Construction & Performance of Jet Grout Supported Soldier Pile Tremie Concrete Walls in Weak Clay

The use of a jet grouted kicker slab provided adequate lateral support for wall construction and simplified bottom excavation by providing a clean, stable invert surface.

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Like many cities in the United States, San Francisco has a combined sewer system that collects urban runoff and sewerage in the same pipe. This system can be a major source of pollution during wet weather. During even the slightest rainfall, untreated wastewater may overflow into San Francisco Bay or the Pacific Ocean, causing officials to close beaches and ban fishing. In the early 1970s, the city and county of San Francisco embarked on a major program to control wet weather sewage overflows. The program, known as the Clean Water Program, consists of a series of facilities, including underground transport/storage structures, pumping stations and treatment plants.

One portion of the Clean Water Program is the Islais Creek Facilities, which consists of a series of underground box structures and tunnels. Ground conditions adjacent to Islais Creek consist of miscellaneous fill overlying thick deposits of weak Bay Mud overlying bedrock belonging to the Franciscan Formation. The deep box structures for the Islais Creek Facilities were built within soldier pile reinforced concrete diaphragm walls (slurry walls) and were excavated through the fill and Bay Mud into the Franciscan bedrock. Jet grouting was used to strengthen the Bay Mud to enable it to provide lateral support at the base of the box structure excavation within the slurry walls.

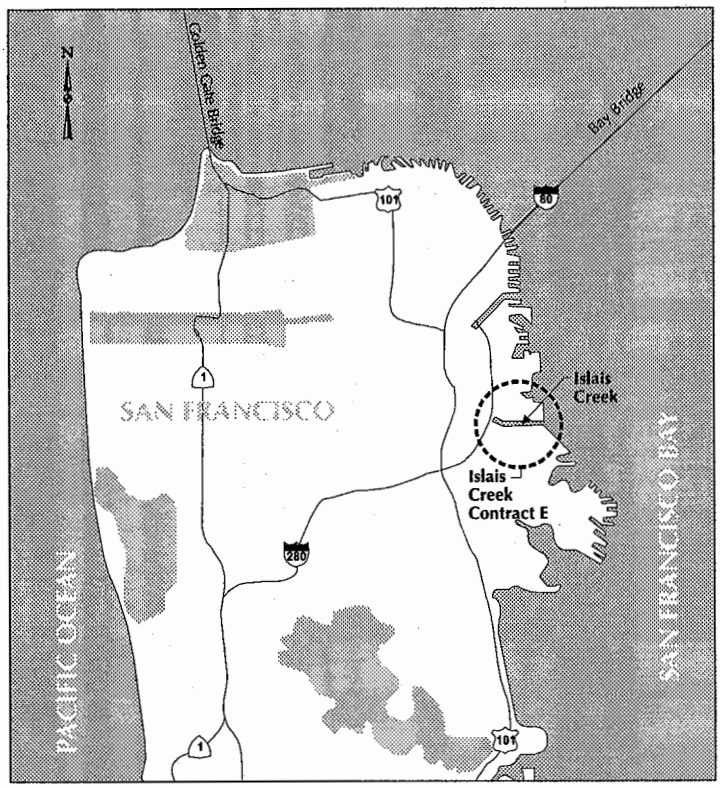

FIGURE 1. Islais Creek Contract E site location plan.

Project Description

Islais Creek Contract E is an 1,100-foot (340-meter) long component of the Islais Creek Facilities, which was constructed between January 1994 and April 1996. The project is located in the southeast corner of San Francisco, adjacent to the west and south sides of Islais Creek. A site location plan is shown in Figure 1.

The challenges that faced the designers and constructors of Contract E were numerous. First, Contract E had to cross beneath the existing Caltrain/Amtrak Railroad embankment and the viaducts supporting Caltrans' Interstate 280 highway (I-280). Second, the system had to be built within the weak Bay Mud that exists in the Islais Creek area. The Bay Mud in this area is recognized as some of the weakest in the entire Bay Area. In fact, the excavation support system for the nearby Davidson Box Sewer, which was built in the late 1970s, failed rather dramatically during construction.[1] Third, the system had to include a control structure to divert flow from the existing Three-Compartment Alemeny Sewer into the new facilities. As a result of these challenges, the design of Contract E included:

- Compressed air tunneling for the undercrossing of the railroad and highway viaducts;
- Deep concrete box structures built within and founded on thick steel and concrete diaphragm walls, laterally supported by jet grouted Bay Mud; and,
- Top-down construction to facilitate connection between the Alemeny Sewer and the new facility.

Contract E consists of the following main features:

- The 510-foot (155-meter) long, 12-foot (3.7-meter) diameter Davidson Tunnel;
- The 252-foot (77-meter) long, 10.5-foot (3.2-meter) diameter Undercrossing Tunnel;
- The East and West Transport/Storage (T/S) structures, which make up 360 feet (110 meters) of open-cut sewer boxes;
- The Davidson Diversion Sewer and the Napoleon Diversion Sewer, which are pile supported box sewers 6.5 feet (2.0 meters) and 5 feet (1.5 meters) in diameter, respectively; and,
- Several smaller sewers that are part of the connection to existing sewers;

A layout showing the various features of Islais Creek Contract E is shown in Figure 2. In that figure, the previously existing structures are indicated with dashed lines. The structures built under Contract E are shown in solid lines. Figure 2 also presents the location of the existing Caltrain/Amtrak Railroad and I-280, which bisect the project.

