**Design And Performance Of Earth Retaining Structures**

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# Design And Performance Of A Deep Excavation Support System In Boston, Massachusetts

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**Meeting Reprint**

# DESIGN AND PERFORMANCE OF A DEEP EXCAVATION SUPPORT SYSTEM IN BOSTON, MASSACHUSETTS

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## ABSTRACT

A deep excavation encompassing most of a Boston, MA city block provided an excellent opportunity to monitor and analyze the performance of a lateral earth support system. The excavation which was approximately 60 feet below street grade utilized three types of earth support; soldier piles and wood lagging, bracket pile underpinning, and a "tangent pile" system, all of which were supported with earth tiebacks. Extensive instrumentation was installed to monitor ground movement and to calculate stresses in the soldier piles above and below the excavation subgrade. A major objective of the investigation was to analyze the moment and load distribution along the pile and to provide information which is useful for designing pile sections and toe embedment lengths. The instrumentation was plannned for research purposes, however, unexpected soil movements on one side of the excavation caused the instrumentation readings to become extremely valuable in understanding the ground response during excavation.

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## INTRODUCTION

The project, known as 125 High Street is a 40 story office building with 5 levels of below grade parking situated in Downtown Boston. The first phase of construction, located in the area of the Old Travelers Building, involves approximately two thirds of the project. Phase 2 of construction begins when Phase 1 is far enough along to permit relocation of the Fire Station. A row of temporary earth support between the two phases is shown as a dashed line on Figure 1. A building of historic significance (135 Oliver Street) remains on the site and is supported in place while the new building wraps around.

The 135 Oliver Street Building has a one level basement and is founded on a continuous wall footing consisting of granite blocks placed on glacial till. Excavation support for the rear or westerly portion of the 135 Oliver Street Building consists of a drilled in "tangent pile" system supported by tiebacks.

The rear portion of the existing fire station is supported by a drilled in bracket pile underpinning system supported by tiebacks, all of which is later removed along with the existing Fire Station.

Lateral support for the remainder of the site consists of a drilled in soldier pile, lagging, and tieback system. The majority of the support system is constructed off wall line (i.e., requiring 2 sided forming for the basement walls).

## SOIL CONDITIONS

The geotechnical information available at bid time and during design indicated that the building site is situated in the Fort Hill area of Boston, mostly within the Fort Hill drumlin deposit, which is considered to be some of the best foundation soils in Boston. The top of the drumlin was removed in the past to provide fill material for Boston's Backbay area. A "till like" material (glacio-marine deposit), consisting of stiff to very stiff silty clay with varying amounts of sand and gravel overlying the glacial till, was encountered along Pearl Street.

The soil conditions as interpreted from the borings (location shown in Figure 2) at the site generally consist of:

- A surface layer of fill consisting of a silty SAND with varying amounts of gravel, brick, concrete, wood, and other demolition debris. The fill varies in thickness from 2 to 13 feet.

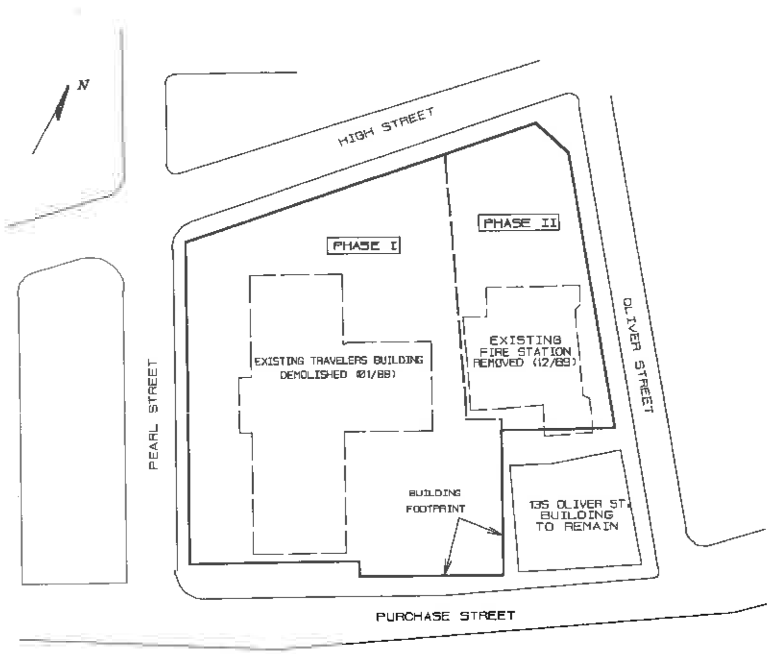



*FIGURE 1: SITE PLAN*

- A glacio-marine deposit consisting of a stiff to very stiff, gray, silty CLAY with trace to little sand to gravel, ranging in thickness from 5 to 15 feet (encountered in two recent borings).

- Below the glacio-marine deposit, and in most instances directly below the fill, lies a deposit of glacial till. The till is generally composed

