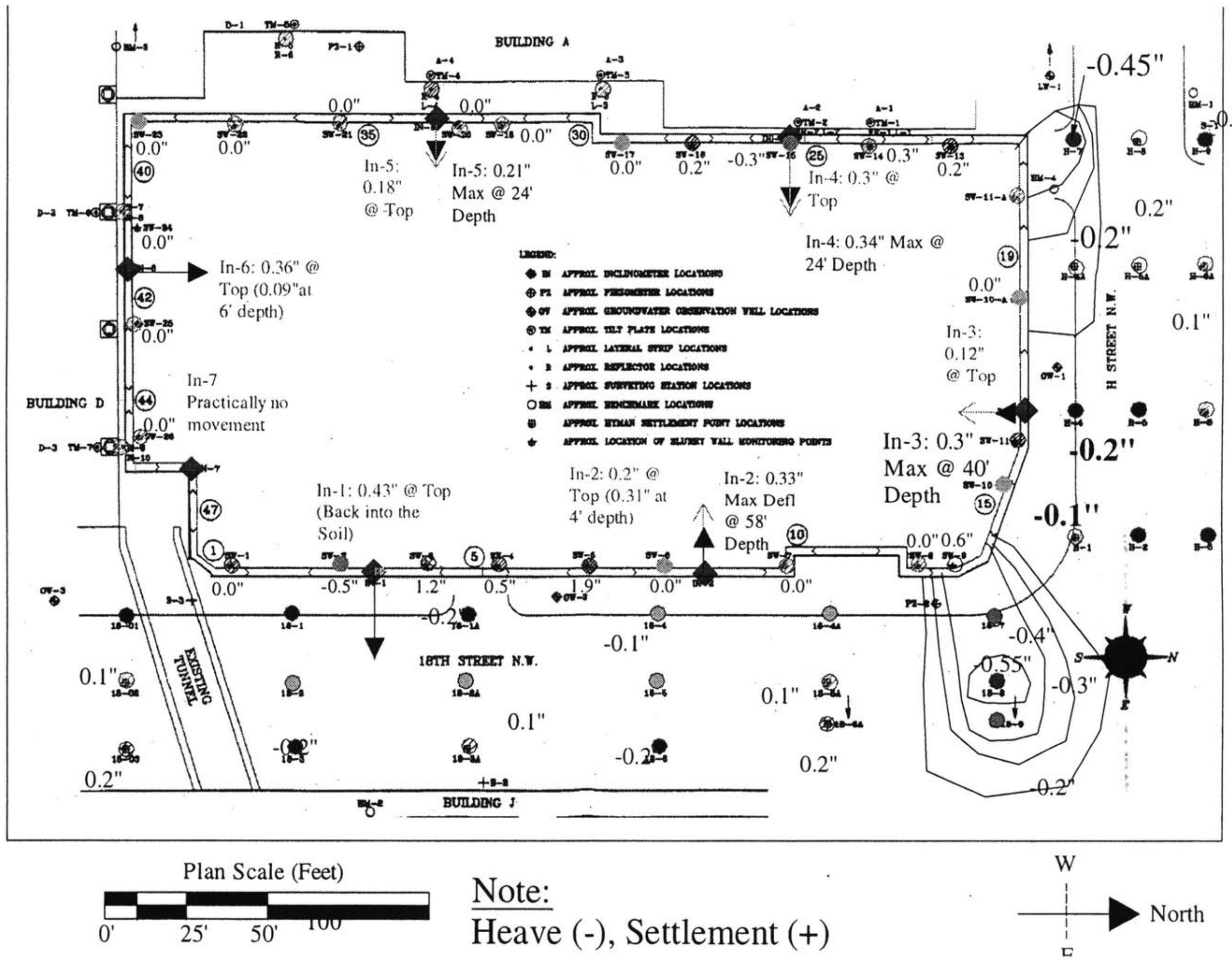


Figure 6.4: World Bank (W-1), site, deflections, and settlements

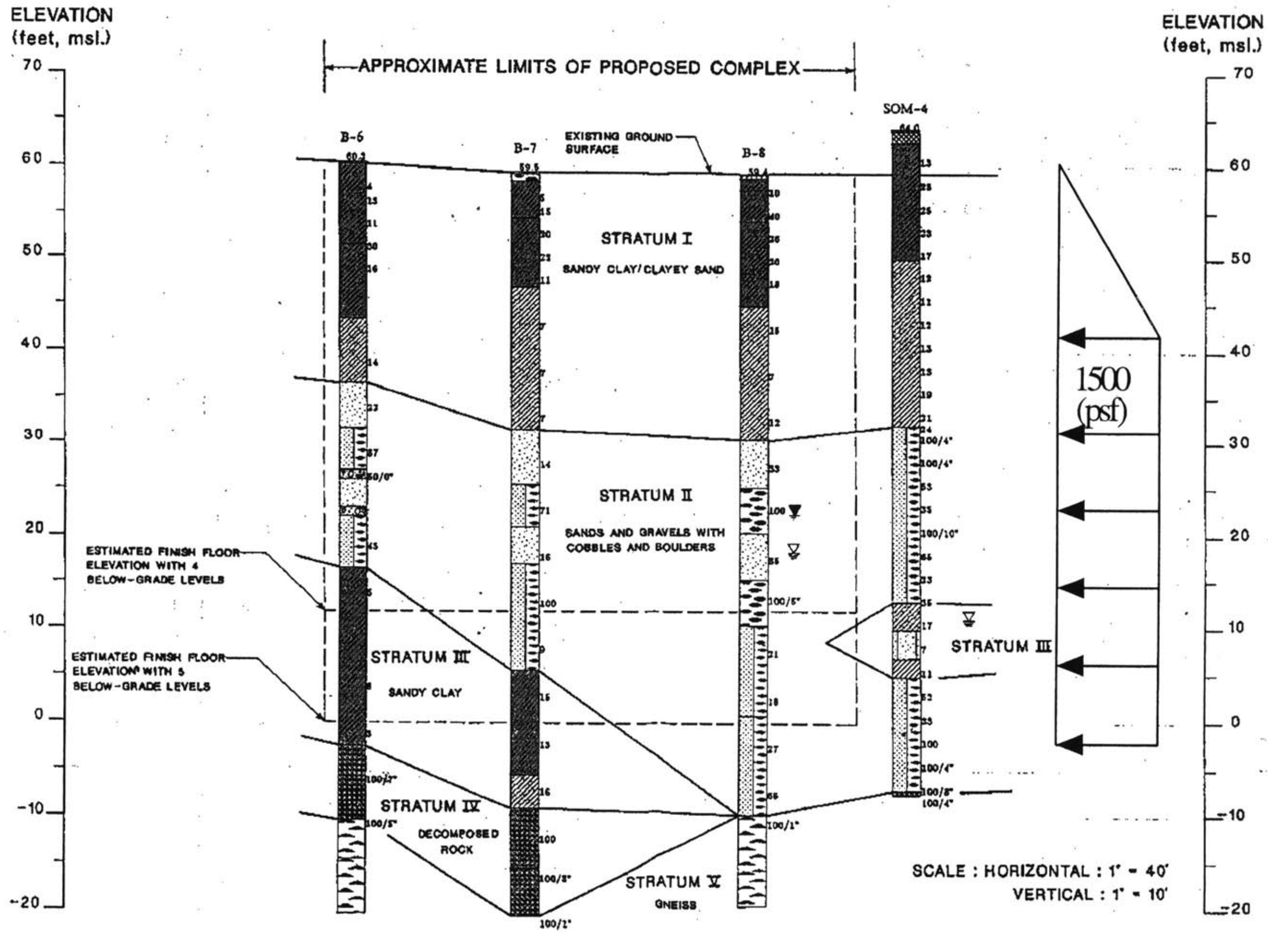


Figure 6.5: Typical soil Profile. World Bank

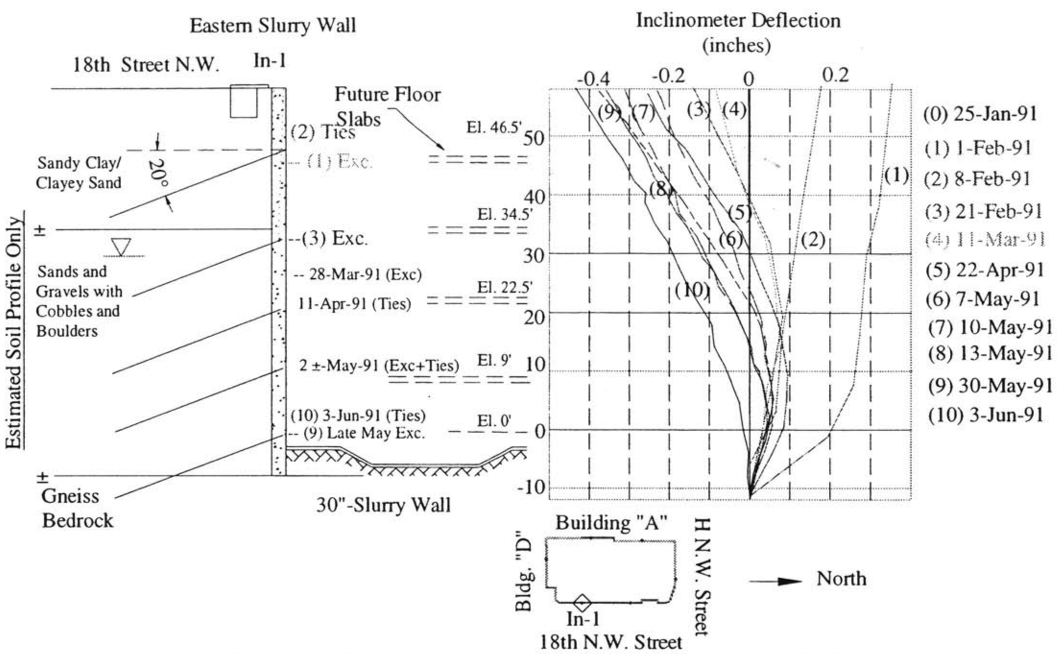


Figure 6.6: Inclinometer In-1, World Bank (W1).

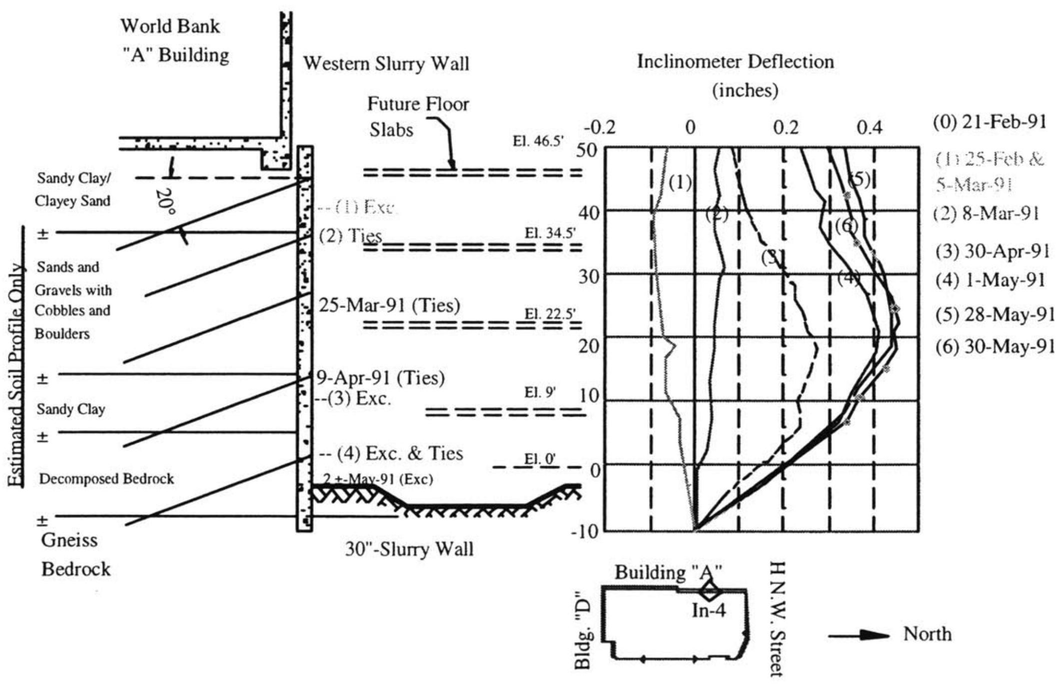


Figure 6.7: Inclinometer In-4, World Bank (W1).

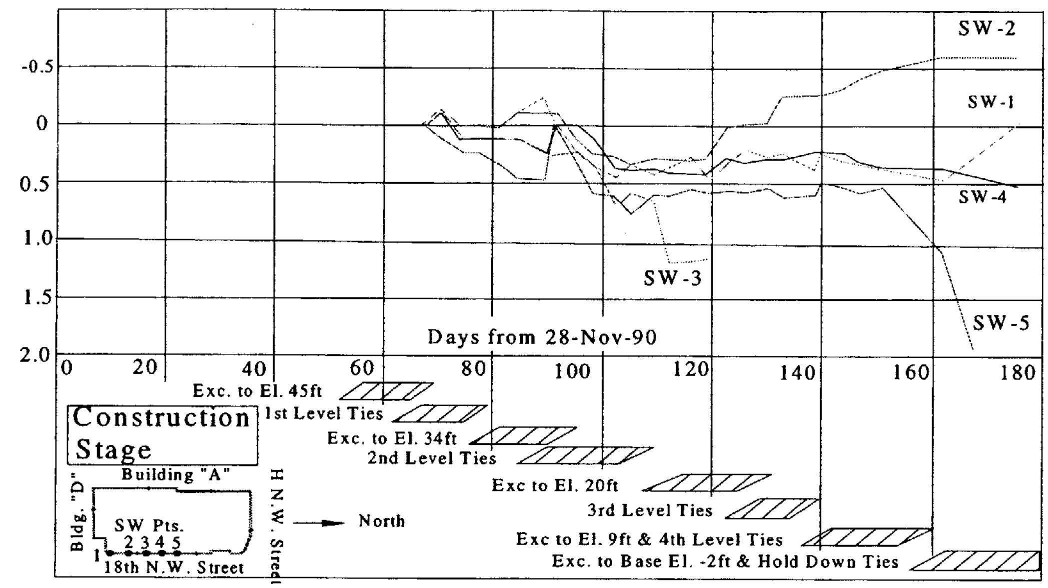


Figure 6.8: Slurry wall settlements and construction activities, (W1).

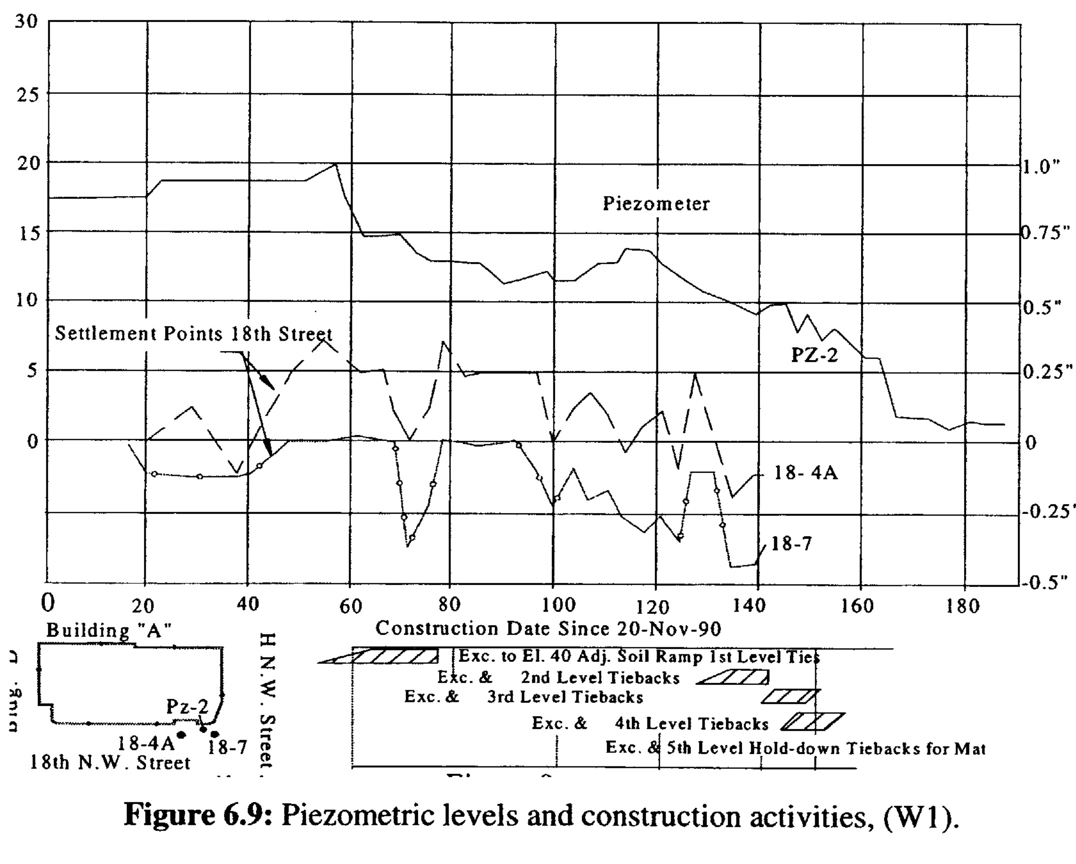


Figure 6.9: Piezometric levels and construction activities, (W1).

6.4.2 Case Study W2, Petworth Subway Station

The 1100 ft long Petworth Subway station lies 60 ft to 100 ft below New Hampshire Ave, includes two entrances which are located on the east and west sides of Georgia Avenue, NW and tie into a passageway leading to the station (Fig. 6.10). Soil retention consists of a 3.0ft-thick permanent slurry wall with cross-lot bracing, while in the south service room, crossover box and entrance soldier beams and lagging wall were used (also with cross-lot bracing). Figure 6.10 also shows the re-routing of New Hampshire Ave. during the three year construction.

Slurry walls were selected in order to mitigate potential for damage to buildings along New Hampshire Avenue and to minimize groundwater drawdown and water inflow into the excavation. Soldier pile and lagging supported a smaller part of the excavation to the south of the site where large diameter utility lines crossed. These remained operational throughout the subway construction. The station was constructed with the cut and cover method.

The site's geology comprised Cretaceous era deposits, but unfortunately a complete subsurface investigation could not be found in the archived data. Internal references mentioned that: Clays (P1) and silty-sands (P2) extended 50 ft to 80 ft to a thin (0 ft to 3 ft) layer of a clay over decomposed to weathered mica-schist bedrock.

The range of horizontal movements was larger than the magnitude of movements itself. Maximum slurry wall movements ranged up to $\delta_w = 0.75$" (Fig. 6.11, 6.13) towards the excavation, and up to $\delta_w = 0.75$" away from the excavation (W2: Fig. 8). Horizontal soil movements were slightly larger for the soldier pile and lagging wall than the slurry wall. Soil 22' behind the soldier pile and lagging wall moved towards the excavation by up to $\delta_H = 0.75$" whereas soil at the interface of the slurry wall and soldier pile and lagging wall 15' from the excavation moved by up to $\delta_H = 0.5$" towards the excavation (Fig. 6.11).

The cross-lot construction sequence clearly affected the bulging wall deflection modes observed in this project (Figs. 6.11, 6.12, and 6.13). Most slurry wall panels bulged the most near the current excavation level.

The excavation at the soldier pile and lagging wall faced major leaking problems in the sandy T2 soils. The dewatering methods used were ineffective, and construction delays resulted. Minor leaks were observed in many panel joints in the slurry wall, where Cretaceous P1 and P2 soils dominate. Grouting easily repaired most leaking joints, except at two panel joints where leaking persisted even after many grouting efforts (Panel 36W).

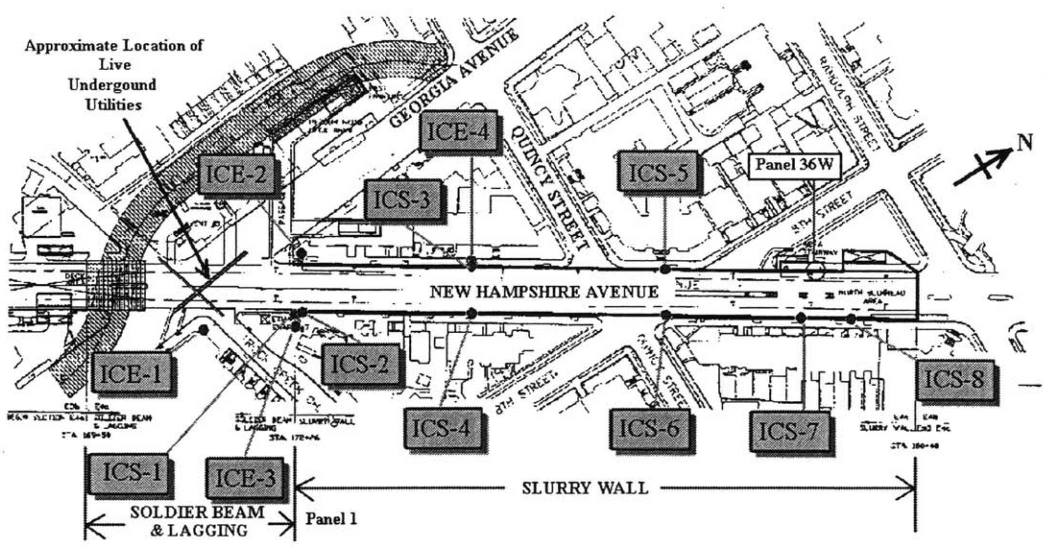


Figure 6.10: Petworth Subway Station site (W2).

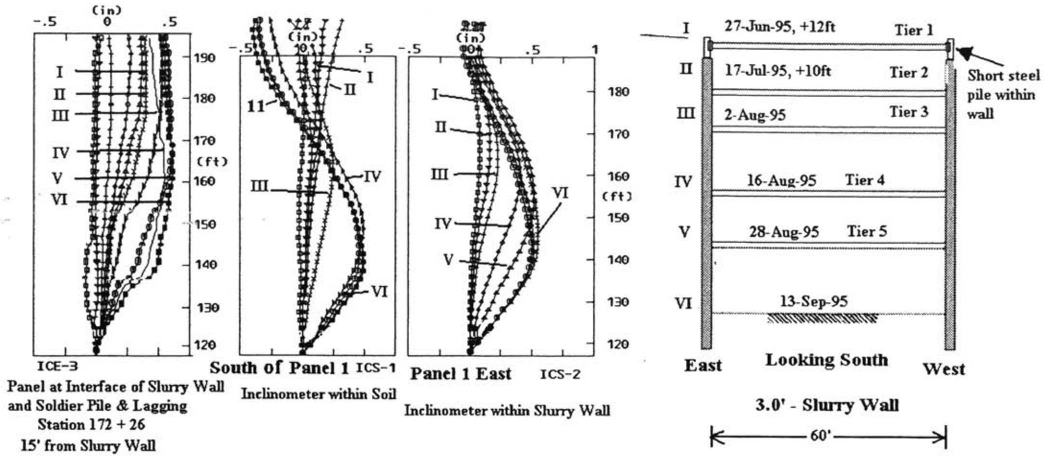


Figure 6.11: Inclinometer deflections at Panel 1 east, (W2).

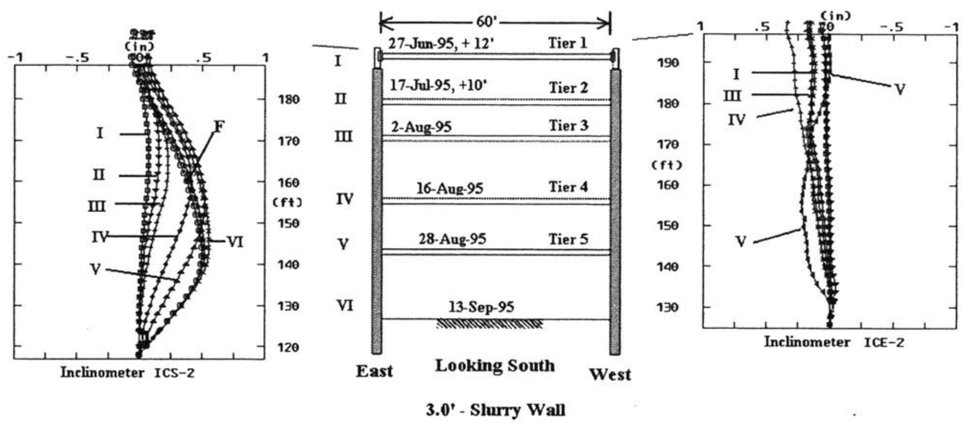


Figure 6.12: Inclinometer deflections at Panel 1, East and West (W2)

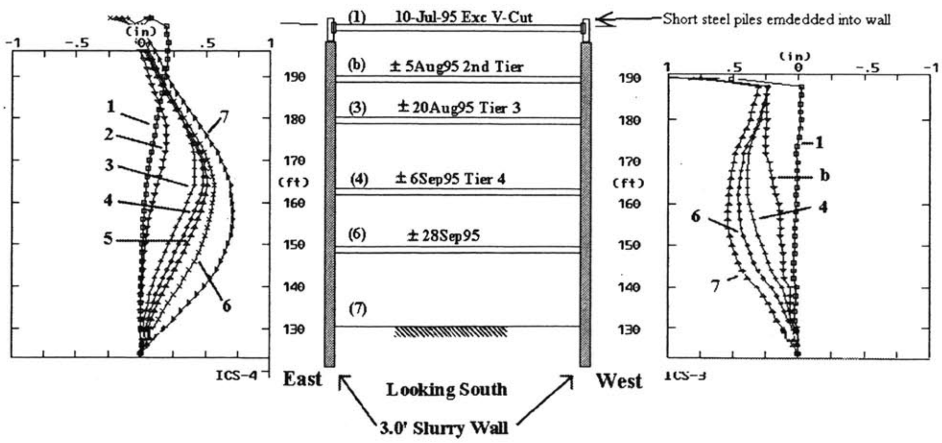


Figure 6.13: Inclinometer deflections at panel 12, (W2).