The West T/S is a 300-foot (91-meter) long, 12-foot (3.7-meter) to 45-foot (13.7-meter) wide, by 46-foot (14.0-meter) deep box sewer

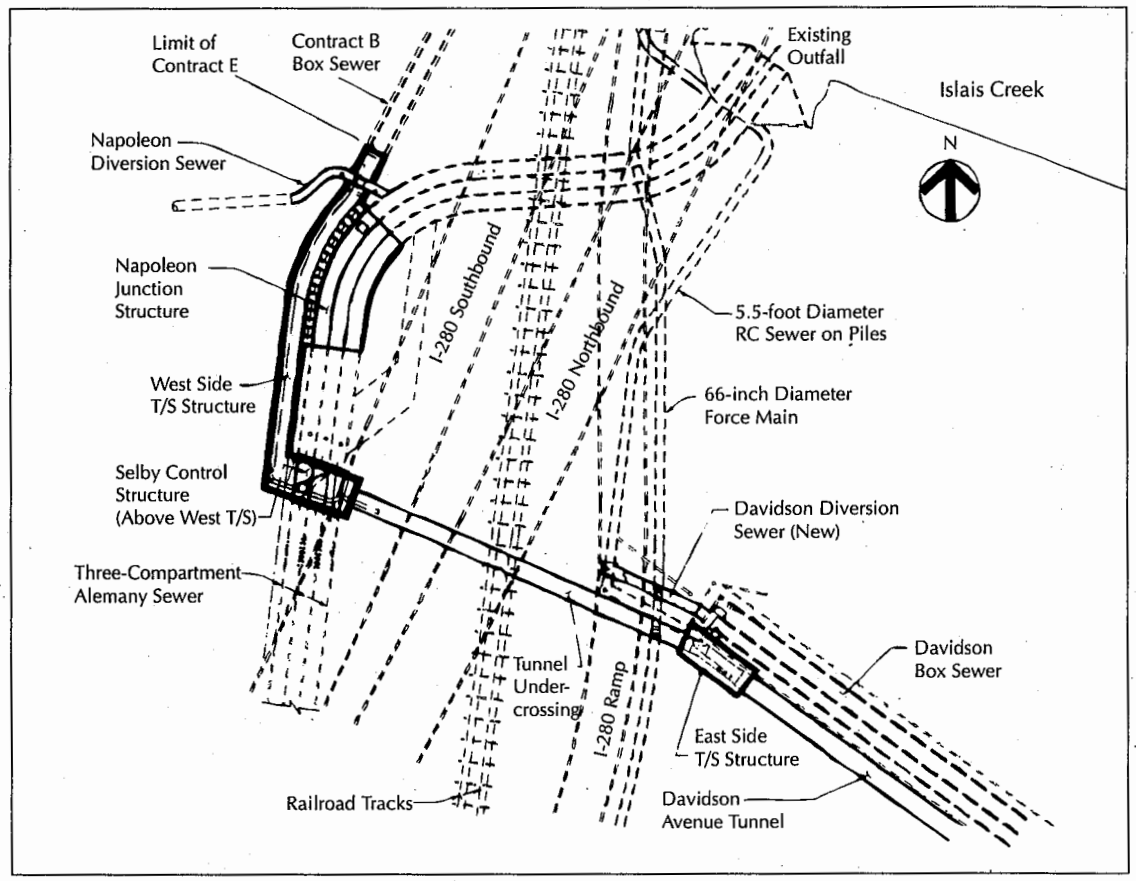

FIGURE 2. Layout plan of Islais Creek Contract E.

that crosses under and connects at two locations with the city's existing Alemeny Sewer. The West T/S abuts the Caltrain/Amtrak commuter railroad embankment and is adjacent to and beneath I-280 viaducts.

The East T/S is a 60-foot (18.3-meter) long, 25-foot (7.6-meter) wide, 45-foot (13.7-meter) deep box structure that connects to the existing Davidson Box Sewer, which runs the length of Davidson Avenue. In addition to providing transport between the new and existing box sewer structures, the East T/S box was designed to be used during construction as the tunnel access shaft, with tunnels being driven from it in two directions.

Prior to about 1890, the site area contained an embayment surrounded by tidal marshland. Islais Creek was a wide meandering waterway surrounded by hills rising 250 to 300 feet (76 to 92 meters) above the basin. A land reclamation operation took place in the 1930s, and most of the area was filled with 10 to 20 feet (3 to 6 meters) of sand, rock fill and miscellaneous waste. Geologic conditions at the site can be summarized by the following strata, listed in the order encountered from the ground surface downward during excavation of the slurry walls:

Fill. The fill is highly variable, consisting of brown, medium dense, poor to well graded sand and gravel, clayey sand, clayey silt and silty sand. Boulders and cobbles are frequently encountered. Fill was encountered to depths ranging from 7 to 10 feet (2 to 3 meters) below the ground surface within the excavation areas.