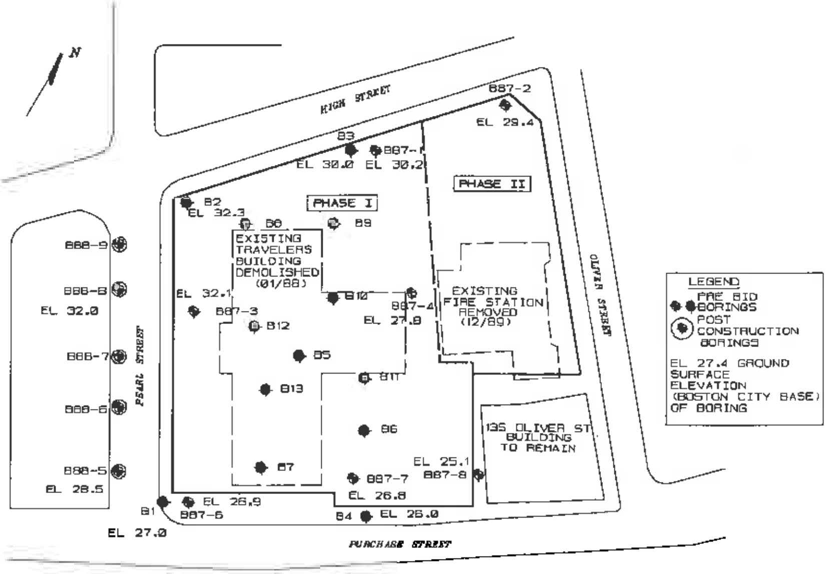



*FIGURE 2: BORING LOCATION PLAN*

of a hard, gravelly to silty CLAY with some to trace coarse to fine sand and gravel. None of the recent borings fully penetrated the till, with the deepest exploration reaching approxmately El.-70. Embedded in the glacial till are lenses and layers of sand, described as a dense, coarse to fine SAND with some to trace gravel and silt, varying in thickness from 6 to 21 ft.

- Based on existing information, bedrock, known as Cambridge Argillite, lies below the glacial till. (1)

The Geotechnical Engineer's report indicates that

"Water levels recorded in the observation wells indicate a perched water condition with levels varying from El. 8.6 to El. 21.8. The piezometric level in the glacial till stratum varied between El. 0 to El. 15 with typical levels between El. 3 to El. 10." (1)

## DESIGN CONSIDERATIONS

Due to the compact nature of the glacial till, a driven pile support system was not feasible. Soldier piles placed in drilled holes backfilled with concrete were used for all three types of support. Lean mix concrete was used as backfill for all piles except the "tangent piles" and the toe portions of the bracket piles in which 2500 psi concrete was used.

Sizing of the soldier piles and tiebacks was based on the loading diagrams shown in Figure 3. The loading depicted in Figure 3a is typical for that used in the design of the Pearl Street and High Street excavation support system. Figures 3b and 3c show loading diagrams for the bracket pile support at the Fire Station and the "tangent pile" support system at the 135 Oliver Street building. A failure plane extending from subgrade at an inclination of 3 vertical: 2 horizontal was used for selecting the tieback lengths (Figure 3a).

## PILES

Soldier piles consist of paired wide flange sections (W 12 x 26 and W 12 x 30) connected with a 13 inch space between webs to allow tieback installation. The piles were designed for combined axial and bending stresses with typical toe penetrations of 5 to 6 feet. The piles were not analyzed as composite sections since the lean mix (1 sack of cement per cubic yard) is not considered strong enough to develop composite action. Wood lagging (3 inch nominal) is used to span between the soldier piles which are spaced 9 to 10 feet on center.

The bracket piles consist of HP 14 x 117 sections placed in drilled holes with structural concrete in the toe and lean mix backfill above subgrade. A steel bracket is welded to the back of the pile to support the building footing (Figure 3b).

The "tangent piles" are installed in 36 inch diameter holes drilled 48 inches apart and backfilled with 2500 psi concrete. This system was selected over a conventional pit underpinning scheme which would have been uneconomical to construct in the compact glacial till. The system stiffness is enhanced by both the close spacing of the piles and tiebacks, and the structural concrete backfill.

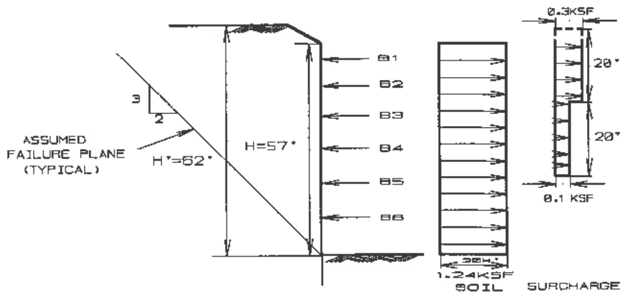



*FIGURE 3a: TYPICAL LOADING DIAGRAM FOR PEARL ST. & HIGH ST.*

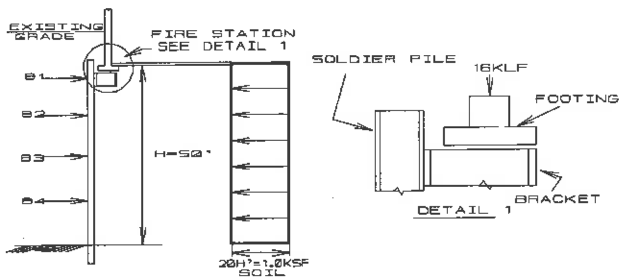



*FIGURE 3b: LOADING DIAGRAM FOR BRACKET PILES*

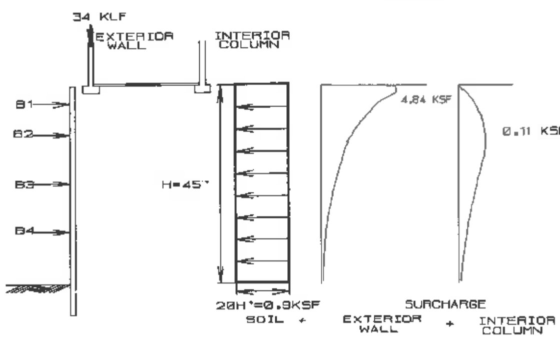



*FIGURE 3c: LOADING DIAGRAM FOR TANGENT PILE SYSTEM*

## TIEBACKS

Tieback tendons consist of prefabricated three and four strand (0.6 inch dia., 270 ksi steel) tendons and are installed by two different methods:

A. Hollow Stem Auger (HSA)

The job was planned initially to be supported with hollow stem auger tiebacks. HSA tiebacks are installed by inserting the tendon inside a 12 inch diameter continuous flight auger prior to drilling. The auger is then drilled into the ground. When the desired depth is reached, a grout (portland cement, sand, water, and fly ash) is pumped through the auger as it is being withdrawn. Because of the unexpectedly high number of boulders and the compactness of the till the method of installation was changed from the HSA to a smaller diameter regroutable tieback on some portions of the work.