6.4.3 Case Study W3, Washington Convention Center

The Washington Convention Center project occupies five and one half city blocks between Mt. Vernon Place NW on the south and N Street on the north and from 7th Street NW on the east into 9th Street NW on the west, in Washington DC (Fig. 6.14). The project is at the present time the largest single basement slurry wall excavation in the United States. The excavation has been partially completed as of this date (7-Mar-2000), as some portions of the site remain unexcavated. The excavation site is approximately 1480' long by 500' wide, and encompasses a 493,700 sq. ft. area (Fig. 6.14). At the north the excavation is 55' deep along N Street, but only 30' deep along the southern side.

Slurry wall panels adjacent to the subway at 7th Street are 3.5'-thick, while at all other location the wall thickness is 3.0'. This was done in order to minimize the impact of the excavation on the adjacent subway along 7th Street. The slurry walls also provide water cut-off and thus minimize the dewatering efforts that are required. Temporary bracing for this excavation consists of a mix of tiebacks and prestressed rakers, depending on location.

Subsurface conditions are typical of north central Washington DC, combining a Pleistocene terrace deposit, over Cretaceous coastal plain sediments, lying on a gently slopping bedrock surface at a depth of 90' to 110' across the site.

Three levels of soil tiebacks along the N-Street and the East alley brace the excavation (Fig. 6.15). Along other locations, one or two levels of tiebacks are used in combination with a raker, and a raker heel block system (Fig. 6.16). Adjacent to tunnels, an upper level of tiebacks and a lower level of rakers was used for temporary support. Two levels of rakers and corner braces will be used at the re-entrant corner where an entrance structure to the subway will be located. The raker block is a concrete mass constructed into a depth of 16' below the excavation base, used for the rakers to transfer the load to (Konstantakos and Whittle, 2000). One level of tiebacks, and rakers was used in most panels along Mt. Vernon Pl. NW. where the excavation is approximately 30' deep (Fig. 6.16). An additional level of tiebacks was installed in deeper sections of the excavation.

Currently, the slurry wall excavation at the Washington Convention center has performed well. Conclusions about the overall performance can not be made because the excavation has finished partially as of the last date when data was available to us. The data in this report only cover up to the completed excavation for the northern and southern project sides. In addition, the accuracy of the available inclinometer data is limited since readings were taken at 5ft intervals instead of the regular 2ft, while none of the inclinometers extended beneath the base of the slurry walls.

The largest slurry wall deflection, δ_H = 0.7", occurred at the deepest section of the excavation along N Street, where the wall was braced by three levels of tiebacks (Fig. 6.15). This deflection was not measured directly but it was reconstructed from data before and after re-initialization of inclinometers took place. The wall slightly bent only below the lowest tieback level, and above that showed very little to no bending. Cantilever movements dominated until the second tieback level was installed.

The largest horizontal soil movement, δ_H =0.75", occurred at the southern side where the excavation was only 30' deep, braced by an upper level of tiebacks and a lower level of rakers (Fig. 6.16). The corresponding wall deflection was not equal to the measured soil deflection because the soil inclinometer extended deeper than the base of the wall. This clearly shows the limitations of measured accuracy when inclinometers lack reliable data. Unfortunately, the inclinometers at that location were damaged before the excavation was completed. Measured surface and building settlements were too small and inconsistent to report on. The subway tunnels along next to the excavation 7th Street did not show any significant settlement.

According to interviews with the field engineer, slurry wall construction did not face any major difficulties, nor were there any serious leakage problems from tiebacks or slurry wall joints.

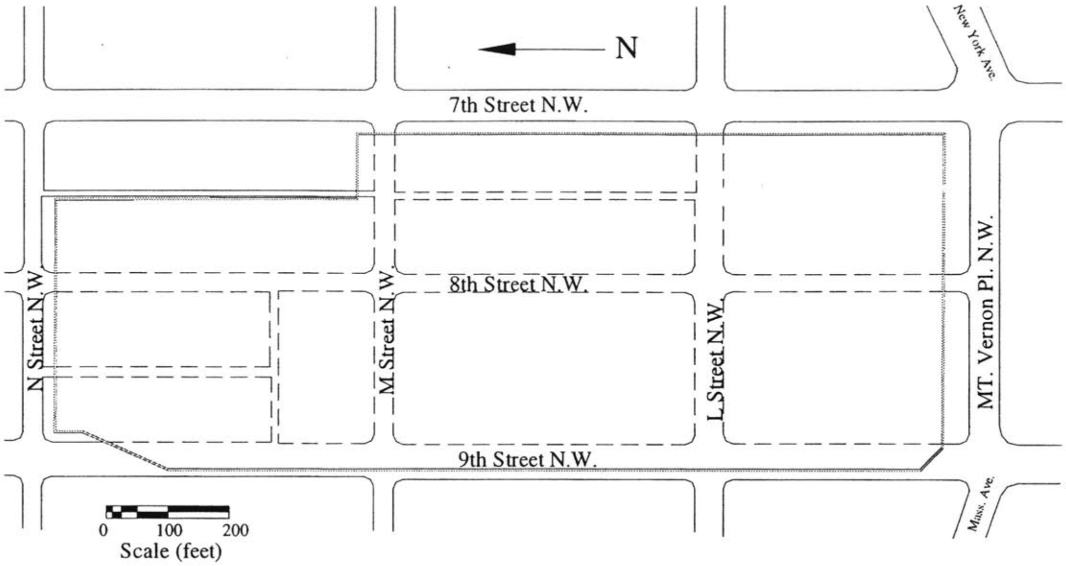


Figure 6.14: Washington Convention Center (W3), project site.

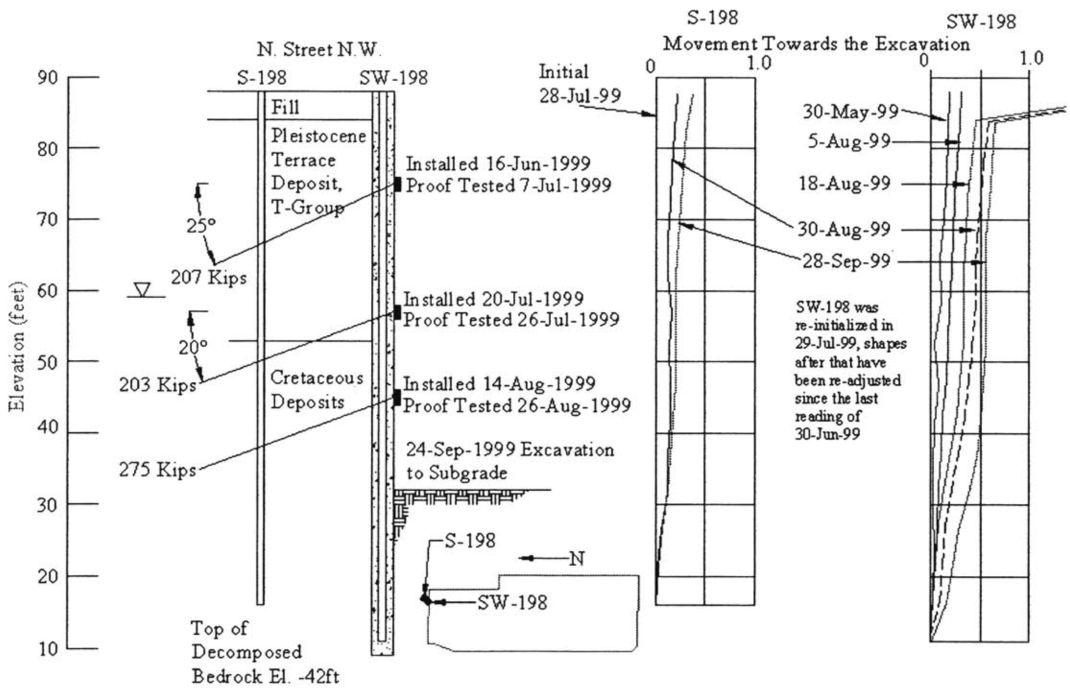


Figure 6.15: Wall and soil deflections at 55'-deep section of Washington Convention Center [W3(b)].

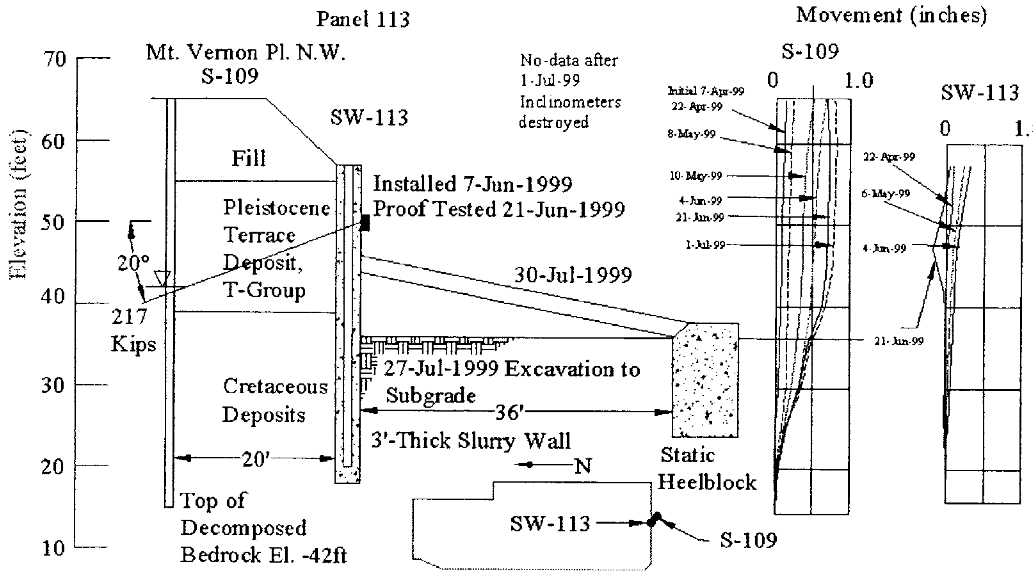


Figure 6.16: Wall and soil deflections at 29'-deep section of Washington Convention Center, [W3(a)].

6.4.4 Case Study W4, Metro Center II

This project involved the construction of a 3-level deep basement next to a historic church, which lies 10' from the excavation. A 2.0'ft-thick diaphragm wall was selected to protect the historic church from excavation induced damage. The soil profile comprises 10ft of fill, underlain by sand to clay to about 50ft to 70ft. Winter et. al. [1991] report that for temporary conditions, the wall was designed with trapezoidal pressures $p = 0.24\gamma H$ which included a 20% increase over normally expected pressured to account for the effect of the existing foundation. The wall was braced by two levels of tiebacks. A typical excavation profile can be seen in Figure 6.17. Winter et al. [1991] report very small bulging deflection less than 0.4" (Fig. 6.18).

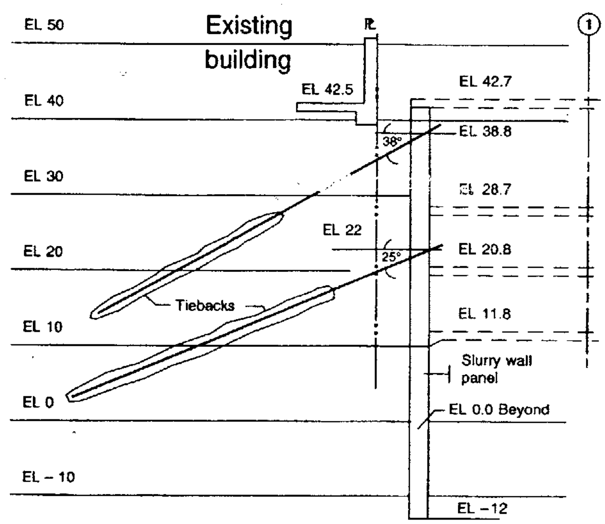


Figure 6.17: Metro Center II (W4), Washington DC (Winter et al., 1991).

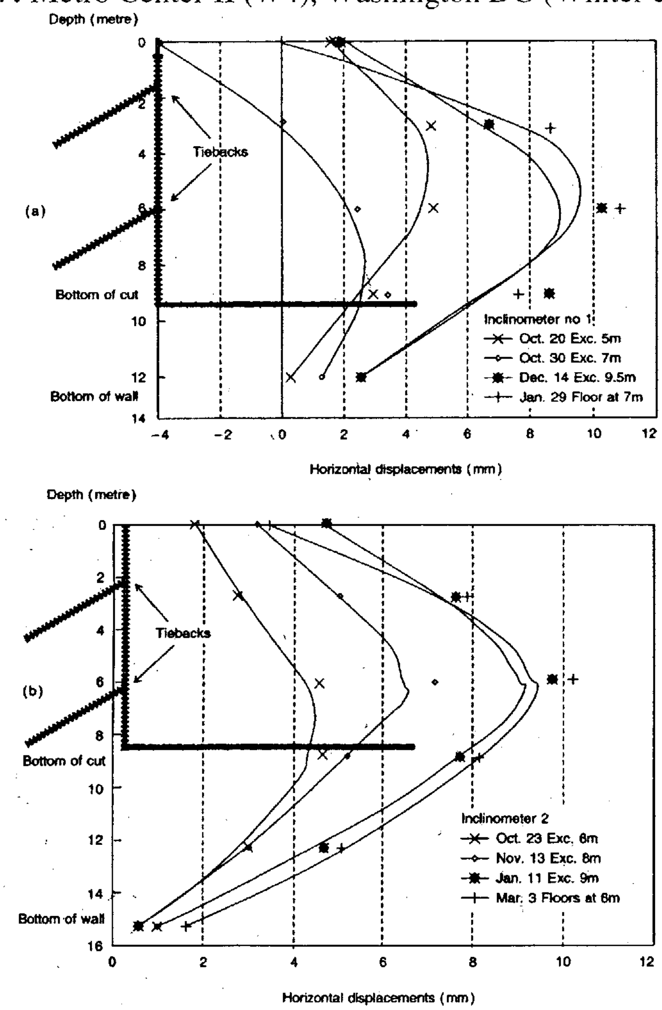


Figure 6.18: Wall deflections at Metro Center II (W4), (Winter et al., 1991)

6.5 Summary of Wall Deflections for Washington DC slurry wall excavations

Figure 6.19 summarizes the maximum wall deflection data after Clough et. al. [1989]. Table 6.8 lists the input parameters used to plot Figure 6.19. Most walls deflected less than δ_W/H=0.1%, with only one project generating movements with $\delta_W/H \geq 0.2\%$. Wall deflection ratio δ_W/H appears to be independent of the system stiffness factor as defined by Clough et al.[1989].

Figures 6.20 and 6.21 show a statistical summary of final and maximum inclinometer deflections from 23 inclinometers. Table 6.7 lists the number of inclinometers per project used to make the statistical summary. Only 6 out of the 70 inclinometers from the Washington Convention Center (W3) were used in order to avoid excessive bias towards the performance of that project. These figures can not be generalized since they are drawn from a limited database. Most inclinometer locations (52.2%) deflected from 0" to 0.5" towards the excavation, while 34.8% deflected from 0.5" to 1.0". In the database all inclinometers deflected less than 1.0", and most likely this is not representative of other projects. There were very small differences in maximum and final wall deflections that did not show up in the frequency plots.

The average values for maximum and final deflections the same namely δ_W/H= 0.07% ± 0.11% or δ_W=0.34" ± 0.38".

Table 6.7: Inclinometers used to derive frequency plots for Washington DC, projects.

ID	Project	Inclinometers
W1	World Bank	7
W2	Petworth Station	8
W3	Washington Covention Center	6
W4	Metro Center II	1
W5	Federal Center Station	1
	Total	23

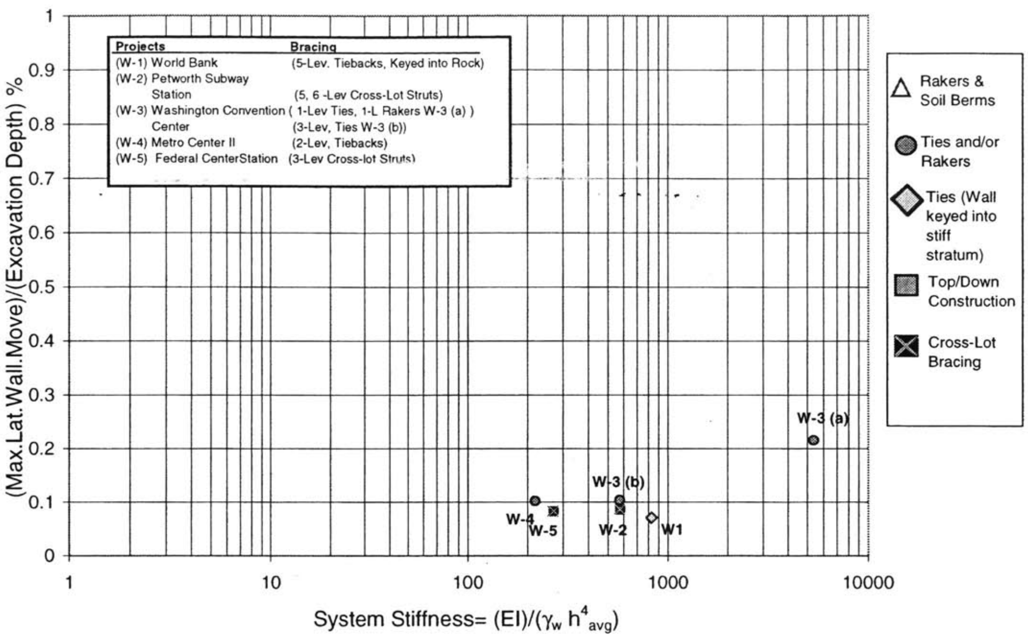


Figure 6.19: Maximum wall deflections for Washington DC projects plotted according to the Clough 1989 approach.

Table 6.8: Summary of system stiffness input parameters and horizontal deflections for diaphragm wall excavations in Washington DC.

	Project	Wall Thick (ft) t	Exc. Depth (ft) H	Vertical Support Spacing (ft) h	f_c' (ksi)	Horizontal Deflections (inches) Caisson+ Other δ_{other}	Total δ_{total}	Exc. Only δ_{Exc}	System Stiffness $EI/\gamma_w\ h^4$	$\delta_{Exc/H}$ %
W1	World Bank	2.5	53	11	5	0	0.45	0.45	827.2	0.071
W2	Petworth Station	3	72	14	5.5	0	0.75	0.75	571.4	0.087
W3 (a)	Washington Covention Center	3	29	8	5.5	0	0.75	0.75	5358.7	0.216
W3(b)	Washington Covention Center	3	56	14	5.5	0	0.7	0.7	571.4	0.104
W4	Metro Center II	2	31	13	5	0	0.38	0.38	217.1	0.102
W5	Federal Center Station	3	60	16	3.5	0	0.6	0.6	267.2	0.083

Note:

Modulus of Elasticity: E (psi)=$57000\sqrt{(f_c'\ (psi)}$ => E(psf)= $395.83\ \sqrt{(f_c'\ (psi)}$ (ACI-8.5.1)

f_c' : 28-Day strength of concrete

Moment of Inertia of Uncracked Section = I

$$I(ft^4)=\frac{(1ft)\ t^3}{12}$$

t (ft) = Diaphragm wall thickness

γ_w = Unit weight of water = 62.4 pcf

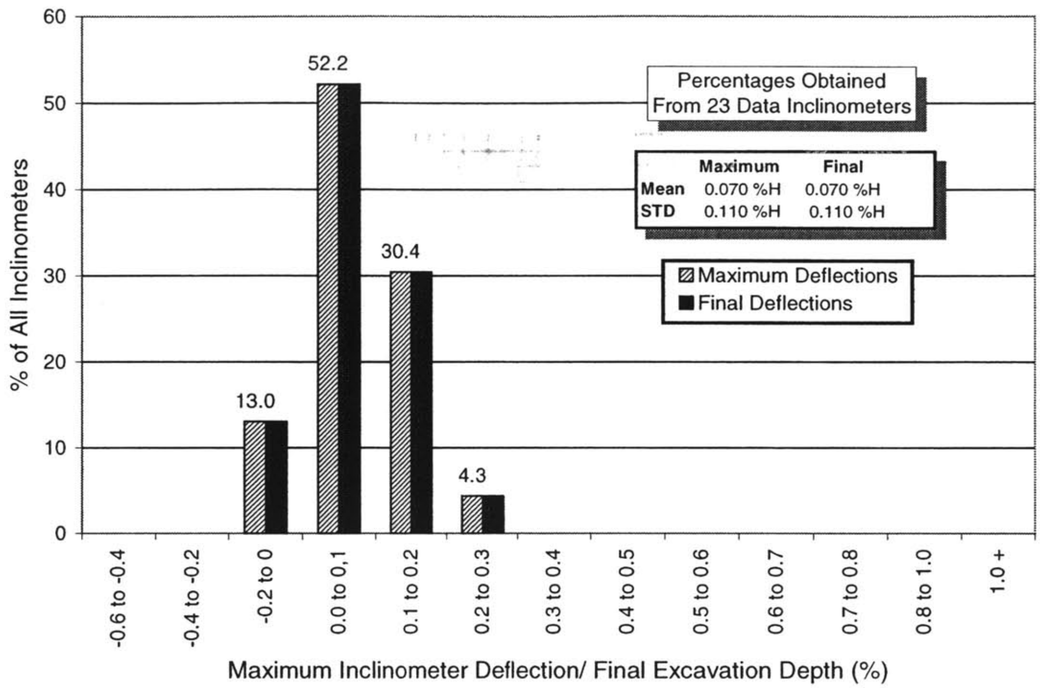


Figure 6.20: Statistical analysis of maximum and final inclinometer deflections as percentages of excavation depth, Washington DC.

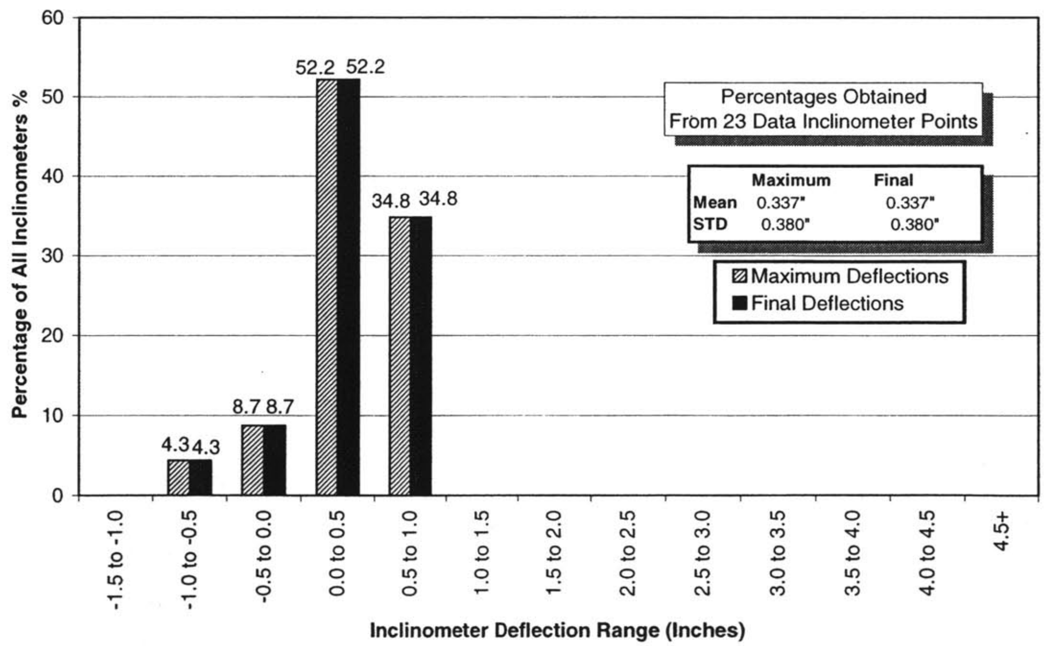


Figure 6.21: Statistical analysis of maximum and final inclinometer deflections for slurry wall excavations in Washington DC.

6.6 Special observations

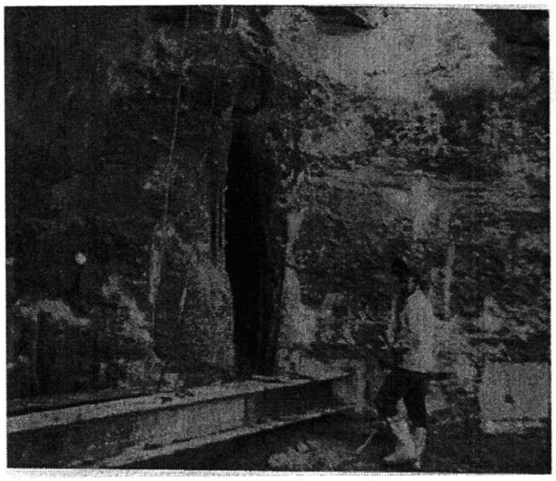
Table 6.9 summarizes the special observations made from three of the five projects in Washington DC. Slurry wall construction in these projects did not face any unexpected difficulties. Occasionally, excess concrete had to be chipped away.