Bay Mud. The Bay Mud is a plastic silty clay or clayey silt, with trace organic fiber, shells and sand. It falls into the MH or CH category of the Unified Soil Classification System. It ranges in consistency from very soft to soft in the upper portions of the de-

posit, to stiff to medium stiff in the lower portions of the deposit. The upper portion, which has a thickness of 20 feet (6 meters), has a higher water content, liquid limit and plasticity index, and a lower unit weight than the lower portion. Geotechnical investigations and testing indicate that undrained shear strength varies considerably along the project alignment. However, the clay is normally consolidated to slightly overconsolidated, with overconsolidation ratios ranging from 1.05 to 1.62. Design shear strengths for the Bay Mud range with depth from 500 to 860 psf (24 to 40 kPa) across the West T/S, and from 375 to 650 psf (18 to 31 kPa) at the East T/S. Bay Mud was encountered from the bottom of the fill to depths ranging from 60 to 118 feet (18 to 36 meters) below the ground surface.

Old Bay Clay. The Old Bay Clay is a dark greenish to bluish gray, moderately plastic, stiff silty clay, with trace amounts of sand and shells. It falls into the CL or CH category of the Unified Soil Classification System. Design shear strengths for the Old Bay Clay ranged from 1,500 to 1,700 psf (72 to 82 kPa). Old Bay Clay was encountered underlying the Bay Mud along about one-third of the West T/S, at depths ranging from 60 to 85 feet (18 to 26 meters).

Bedrock. The bedrock across the entire site consists of highly weathered to moderately weathered, hard serpentinite belonging to the Franciscan Formation. The serpentinite is fractured in some locations, with infilling present in the joints. Rock Quality Designations (RQD) for samples ranged from 0 to 66 percent. Bedrock was encountered at depths ranging from 65 to 118 feet (20 to 36 meters) below the ground surface.

Groundwater is present at depths ranging from 5 to 7 feet (1.5 to 2.1 meters) below the ground surface across the site. It was not clearly determined during construction if the groundwater levels were affected by tides.

Design Requirements of the T/S Structures

Both the West T/S and East T/S structures are designed to connect flows from existing pile supported box sewers to the new Islais Creek Facilities. At the West T/S, the connections are made through a system of three bulkheads and two weirs located within the Alemeny Sewer at the Selby Control Structure and through eighteen weirs connecting to the Alemeny Sewer at the Napoleon Junction Structure. The connection of the West T/S to the existing sewers is illustrated in Figure 3 in both plan and profile. At the East T/S, the connections are made through three flow control chambers connected to the Davidson Box Sewer.

The existing sewers that connect to the new T/S structures have invert elevations within 12 to 20 feet (3.7 to 6.1 meters) of the ground surface. The inverts of the tunnels and T/S structures are approximately 45 feet (14 meters) below the ground surface. During rainfalls, the Alemeny and the Davidson Box sewers will fill to capacity and overflow through the weirs and flow control chambers into the West T/S and East T/S structures. Flows will be collected and stored within these structures and the remaining Islais Creek Facilities until capacity is available to treat the flow at the Southeast Treatment Plant.

Structural System. Both T/S structures were designed to be supported by, and constructed within, soldier pile tremie concrete (SPTC) walls. The T/S structures were also designed to be laterally supported at the base of excavation by a zone of jet grouted Bay Mud, installed prior to excavation.

SPTC walls are a variation of reinforced concrete diaphragm walls, or slurry walls. They are constructed by excavating a linear trench in the ground. As excavation proceeds, the trench is filled with slurry, which supports the walls of the trench. With conventional reinforced concrete diaphragm walls, reinforcing steel is placed in the completed trench excavation. Rebar placement is followed by the tremie concrete operation, in which concrete is placed from the bottom of the trench through tremie pipes while the slurry is displaced by the rising concrete. An SPTC wall is unique in that the reinforcing steel is replaced by wide-flange steel beams, or soldier piles. Figure 4 shows the simplified construction sequence for the SPTC walls.

Jet grouting is a soil improvement technique that utilizes fluids jetted at very high pressure