B. Regroutable Tiebacks

Regroutable tiebacks are installed in 6 inch diameter augered holes or in 6 inch diameter cased holes if caving soils are encountered. The prefabricated tendons and regrout tubes are inserted in tremie grouted holes and regrouted once or twice to achieve capacity. Tieback design loads ranged from 93 kips to 141 kips. All tiebacks were tested to at least 125 percent of design load and locked off at the design load.

## RAKERS

Inclined rakers to concrete heel blocks were installed at selected soldier piles along Pearl Street. The rakers were installed and wedged in place as the excavation approached subgrade. The purpose of the rakers was to attempt to limit further lateral movement of the sheeting on Pearl Street.

## INSTRUMENTATION

The focus of this paper is on the contractor installed instrumentation (Figure 4) described below. Reference will also be made to readings obtained from various optical survey monitoring points established on soldier piles, buildings, and streets around the site.

1. SLOPE INCLINOMETERS

Slope inclinometers (See Figure 5) were installed on two pairs of soldier piles (4 total) to a depth of approximately 15 feet below the tip of each soldier pile. The inclinometer casing within the soldier piles was installed by:

a. Placing a PVC sleeve within the lean mix concrete backfill.
b. Drilling through the sleeve to the required depth.
c. Setting the inclinometer casing.

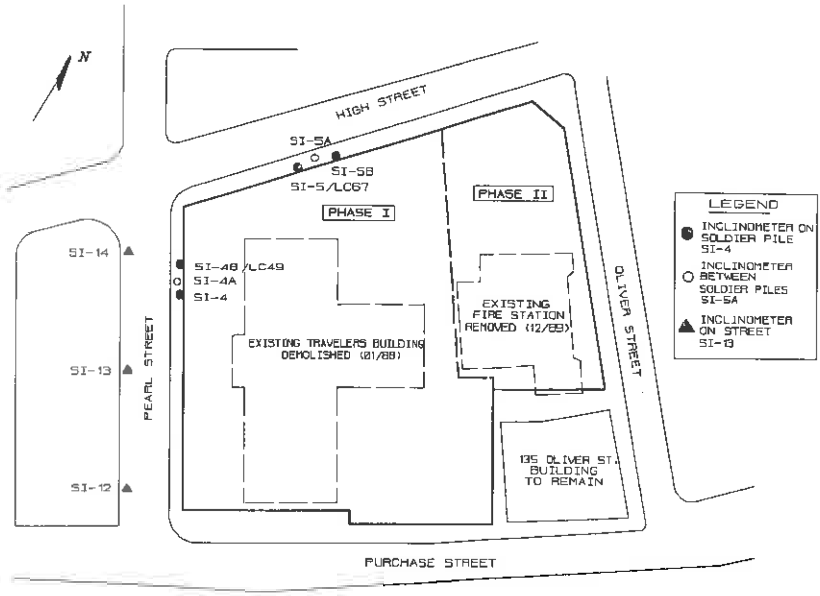



***FIGURE 4:*** *INSTRUMENTATION LOCATIONS*

d. Grouting the inclinometer casing in place as the drill casing is withdrawn.

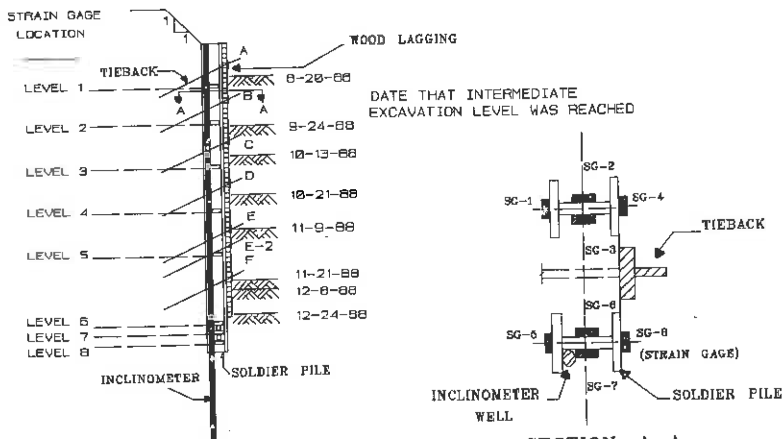
Additionally, inclinometers were installed between the pairs of instrumented soldier piles (48 and 49, 67 and 68) to depths of 15 feet below the tips of adjacent piles. The inclinometer casing between soldier piles was installed similarly except that drill casing through the soils was used for the full depth. These inclinometers were installed after the piles were placed.

2. STRAIN GAGES ON SOLDIER PILES

Clusters of Geokon Model VSM-4000 vibrating wire strain gages (total of 250±) were installed on the 4 soldier piles between tieback locations (Figure 5) and in the soldier pile toes. The gages were installed by welding attachment brackets to the pile and fastening the gage to the brackets after cooling. The gages were then protected by tack welding a protective cover before pile installation.

3. LOAD CELLS

Thirteen Geokon Model 3000 center hole load cells were installed on the tiebacks for soldier piles 49 and 67 to monitor change in load with time.




SOLDIER PILE DETAIL

SECTION A-A

*FIGURE 5: TYPICAL INSTRUMENTED PILE*

4. STRAIN GAGES ON RAKERS

Two vibrating wire strain gages (one on each side of the web) were installed on each raker.

## OBSERVED PERFORMANCE

Figures 6 and 7 indicate the lateral movement that occurred during excavation on Pearl Street and High Street respectively. The magnitude of ground movement on Pearl Street was several times that observed on the High Street side.

The axial load, moment and combined stress as computed from the strain gage readings are plotted for two piles in Figure 8. A family of plots representing information gathered at different times in the excavation sequence is presented for each parameter.

Optical survey readings on the streets and adjacent buildings taken by the geotechnical engineer showed greater vertical movement on the Pearl Street side than was observed on High Street. The difference in ground movement became noticeable and pronounced as the excavation approached mid depth.