Small leakage was observed through some tieback holes and in some joints. In the World Bank project (W1) a panel near a re-entrant corner was not cleaned properly and as a result a large soil pocket formed in the panel (Fig. 6.23). When this pocket was cleaned it was large enough to fit a single person. Surprisingly, there was very little water infiltration through the open hole at that time. It is most likely that a local zone of low permeability material restricted the water inflow through the open hole.

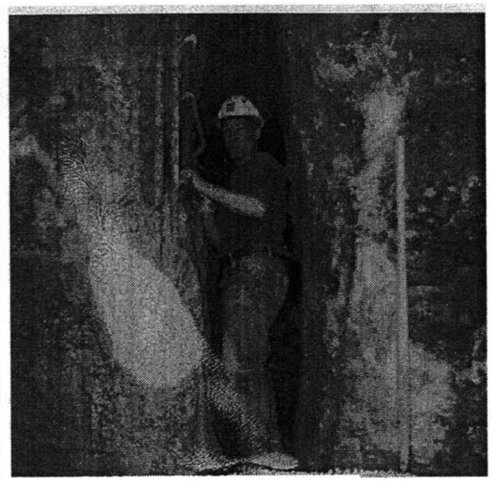
On the other hand, one panel joint in the Petworth Subway Station, leaked repeatedly despite many sealing efforts. In the same project, a soldier pile and wood lagging also faced serious leakage and caused delays in construction

Table 6.9: Special observations in Washington DC slurry wall excavations

Job	Year	Soil Profile	Thickness (ft)	Typical Panel Depth (ft)	Typical Panel Length (ft)	Gs Bentonite mix	Cave-ins Reported	Other Special Observations
(W1) World Bank	1991	Sand and Clay, with gravel and cobbles	2.5	65	18	1.1	no cave-ins	Poor cleaning in one panel resulted in a large soil pocket, which when cleaned was large enough to fit a person. Small leaks though tiebacks.
(W2) Petworth Subway Station	1995	Lower Cretaceous sands and clays, Kps	1.75	75 - 98	25	-	one cave-in mentioned	Excess concrete had to be chipped in many locations. One panel joint was very hard to seal despite many grouting efforts. Small excess concrete overpour of 2% was a major success.The slurry wall leaked far less than a soldier pile and wood lagging wall for the same subway
(W3) Washington Convention Center	1999, 2000	Cretaceous & Pleistocene, Gravel, sands, silt and clay	2.5	54	25	-	none mentioned from interviewing field engineers	Small leaks through tiebacks.

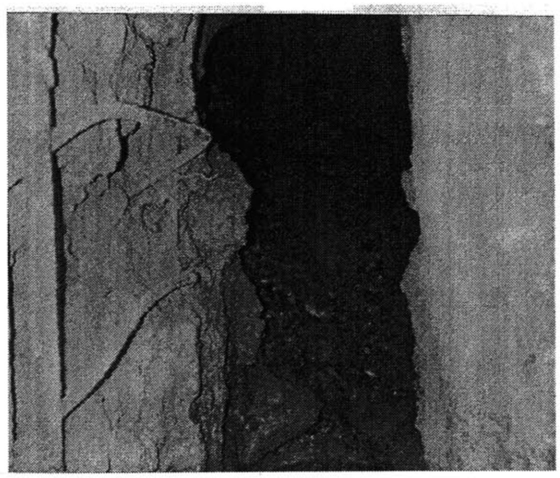


(A)



(B)

(C)



(D)

Figure 6.22: Large void at a slurry wall panel due to poor cleaning (A) Large void, (B) A person inside the void, (C) Water leaking through the void, (D) Distorted panel reinforcement

6.7 Summary

The five slurry wall excavations in Washington DC have performed very well, as the excavation induced deformations were small. Maximum wall deflections did not exceed 1.0" and were typically in the order of 0.5" to 0.75". Wall deflections varied a lot even within the same project, since in some panels in the same job deflected back into the retained soil by as much as other panels deflected towards the excavation. Walls subjected to building surcharge deflected in a bowing mode (Type II), whereas walls not subject to building surcharge tended not to bow.

Measured building and surface settlements were generally smaller than wall deflections. Settlement of individual slurry wall panels depends on construction or local soil variations (like softer soil zones at the bottom of the wall).

Small water leakage or dampness was occasionally observed through panel joints or tieback holes. In most cases, grouting easily repaired leakage through panel joints. Other for utilities buried in shallow soil, slurry wall construction in the studied projects did not face any major difficulties.

The major lesson learned from Washington DC projects is that when deformations caused by the excavation are very small, individual panel behavior can be affected by local details in construction, soil profile, and adjacent surcharge loading.

Chapter 7

San Francisco, Other US. Projects, Review of Existing Literature

Chapter 7

San Francisco, Other US. Projects, Review of Existing Literature

7.1 Scope of this Chapter

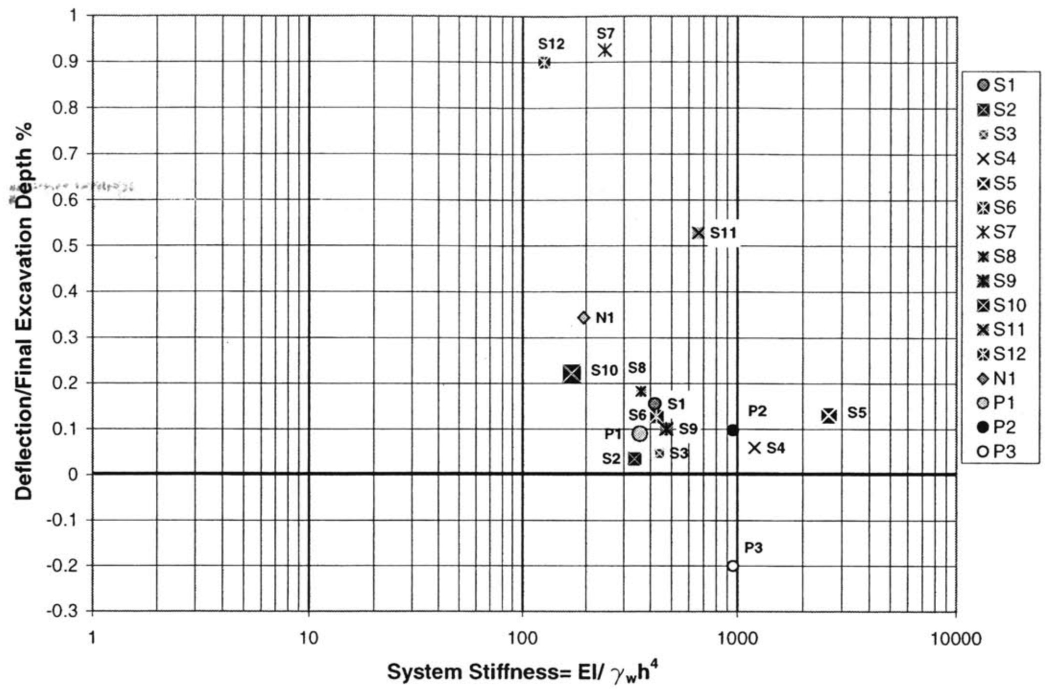
This chapter discusses the performance of slurry wall excavations in cities other than Boston, Chicago, and Washington DC. Only one of these projects, Yerba Buena Tower (S9, Konstantakos & Whittle, 2000) in San Francisco, represents a new case study. Information for the rest of the projects has been obtained from the published literature. The list of these projects can be seen in Table 7.1. Figure 7.1 shows the maximum wall deflections from these projects plotted as a function of system stiffness as proposed by Clough et al. [1989]. Table 7.2 summarizes the input parameters used to plot Figure 7.1. The majority of the projects plotted beneath $\delta_H/H=0.3\%$, and the ratio δ_H/H decreases slightly with increased system stiffness. However, the system stiffness does not seem a sensitive factor for the measured final deflections.

Table 7.1: List of San Francisco and Other Projects

	Project	Wall Thick (ft)	Exc. Depth (ft)	Vertical Support Spacing (ft)	t_c' (ksi)	Horizontal Deflections (inches) Other	Total	Exc. Only	System Stiffness		Factor of Safety against Basal Heave
		t	H	h		δ_{other}	δ_{total}	δ_{Exc}	$EI/\gamma_w h^4$	$\delta_{Exc/H}$ %	FS
	San Francisco										
S1	Security National Bank	3	48.5	11	3	0	0.9	0.9	1107.2	0.155	>2
S2	Embacardero Bart Zone 1	3.5	70	10	3	0	0.3	0.3	2574.2	0.036	>2
S3	Embacardero Bart Zone 2	3.5	70	10	3	0	0.4	0.4	2574.2	0.048	>2
S4	Embarcadero Bart Zone 3	3.5	70	10	3	0	0.5	0.5	2574.2	0.060	>2
S5	Embarcadero Bart Zone 4	3.5	70	10	3	0	1.1	1.1	2574.2	0.131	1.8
S6	Civic Center Bart Station	3	78	11	3	0	1.2	1.2	1107.2	0.128	
S7	One Market Plaza	2.5	36	10	3	0	4	4	938.1	0.926	1.7
S8	China Basin Pump Plant	3	64	14.5	3	0	1.4	1.4	366.7	0.182	>2
S9	Yerba Buena Tower	3	66	14.5	5	0	0.8	0.8	473.4	0.101	>2.4
S10	Islais Contract E	3.333	45	20	4.5	0	1.2	1.2	170.2	0.222	
S11	MUNI Metro Turnback	3	36	10	4.5	0	2.3	2.3	1985.4	0.529	
S12	Southern Pacific	2.5	37	20	3	0	4.0	4.0	58.6	0.901	
	New York										
N1	World Trade Center	3	70	17	3	0	2.88	2.88	194.1	0.343	
	Arkansas										
P1	Wilbur Mills. D. Dam	3-T	79	49	5	0	0.85	0.85	356.5	0.090	
	Boston										
P2	Harvard Square Station	3	38.5	12	4.5	0	0.45	0.45	957.5	0.097	
	Oregon										
P3	Bonnevile Navigation Lock	3	50	12	4.5	0	-1.2	-1.2	957.5	-0.200	

Note: **TB** -Tiebacks, **CLB** - Cross-Lot Bracing, **TD** - Top/down, **R** – Rakers, **SB**-Soil Berms, **CB** – Corner Bracing, **IB**-Internal Bracing, **PC** – Precast, **PT**–Post Tensioned, **SP** – Soldier Piles

Project	Authors
S9	Konstantakos & Whittle, 2000
S1-S8	Clough and Buchignani, 1981
S11	Adams and Robinson, 1996
S10	Koutsoftas et al., 2000
N1	Saxena, 1974
P1	Berger and Tryon, 1999
P2	Hansmire et al., 1989
S12	Clough, 1975
P3	Munger et al., 1990



Cross Lot Bracing Internal Bracing Tiebacks Tiebacks, keyed into rock

Figure 7.1: Maximum wall deflections for San Francisco and other diaphragm wall excavations plotted according to the Clough et al. (1989).

Table 7.2: Summary system stiffness input parameters for various projects.

	Project	Wall Thick (ft)	Exc. Depth (ft)	Vertical Support Spacing (ft)	t_c (ksi)	Horizontal Deflections (inches) Other	Total	Exc. Only	System Stiffness		Factor of Safety against Basal Heave
		t	H	h		δ_{other}	δ_{total}	δ_{Exc}	$EI/\gamma_w h^4$	$\delta_{Exc/H}$ %	FS
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S3	Embacardero Bart Zone 2	3.5	70	10	3	0	0.4	0.4	2574.2	0.048	>2
S4	Embarcadero Bart Zone 3	3.5	70	10	3	0	0.5	0.5	2574.2	0.060	>2
S5	Embarcadero Bart Zone 4	3.5	70	10	3	0	1.1	1.1	2574.2	0.131	1.8
S6	Civic Center Bart Station	3	78	11	3	0	1.2	1.2	1107.2	0.128	
S7	One Market Plaza	2.5	36	10	3	0	4	4	938.1	0.926	1.7
S8	China Basin Pump Plant	3	64	14.5	3	0	1.4	1.4	366.7	0.182	>2
S9	Yerba Buena Tower	3	66	14.5	5	0	0.8	0.8	473.4	0.101	>2.4
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S11	MUNI Metro Turnback	3	36	10	4.5	0	2.3	2.3	1985.4	0.529	
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	New York										
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	Oregon										
P3	Bonnevile Navigation Lock	3	50	12	4.5	0	-1.2	-1.2	957.5	-0.200	

7.2 San Francisco

Clough and Buchignani [1981] carried out a major review of slurry wall excavations in San Francisco. The local slurry wall excavation practice was dominated by soldier pile and tremie concrete walls. The soil conditions in these projects vary from sand to deep soft clay, where basal stability is a major design issue.

The San Francisco Bay Area is in the California Coastal Range Province, a region characterized by northwest-trending ridges and valleys that generally parallel the major geologic structures such as the San Andreas and Hayward fault systems. The Bay Area bedrock is composed of highly consolidated, tectonically deformed, sedimentary, volcanic, and metamorphic rocks of the Franciscan assemblage. Franciscan rocks are closely associated with large bodies of serpentine. The Franciscan rocks usually consist of sheared shale and sandstone, with isolated masses of other rock types that are referred to as melagne.

There are three major active faults in the San Francisco area: San Andreas, Hayward, and Cavaleras faults. The closest of these faults to the site (8.5 miles) is the San Andreas fault which is capable of Richter scale 7 devastating earthquakes, thus seismic loading is a major consideration in design.

In the eastern part of the city, soft Bay Mud soils dominate, while sands are present further inside. The general soil profile in eastern San Francisco consists of: a) 20' of Rubble Fill, b) 50' Recent Bay Mud with Undrained shear strength increasing linearly with depth from s_u = 0.6 ksf at the top to s_u = 1.25 ksf at the base of the layer, c) a variable thickness dense sand layer, d) and a variable thickness Old Bay Mud. The Recent Bay Mud thickness increases from 0' at 2200' from the Bay to 100 ft near the Bay in the vicinity of Market Street (Fig. 7.2). Other for the high water table the sands do not create any other difficulties.

7.2.1 Reported Performance of Excavations in San Francisco

Clough and Buchignani [1981] report the following regarding the slurry wall excavations that they have studied:

1) No problems were associated with wall movements or street settlements where slurry walls were used in the Bay area.
2) Wall movements associated with slurry walls have been relatively small although the system stiffnesses were similar to those used in sheet pile systems.
3) Lack of problems with movements were attributed to: a) the low level of ground disturbance during slurry wall installation; b) the ability of the slurry wall to cut off water flow during the excavation; c) the intimate contact created between the soil and the slurry wall by tremie concrete pressure.
4) Leakage: minor amounts of leakage occurred through panel joints in almost all cases below the water table.

The walls deflected in a bowing shape in all the cases for which Clough and Buchignani [1981] reported deflection patterns (Fig. 7.3). The largest wall deflections occurred in the 36'-deep One Market Plaza project, reaching δ_H = 4.0" slightly below the excavation base. In this project, only one level of internal braces was used in combination with 25'-deep soil berms adjacent to the wall face. Deflections reached δ_H = 3.0" when the excavation base was reached at the center of the site, with an additional δ_H = 1.0" movement caused by the removal of the soil berm. The braces where effective at restricting movements at the top of the wall but the soil berm was not able to restrict deformations since the Recent Bay Mud is particularly soft at the upper 25'.

In the Islais Creek Contract E (Adams, and Robinson, 1996), a jet grout kicker slab was constructed below the final grade before any excavation took place (Fig. 7.4). The SPTC wall was pushed back by the grouting operations for the kicker slab installation. The kicker slab was effective in restricting movements as the wall deformed primarily by bending between the lowest strut and the kicker slab. All soldier piles were keyed into the underlying rock at a depth of 100'.

It is interesting to note that the outer soldier piles of the first constructed panels showed considerable axial bending about the weak axis during concreting, and in some cases broke out of their restraints at the guide walls. Inclinometers that were attached on the soldier piles indicated that the piles had deflected at their bases initial position by as much as 4" at the middle of the Bay Mud. It was believed that this was caused by the unbalanced pressures between the concrete tremie and the soil acting on the weak axis of the piles.

In the 42'-deep MUNI Metro Turnback project (Koutsoftas et al, 2000), the tremied part of the SPTC wall extended to 66' beneath the surface, and the soldier piles extended to 135'. The 3 levels of bracing effectively restricted deformations above the excavation base. However, deformations caused by the excavation were larger beneath the base of excavation, reaching δ_H =2.3" near the base of the concreted part of the wall (Fig. 7.5). The net effect of concreting and pile driving construction caused soil to move away from the excavation by 1.1". This suggests that the pressures from the concrete tremie are larger than those exerted by the soft Bay Mud.

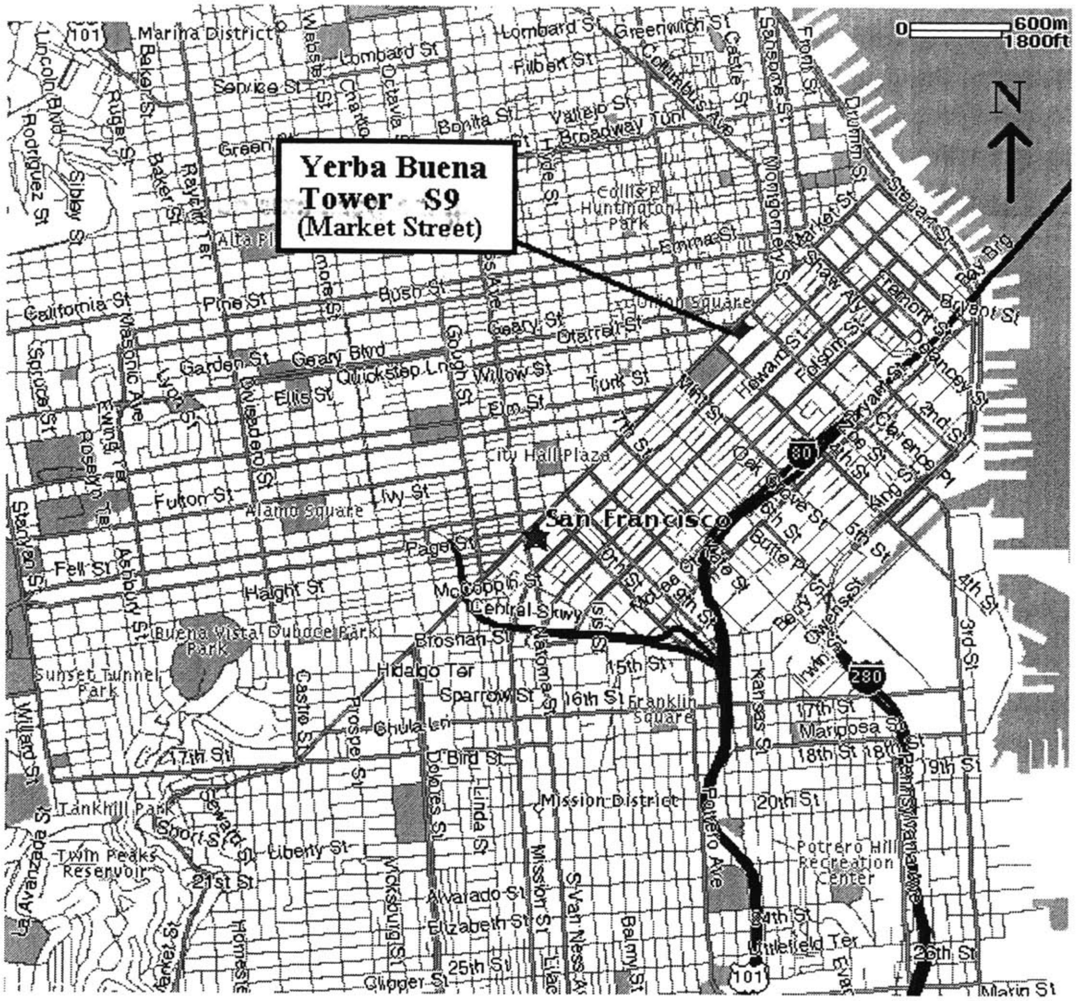


Figure 7.2: Map of downtown San Francisco, and Yerba Buena Tower (S9).

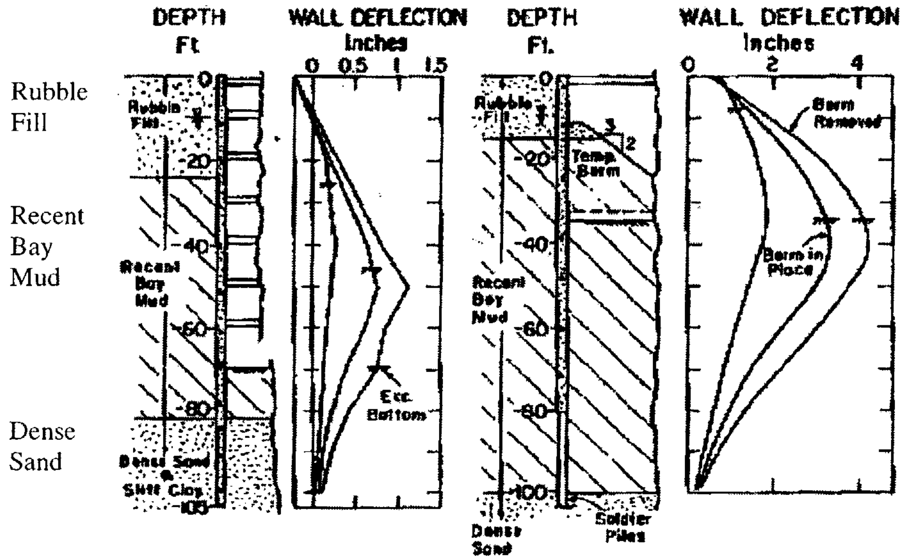


Figure 7.3: Measured lateral wall movement profiles for Embacedero BART Station and One Market Plaza Building slurry walls, Clough and Buchignani, 1981.

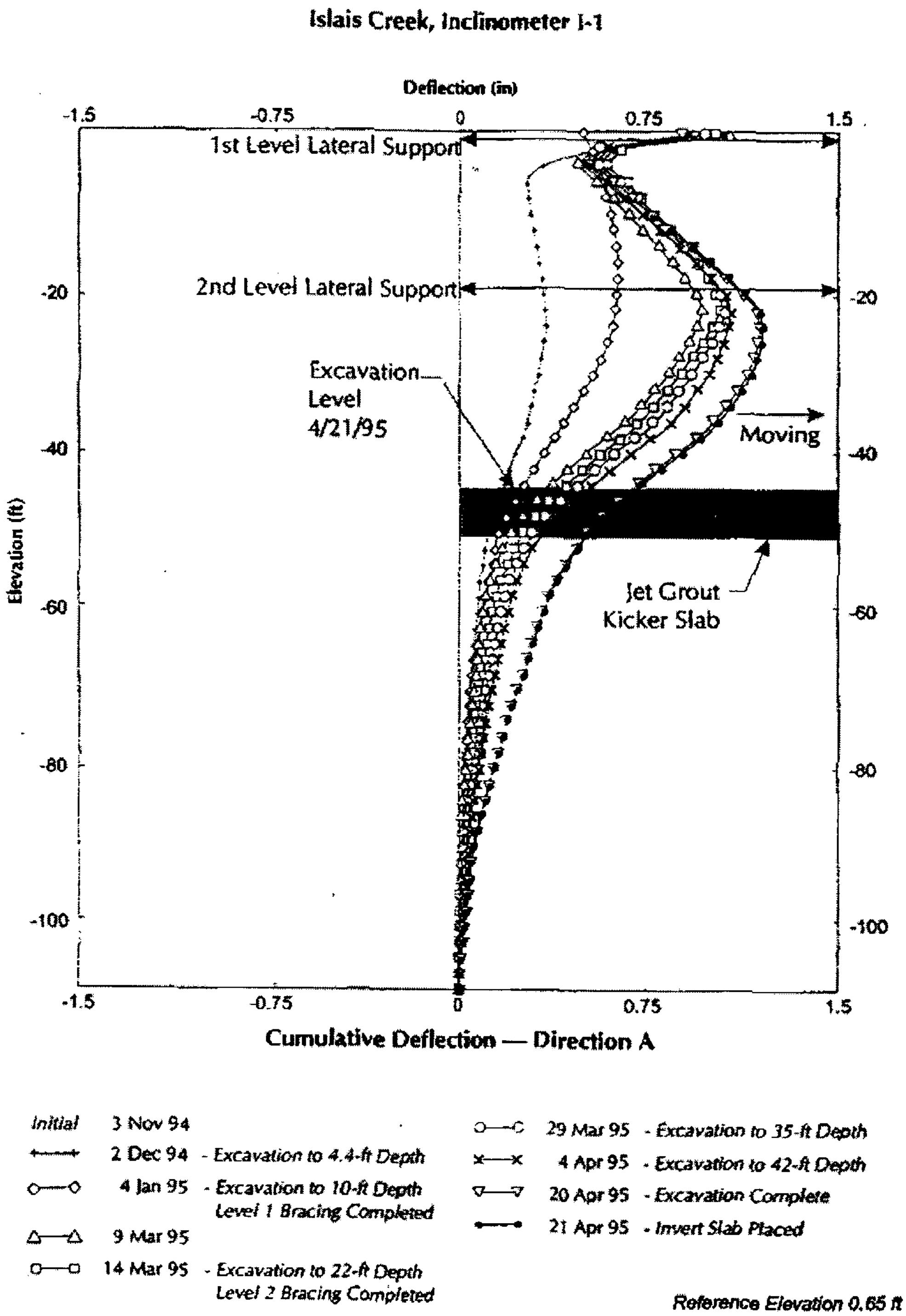


Figure 7.4: Islais Contract E, (Adams and Robison, 1996).