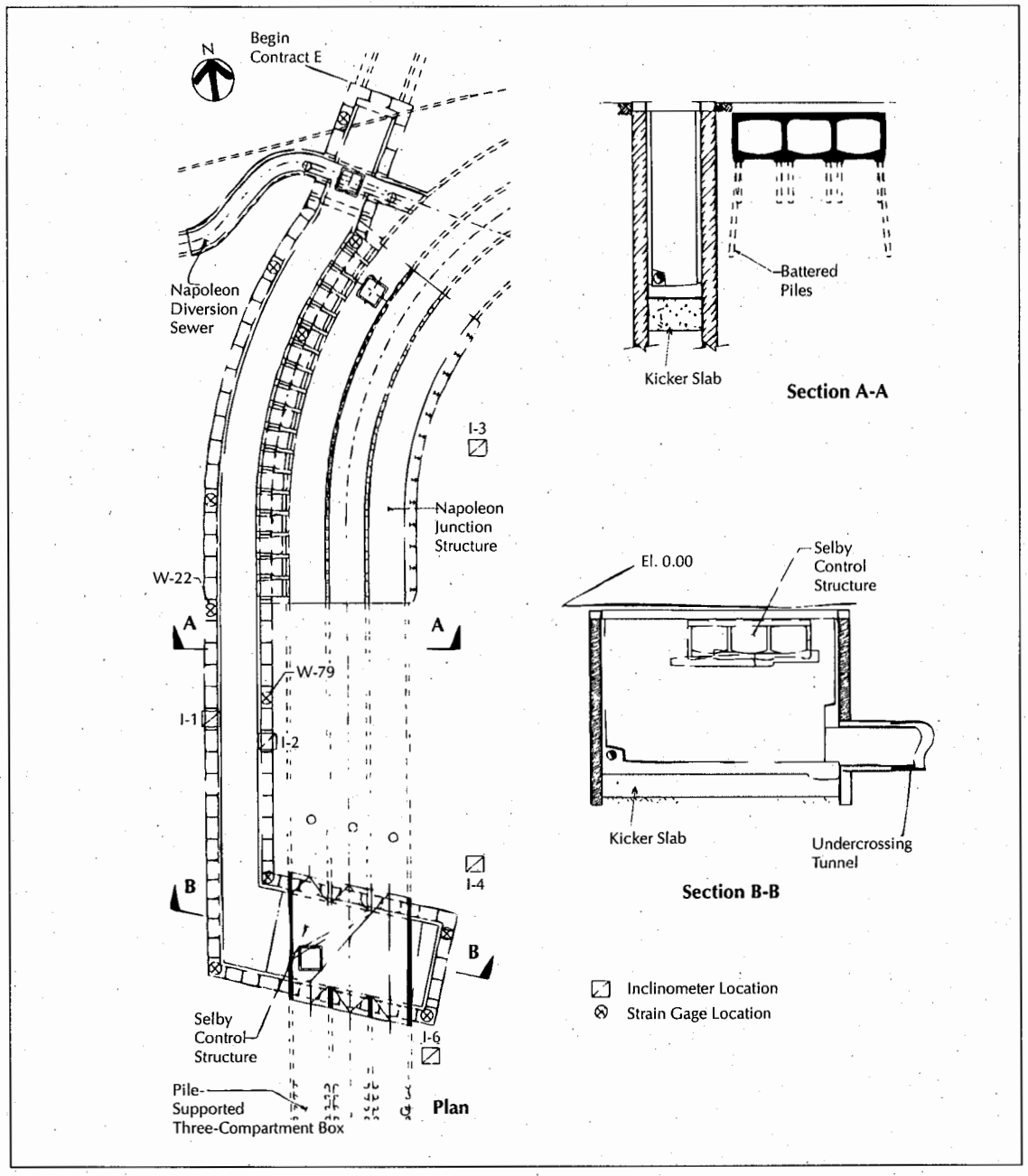

FIGURE 3. West T/S plan and sections.

to fracture, partially replace and simultaneously mix soil in-situ with a stabilizing agent. With weak clays, jet grouting has the ability to increase the soil's in-situ shear strength tenfold. On this project, after completion of the SPTC walls and before excavation began, a thick zone of jet grouted Bay Mud was created between the SPTC walls at the base of the planned excavation. The purpose of this zone of jet grouted Bay Mud, referred to as the *jet grout kicker slab*, was to provide lateral support of the SPTC walls during excavation. The location of the jet grout kicker slab relative to the SPTC walls is illustrated in Figure 3.

Vertical Support. The Islais Creek Facilities are designed to be empty during the dry season