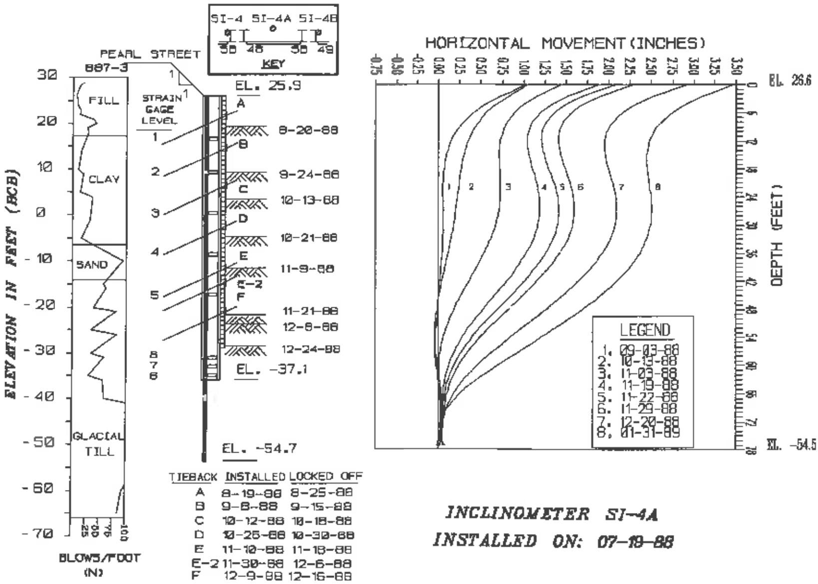



*FIGURE 6a: LATERAL MOVEMENT FOR SOLDIER PILE 49*

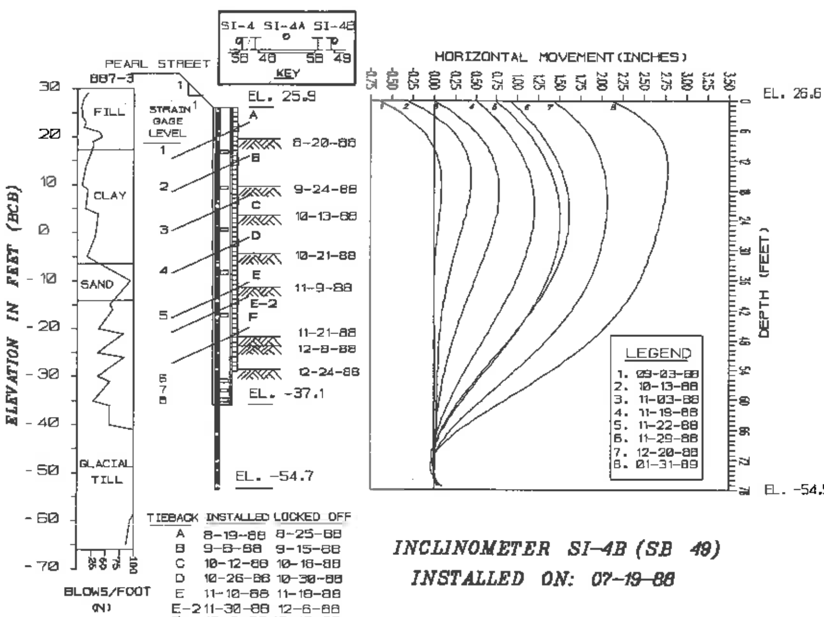



*FIGURE 6b: LATERAL MOVEMENT FOR SOLDIER PILE 49*

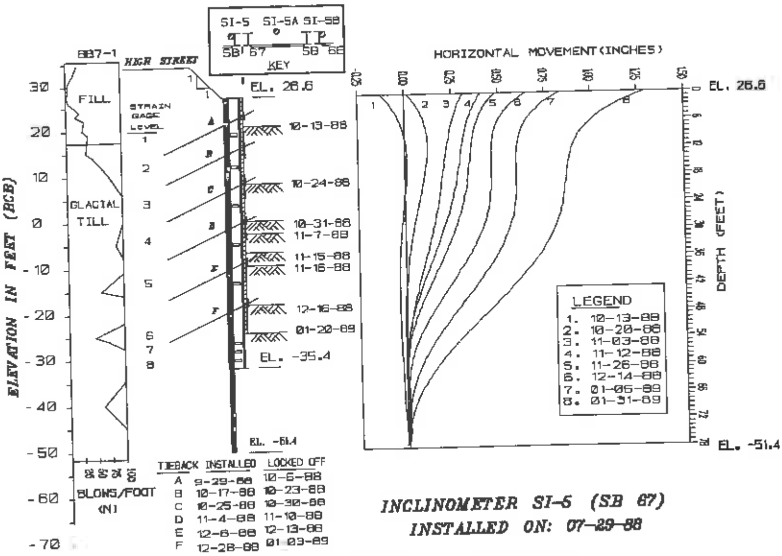



*FIGURE 7a: LATERAL MOVEMENT FOR SOLDIER PILE 67*

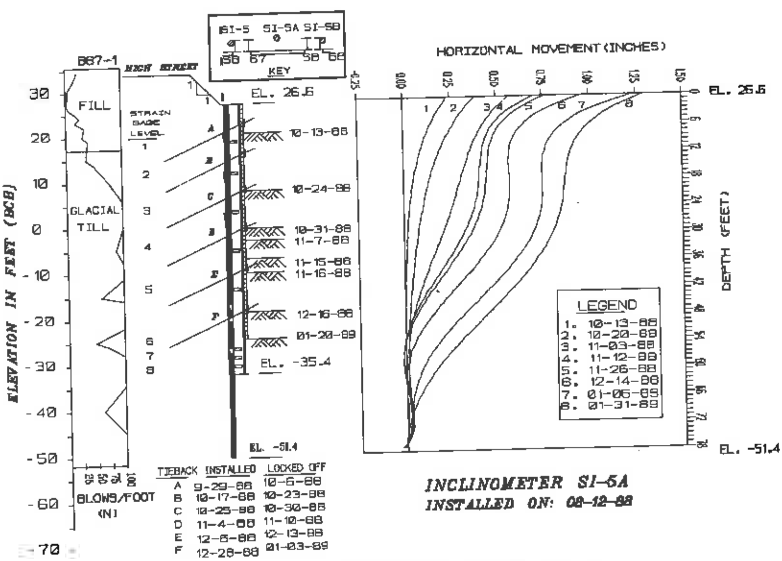



*FIGURE 7b: LATERAL MOVEMENT FOR SOLDIER PILE 67*

Table 1 compares performance of the piles on Pearl Street and High Street as the depth of excavation approached the midway point at the respective locations:

TABLE 1

| | Pearl Street Pile 48(10/21/88) | High Street Pile 67(11/12/88) |
|---|---|---|
| Maximum Lateral Movement (in.) | 3/4 | 3/8 |
| Maximum Axial Load (kips) | 75 | 90 |
| Maximum Moment (kips - ft.) | 80 | 45 |
| Combined Stress (ksi) | 15 | 10 |
| Maximum Building Settlement (in.)* | 0.4 | 0.1 |
| Maximum Street Settlement (in.)** | 1.0 | 0.25 |

* Building reference settlement readings taken by Geotechnical Engineer. (Buildings are located approximately 45 feet and 50 feet from the Pearl Street and High Street sheeting respectively).
** Surface street settlement readings taken by Geotechnical Engineer. (Taken at points ranging from 5 feet to 35 feet from the face of sheeting).

Visual observation of the street and existing buildings revealed that little or no cracking or street subsidence was evident on High Street. Pearl Street, on the other hand, showed signs of distress in the form of non-uniform pavement settlement, cracking in the pavement, and separation between the pavement and curb. While some of the Pearl Street settlement is associated with sheeting deflection, a portion is attributable to other causes. A previously installed deep utility had apparently been backfilled carelessly and street settlements were accentuated over the trench excavation. Leaky sewer and drain lines caused a continuous recharge of water with constant seepage through the lagging boards which was not evident on the other excavated faces.

A row of three story buildings on the West side of Pearl Street began to settle as the excavation reached a depth of 15 to 20 feet. The buildings contain a one level basement below grade and are presumably supported on spread footings.

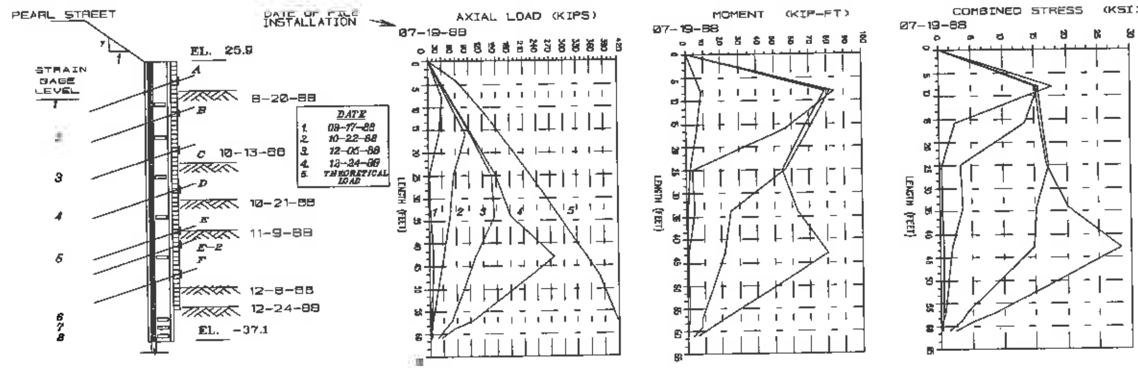



FIGURE 8a: STRAIN ANAYLSIS FOR SOLDIER PILE 49

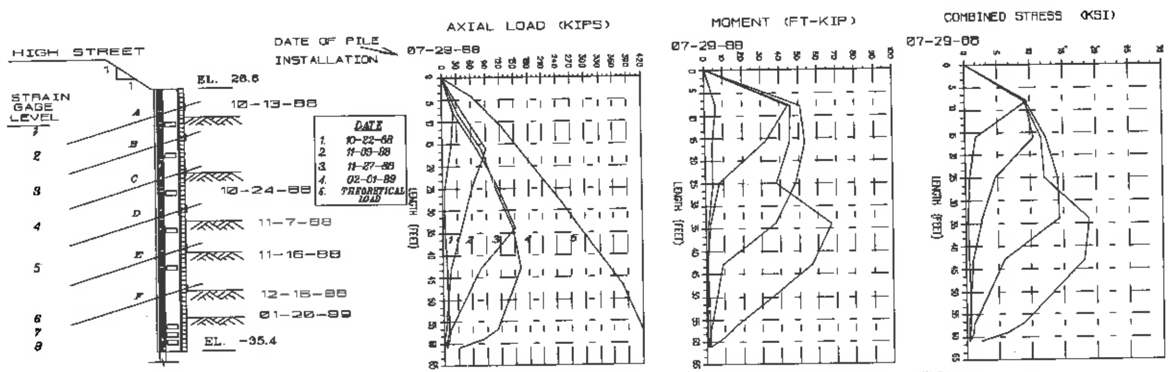



FIGURE 8b: STRAIN ANAYLSIS FOR SOLDIER PILE

The Pearl Street inclinometers (Figure6) show successive bulges in lateral movement as each new cut was made. The magnitude of building settlement also increased as each new lift was excavated. The lateral movement peaks at approximately 2 3/4 inches. The load cell readings (Figure 9), however, generally show an initial loss of load followed by little or no load increase with time. Because of the observed difference in performance between High Street and Pearl Street, and the apparent inconsistency of increasing lateral deflection without accompanying load increase in the tiebacks, it was decided to obtain additional subsurface information.