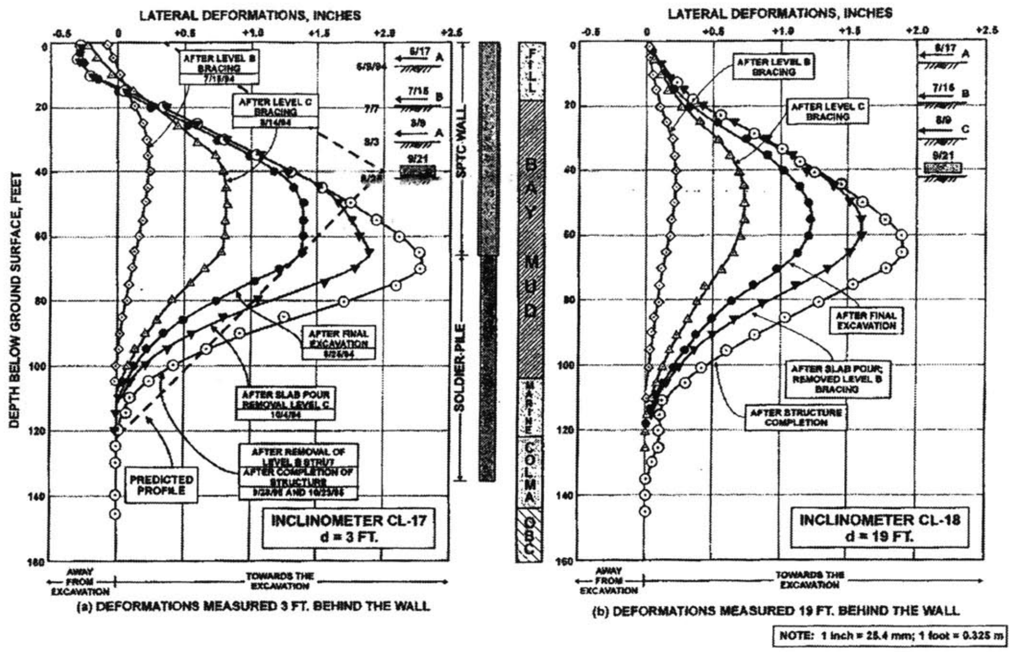


Figure 7.5: Deformations measured at Muni Turnback project (Koutsoftas et al. 2000).

7.2.2 Case Study C9, Yerba Buena Tower

The Yerba Buena Tower (S9) (or CB-1 Tower) is the only known recent project in the city to be constructed as a standard reinforced concrete diaphragm wall without soldier piles. The project is located on the south side of Market Street between Third and Fourth Streets, in San Francisco (Fig. 7.2). The excavation encompasses a flat area of 205' by 210' in dimension, where a 7-level, 66'-deep basement has been constructed (Fig. 7.6). The construction of the 39-floor tower was not completed as of 2-Feb-2000, but the base of the excavation has been reached and the basement floors have been constructed.

Slurry walls were selected for temporary and permanent support of the excavation, mainly because important adjacent structures (BART and MUNI tunnels) are located beneath Market Street to the north of the project (Fig. 7.8). The Marriott Hotel is located to the west of the project and it includes two underground parking levels. A six story brick building with one basement level

(735 Market Street) is located along the northeastern project border and a two-story former PG&E substation, is located along the southern project side. Figure 7.6 shows the site plan.

A general subsurface profile can be seen in Figure 7.7. The static groundwater table is located 36 ft beneath the surface. The general subsurface profile at the site consists of:

a) 4 ft-15ft of miscellaneous fill.
b) 5 ft-14 ft dune sand.
c) Thin 3 ft to 5 ft-thick dark brown marsh deposit (loose to medium stiff to stiff, silty clay to clayey silt, trace sand and gravel).
d) 15 ft to 17ft of medium dense to dense clayey sand layer with interbedded sandy clay lenses.
e) 45 ft to 60 ft-thick dense to very silty sand.
f) 30 ft to 50 ft-thick (at 90 ft to 143 ft depth) Old Bay Clay (OBC). This clay is stiff, slightly overconsolidated, moderately compressible, with high plasticity.
g) Dense to very dense silty sand interbedded with very stiff to hard clay lenses.
h) Franciscan bedrock underlies the site at about 230 ft depth.

Slurry walls 3'-thick, approximately 102' deep, embedded a minimum of 10' into the Old Bay Clay were used to form the deep basement walls. The base of the excavation reached from El. –32 ft to El. –36ft depending on location (approximate depth of 65ft). Internal bracing was used to support the northern side of the project since tiebacks were not possible (obstructed by BART and MUNI tunnels). Temporary tiebacks braced part of the eastern, western, and the whole of the southern slurry walls, with the fixed length installed within the silty sand layer. Typical tieback lock-off loads were about 100 kips. Preloaded inclined rakers, installed within soil berms provided the lowest level of support. Four levels of tiebacks were used along Stevenson Street, while one level of tiebacks was used along the Marriott Hotel. Bracing plans can also be seen in Figure 7.6.

The slurry wall along Marriott was constructed from El. 0ft (surface at El. 32ft) and a concrete wall extending to the surface was thereafter constructed.

The excavation performed well, as deflections and settlements were kept to moderate values. Deformations in this job occurred mostly below the base of the excavation, since the internal bracing was effective in restricting deflections above the excavation base. Maximum horizontal soil movements according to inclinometers ranged up to δ_H=0.85" along Market Street at 5.5' from the excavation, where BART & MUNI tunnels run under (Fig. 7.8). At 22.5' from the excavation the maximum horizontal soil movement decreased to δ_H=0.75".

When the excavation base was reached, walls along Market Street deflected the below the excavation base. The embedded portion of the wall simply translated with little to no signs of bending (12-Aug-99: Fig. 7.8). Thereafter, wall deflections increased as basement floor construction progressed.

Adjacent points along the Marriott Hotel heaved by up to 1.2", while points along Stevenson Streets and 735 Market Street building settled by as much as 1.8". The majority of the settlement at the 735 Market Street building occurred before any excavation took place and was probably caused by slurry wall construction, and fill sliding under the guidewalls. Surprisingly, one point 57' away from the excavation settled by δ_V=1.55" most likely most likely because of tieback disturbance (Konstantakos and Whittle, 2000, S9). Surface and building settlements ranged from δ_V/V=+0.2% to δ_V/V=-0.15% (heave), and almost no movement occurred more than 60' from the excavation (Fig. 7.10). Along Market Street settlements were smaller reaching δ_V=0.85" (Fig. 7.9). Before the excavation base was reached, settlements at Market Street were in the order of δ_V=0.2", but they increased as wall movements increased during basement floor construction.

Inclinometers indicated that slurry wall construction caused larger soil movements in the upper 10' of the fill layer (δ_H=2.7"). Sliding of the fill under the guidewalls probably caused these deformations. Horizontal soil movements that were caused by slurry wall construction were almost constant for the rest of the wall depth (δ_V=0.1"-0.2").

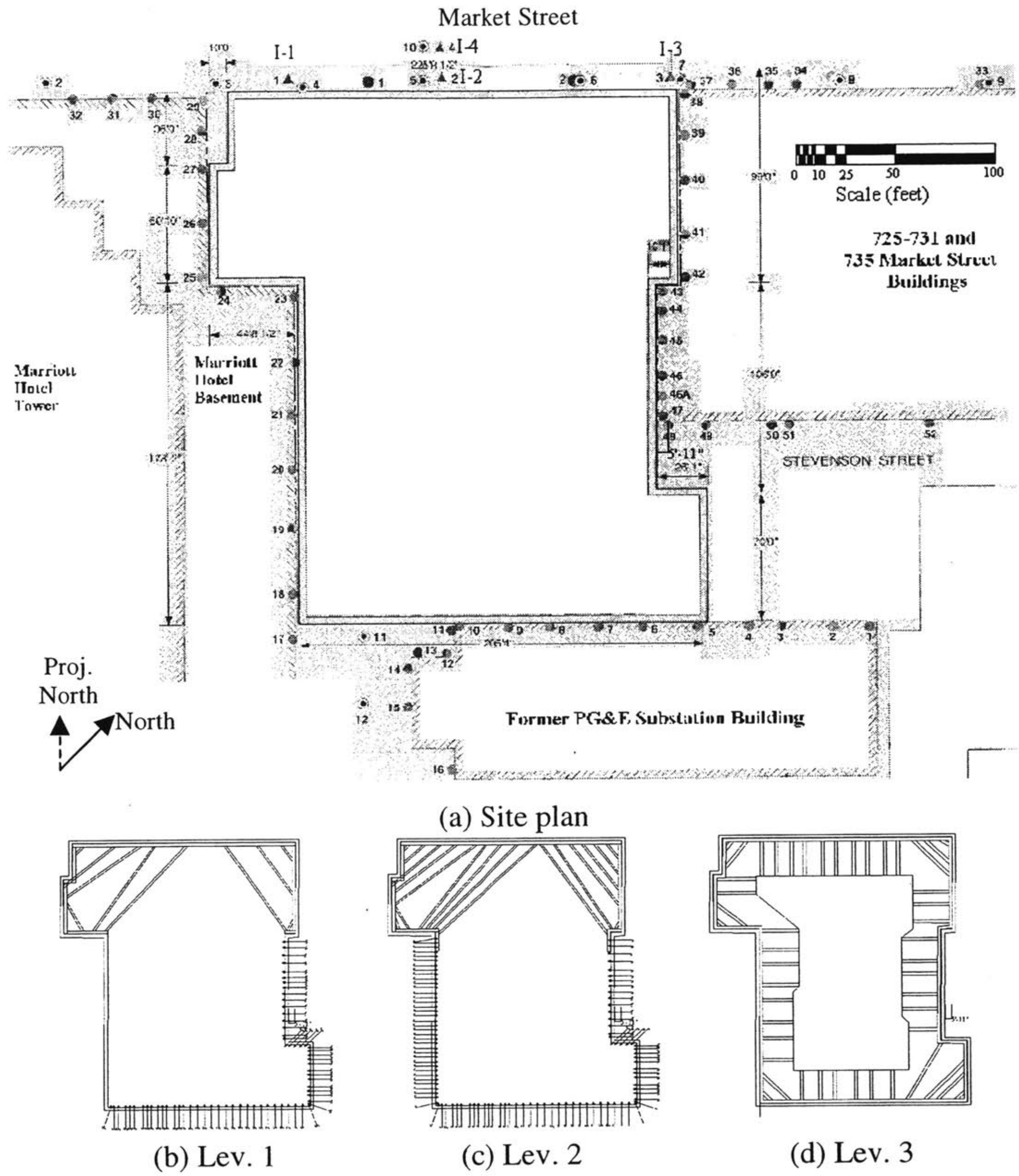


Figure 7.6: Yerba Buena Tower (S9): a) site, b) level 1 bracing at El. 18ft, c) level 2 bracing at El. –5 ft, and d) level 3 bracing (rakers) at El –19.75ft, surface at approx. El. 32ft, excavation base at El. –32 ft to El. –36 ft, (SFCD).

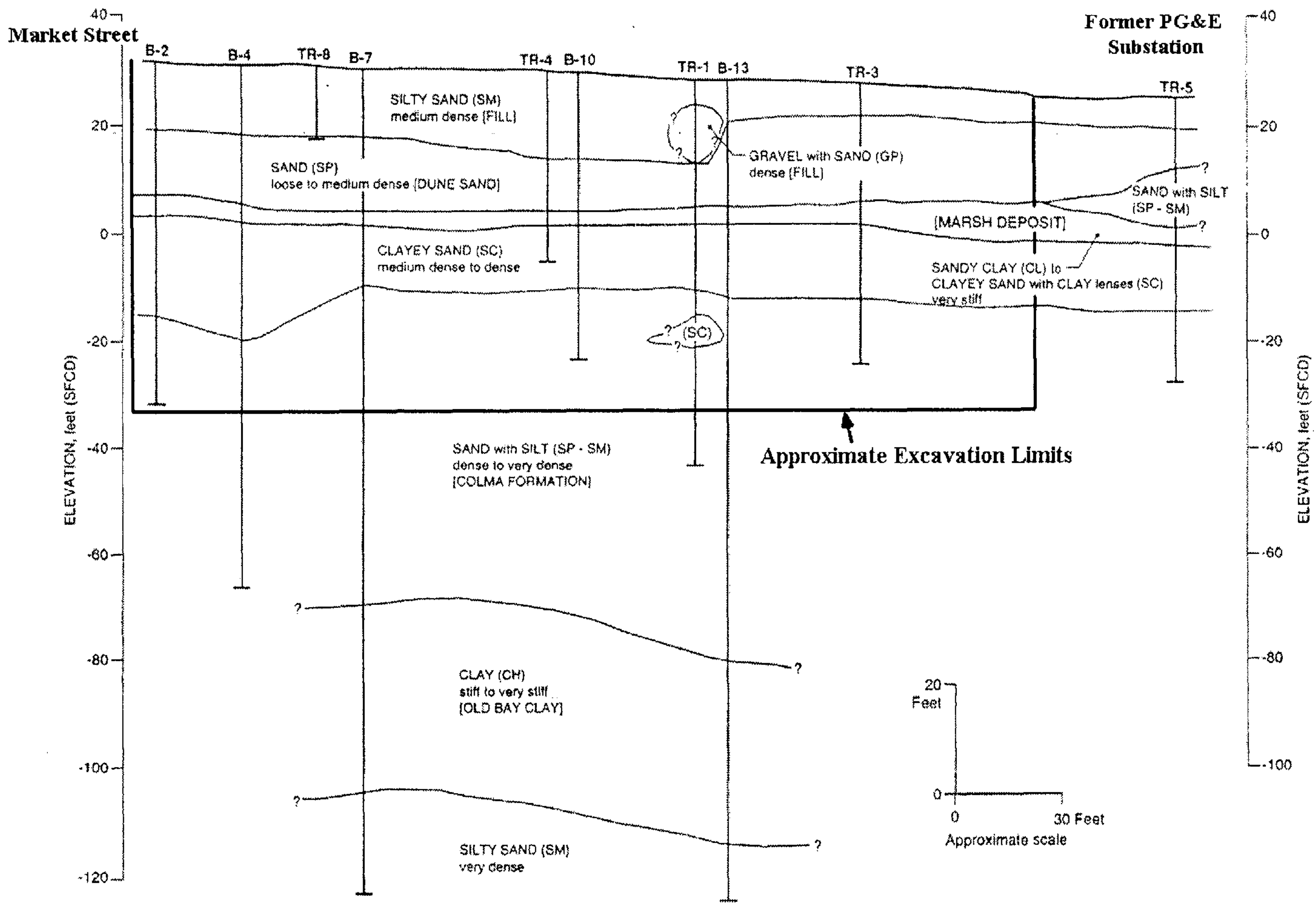


Figure 7.7: Generalized soil profile, Yerba Buena Tower (S9).

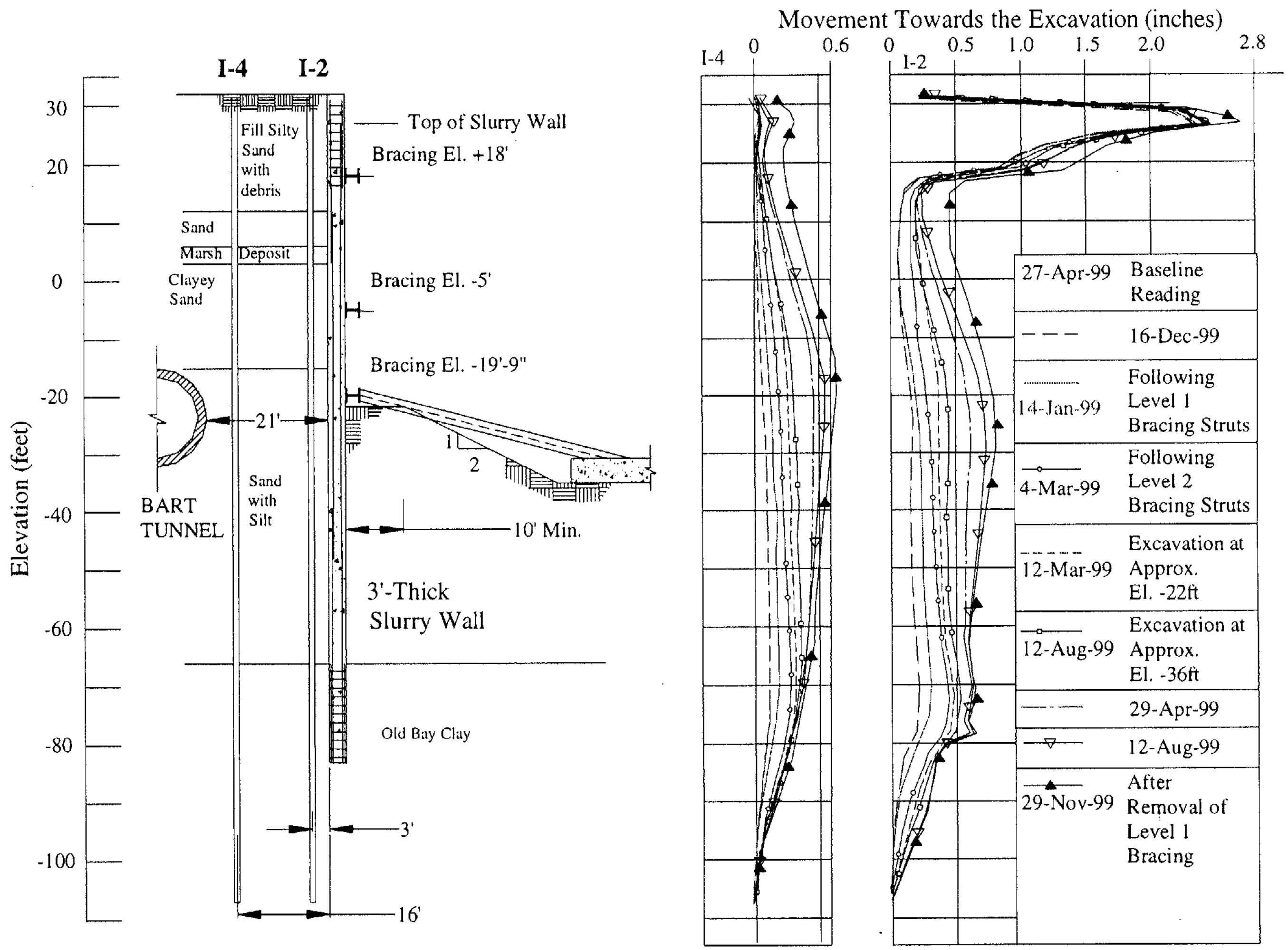


Figure 7.8: Inclinometers I-2 & I-4 at Market Street, Yerba Buena Tower (S9)

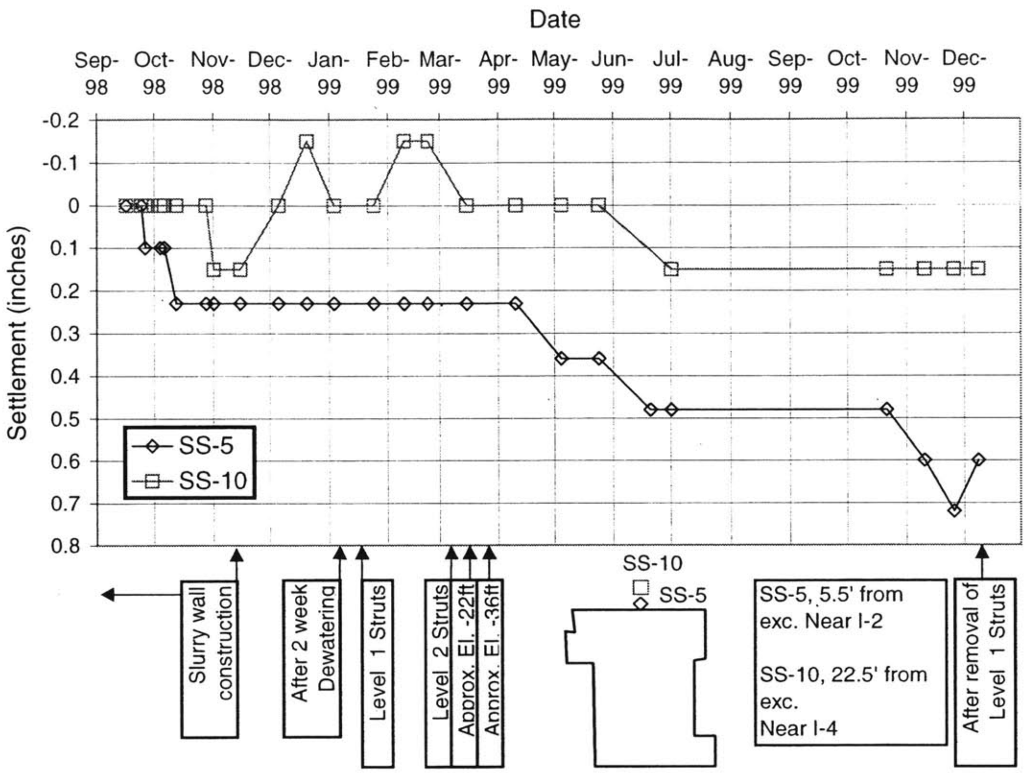


Figure 7.9: Settlements vs. time for points at Market Street, (S9).

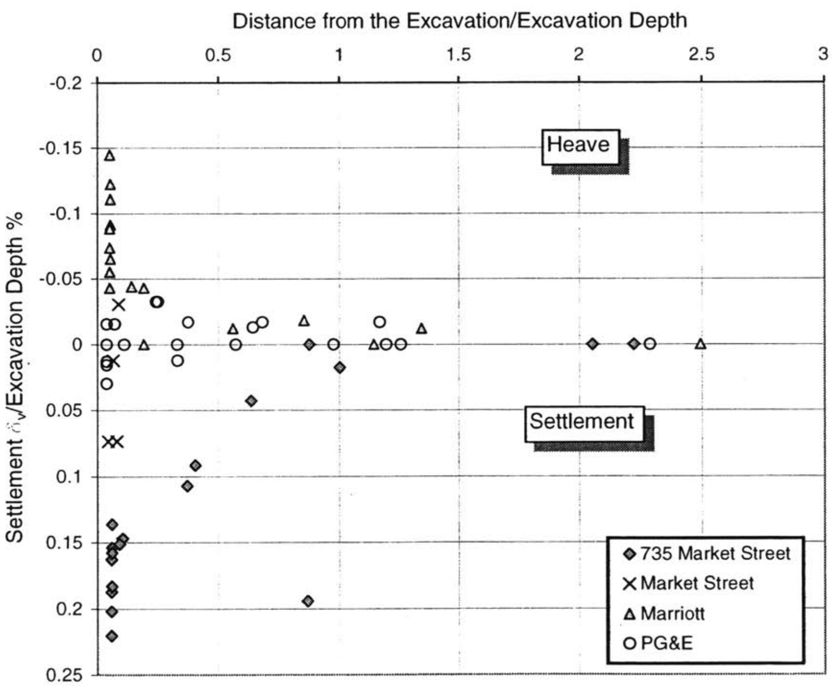


Figure 7.10: Maximum settlements/excavation depth (%) vs. distance from excavation/excavation depth, (S9).

7.2.3 Summary of San Francisco Experience

Small deformations have been measured in excavations supported by diaphragm walls in San Francisco. Basal stability is very important in these jobs since the dominant Bay Mud Clay is very soft. The factor of safety against basal heave for the studied San Francisco projects was generally higher than 2 (FS=2.4 for Yerba Buena, S9). The soft Bay Mud causes the majority of the deformations to occur beneath the base of the excavation even when very deep soldier piles have been used. The pressure from the concrete tremie tends to push back the soft Bay Mud away from the excavation. Thus, in some cases, slurry wall construction partially counterbalances the deflections induced by the excavation.

7.3 Other Projects in the US.

The World Trade Center (Saxena, 1974), New York, is one of the deepest early diaphragm wall projects in the US. The slurry wall was keyed into bedrock or hardpan by 8'. Six levels of rock anchors braced the excavation in most locations except at a section adjacent to a subway tunnel where only four levels were used. The wall cantilevered up to 6" before the first bracing level was installed, and was thereafter pushed back into the soil by up to 2.4" when all anchors were installed. Only, at the section adjacent to the subway did the wall moved towards the excavation (2.4") in a cantilevering mode. In that location, the first anchor level was installed under the subway, 35' below the surface.

The Bonneville Navigation lock (Hanshmire et al., 1989) used a temporary slurry wall keyed into diabase bedrock by 10' more or less, and braced by four levels of tiebacks. The diaphragm wall in this project was also pushed back into the retained soil by the large tieback forces that were applied.

Special attention must be given to the 84-ft-deep excavation for the Wilbur D. Mills Dam in Arkansas (Berger and Tryon, 1999). Huge T-diaphragm wall with 9' long panels, 3'-thick at the face, and with a web 14'-long and 3'-thick, stem were used to form the headrace and tailrace channel training walls and a portion of the powerhouse retaining walls for the dam. The headrace and tailrace channel walls were braced by a single level deadman anchor system with loads from 300 kips to 1866 kips. Soils at the site horizontally bedded with the upper 55ft consisting of dense sand, underlain by 10ft of intermediate clay, 15ft of dense sand, 10ft of intermediate clay, and 20 ft of dense sand, over tertiary clay. The T-panels were embedded by 49' since only one bracing level was used and water levels between the retained soil and the channel can vary significantly. The T-wall slightly curved towards the excavation by up to 0.85".