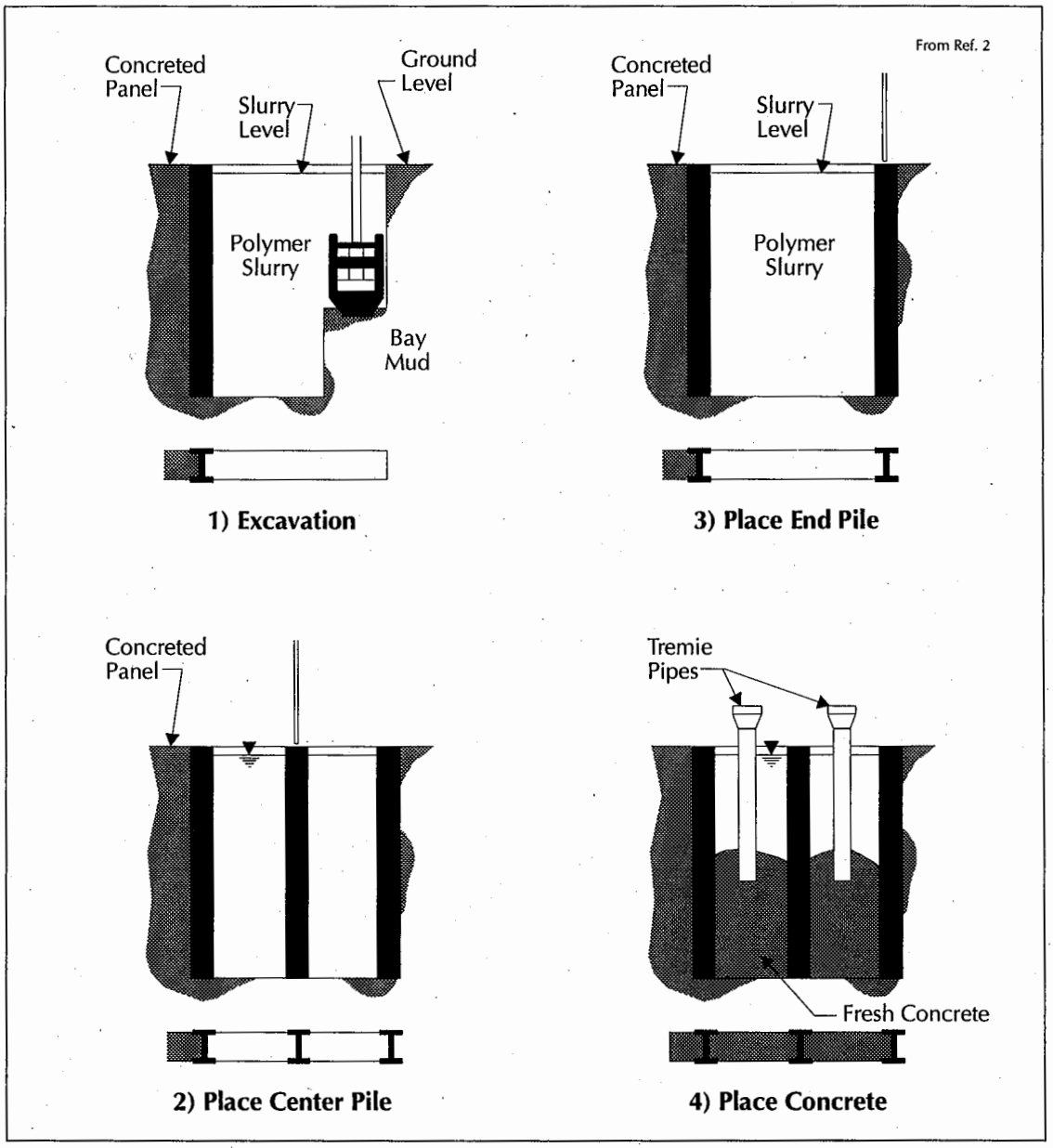

FIGURE 4. Simplified SPTC wall construction sequence.

and to be full during periods of rainfall. Because of these conditions, the design of the structures had to accommodate large downward vertical loads for the full condition and significant uplift pressures for all operating conditions.

To provide the necessary vertical support of the structures and permanent lateral support of the surrounding weak Bay Mud, both the West T/S and East T/S were designed to be constructed within SPTC walls. The West T/S design required 40-inch (1,000-millimeter) thick SPTC walls. The vertical and lateral support is provided by 40-inch (1,000-millimeter) deep steel beams located at a maximum spacing of 6.5-foot (2.0-meter) centers, placed within 4 ksi (28 mPa) concrete walls. The East T/S design required 36-inch (900-millimeter) thick SPTC walls, with 36-inch (900-millimeter) deep steel beams, located at a maximum spacing of 6-foot

(1.8-meter) centers. The design of the SPTC walls for both the West T/S and the East T/S required that the steel beams be socketed a minimum of 5 feet (1.5 meters) into bedrock.

Shear studs were required to ensure an adequate bond between the steel beams and the concrete between them. The shear studs consisted of 4-inch (100-millimeter) long, 0.5-inch (13-millimeter) diameter Nelson studs, which were required to be included on the web of each soldier pile on both the East T/S and West T/S structures.

Jet Grout Kicker Slab. An analysis of the strength of the Bay Mud stratum indicated that it would not provide sufficient passive resistance to lateral earth pressures during the excavation of the West T/S and East T/S structures. Thus, the design specified that a zone of jet grouted Bay Mud be installed below the invert elevation between the SPTC walls prior to the excavation of the T/S structures. This jet grout kicker slab was designed to reduce lateral movements and the bracing requirements for these relatively deep excavations. An added benefit of the kicker slab was that it formed a working platform on which to construct the base slabs of both T/S structures.

The design assumed that the soldier piles within the SPTC walls would carry the entire lateral earth pressure load. The jet grout kicker slab was specified with a performance-based specification, with the requirement that the contractor provide sufficient jet grout improved soil to resist a maximum horizontal reaction of 100 kips/ft. (1,460 kN/m) of running wall. In addition to this design requirement, the maximum allowable compressive stress in the jet grout kicker slab was specified to be 25 percent of the contractor's anticipated design strength. This was meant to provide for a factor of safety of 4.0.