Several additional borings were taken during construction along the West side of Pearl Street (Figure 2) closer to the area where the tieback anchors were being made. Inclinometers were installed in some of the boreholes to check for subsurface movement. The new borings revealed the "glacio-marine" deposit to be both softer (N values as low as 4) and thicker (up to 20 feet thick) than encountered in the previous borings. It appears that Pearl Street marks a dramatic change in soil conditions as the West side of the Fort Hill Drumlin falls off steeply. Discontinuous granular deposits within the layer tend to trap water and cause localized problems with soldier pile and tieback installation during construction.

Construction was still progressing as the subsurface exploration and analysis began. As the excavation approached subgrade on the southerly portion of Pearl Street, building settlements (on the West side of Pearl Street 45 feet from the sheeting face) of 1 inch or greater were measured. Inclined pre-stressed rakers to heel blocks and an additional row of tiebacks were installed. Readings from strain gages installed on Rakers 31, 33, 35, 37, and 39 in the southerly portion of Pearl Street indicate that load continued to increase after subgrade was reached. Table 2 compares the performance of the piles on Pearl Street and High Street after subgrade has been reached.

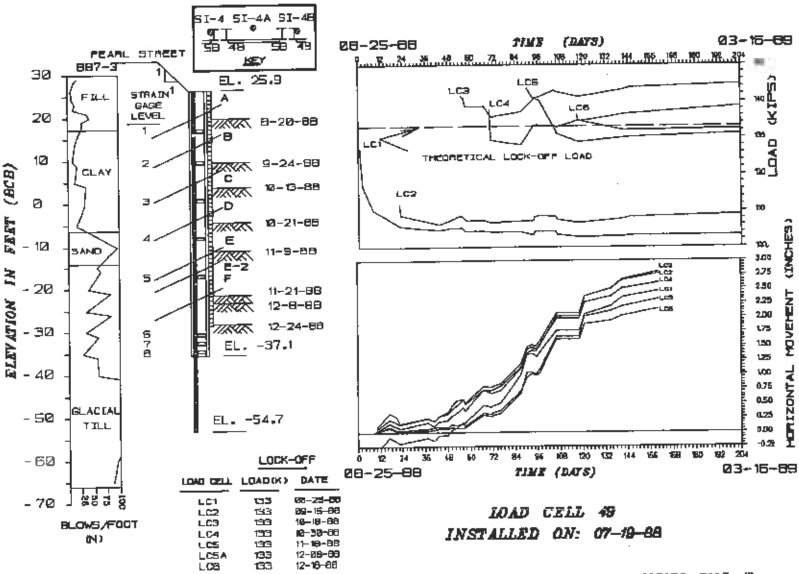



FIGURE 9a: HORIZONTAL MOVEMENT AND LOADCELL INSTRUMENTATION FOR SOLDIER PILE 49

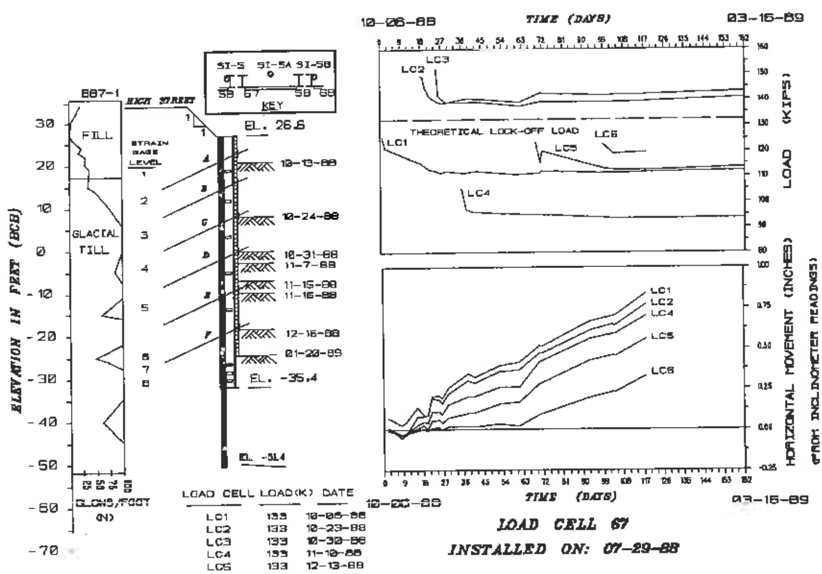



FIGURE 9b: HORIZONTAL MOVEMENT AND LOADCELL INSTRUMENTATION FOR SOLDIER PILE 67

TABLE 2

| | Pearl Street Pile 48(12/24/88) | HighStreet Pile 67(2/1/89) |
|---|---|---|
| Maximum Lateral Movement (in.) | 2.75 | 0.85 |
| Maximum Axial Load (kips) | 275 | 160 |
| Maximum Moment (kips - ft.) | 80 | 68 |
| Combined Stress (ksi) | 28 | 18 |
| Maximum Building Settlement (in.) | 1.4 | 0.2 |
| Maximum Street Settlement (in.) | 4 1/4 | 3/4 |

Extra tiebacks and rakers (44, 46, 51, and 53) were installed in the northerly portion of Pearl Street (see Figure 10 for location) before the subgrade elevation was reached. Data from strain gages installed on these rakers also show that increasing load was experienced by