Chapter 8
Summary Measured Performance of Slurry Walls

Chapter 8

Summary Measured Performance of Slurry Walls

8.1 Introduction

This chapter evaluates the measured performance of the slurry wall projects presented in chapters 4 through 7. The evaluation focuses primarily on inclinometer deflections for comparison of projects since these are both widely available and more reliable and available than other measured data. Settlements were not available or reliable for all jobs and therefore extensive conclusions can not be drawn from them. The exception is Boston where settlement data have been extensively recorded and archived.

The case studies have been divided into four categories:

I) **Floating walls**: When the toe of the wall is embedded within a soft stratum. Most of these projects were supported using tieback anchors, and/or in combination with rakers.

II) **Keyed, tieback walls**: Cases where the toe of the wall is embedded into a stiff stratum like glacial till or bedrock, and the wall is braced tiebacks or rock anchors.

III) **Top/Down (Up/Down):** The basement floors are constructed as the excavation progresses (Thesis section 2.3.3). In all cases the slurry wall extended into a stiff stratum.

IV) **Cross-Lot, Internally braced walls:** Cross-lot excavations are typically braced by preloaded large diameter steel pipes spanning across opposite walls, in narrow excavations (<120ft). For wider excavations, the bracing usually is provided by corner strut systems. The site layout determines the arrangement of the cross-lot or the internal braces. The walls in most projects extend into a stiff stratum.

8.2 Measured performance of floating slurry walls

Table 8.1 lists the floating excavations studied in this thesis. Profiles of these excavations can be seen in the previous chapters. These excavations are relatively shallow, with most 35'-deep and supported by two or three levels of bracing. Thickness of these diaphragm walls ranges from 2' to 2.5', and all of them are embedded into clay. In Chicago, the designs called for the walls to extend by a minimum of 5' into a stiff clay stratum (1.0 tsf to 4.0 tsf) that underlies softer clays and fills, at 50' to 55' beneath the surface. In Boston, the designs aim to extend the diaphragm walls into either the desiccated clay crust, or in sand lenses between the crust and the lower BBC. Tiebacks were the preferred bracing type for these excavations, but some projects used rakers.

Table 8.1: Floating slurry wall excavations

ID	Year	Project Name	Soil Type	Excavation Bracing/ Excavation Method	Exc. Depth (ft)	Slurry Wall Thick. (inches)	δ_{Hmax} (inches)	δ_{Vmax} (Inch)	Defl. Shape Type	Fig.
		Boston								
B3	1982	State Transportation building	A*	2 Levels TB, R, Corner B	27 36	24	1.25	1.2	I, II	4.14
B6	1985	One Memorial Drive Building	A*	2 Levels TB	30	24	1.3	1.2	I	4.21
B7	1987	500 Boylston Building	A*	4 Levels TB, Rakers	42	24	3.3	4.5	IV	4.26
		Chicago								
C3	1971	Amoco Standard Oil	Soft clays, over stiff	1 Level Ties, Soldier piles, Soil berms	23 44 at center	30	4.6		IV	5.10
C4	1973	Water Tower	Down town	1 Level TB & 1 Rakers	44	24	2.5	1.5	I, II	5.13 5.14
C6	1987	Prudential Two	Down town	1 Level Ties, 1 Level R,	25	27	0.45		I	5.16
C7	1987	AT&T corporate center	Down town	3 Levels of rakers and corner braces	27	30	1.55	1.5 5	II	5.17
C9	1993	Northwestern University Memorial Parking Garage	Down town	1 Level of Tiebacks	23	24	0.45		I	5.23
C10	1996 1997	Museum of Science and Industry Parking garage	Stiffer clays	3 levels TB	34	30	0.85		I, II	5.27 5.28

Notes: * According to Johnson see section 4.2

With the exception of three projects (B7, C3, & C4), wall deflections and surface settlements were small and both ranged up to 1.55". Shallow excavations in Chicago showed very little wall bending with the wall base translating slightly towards the excavation (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 settlements were generally on the same order as maximum wall deflections. Figure 8.1 displays selected typical inclinometer deflection data for most of the projects included in Table 8.1. Maximum settlements were in the same order as maximum wall deflections.

The C3 and C4 projects were early slurry wall excavations in Chicago in the 1970s, and thus experience had not fully accumulated at that time. Caisson construction caused soil softening in these projects. Tieback creep and load loss problems in combination with ground softening due to pile extraction caused the large deformations in the 500 Boylston (B7 case study). The tiebacks in this project were not long enough to extend beyond the active zone of soil movements and in addition, the fixed lengths of some ties were located within the fill layer. In addition, the proximity of ties led to inefficiencies due to interactions between the grouted zones of adjacent anchors. Small wall embedment (10ft) did not prove adequate to restraint movements as the wall rotated as a rigid body despite being braced by four levels of tiebacks.

Figure 8.2 shows statistical analysis of measured deformations by inclinometers installed in all the floating projects. Figure 8.3 uses the same data but plotted over the final excavation depth. We can clearly see that 75% of all monitoring locations deflected less than 1.0", with only 14% deflecting more than 1.5" towards the excavation. There were only small differences in the maximum and final measured wall deflections. Most of the monitored wall sections (81%) deflected from $\delta_W/H=0.1\%$ to 0.4%. For a 30'-deep excavation this would correspond to a maximum 1.5" wall deflection.

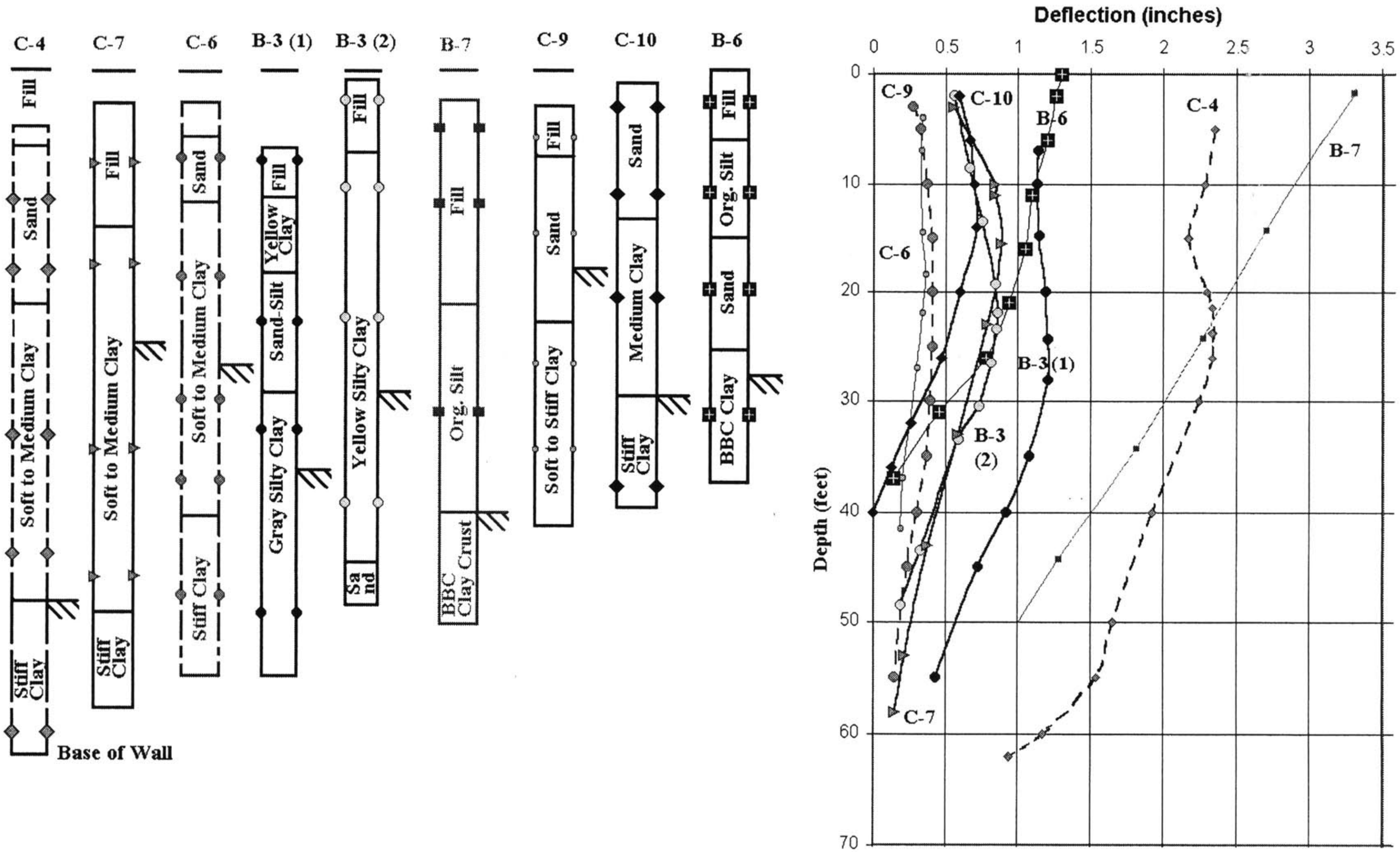


Figure 8.1: Inclinometer deflections for floating slurry wall excavations

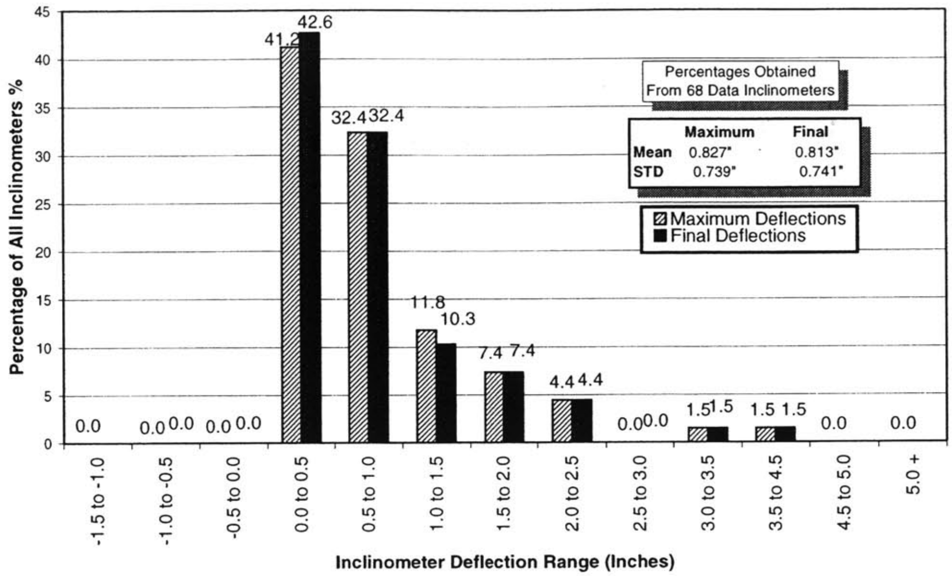


Figure 8.2: Statistical analysis of inclinometer deflections for floating slurry wall excavations

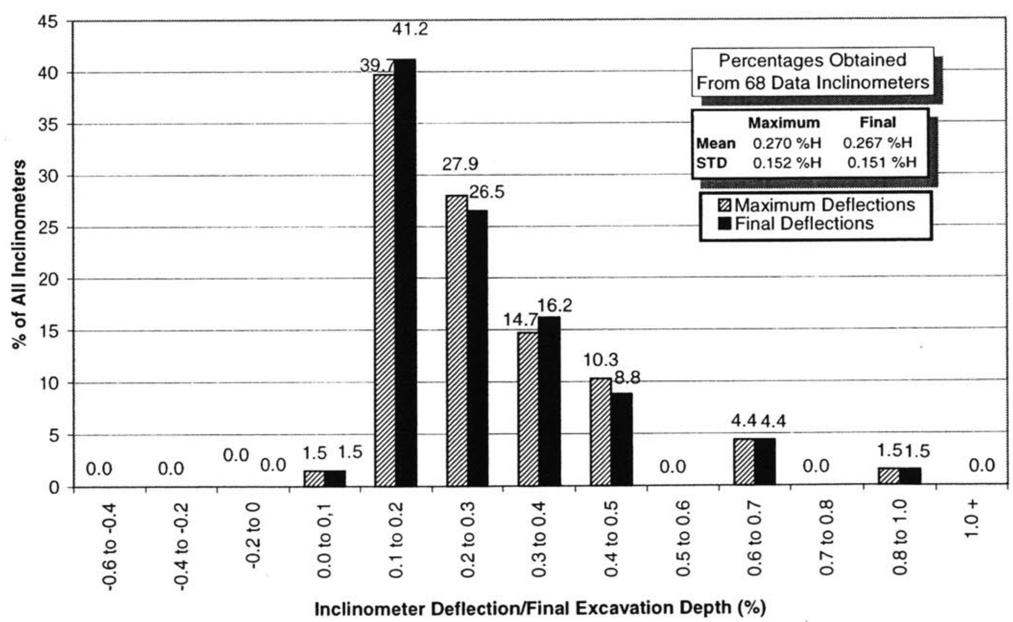


Figure 8.3: Statistical analysis of inclinometer deflections/Final Excavation depth %, for floating slurry wall excavations

8.3 Measured Performance of keyed tieback slurry walls

Table 8.2 lists four projects where walls were keyed into underlying bearing layers. The 60-State Street project is keyed into glacial till, while the Dana Farber excavation is the deepest tieback slurry wall excavation in Boston. In Washington, the World Bank is keyed into decomposed bedrock, while the Washington Convention Center is keyed into very stiff sands and clays.

Table 8.2: Keyed tieback walls

ID	Year	Project Name	Exc. Depth (feet)	Wall thickness (inches)	Bracing Type	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Defl. Type	Figure
		Boston							
B2	1975	60-State Street	32	30	3 or 2 Levels TB, wall keyed into glacial till	1.3 0.92**	1.2	I, II	4.11
B12	1995	Dana Farber Tower	65 - 90	36	6-Levels Permanent Rock tiebacks	0.72 0.4	0.64 2.8	I, V	4.53
		Washington							
W1	1991	World Bank	60	30	5-Lev. TB	0.45 -0.45	0.45	II IV	6.7 6.6
W3	1999 2000	Washington Convention Center	30 – 55	36, 48 36	1-Ties, 1 Rakers. 3-L. Ties	0.75 0.70	0	I I	6.16 6.15

Overall, maximum and final wall deflections and settlements were small in most cases. Only in the B2 case study did the wall deflections exceed 1.0", in all other projects wall deflections were smaller than 1.0". However, the maximum wall movements shown in Table 8.2 do not reflect the range of measured deflections which was larger than the absolute magnitude of deformations towards the excavation. In most projects, some sections of the slurry wall deflected back towards the retained soil by as much as other wall sections deflected towards the excavation. In these projects, deformations were eventually controlled by minor variations in construction and site conditions.

Figure 8.4 compares selected inclinometer deflections from these keyed tieback excavations. The effect of building surcharge can clearly be seen in the

World Bank and the Dana Farber Tower deflections were the walls bent slightly above the excavation base.

The distribution of measured final and maximum wall deflections is shown in Figure 8.5. The statistical distribution of inclinometer deflections as a percentage of the final excavation depth can be seen in Figure 8.6. The data shown in these figures can not be generalized because the distributions have been derived from only 21 inclinometers. Conclusions can thus be made only for the studied projects.

Half of the monitored wall sections deflected from 0" to 0.5" towards the excavation, while 14% of all inclinometers deflected back into the soil from 0' to 0.5". The mean maximum deflection was δ_W=0.393" ± 0.412", and average δ_w/H= 0.086% ± 0.124%. For the final deflections had was δ_W=0.274" ± 0.326", and δ_w/H= 0.055% ± 0.097%, while for the final readings they were δ_W=0.274" ± 0.326". Inclinometers used in these four excavations (B2, B12, W1, W3) (represent 34% of the total wall sections that were monitored. About 60% of the monitored inclinometers deflected from 0% to 0.1% of the final excavation depth.

In the Dana Farber Tower, soil losses during tieback drilling caused most slurry wall panels to deflect back into the retained soil, while the surface settlement ranged up to 2.8". The scatter in the measured settlements in the Washington projects was in most cases larger than the settlement itself.

Embedment strain gages installed within the slurry wall in the Dana Farber project (B12) indicate that when a wall is keyed into a stiff stratum like bedrock then the full vertical force of the tiebacks is carried to the bottom of the wall (thus there is very little side friction between the slurry wall and the retained soil).

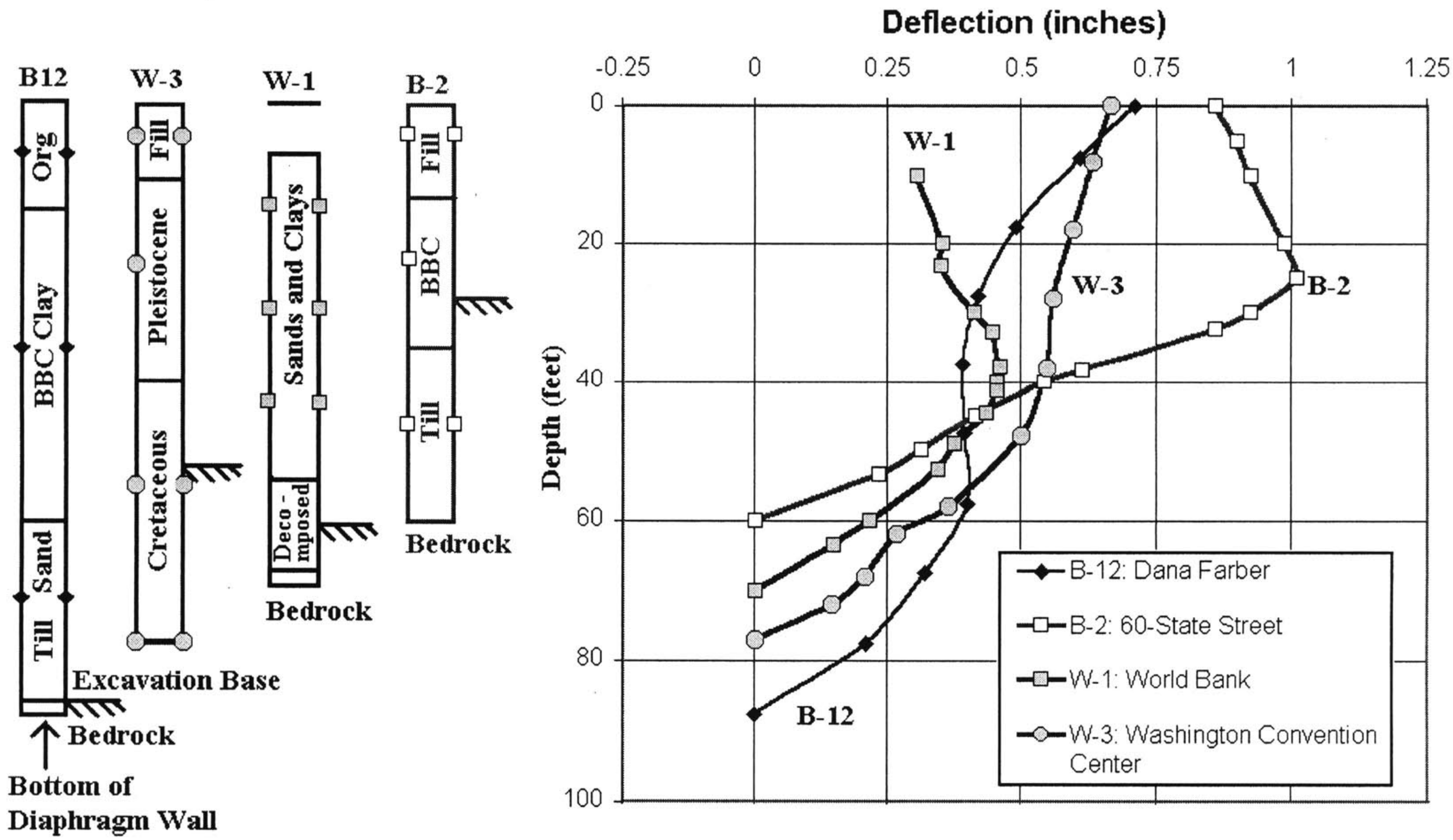


Figure 8.4: Wall deflections for keyed tieback slurry wall excavations

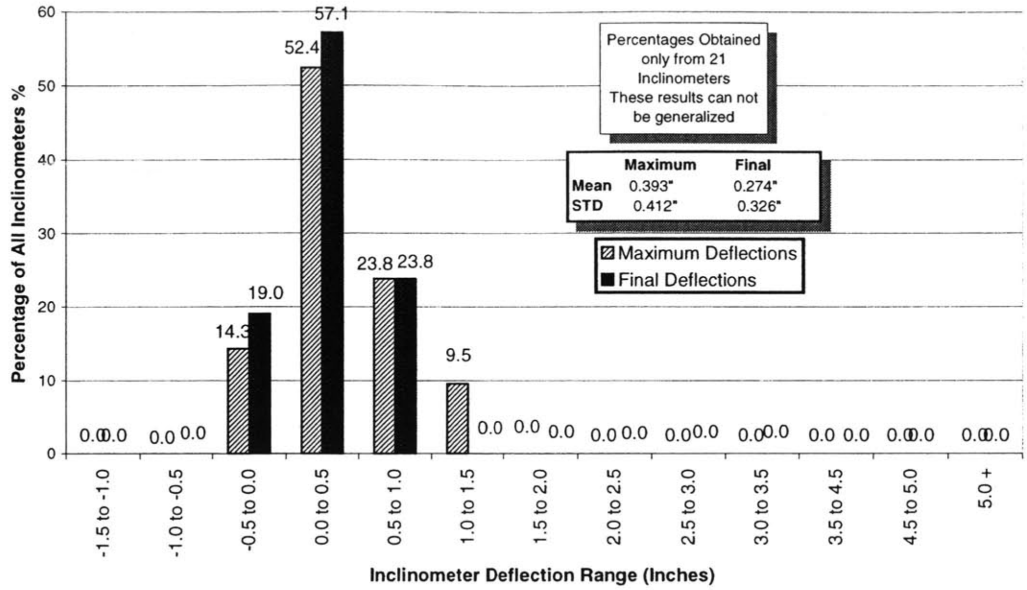


Figure 8.5: Statistical distribution of wall deflections for studied keyed tieback slurry wall excavations.

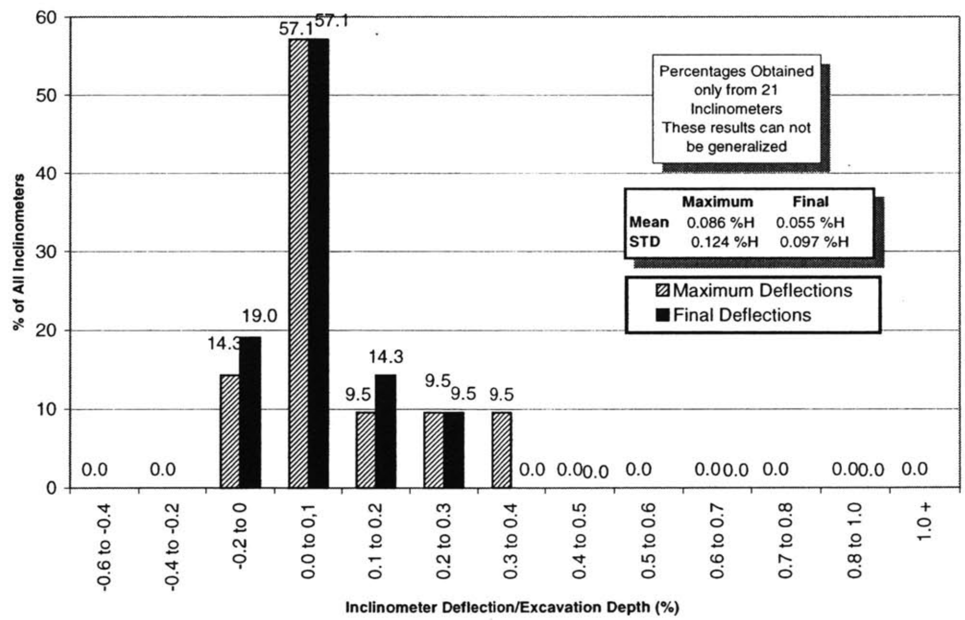


Figure 8.6: Statistical distribution of wall deflection as a percentage of the final excavation depth for the studied keyed tieback slurry wall excavations.

8.4 Measured performance of Top/Down slurry wall excavations

Table 8.3 lists the top/down excavations studied in this research, five out of the six projects are in Boston and one is in Chicago. Excavation profiles for these projects except for B9 can be seen in the referenced figures. The top/down excavations in Boston are 55' to 75' deep, while the project C8 in Chicago is only 35' deep. All the walls in Boston are keyed into either glacial till or bedrock.

Maximum wall deflections ranged up to 2.2" (B10) while maximum settlements induced by the excavation only where roughly in the same order as maximum wall deflections. In the B4 case, pile extraction caused ground softening and thus surface settlements were larger. In B10, larger than expected wall deflections at some locations were attributed to poor control on the backfilling for Load Bearing Elements (LBE's). In all other projects, maximum wall deflections and settlements were typically less than 1.0".

Figure 8.7 shows selected inclinometer deflections of top/down excavations in Boston and in Chicago. In Boston projects where profile A dominated (Boston Blue Clay), the walls tended to bow towards the excavation with the 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 in excavations within B soil profiles did not have that profound bending as walls in soil profiles where Boston Blue Clay dominated.