The soldier beams within the SPTC walls are connected to the T/S structure floor slabs and 12-inch (300-millimeter) thick finish walls with steel shear studs welded to the face of each soldier pile. The base slab and finish walls are structurally connected by standard reinforcing steel details. The roof of each structure consists of a 24-inch (600-millimeter) thick reinforced concrete slab that was connected as a cap beam (poured over and around each soldier pile) onto the top of the SPTC wall and interior finish wall.

Instrumentation. Contract documents specified instrumentation to monitor the SPTC wall performance during the construction of the T/S structures. Instruments used on the project included:

- Surface settlement points located adjacent to the excavation and on the nearby I-280 viaduct supports;
- Lateral deformation points located at the top of the SPTC walls and on the I-280 supports;
- Inclinometers cast within the SPTC walls and socketed into rock; and,
- Vibrating wire strain gages.

Figure 3 presents a generalized plan noting instrumentation locations for the West T/S. Vibrating wire strain gages were installed in pairs, opposite each other on the inside flanges of selected beams, at the mid-span, invert and embedment locations, at depths as shown schematically in Figure 5. The strain gages and inclinometers were installed on selected soldier piles in both the West T/S and East T/S structures and initialized prior to placing tremie concrete within the beams.

Construction of the SPTC Walls

A total of about 78,500 square feet of SPTC walls had to be installed project-wide on two different sides of an existing railroad embankment and through existing pile supported reinforced concrete sewer boxes located below the ground surface (as shown in Figure 2). To accomplish construction, the contractor utilized a rather complex sequencing scheme, which included multiple mobilizations of excavating and servicing cranes, and a large slurry distribution/handling system.

SPTC wall installation on the East T/S commenced on June 6, 1994, and continued until 7 of the 10 panels were completed, whereupon the SPTC wall operation moved to the West T/S. The partially completed East T/S walls were then used, along with sheet piles and the existing Davidson Sewer, to support the ground while construction of the new Davidson Diversion Sewer and demolition and re-

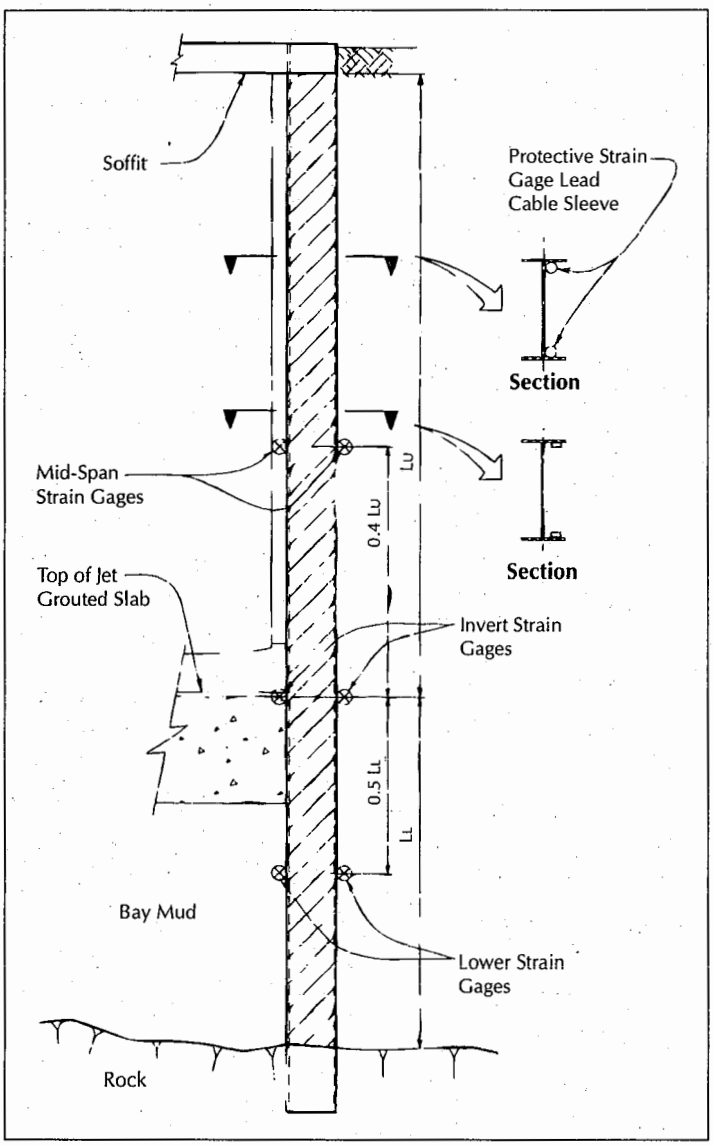

FIGURE 5. Schematic view of a pile with strain gages.

moval of the existing Davidson Sewer took place. When the Diversion Sewer was completed, the SPTC wall operations moved back to the East T/S, completing the wall installation there on October 19, 1994.