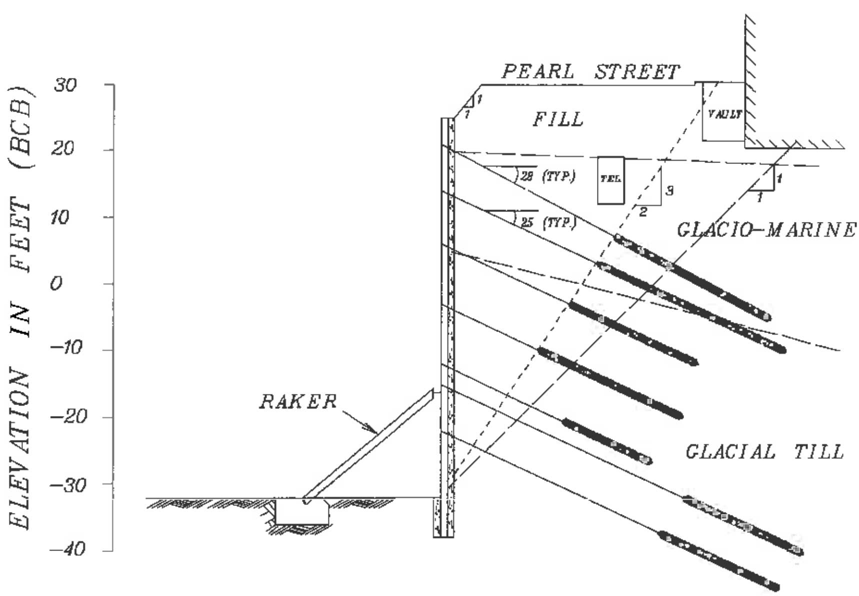



FIGURE 10: TYPICAL CROSS-SECTION OF PEARL STREET

the rakers as the excavation proceeded to subgrade. The magnitude of loads summarized in Table 3 are net loads experienced after prestressing the rakers. The benefit of the rakers is very doubtful since the pattern of settlement and deflection continued to occur after the rakers were installed.

TABLE 3

RAKERS

| RAKER# | DATE OF READING/LOAD (KIPS) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 12/8/88 | 12/13 | 12/21 | 12/28 | 12/30 | 1/6/89 | 1/10 | 1/13 | 1/16 |
| 31[1] | 27 | 51 | 84 | 74 | 74 | 67 | 62 | 101 | 112 |
| 33[1] | 31 | 64 | 94 | 88 | 88 | 67 | 93 | 120 | 132 |
| 35[1] | | 33 | 62 | 58 | 58 | 65 | 63 | 83 | 96 |
| 37[1] | | 26 | 45 | 45 | 45 | 47 | 44 | 65 | 71 |
| 39[1] | | 34 | 54 | 51 | 51 | | 54 | 84 | 91 |
| 44[2] | | | | | | | | 12 | 27 |
| 46[2] | | | | | 51 | 90 | 91 | 126 | 143 |
| 51[2] | | | | | 29 | 47 | 143 | 180 | 202 |
| 53[2] | | | | | | 40 | 50 | 94 | 100 |

1. Rakers were installed without walers and pre-stressed on November 27, 1988. Subgrade elevation (-30) was reached approximately November 29, 1988.
2. Rakers were installed on walers and pre-stressed on December 29 - 30, 1988. Excavation level was approximately elevation -25 on December 29, 1988. Subgrade elevation (-30) was reached January 4, 1989.

## ANALYSIS

The contradictory behavior of increasing deflection with decreasing tieback load build-up (Figure 9) provides an interesting dilemma. The most likely explanation is that a mass stability type movement occurred. Figure 10 shows the assumed failure plane used for design. On all streets except Pearl Street the tiebacks were anchored exclusively in the till. On Pearl Street a different set of soil conditions was encountered. The "glacio-marine" clay layer was found to be softer and thicker which suggests that the thickness of the clay layer increases toward the west as the steeply sloping drumlin side drops off. Table 2 shows that measured movements on Pearl Street were greater than on High Street.

The inclinometers (SI 12, and SI 14) installed on the West side of Pearl Street, after movement problems developed, indicate lateral deflections of approximately 1/2 inch shown in Figure 11. The extent of the deflection shown in Figure 11 indicates that once the glacio-marine deposit began to move the till beneath was also mobilized to displace laterally. Thus, the mechanism of failure involves more than the clay sliding across the top of the till. This conclusion seems to be supported by the fact that the load cells on the lower tiers behaved similarly to the upper ones, i.e. no increase in load was associated with continuing lateral deflection.

The choice of the assumed failure plane for design now appears invalid for Pearl Street when the additional thickness of the softer glacio-marine clay is considered. The upper tiebacks were not long enough to prevent a mass movement of the clay. By assuming a flatter failure plane ($45^{o}$) it is apparent in Figure 10 that 15 to 30 percent of the upper four tieback anchors are located within the failure zone. Since the other side of the excavation did not experience similar magnitudes of deflection, the mass movement on Pearl Street is attributed to the thicker clay layer in the area where the ties are anchored.

The combined stress plot (Figure 8a) for soldier pile 48 shows higher than expected stress levels (approximately 30 ksi) above subgrade level. The mass movement apparently affects the piles differently at different levels as the piles adjust to the ground movement. Stresses in the lower portions of pile 48 are much less, and the magnitude of lateral movement is also less. Combined stresses in the High Street pile (67) show values peaking at less than 20 ksi (Figure 9b).

Of additional interest is the magnitude of lateral movement below subgrade, both at and between soldier piles. The inclinometers were installed to depths of 15 feet below the tips of the soldier piles. The Pearl Street side of the excavation experienced lateral movements below subgrade up to 1 1/4 inches as compared to 1/4 inch lateral movements below subgrade measured on the High Street side. The lateral movements below subgrade were uniform in each set of three inclinometers. This appears to indicate that the ground moves as a unit below subgrade and that the pile may not develop significant lateral load carrying capacity below subgrade. This also indicates that the soldier