About 71% to 80% of all the inclinometers installed in top/down excavations deflected from 0" to 1.0" towards the excavation (Fig. 8.8). The remaining 20% to 30% of all the measured wall deflections reflect data from the 75 State Street and the Post Office Square projects. For the maximum deflections the average was δ_H=0.689" ± 0.556". while the average final deflections were δ_H=0.566" ± 0.371". The statistical distribution of inclinometer deflections as a percentage of the final excavation depth is shown in Figure 8.9. The average deflections were δ_H/H=0.126% ± 0.076% for the maximum deflections, and δ_H/H=0.121% ± 0.073% for the final deflections respectively. Close to 80% of the monitoring locations deflected from 0% to 0.2% of the final excavation depth, while the remaining 20% deflected from 0.2% to 0.3%. This would imply that there is an 80% chance that 60' -deep top/down slurry wall excavation would deflect by up to 1.5%.

Table 8.3: List of top/down slurry wall excavations

ID	Year	Project Name	Soil Typ	Exc. Depth (feet)	Wall thick. (inches)	Bracing Type	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Deflection Type	Figures
		Boston								
B4	1983	75 State Street	A*	65	30	6-Levels	1.85	4.0	II	4.17
B5	1984	Rowes Wharf	B	55	30	5 Levels	0.41	___	II	4.19
B9**	1990	125 Summer St	A*	60	30	6-Levels	0.6	0.38	---	---
B10	1989	Post Office Square Garage	A*	75	36	7-Levels	2.15	2.75	II	4.37
B11	1994	Beth Israel Deaconess	A*	55	36	5-Levels	0.85	0.7	II	4.43
B13	1998	Millenium Place	B*	55	36	5-Levels	0.7	.45	I, II	--
		Chicago								
C8	1989	Guest Quarters Hotel	Typ/ Down town	35	24	3-Levels	0.65		I	5.20

* A: Fill, Organic Silt, Boston Blue Clay, Glacial Till, Bedrock, B: Fill, Glaciomarine, Glacial Till, Bedrock, according to Johnson 1989,

** Only referenced

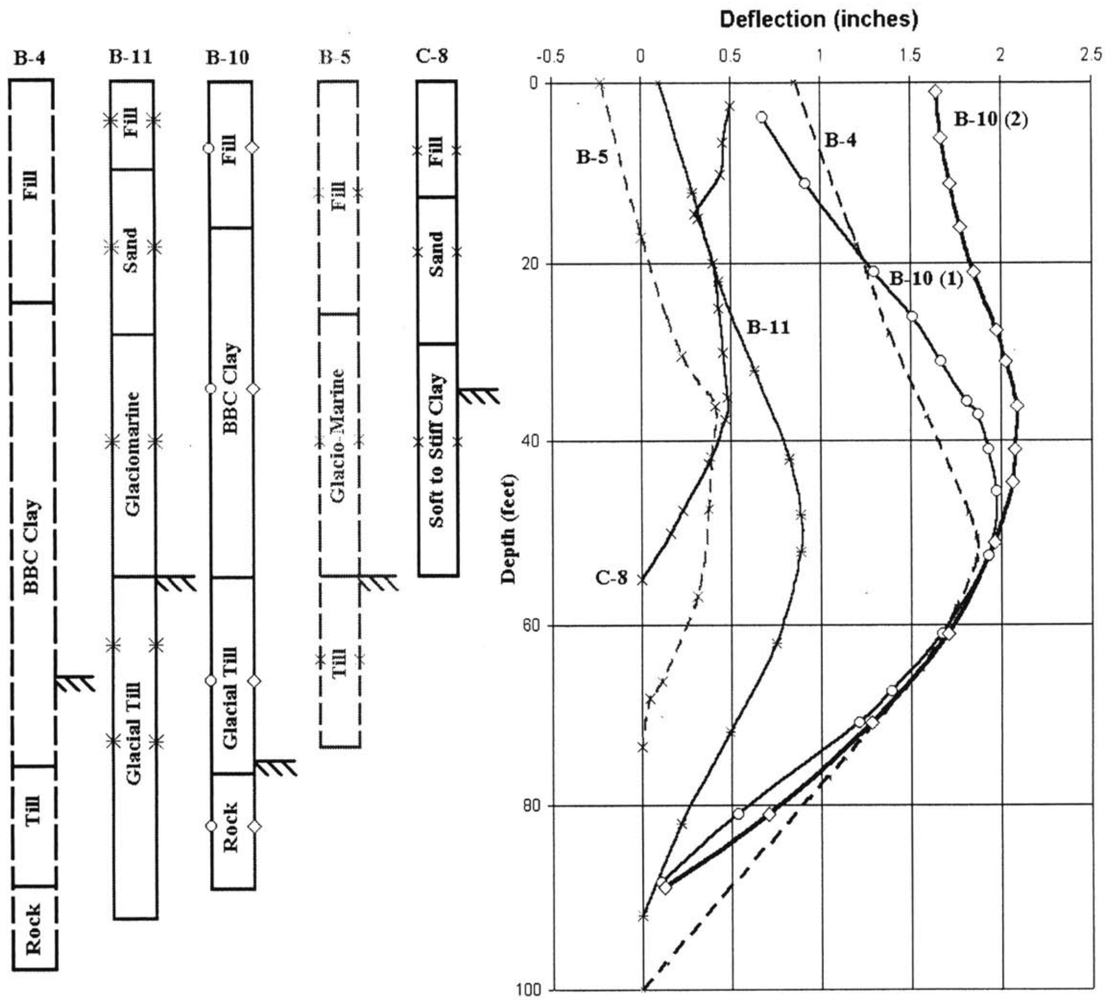


Figure 8.7: Selected inclinometer deflections from top/down slurry wall

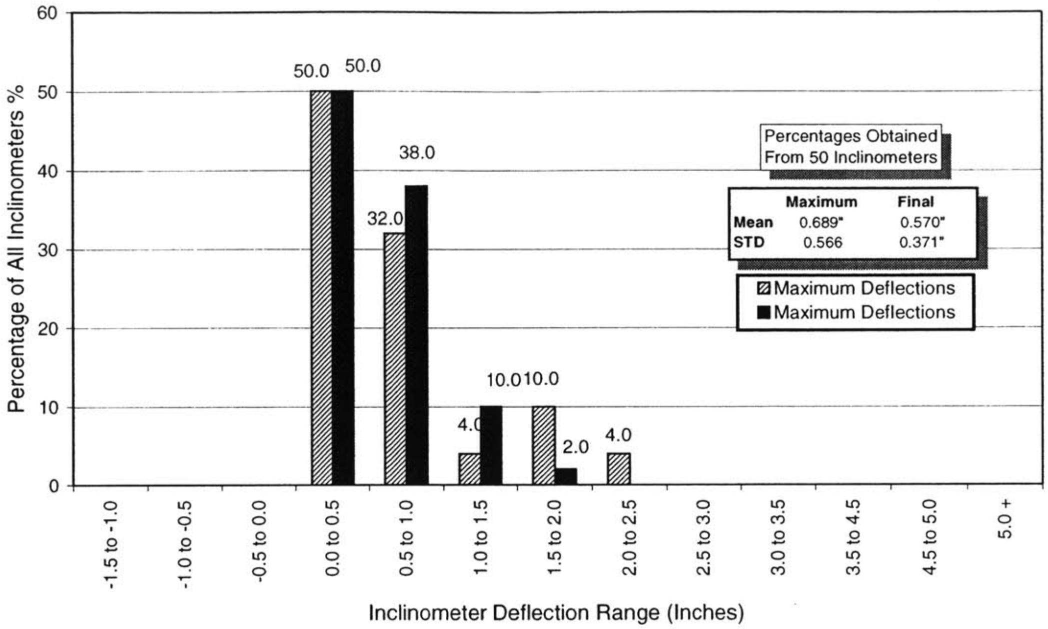


Figure 8.8: Statistical distribution of maximum and final inclinometer deflections for top/down slurry wall excavations.

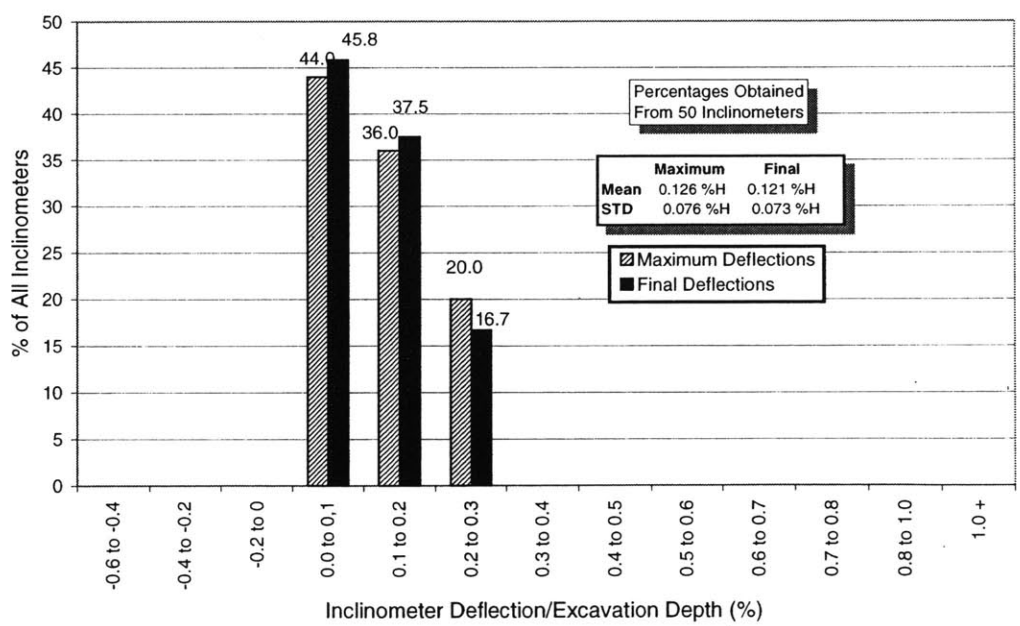


Figure 8.9: Statistical distribution of maximum and final inclinometer deflections as a percentage of the final excavation depth, for top/down slurry wall excavations.

8.5 Measured Performance of Cross-lot and internally braced excavations

Table 8.4 lists slurry wall excavations supported by cross-lot braces or internal bracing. Cross-lot bracing was preferred in relatively narrow sites where the opposite walls were 50ft to 120ft from each other. All of the cross-lot braced walls were keyed into a stiff stratum (glacial till or bedrock). Cross-lot excavations in Boston and in Washington ranged from 47ft to 100ft deep. Complicated corner bracing was used in the 66'-deep Yerba Buena Tower excavation, where the slurry wall was 115' deep. The two Chicago excavations are not representative of modern practice since they were amongst the first slurry wall excavations constructed in the US.

Table 8.4: List of cross-lot and internally braced slurry wall excavations

ID	Year	Project Name	Soil Typ	Exc. Depth (feet)	Wall thick. (inches)	Bracing Type	δ_{Hmax} (inches)	δ_{Vmax} (inch)	Deflection Type	Figure
		Boston								
B1	1973	MBTA South Cove	A*	53	36	3 Levels Cross-lot	1.35	0.5	I	4.8
B8	1989	Flagship Wharf	C	47	30	3 Levels Cross-lot	1.81	1.7	II	4.30
		Washington								
W2	1995	Petworth Subway Station	Sands and Clays	60 – 100	36	5-6 Levels Cross Lot	0.75	-	II	6.11 6.12 6.13
		Chicago								
C1	1970	CNA	Typ. Downtown	31	30	(1Rakers+ Berms, Cross Lot permanent floor steel mixed)	3.3	5	-	5.7
C2	1971	Sears Tower	Typ. Downtown	32	30	3 Levels Rakers, soil berms	6+		IV	--
		San Francisco								
S9	1999	Yerba Buena Tower	Fill, Sand, Clay	66	36	2 Lev, Int Brac, 1 Lev. Rakers	0.8	0.5	V	7.8

Figure 8.10 shows selected inclinometer deflections from the B8, W2, and S9 excavations. Wall deflections for the cross-lot braced excavations were moderate, ranging up to 1.8" for the B8 project. In the deep Petworth Subway wall deflections were smaller and ranged up to 0.75" towards the excavation. In the Yerba Buena Tower, the underlying clays were not able to restraint the base of the wall and thus soil deformations occurred throughout the depth of the slurry wall. None the less, wall deflections were small and ranged up to 0.8" towards the excavation.

The range of wall deflections was almost double the maximum wall movement for B8 and W2, since some panels deflected back into the retained soil by as much as other panels deflected towards the excavation. This can be seen in Figure 8.11, which shows the statistical distribution of wall deflections for the B8, W2, and S9 excavations. The average deflection was δ_W=0.409" ± 0.610" (for both maximum and final conditions). General conclusions can not be drawn because these percentages are derived from a limited number of inclinometers. About 75% of all the monitored sections deflected towards the excavation from 0" to 1.0", and close to 16% of all walls moved back into the retained soil by as much as 1.0". There were very small differences between the maximum and the final wall deflections.

The statistical distribution of inclinometer deflections as a percentage of the final excavation depth can be seen in Figure 8.12. The average was δ_W/H=0.082% ± 0.112%" for both maximum and final deflections. Close to 72% of all the inclinometers deflected from 0% to 0.2% of the final excavation depth, and 9% of the inclinometers deflected from 0.2% to 0.4%. For a 65'-deep excavation this would imply that there is a 72% probability that a wall panel will deflect up to 1.5" towards the excavation.

Thermal expansion and contraction can be very important when the bracing struts are too long. Flagship Wharf (B8) was the only project where inclinometer deflections at both ends and raking loads were measured.

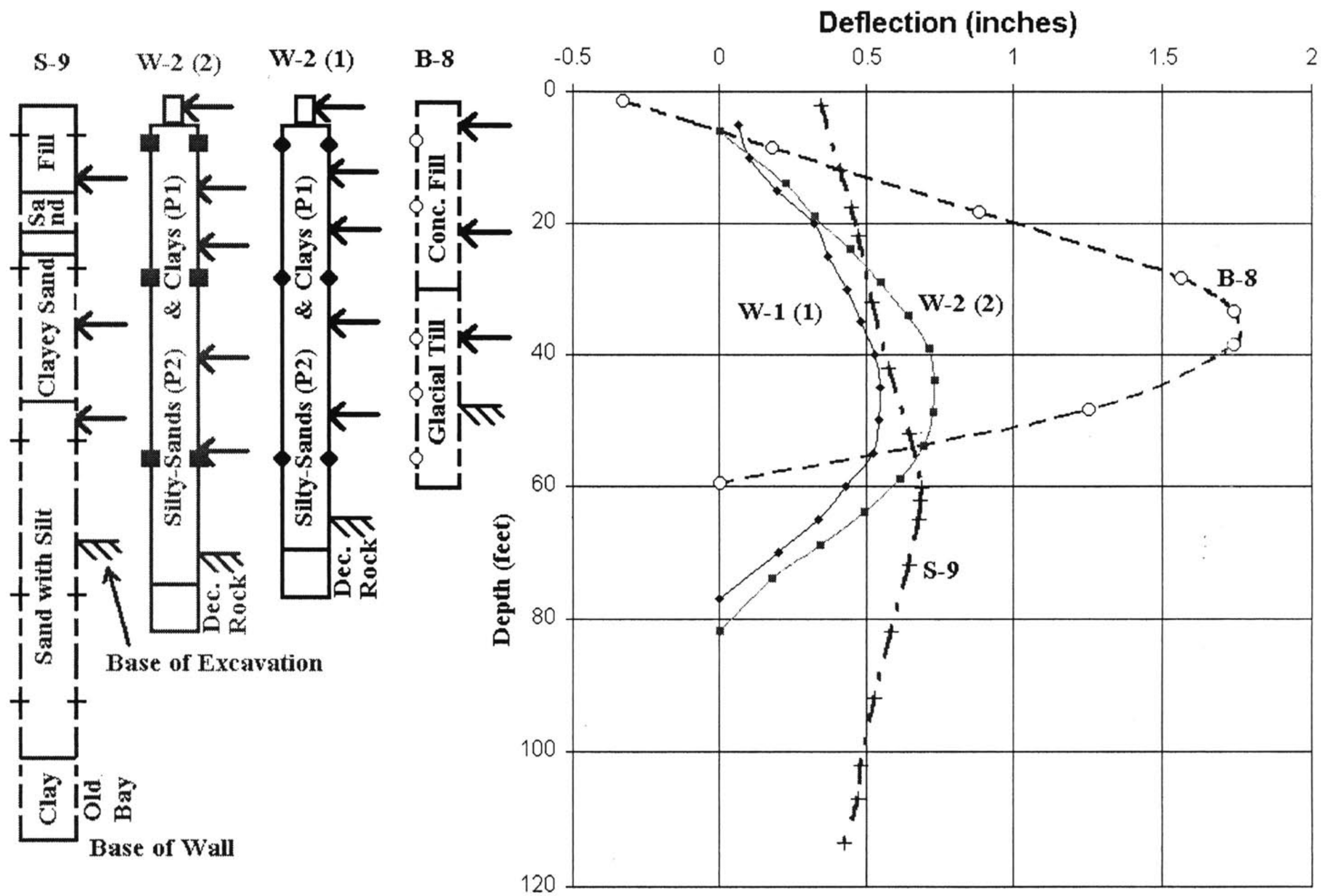


Figure 8.10: Selected inclinometer deflections for cross-lot and internally braced slurry wall

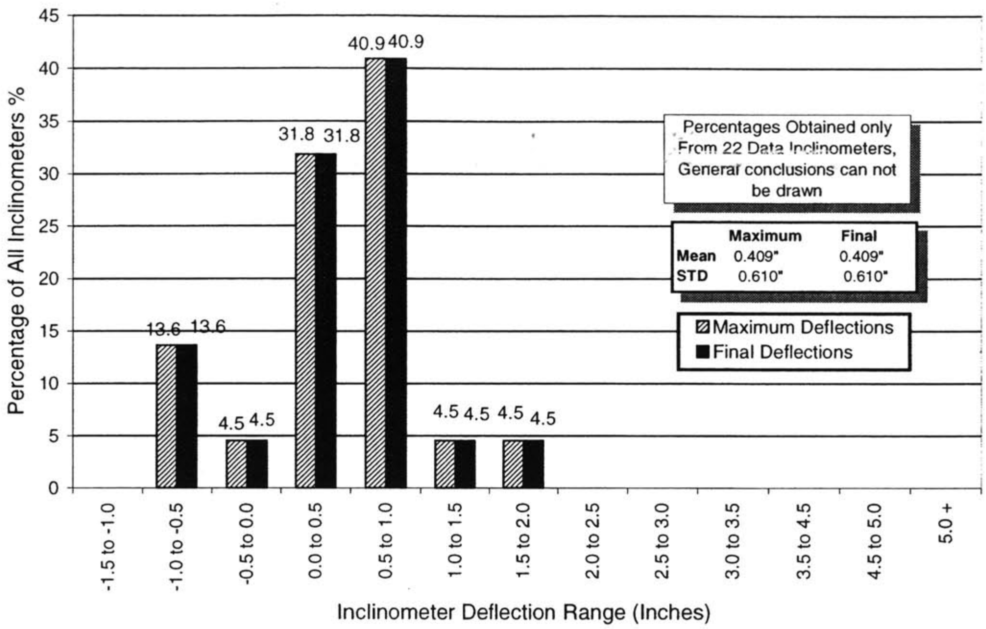


Figure 8.11: Statistical distribution of inclinometer deflections for projects B8, W2, and S9.

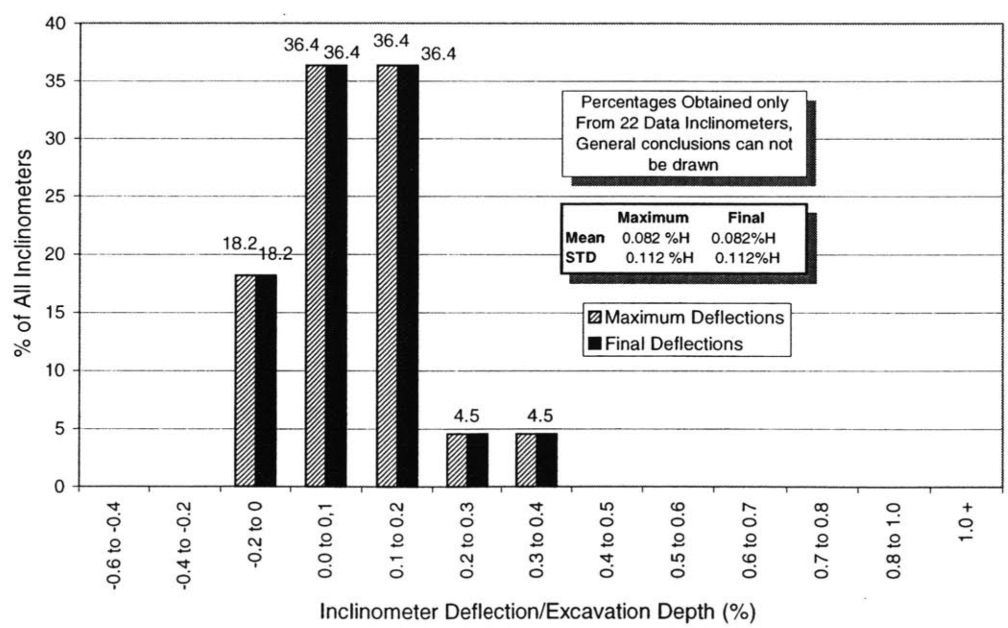


Figure 8.12: Statistical distribution of inclinometer deflections as a percentage of the final excavation depth for projects B8, W2, and S9.

8.6 Statistical analysis of all Inclinometer deflections

This section discusses the statistical distribution of inclinometer deflections in slurry wall excavations. Figures 8.13 and 8.14 plot the statistical distributions of inclinometer deflections for all the case studies included in the data report, while Figures 8.15 and 8.16 show similar plots that include other projects referenced from Chapter 7. Wall deflections induced by the excavation were overall small.

From Figure 8.13 we can see that the overwhelming 75% of all the monitored walls deflected from 0" to 1.0" towards the excavation. Movements from 1.0" to 2.0" represent only 10% to 15% of all the inclinometers. Deflections above 2.0" are very rare and mostly occurred in earlier jobs, or when the bracing was not adequate, or when caisson construction or pile extraction caused ground softening. The average deflection was δ_W=0.70" ± 0.70" for maximum deflection (dor final excavated grades the average was slightly smaller δ_W=0.64" ± 0.66").

Most monitored panels (30% to 36%) deflected from δ_W/H=0.1% to δ_W/H=0.2%, with average δ_W/H=0.184% ± 0.158% (Maximum deflections). For the final deflections the average was δ_W/H=0.177% ± 0.157%. Close to 78% of all the inclinometers deflected from 0% to 0.3% of the final excavation depth.

The statistical distribution of inclinometer deflections changed slightly when the performance data from archival projects were added (e.g. Ch. 7: Clough & Buchignani, 1981). About 70% of all the inclinometers deflected from 0" to 1.0" towards the excavation. Deflections from 1.0" to 2.5" make up 20% of the all the monitored inclinometers. In both Figures 8.13 and 8.15 we can see that percentages decrease as the deflection range increases. Very few of the mentioned walls were pushed back into the retained soil.

Table 8.5 summarizes the average deflections and variances for all the studied projects. The final deflections are smaller than the maximum deflections for all categories. The average deflection and variance for floating excavation was larger than any other type of excavation (δ_W= 0.83" ± 0.74", and δ_W/H=0.27% ± 0.15%). Cross-lot and internally braced excavations and keyed tieback walls had the same average deflection (δ_W=0.41" and δ_W=0.39"), but the standard deviation was larger for the cross-lot excavations. For top/down projects the average deflections were δ_W=0.689" (δ_W/H=0.126%) for maximum deflections and δ_H=0.57" and δ_H/H=0.121% for final deflections. For all the studied projects, the deviations of the deflections were almost equal or slightly larger than the actual deflections (δ_W=0.70" ± 0.70" for maximum, δ_W=0.64" ± 0.66" for final). By including the archival projects the averages were increased slightly (δ_H = 0.78" ± 0.84" maximum, and δ_H = 0.70"±0.77" for final). Thus, the variance of measured deflections was just as large or slightly larger than the average measured deflections.

Table 8.5: Summary of deformations for studied projects, by category.