Construction of the SPTC walls for the West T/S commenced on June 25, 1994. Generally throughout the West T/S SPTC wall construction, two excavation cranes, one service crane and one all-terrain crane were used during the operations. Like the East T/S, the West T/S SPTC wall excavations also had to cross through an existing box sewer. However, the West T/S sewer crossing was much more challenging since this section of the box (referred to as the Selby Control Structure) is the new overflow connection into the lower tunnel system. The West T/S SPTC walls were completed on October 10, 1994.

Excavation of the 5-foot (1.5-meter) rock socket was performed using drop chisels — both star shaped and rectangular — combined with an 18-ton toothed clam. Panels ranged in length from 9.5 feet (3 meters) to 21 feet (6.4 meters) long. Typically, throughout the project, slurry wall excavation rates averaged 78 square feet per hour in the clay/soils and 14.3 square feet per hour in the rock. Overall, with two excavator cranes working the West T/S (one of which was run around the clock for some of the operation), the contractor averaged 3.2 panels per week during the course of the work. For the East T/S, with production interrupted as described above, the contractor averaged 1.7 completed panels per week. These rates do include typical down time associated with the maintenance/breakdown of equipment, traffic delays, break periods and survey time; however, they do not include mobilizations.

Use of Bentonite & Polymer Slurries. Slurry wall panels throughout the project were excavated using both bentonite and polymer slurries to support the excavation. During the initial excavation within the fill, bentonite slurry was used to support the excavation. As the excavation proceeded into and through the clays, polymer slurry was introduced into the trench for the remainder of the excavation. Although the action of the bucket tended to mix both the polymer and bentonite slurries together during the deeper portion of the excavation, this system was successful in forming a cake on the inside face of the sandy fill soils to

control loss of ground into the excavation within the fill. Typically, by the time the excavation had reached depths greater than 60 feet (18 meters), the percentage of bentonite in the slurry was negligible.

Polymer slurries allow for sand and other larger solid particles to settle out of the slurry at a much quicker rate than bentonite slurries. Throughout the Contract E slurry wall operations, this behavior was observed to be true. Typical panel excavation was completed at the end of a day, or by early evening. Prior to covering the excavation for the night, the depth and cleanliness of the bottom were checked using a heavy steel weight affixed to a tape. A clean rock surface was confirmed by bouncing the steel weight off the bottom of the trench across the entire length of the excavation (a process referred to as *sounding the hole*). At the beginning of the next day's shift, the panel was cleaned out with a special smooth-edged "clean-out" bucket and sounded again by an inspector who confirmed depths from the night before and checked again for a clean bottom. Typically about 2 feet (600 millimeters) of material was removed from the bottom of the panel on the morning of concrete placement. During the clean-out of each panel, a sample of slurry was taken from within the clean-out bucket and tested for sand content, viscosity and density. Sand contents in the slurry were well below specified limits during the entire project. Consequently, conventional desanding was never performed.

Soldier Piles & Welding. Soldier piles on the project were large and had to undergo several field modifications prior to being set into the slurry wall excavation. Beam sizes ranged from W36X210 to W40X324, and ranged in length from 80 to 128 feet (24 to 39 meters) long. Welding of shear studs within the webs at socket level and mid-box level, and sand blasting and epoxy coating the top 30 feet (9.1 meters) of outside flanges were required of every beam prior to installation. In addition, the installation of instrumentation was required on 18 beams prior to placement. For primary panels, in which the web area on the outside would later be excavated, the outside web shear studs were transposed to the inside face of the next beam (on the secondary panel). Interior beams, and all beams installed within a secondary panel, had shear studs welded to both sides of the web.

The varying size/location requirements and the overall immensity of the beams required a rather detailed organizational effort by the contractor in order to place shear studs, epoxy coating and instrumentation in the right location on the beams. Many beams required splicing to achieve the required installation lengths. The specifications required that any splices in the SPTC wall piles be full-penetration butt welds. Full-penetration butt welding of beams of this size required about six hours of welding time per weld. All splices were X-ray inspected prior to beam placement.

Figure 6 shows workers welding shear studs to the socket portion of a beam. Figure 7 was taken during beam placement beneath the I-280 viaducts at the West T/S.

Placement of Piles & Tremie Concrete. Once the panel had been excavated and cleaned, the soldier beams were inserted into the panel. For the larger primary panels, typically three or four beams were installed — two being end beams, which served initially as "end-stops" during the concrete placement of the primary panels, and then as excavation guides during secondary panel excavation. Typically, one or two piles were placed within secondary panels. At several low head-room panel locations, beams had to be welded as they were set into the trench. Due to the addition of two welds to the soldier piles, this activity added about 12 hours to the cycle on panels beneath the I-280 viaducts.

Beams were typically measured on the ground surface, set into the hole, aligned into position and dropped the final 12 feet (3.7 meters) into the rock socket. This method served as a good way to confirm the bearing quality on the rock surface below. After the beam had been set, it was secured into the top of the guide wall with steel channels and anchored by rock dowels in the guide wall. The outside webs of the end beams of the primary panels were filled with styrofoam and plywood block outs. Steel straps were used to hold the block outs in place.