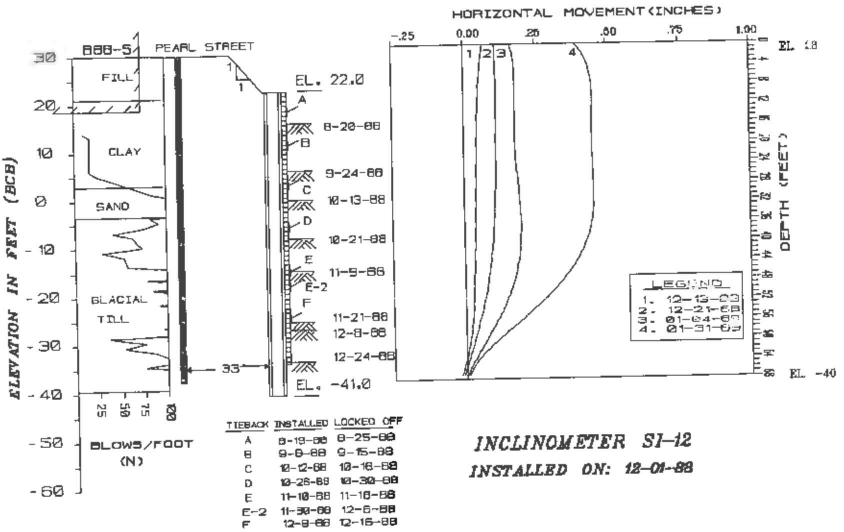



**FIGURE 11a:** LATERAL MOVEMENT OF PEARL ST. (33' OFFSET FROM EXCAVATION)

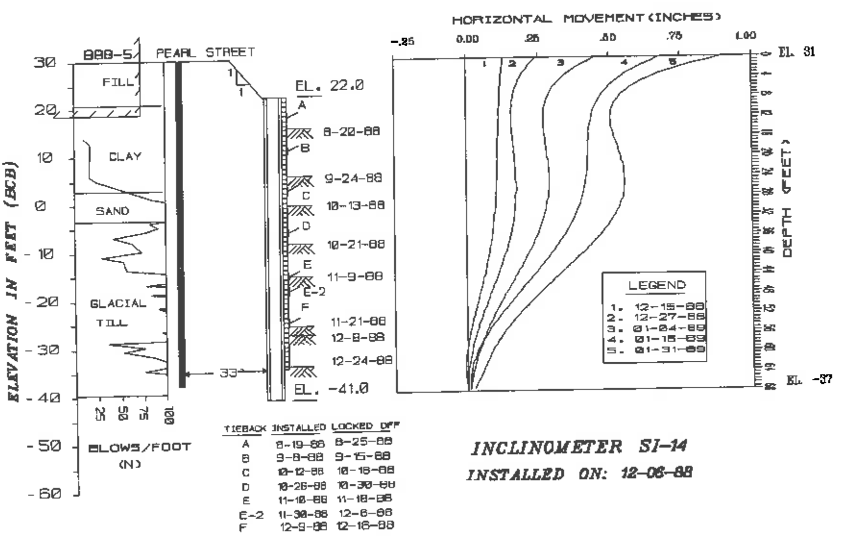



**FIGURE 11b:** LATERAL MOVEMENT OF PEARL ST. (33' OFFSET FROM EXCAVATION)

pile may function as a load distribution member (like a waler) which helps impart the tieback load to the soil.

One of the prime reasons for instrumenting the piles is to analyze the load distribution in the pile toes to verify design assumptions. The axial load distribution computed from the strain gages in the soldier piles is shown in Figures 8a and 8b along with the theoretical maximum axial load line derived by summing the vertical components of the tieback load with depth. The data clearly indicates that the axial load dissipates before the toe of the pile is reached.

Our experience is that soldier piles do not fail by the toe kicking out, except in cases of accidental overexcavation or soft ground at subgrade. The data confirms that load is transferred above subgrade from the pile to soil through friction and that the soldier pile does not provide significant lateral restraint below subgrade in a stiff soil.

## RECOMMENDATION FOR FUTURE STUDY

The opportunity still exists to improve design techniques for tiedback support systems. The following areas of research are suggested to the reader as topics for future research:

1. Instrumentation and analysis of soldier pile toe design for driven piles.
2. The presence or absence of composite action between the soldier pile and concrete backfill for drilled in piles.
3. The necessity for pile toe penetration in good soils.
4. Analysis of the design of soldier piles as load distribution members rather than moment resisting members.
5. Analysis of the effect of stiffness on the performance of drilled and driven pile excavation support systems.
6. Finite element studies of deep seated ground movements including those between piles and below subgrade which could lead to better understanding of support systems.
7. Analysis of the appropriate earth pressure diagrams for different soil conditions.
8. Stability considerations for tiedback soldier pile support system.

## SUMMARY AND CONCLUSION

Previous work (References 2, 3, and 4) has shown that minimum lateral displacement needed to achieve the

Active state is between 0.1% to 0.5% H depending on the soil type. The lateral support system on 125 High Street performed well within these bounds.

| | Lateral Movement/Depth of Excavation |
|---|---|
| Pearl Street (clay) | 0.38% |
| High Street | 0.10% |
| "Tangent Pile" | 0.08% |
| Purchase Street | 0.08% |

The merit of the observational approach in geotechnical engineering is proven again. What started as research on such items as soldier pile toe design and soil - pile interaction resulted in the acquisition of very timely and critical data which was needed to understand the movements that resulted.

While it can usually be said that more subsurface investigation is desirable on any given site, this project illustrates the need for proper boring coverage in areas where conditions are known to be changing. Borings are seldom taken in the area where tiebacks will achieve their anchorage. On deep excavations this oversight can be costly to the owner, engineer, and contractor. The character and thickness of the clay overlying the glacial till produced unanticipated results on the Pearl Street side. It is clear from the data presented that the assumed failure plane location for Pearl Street was incorrect for the soil conditions that existed under Pearl Street. Longer tiebacks at the upper levels would have reduced, but not eliminated, the lateral movement. The additional lower tiebacks and braces, while not adversely affecting the system, did not appreciably enhance the performance.

Clearly, design procedures which result in extremely long toe penetrations in good soils should be reviewed based on the data presented herein. Much of the axial load is lost above subgrade in friction and that load which reaches subgrade dissipates rapidly as shown. Approximately 50% of the load was dissipated on the High Street side, while about 15% of the load was taken out in friction on the Pearl Street side.

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## CONVERSION TO SI UNITS

| To Convert | To | Multiply by |
|---|---|---|
| in. | m | 0.0254 |
| ft. | m | 0.3048 |
| ton | kN | 8.896 |
| ton/sq. ft. | kPa | 95.76 |