	Inclino-meters	Deflections				Deflections divided Final Excavation Depth			
		Maximum		Final		Maximum		Final	
		δ_H (inches)	Variance of δ_H (inches)	δ_H (inches)	Variance of δ_H (inches)	δ_H/H %	Variance of δ_H/H %	δ_H/H %	Variance of δ_H/H %
Floating Excavations	68	0.827	0.739	0.813	0.741	0.27	0.154	0.267	0.151
Cross-Lot & Internal Bracing	22	0.409	0.61	0.353	0.547	0.082	0.112	0.082	0.112
Keyed Tiebacked Walls	21	0.393	0.412	0.274	0.326	0.086	0.124	0.055	0.097
Top/Down	50	0.689	0.566	0.57	0.371	0.126	0.076	0.121	0.073
Total	161								
Previous Total + Database, Including W4, & C2	164					0.184	0.158	0.177	0.157
+C5	167	0.699	0.698	0.638	0.662				
+Referenced without C5	192					0.191	0.19	0.181	0.186
+Referenced + C5	195	0.778	0.843	0.704	0.772				

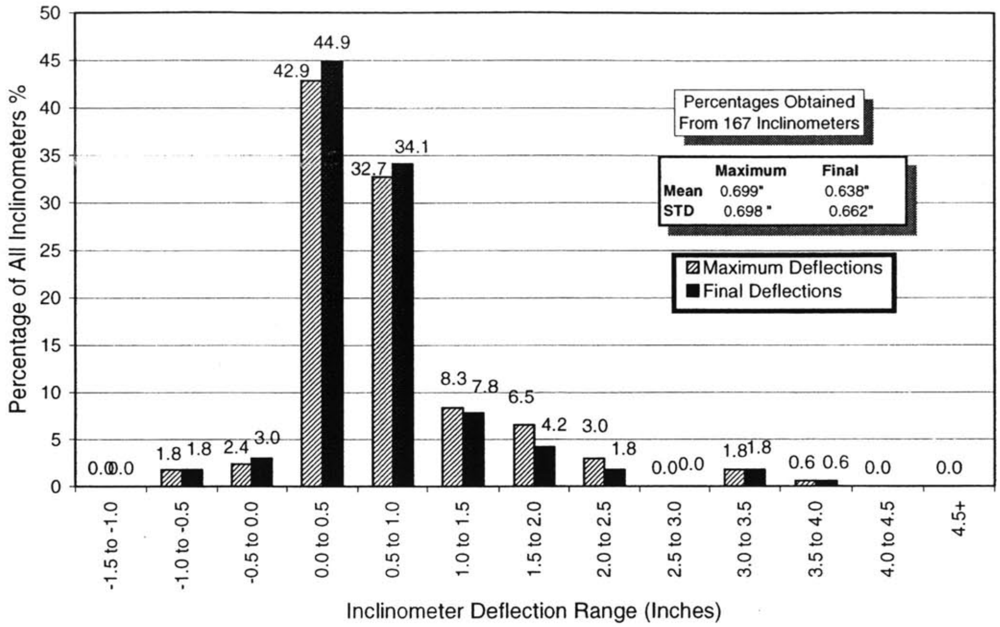


Figure 8.13: Statistical distribution of maximum and final inclinometer deflections for projects studied in this thesis, (Chapters 4, 5, and 6).

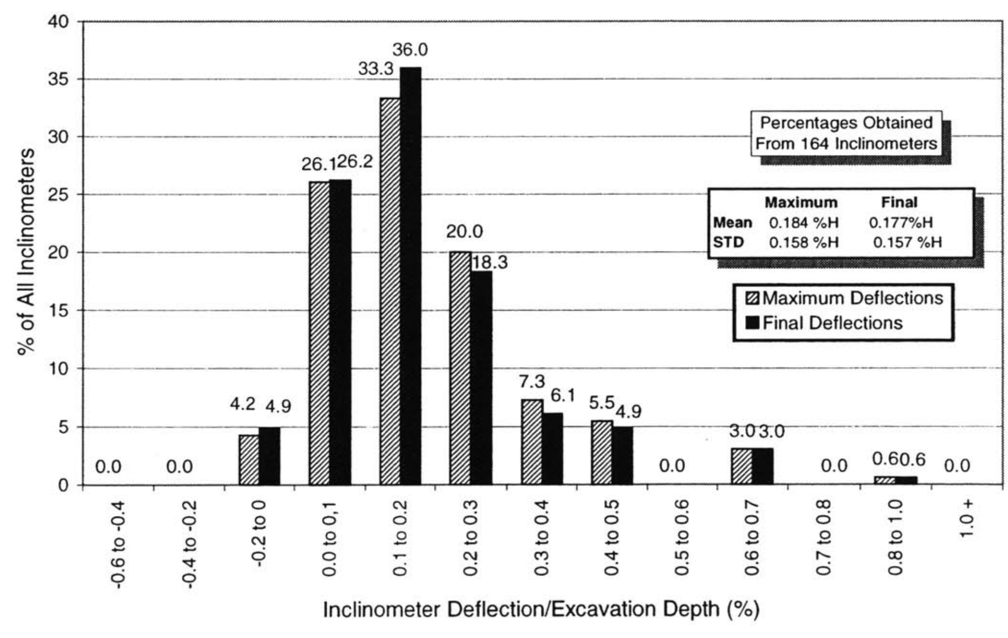


Figure 8.14: Statistical distribution of maximum and final inclinometer deflections as a percentage of the final excavation depth for projects studied in this thesis, (Chapters 4, 5, and 6).

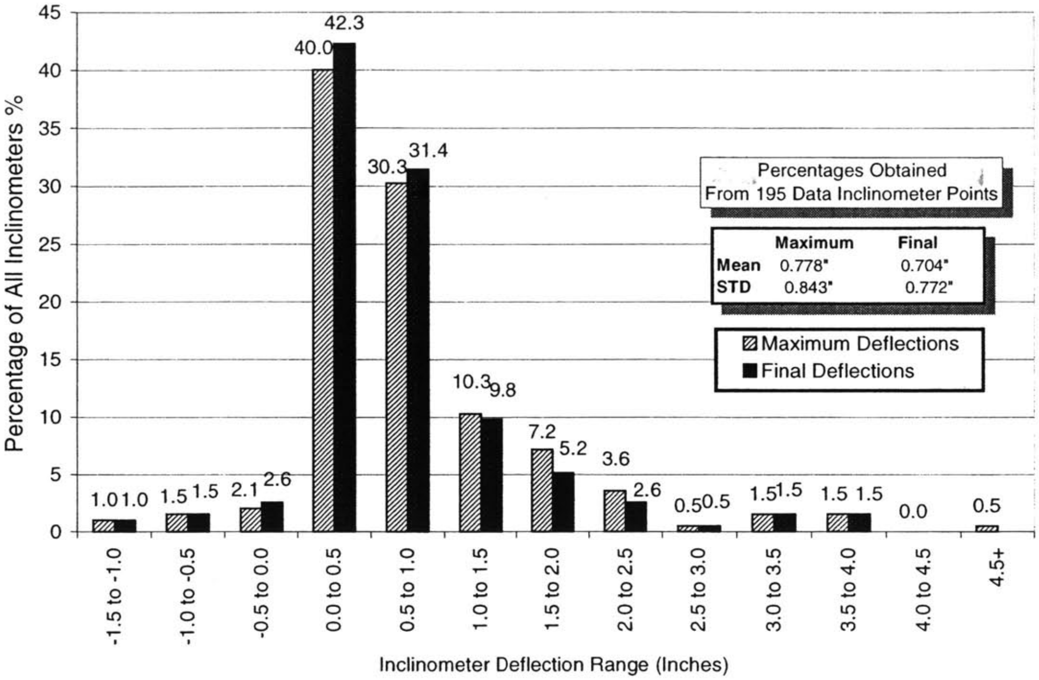


Figure 8.15: Statistical distribution of maximum and final inclinometer deflections for projects studied in this thesis and referenced projects, (Chapters 4, 5, 6, and 7).

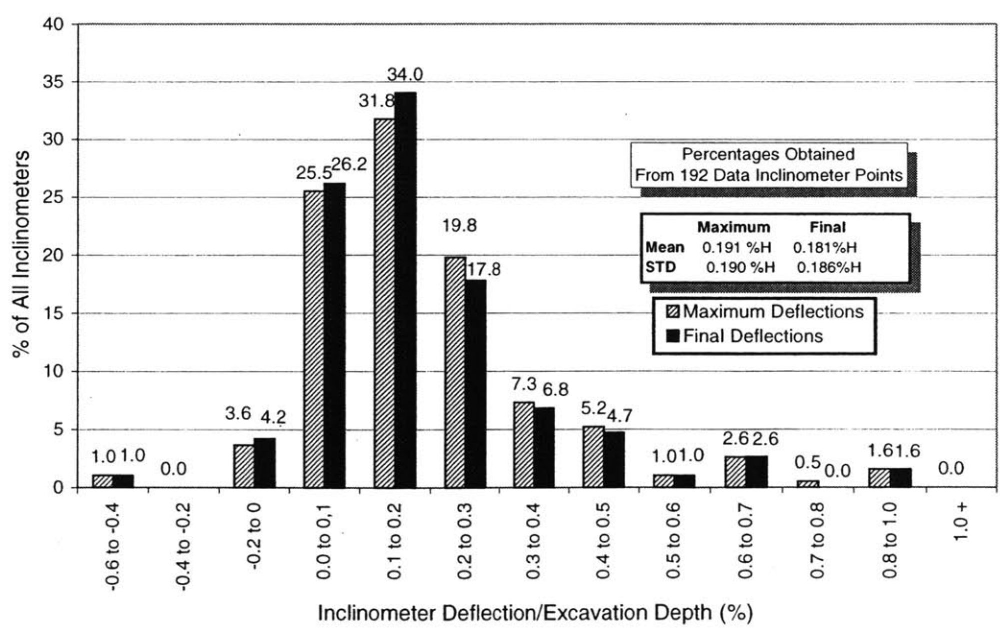


Figure 8.16: Statistical distribution of maximum and final inclinometer deflections as a percentage of the final excavation depth for projects studied in this thesis and referenced projects, (Chapters 4, 5, 6, and 7).

8.7 Measured maximum wall deflections plotted according to Clough

Figure 8.17 plots maximum inclinometer deflections as a percentage of the final excavation depth versus the system stiffness factor (Clough et al., 1989). The data was plotted by assuming an uncracked concrete section and ignoring wall reinforcement. The modulus of elasticity of the tremie concrete was estimated from the 28-day peak strength f_c' according to an equation specified by the American Concrete Institute. The deflections reported in this figure exclude movements that were associated with activities such as caisson construction.

This approach of plotting the data has some obvious limitations because a) the spacing between vertical supports varies little from project to project average 9ft to 11ft with 7ft minimum and 17ft maximum, b) the wall thickness typically varies from 2' to 3', c) the effects of bracing forces are totally ignored, and d) the effect of soil conditions is accounted in the basal stability factor, while the majority of the walls are keyed into a stiff stratum.

With a few notable exceptions all the projects generated $\delta_W/H \leq 0.35\%$. Data from cross-lot projects suggest that deflections increase as the system stiffness decreases, while tieback excavations show a lot of scatter.

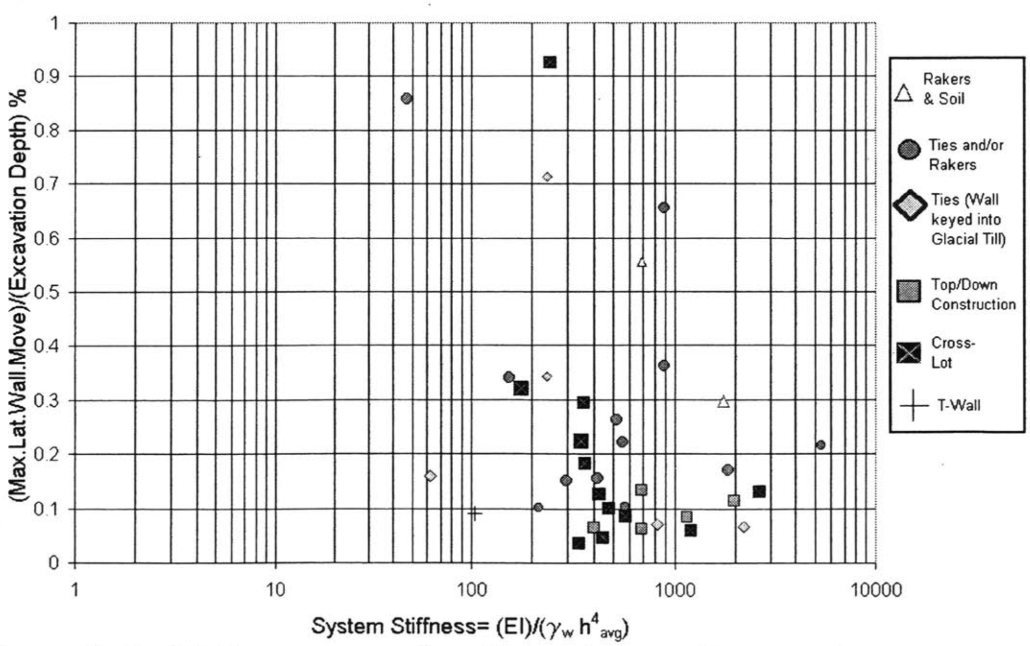


Figure 8.17: Wall movements for all the slurry wall excavations vs. system stiffness (Clough et al, 1989 approach).

8.8 Measured settlement performance

Figure 8.18 plots surface and building settlements for slurry wall excavations versus the distance behind the wall standardized by the final excavation depth. The largest settlements occurred between 0 and 0.5 times the excavation depth, while the maximum settlement generally occurred at a distance from 0.2 to 0.25 times the final excavation depth. Almost all the measured settlements fell within Zone I (Peck, 1969). Settlement data in this Figure reflect mostly on the Boston slurry wall excavations which were very well instrumented. Settlements in Washington DC were too small to be considered.

Most settlements were generally less than 0.2% of the final excavation depth. Larger movements measured in the 500 Boylston (B7) and in the 75 State Street (B4) projects are not representative of the vast majority of projects. Tieback problems and ground softening due to pile extraction were the main causes of these data.

One should expect settlements in the order of $\delta_V/H=0.1\%$ to 0.2% within 25% of H from the face of the excavation. The ratio δ_V/H gets smaller for deeper excavations (B10, B12). Settlements rapidly decrease at a distance 70% of H behind the wall. For a 60'-deep excavation this would generate settlements in the order of $\delta_V=0.7"$ to 1.5" within 42 ft from the excavation face.

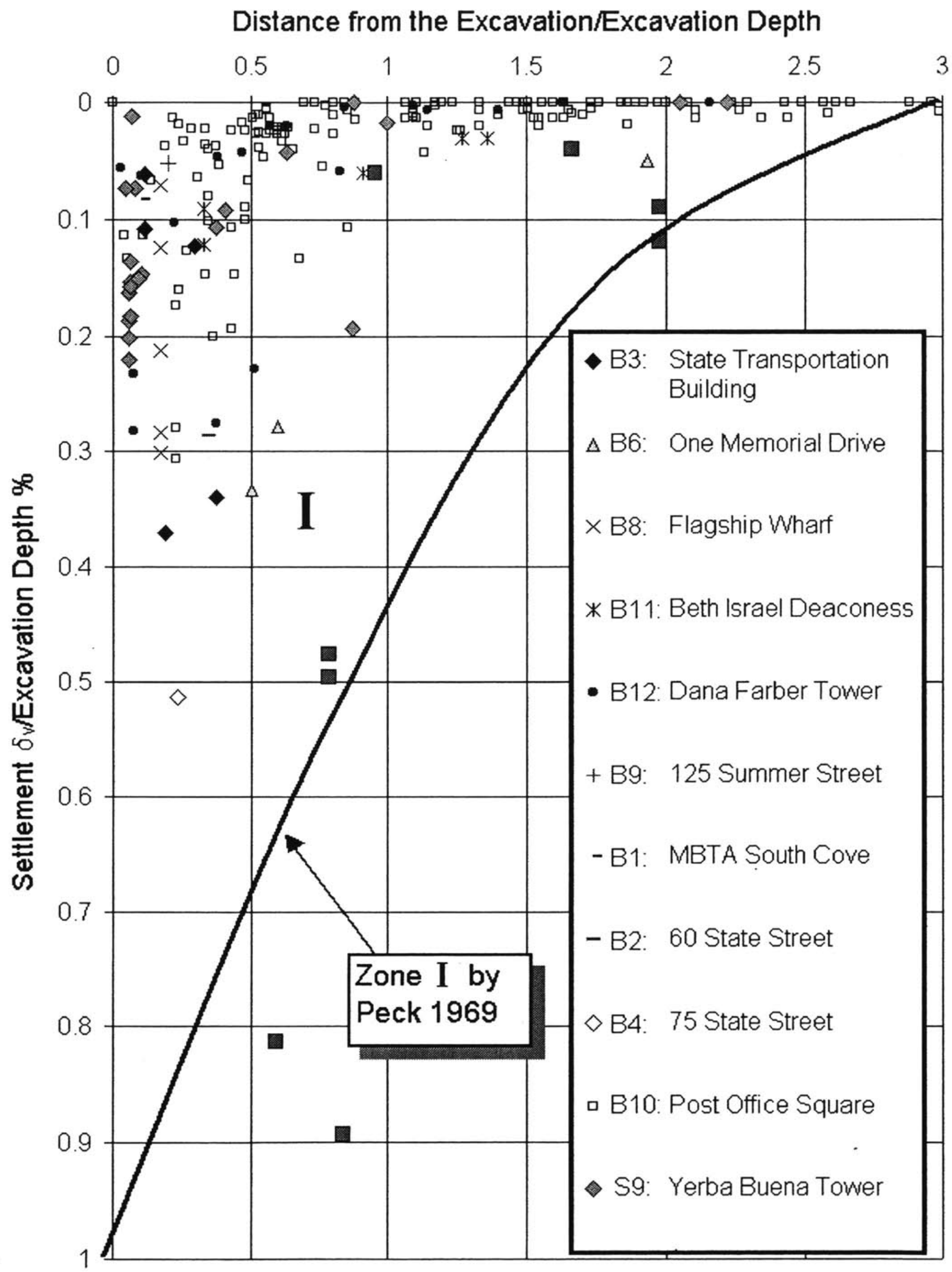


Figure 8.18: Summary of settlements for slurry wall supported excavations.

8.9 Wall translation, rotation, and bending.

In order to compare wall deflections from different projects the final movements of the wall have been subdivided into three basic modes corresponding to rigid body translation, rigid body rotation, and bending (Fig. 8.19), characterized by modal components C_t, C_R or θ, and C_m, respectively. The excavations are described by the wall depth H_W and embedment H_d, the excavation depth H, and the average vertical support spacing h (Figures 8.20 to 8.28). Table 8.7 summarizes all the derived parameters corresponding to these deflection modes.

Figure 8.20 correlates the ratio of C_t/H_d (%) to the ratio H/H_d for floating tieback excavations. The linear trendline fitted through the data is very accurate (R^2 =0.965). Hence, the maximum rigid translation C_t is related directly to the excavation depth, H, and to the wall embedment, H_d. Rigid body translations C_t for floating walls ranged from 0.14" (C6) to 1.05" (B7) maximum.

Figure 8.21 correlates the ratio C_R/H_W (%) (= tanθ) measured at the top of the floating walls to the ratio $H/(H_d+h)$. A linear trendline fitted through the majority of the data produced very satisfactory results. Only two were excluded from the linear trendline fit projects (C3 because of caisson construction, and B6 to provide a better fit). It can thus be concluded that the rigid body rotation C_R measured at the top of floating walls is directly related to the ratio of the excavation depth H to the depth H_d below the lowest level of support (H_d+h). Wall rotations C_R for floating excavations ranged from 0.13" (C6) to 3.65" (C3).

Rigid body rotations for keyed tieback excavations generated inconclusive results. The magnitude of the bracing forces most likely controlled the rigid rotations for these walls. Rigid body rotations for keyed tieback projects ranged from C_R=0.3" (W1) to C_R=0.86" (B2).

Figure 8.22 plots the ratio of C_R/H_W versus $H/(H_d+h)$ for top/down diaphragm wall excavations. The data shows some scatter but a line can be fitted accurately through the C8, B10 (1), and B10 (2) data points. This fitted trendline can only serve as an upper limit for C_R for top/down projects. Rigid rotations for top/down excavations ranged from C_R=-0.22" (B5) to C_R=1.52".

Rotational movements were almost zero for four out of the five projects supported by cross-lots and internal braces. Bending deflections (C_m) for keyed tieback diaphragm walls did not generate any trend because the data was very limited. Wall bending deflections for keyed tieback walls ranged from C_m=0.16" to C_m=0.51".

Figure 8.23 correlates C_M/H_M and the ratio $H_w/(H_d+h)$ for floating excavations. A linear correlation can be fitted through the data with good accuracy (R^2=0.81). Bending deflections for floating walls were generally small ranging from C_m=0.0" (B7, C3) to C_R=0.51" (C4).

Figure 8.24 shows that there is a good new correlation (R^2=0.94) between C_M/H_M and the ratio $H_w/(H_d+h)$ for top/down excavations. The B4 project was excluded from the trendline fit because it plotted too high. This suggests that there is a strong relation between maximum wall bending and the ratios H_W/H_d and H/H_w. Maximum bending deflections for top/down projects ranged from C_m=0.3" (C8) to C_m=1.58" (B10).

Figure 8.25 plots C_M/H_M as a function of $H_w/(H_d+h)$ for cross-lot and internally braced diaphragm wall excavations. The trendline on this figure is based only on four projects. This trendline suggests that bending deflections for these projects generally increase as the ratio $H_w/(H_d+h)$ increases. Maximum bending deflections for cross-lot and internally braced walls ranged from C_m=0.3" (S9) to C_m=1.9" (B8). The larger scatter in the data compared to other types of excavations is most likely the result of bracing forces.

The bending mode shape (C_m/C_{mMax}) is plotted as a function of the depth ratio y/H_W, for 16 of the slurry wall excavations in Figure 8.26. Wall bending can be standardized by this method for a variety of projects (R^2=0.73). Floating walls in Chicago generated bending mode shapes that did not follow the general pattern and were excluded from this graph. Maximum wall bending occurs between 30% and 60% of the wall depth but the average maximum bending occurs slightly below 50% of the wall depth. For the studied projects, the average maximum deflection was at 54% of H_w as measured from the top of the wall or 46% of H_w as measured from the base of the wall. This result is consistent wit the findings of DiBiagio and Roti [1979] who reported that the total resultant force on a diaphragm wall in Oslo was located between 44% and 45% of the wall height from the bottom.

Two sinusoidal curves were fitted through the data with reasonable accuracy. The peak of the two curves was at the location where the average maximum wall bending C_m occurred. These sinusoidal curves yielded effective wall lengths of 66%H_w and 63%H_w for the part of the walls above and below the point of maximum wall bending respectively (where $dC_m/dy = 0$ if the curves where extended). The sinusoidal equations used to fit the data and their input parameters are listed in Table 8.6.

Table 8.6: Equations used to standardize wall bending deflections

For, $d/H_w \leq 54\%$

$C_1 = C_m/C_{mMax} = (\sin\theta_1 - \sin(-\lambda))(1-\sin(-\lambda))$

with

$$\theta_1 = \frac{90^\circ + \lambda^\circ}{\mu}\,(d) - \lambda^\circ$$

For $d/H_w \geq 54\%$

$C_2 = C_m/C_{mMax} = (\sin\theta_2 - \sin(180^\circ + \rho))(1-\sin(\rho))$

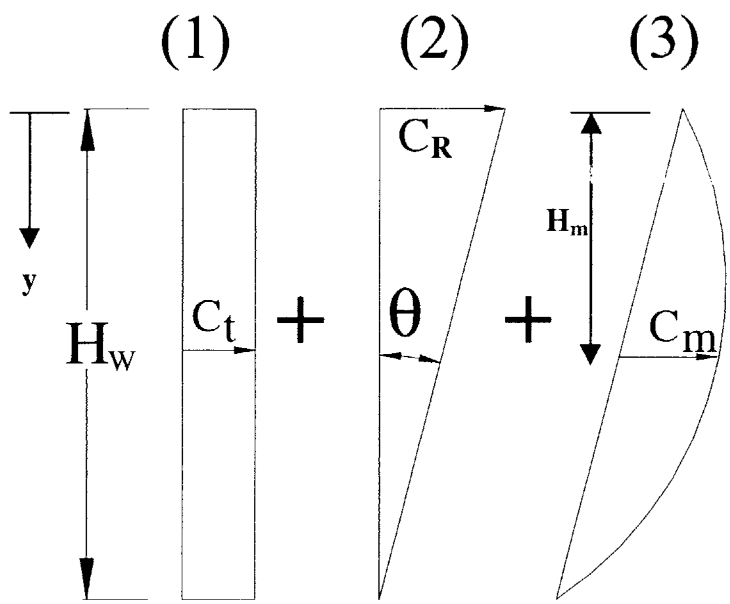
with

$$\theta_2 = \frac{90^\circ + \rho^\circ}{(1-\mu)}\,(d-\mu) - \lambda^\circ$$

<u>For the projects in Figure 8.29:</u>

$\lambda = 57.5^\circ$, $\rho = 65.5^\circ$, and $\mu = H_m/H_w = 54\%$ for the location of the Maximum Bending point from top of wall.

$d =$ depth divided by the wall depth (y/H_w).



$C_R = \tan(\theta)\ H_W$

Figure 8.19: Separation of wall deflections and standardization approach: (1) rigid translation, (2) rigid rotation, (C) Bending, H is the wall depth.

Table 8.7: Summary of excavation basic components describing deflection modes.

ID	Project	H_w (ft)	H (ft)	H_d (ft)	h (ft)	C_t (inches)	θ Degrees	C_R (inches)	C_m (inches)	H_m/H_W %
Floating Tiedback Walls										
B3 (1)	State Transportation1	48	27	19	10	0.425	0.0711	0.715	0.378	44.0
B3 (2)	State Transportation2	46.5	25	19	10	0.190	0.0374	0.364	0.461	46.2
B6	One Memorial Drive	47	30	24	9	0.140	0.1178	1.160	0.298	56.8
B7	500 Boylston	47	42	14	9	1.050	0.2235	2.200	0.000	0.0
C3	Amoco (Standard Oil)	40	23	15	17	1.000	0.4357	3.650	0	0.0
C4	Water Tower	57	44	22	15	0.940	0.1181	1.410	0.511	43.9
C6	Prudential Two	52	25	28	13	0.143	0.0120	0.131	0.116	61.5
C7	AT&T	55	27	30	9	0.147	0.0353	0.407	0.427	22.7
C9	NU Parking Garage	37.5	23	24	7.5	0.185	0.0193	0.152	0.0845	38.3
C10	Museum Science Industry	40	34	10	12	0.000	0.0704	0.590	0.306	31.6
Keyed Tiedback Walls										
B2	60 State Street	60	35	27	15	0	0.0684	0.859	0.509	41.7
B12	Daba Farber Tower	50	88	2	10	0	0.0372	0.390	0.166	40.0
W1	World Bank	60	60	7	11	0	0.0241	0.303	0.309	52.2
W3	Washington Convention Center	77	55	23	14	0	0.0414	0.667	0.246	62.3
Top/Down Excavations										
B4	75 State Street	100	65	30	11	0	0.0408	0.854	1.458	53.0
B5	Rowes Wharf	74	55	30	11	0	-0.0143	-0.222	0.525	49.0
B10 (1)	Post Office Square1	91.8	75	12	10	0.100	0.0300	0.577	1.583	57.6
B10 (2)	Post Office Square2	88	75	16	10	0.127	0.0824	1.519	1.179	56.8
B11	Beth Israel Deaconess	98	65	24	11	0	0.0050	0.103	0.842	56.5
C8	Guest Quarters Hotel	52.5	35	24	11	0	0.0457	0.503	0.3	66.7
Cross-lot and Internally Braced Excavations										
B8	Flagship Wharf	64	47	13	16	0	-0.0246	-0.330	1.888	55.2
W2 (1)	Petworth Subway Station1	72	60	12	14	0	0.0043	0.065	0.5465	58.4
W2 (2)	Petworth Subway Station2	76	60	16	14	0	0.0000	0.000	0.7325	50.0
W5	Federal Center Subway Station	74	60	14	16	0	0.0000	0.000	0.625	46.7
S9	Yerba Buena Tower (CB1)	111.5	66	46	15	0.426	-0.0034	-0.080	0.303	52.2

H_w Wall depth

H Excavation depth

H_d Embedment depth

H_m Depth to Maximum Bending Deflection Measured from Top of Wall as a% of Wall Depth

h Spacing from lowest support level

C_t Rigid Body Translation

C_R Rigid Body Rotation

C_m Bending deflection

θ Rigid Body Rotation

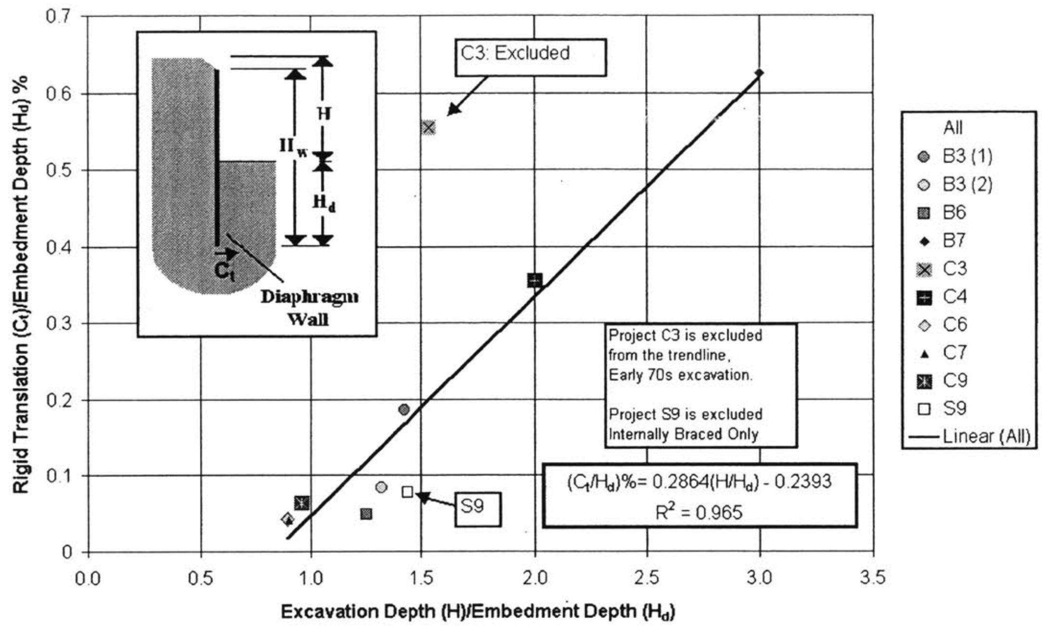


Figure 8.20: Rigid translation deflections for floating tieback excavations.

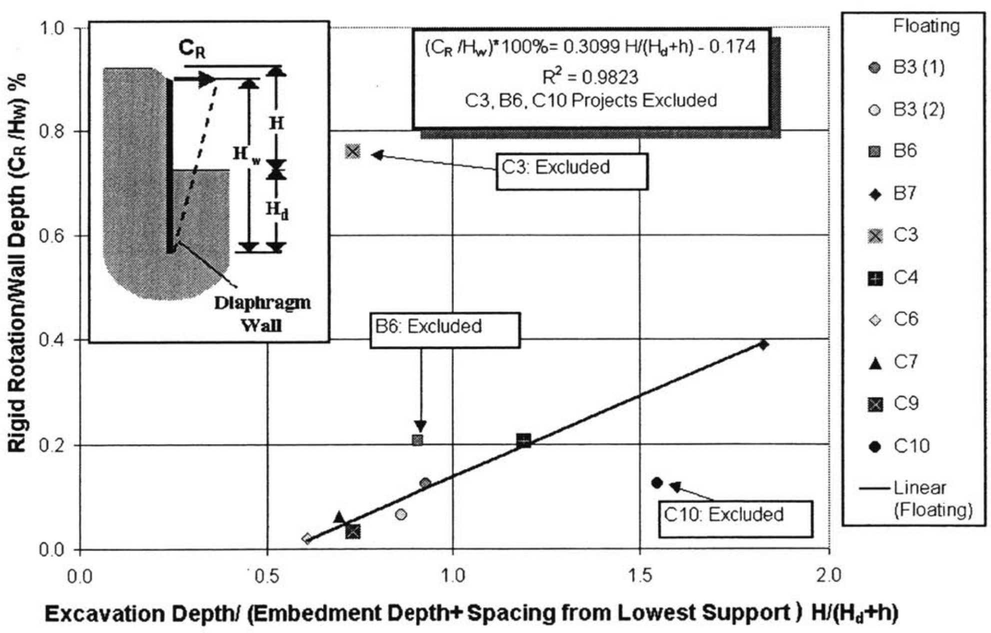


Figure 8.21: Rigid Rotational Deflections C_R for floating tieback diaphragm walls.

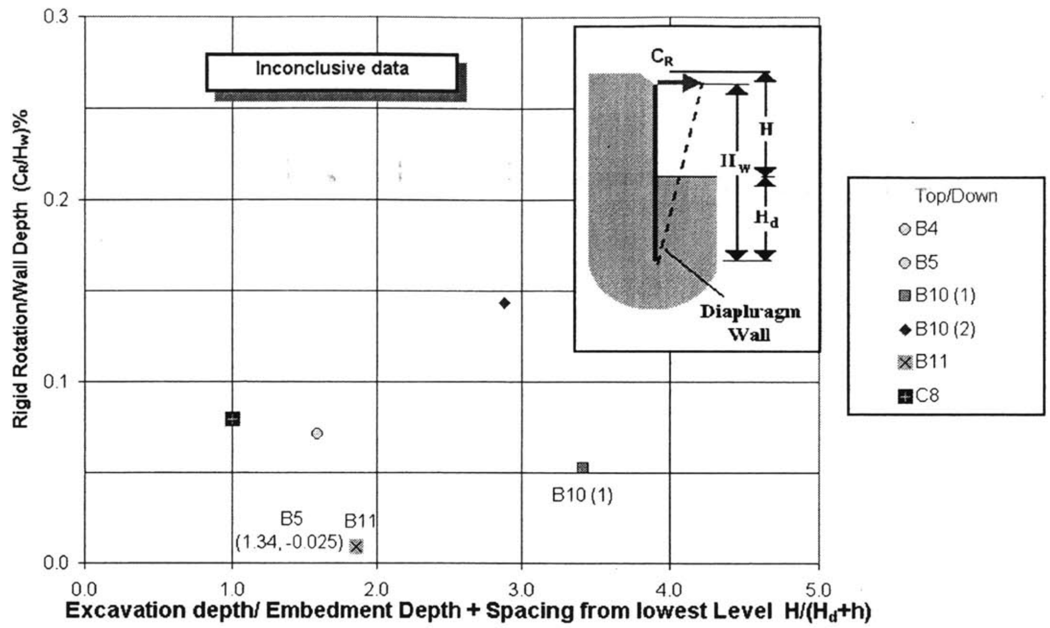


Figure 8.22: Rigid Rotational Deflections C_R for top/down excavations.

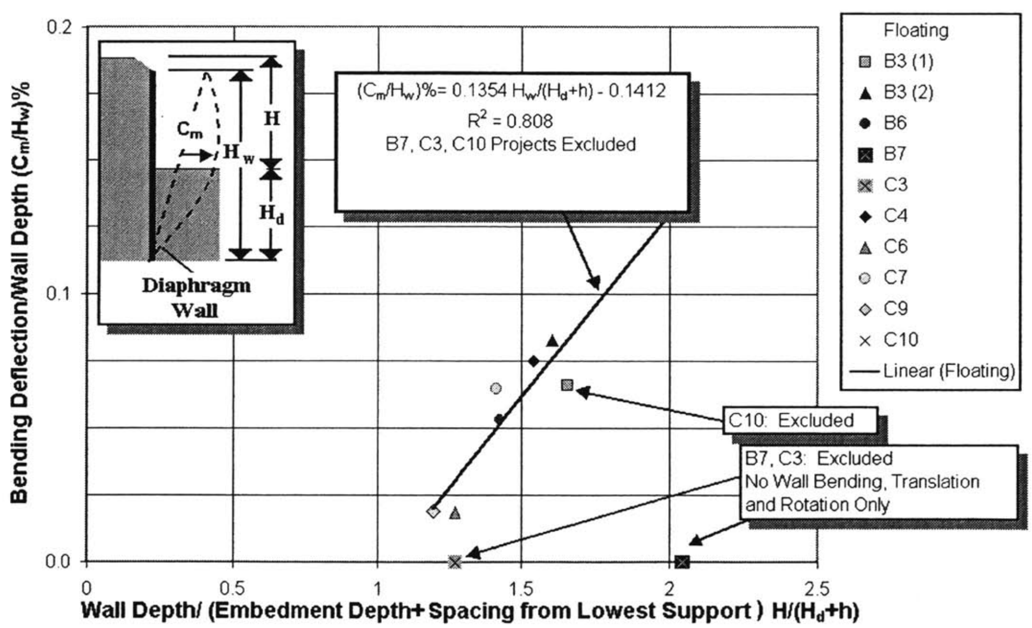


Figure 8.23: Wall Bending Deflections C_M for floating tieback diaphragm walls.

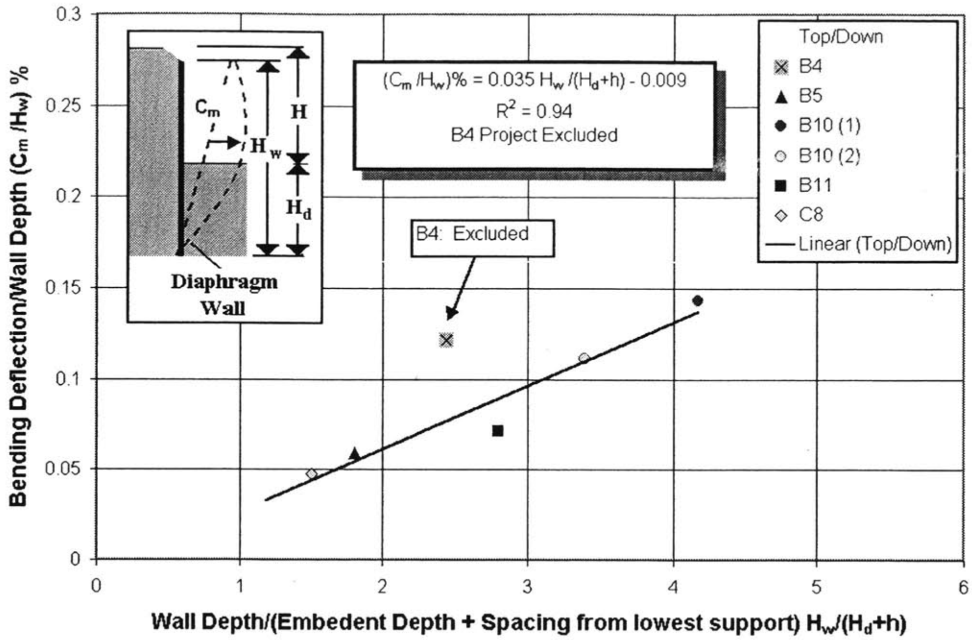


Figure 8.24: Wall Bending Deflections C_M for top/down diaphragm wall excavations.

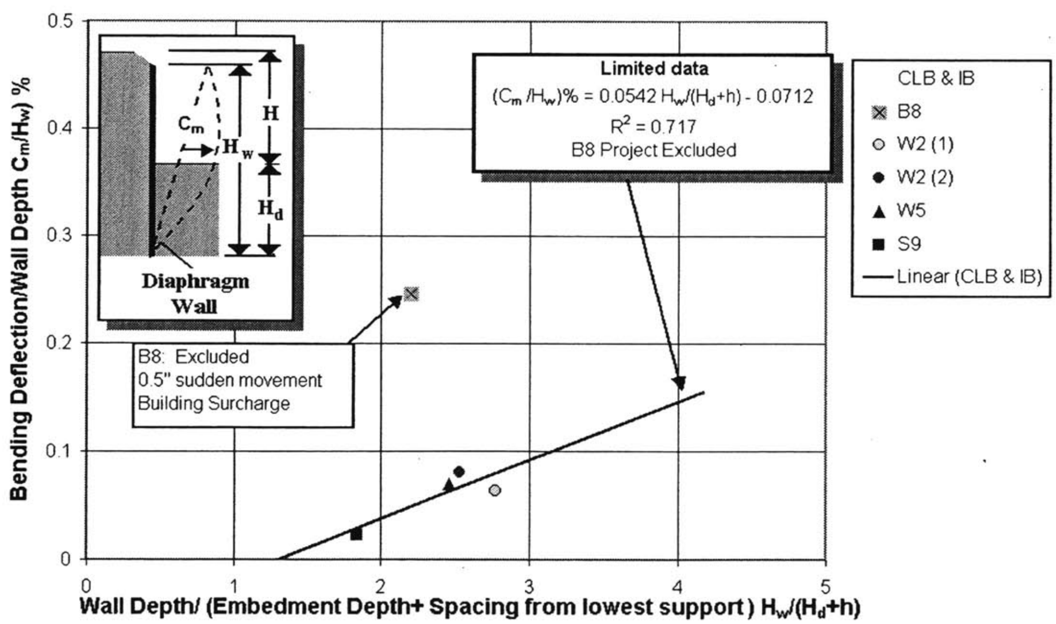


Figure 8.25: Wall Bending Deflections C_M for cross-lot and internally braced diaphragm wall excavations.

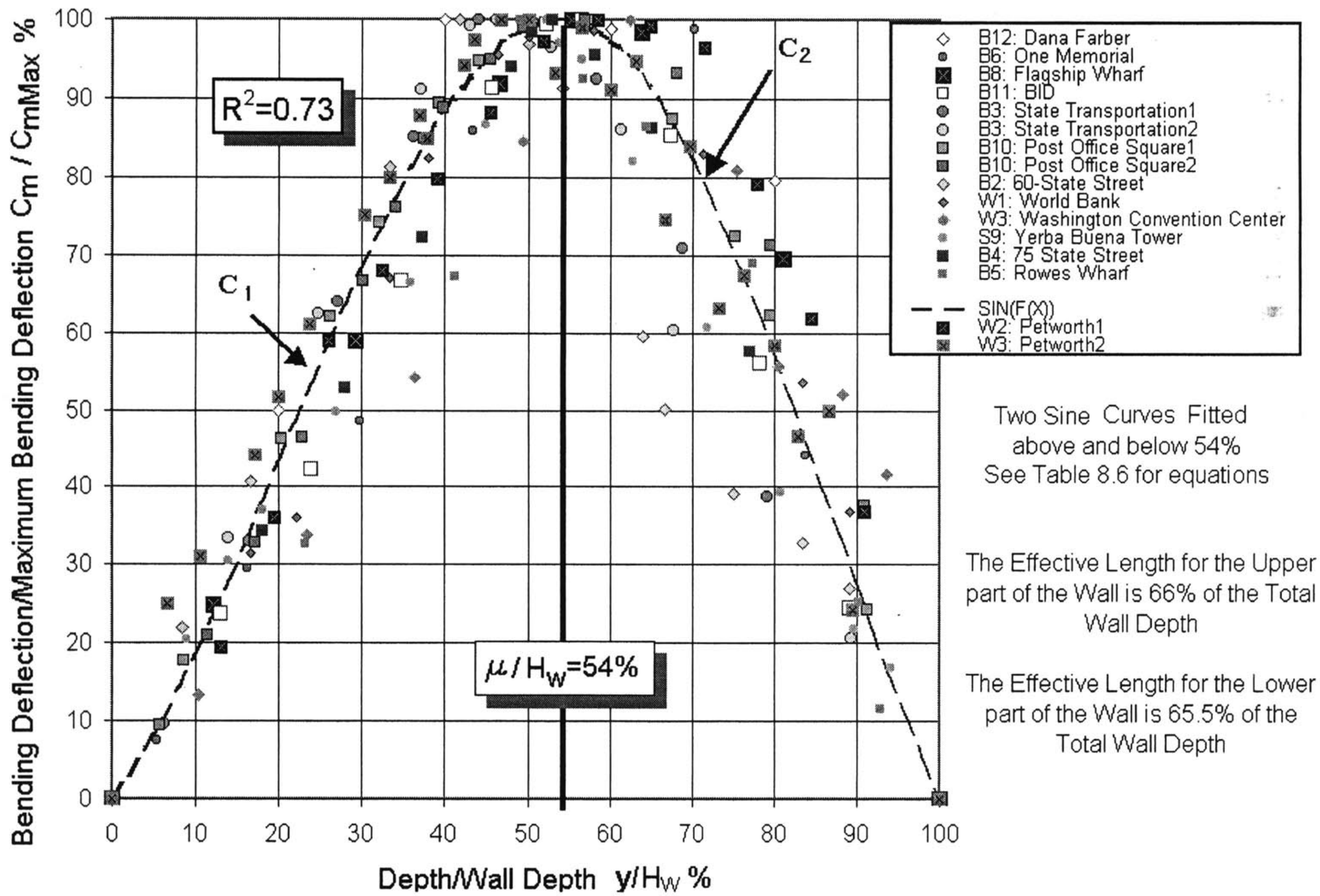


Figure 8.26: Results from proposed method of standardizing wall bending

Chapter 9
Summary, Conclusions, and Recommendations

Chapter 9
Summary, Conclusions, and Recommendations

9.1 Summary

This thesis studied the performance of 29 slurry wall excavations in Boston, Chicago, Washington DC, and San Francisco from 1970 to 2000. Of the 29 excavations, 25 were constructed after 1980 and a few have just recently been completed. Archived performance data was gathered for each project and a complete case study was constructed for each excavation. Published data was incorporated into the thesis wherever possible. Each case study provides information about a) soil and site conditions, b) excavation profiles at various stages of construction, c) measured performance data for various construction stages (deformations, settlements, brace loads, water levels etc.), d) qualitative data (water leaks, problems with slurry wall construction etc.), and e) a summary of each case study. The complete case studies can be found in the MIT Research Report by Konstantakos & Whittle [2000] that will be published in the summer of 2000. The thesis itself contains a brief summary of each case study. The database will also be posted on the Internet during summer 2000.

Once the complete database was constructed, projects were grouped into four categories according to bracing type and wall fixity. The four categories were 1) floating tieback walls, 2) keyed tieback walls, 3) top/down excavations, and 4) cross-lot and internally braced excavations.

Wall deflections for each category were subdivided into three types namely: 1) rigid body translations, 2) rigid body rotations, and 3) bending. Frequency plots were constructed for the wall deflections for all different excavation types. Deflection data was also plotted versus the system stiffness as proposed by Clough [1981]. Settlements were plotted for all the projects for which they were available.

9.2 Conclusions

The vast majority of the studied slurry wall excavations cause very small deformations, which in most cases are less than 1.0". Settlements are in the same order as wall deformations, but they are also affected by other factors such as the drilling method of tiebacks if used. Variations of deformations can be large even within the same project. In all cases, the maximum deformations represented only a small fraction of all the slurry wall locations that were monitored. Small local variations in construction and in the site conditions seem to control the occurrence of maximum deformations in a given project. The effect of the increased wall rigidity was evident in most cases since the walls did not bend a lot between significantly between supports.

Deformations were larger in earlier projects and in projects were other factors induced large soil movements. Such factors were: a) ground softening due to caisson construction, b) ground losses through tiebacks, c) ground softening due to pile extraction, d) ground softening due to soft backfill material for Load Bearing Elements, e) inadequate wall embedment, and f) inadequate bracing and load loss in the bracing.

Rigid body translations are most important in floating excavations. The translative movements at the base of the wall were found to be related to the excavation depth and to the wall embedment depth. Translative movements ranged up to 1.1" (C3, B7) when adequate embedment was not provided. Rigid body wall rotations are nearly correlated to the ratio of excavation depth to embedment depth plus the vertical spacing from the lowest support level to the excavation grade. For floating tieback walls the correlation between the two is linear with great accuracy. Top/down excavations showed larger scatter in wall rotations than floating walls, and the vertical support did not affect the magnitude of movements. Cross-lot and internally braced walls showed little to no rotation due larger bracing forces in the upper bracing levels. Rigid body rotations were the largest for floating walls, reaching 3.7" and 2.2" for the C3 and the B7 projects respectively.

The ratio of bending deflection to wall depth is closely related to the ratio of excavation depth to wall depth plus the vertical spacing from the lowest support level to the excavation grade. The linear relations that were derived provided an excellent fit for top/down excavations, a good fit for floating walls, and a moderate fit for and cross-lot braced excavations. Bending deflections were generally the largest in top/down excavations reaching 1.6" at the B10 excavation. The B8 cross-lot project had the largest bending with roughly 1.9" towards the excavation. Floating tieback excavations generate small wall bending in the order of 0.5".

For many projects, wall bending can be standardized by the wall depth and by the maximum bending deflection. It was found that two sinusoidal curves could describe with reasonable accuracy the bending deflections for many projects. The maximum bending deflection generally occurred at 54% of the diaphragm wall depth as measured from the top (or 46% from the base of the wall).

Water leakage is very difficult if not virtually impossible to avoid. Minor leakage between panel joints will occur even if the best precautions are taken. Attention must be given to water-stopping details at tieback holes or when caissons are constructed between slurry wall panels. Generally, increasing the number of openings and the number of joints in a diaphragm wall increases the chances of water leakage.

Slurry wall construction faces some other minor problems. It is usual that a panel or two will collapse during trenching in each job. Some of the collapses were caused by fill that slid under the guidewalls, while others were due to panels being left open for an extended period of time. In two cases (B3, B8), panels collapsed when the contractor trenched without slurry.

Occasionally, zones of soft soil-slurry material get entrapped between the tremied concrete. This can cause major problems when the bottom of the panels is not cleaned adequately, since the flow of the tremied concrete will entrap a large portion of this material within the panel. These pockets of soft material will leak excessively when the excavation exposes them.

Concluding, the slurry walls studied in this thesis performed very well and induced small wall deflections and settlements except for cases when other construction practices caused problems. Thus, diaphragm walls can be very effective when deformation control is important providing that bracing and wall embedment are adequate.

9.3 Recommendations

This thesis and the associated data report (Konstantakos & Whittle, 2000) have attempted to develop a perspective on the performance of slurry wall supported excavations, based mainly on projects carried out during the last 20 years. The project has illustrated most clearly the limitations of data mining from projects archived in the files of engineering companies (consultants and contractors). Of the 29 projects investigated during this research, only a small subset have archived all of the performance monitoring data that were obtained during the construction. This reflects the limited storage capabilities (in the pre-digital age) and the progressive pruning of the archives over time. It is clear that the most reliable data have been obtained from projects that are either on-going or recently completed (perhaps the most notable example is the Dana-Farber project, B12) or those that have been perceived as a particular technical challenge (e.g., Post Office Square, B10). Given this situation, the geotechnical profession should invest more energy in reporting results from well documented case studies, or disseminate these data through some more centralized database. The Author plans to provide access to the current database through the internet (work to be completed during the Summer 2000).

The interpretation of performance data for the slurry wall projects described in this thesis has focused heavily on lateral wall deflections, based on inclinometer records. In contrast, there is much less archival data on ground movements and their effect on adjacent structures (data was notably absent from projects in Chicago). There is even less data for measuring the actual structural loads in the bracing (only found in 5 out of the 29 case studies) and only qualitative records of leakage problems. Hence, the current database provides only a modest contribution to the empirical prediction of surface settlements. However, the data have confirmed the importance of tieback installation procedures, and other ancillary construction activities (caisson or LBE construction, pile extraction etc.) on surface settlements. The data have offered no new insight into the structural design of bracing systems. However, recent experience in the CA/T project (Whittle, 1999) has confirmed that these bracing systems are generally overdesigned using apparent earth pressure envelopes. Future projects should place greater emphasis in recording ground movements and relating them to construction activities and in measuring forces in key structural elements.

The current study has focused exclusively on diaphragm walls. There remains almost no quantitative data to compare the performance of different types of wall systems (i.e. soldier pile and lagging, sheet pile, contiguous bored pile, soil-mix etc.). This type of comparison could be carried out by numerical analyses (by finite element methods), but this type of computational study will be less persuasive to the practice than well documented field data.

The current database includes several well documented projects that deserve further study and can serve as useful sources for validation of analyses.

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