Concrete was placed between the beams using two or three tremie pipes (depending on the panel size), which were inserted into the

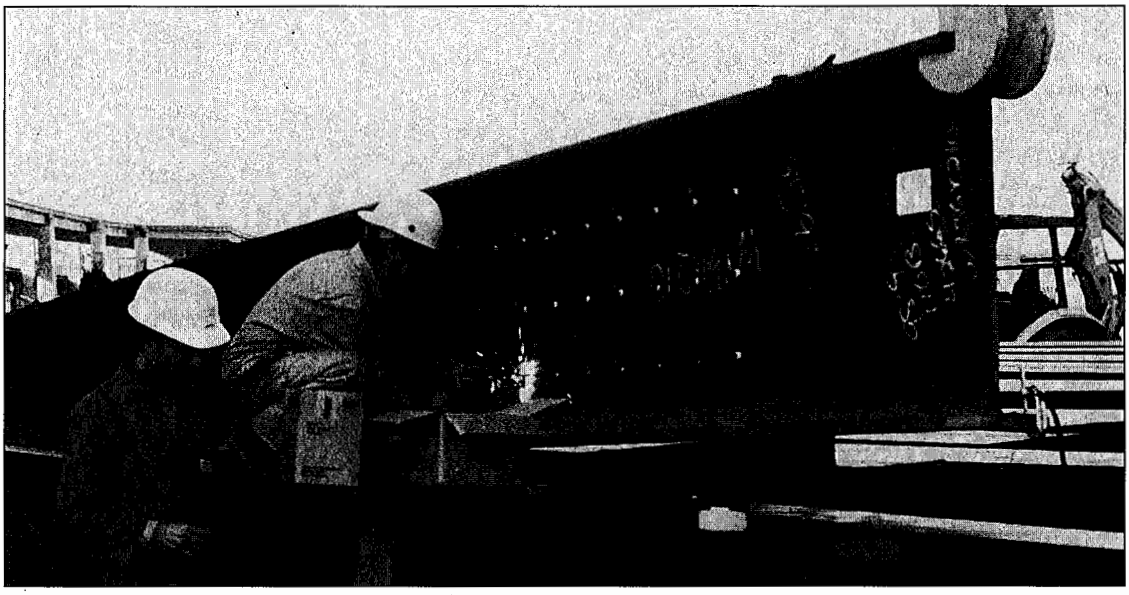

FIGURE 6. Welding shear studs to the socket portion of a beam.

panel between the beams and lowered to a maximum of 3 feet (1 meter) off the bottom of the trench (see Figure 8). At several locations, the tremie pipes were serviced by three cranes (excavating, service and all-terrain cranes) due to the tight spacing of piles and rapid rise of concrete, which was installed from two or three ready-mix trucks at each panel. As concrete was placed, slurry was pumped off the top of each panel and stored for future use.

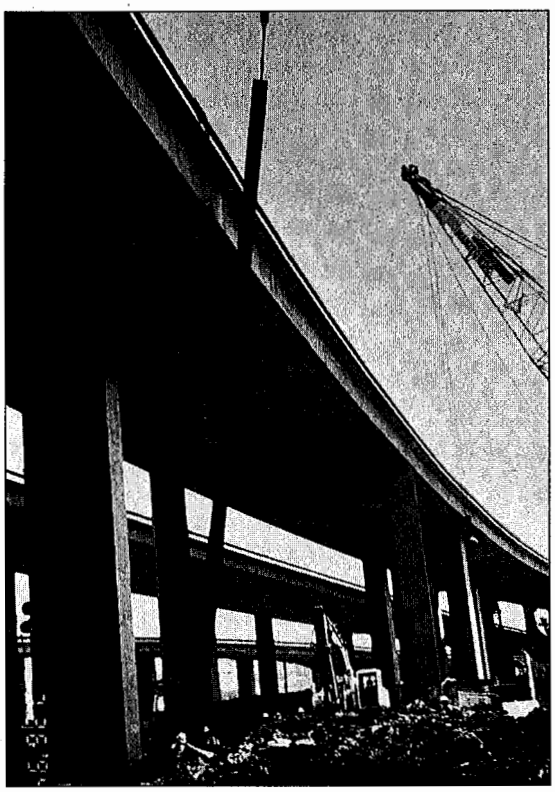

FIGURE 7. Beam placement beneath the I-280 viaducts at the West T/S.

Axial Bending of End Piles During Concrete Placement. During concrete placement of the first several panels, the tops of the end soldier piles were observed to break out of their restraints in the guide walls. In some cases, this deformation was observed to occur abruptly, while in others it occurred slowly over the duration of the concreting operation. It was suspected that this situation was occurring because the tremie concrete and soldier pile were compressing against the Bay Mud in an unbalanced loading condition.

In an effort to both determine what was happening to the soldier pile and to see if adding steel would improve the condition, one panel was constructed with inclinometers installed in the webs of the end soldier piles. In addition, one of the end soldier piles had a 60-foot (18.5-meter) long HP14X117 "strong-back" section welded to the inside center of its web. Inclinometers installed on the soldier piles were read prior to tremie concrete placement and

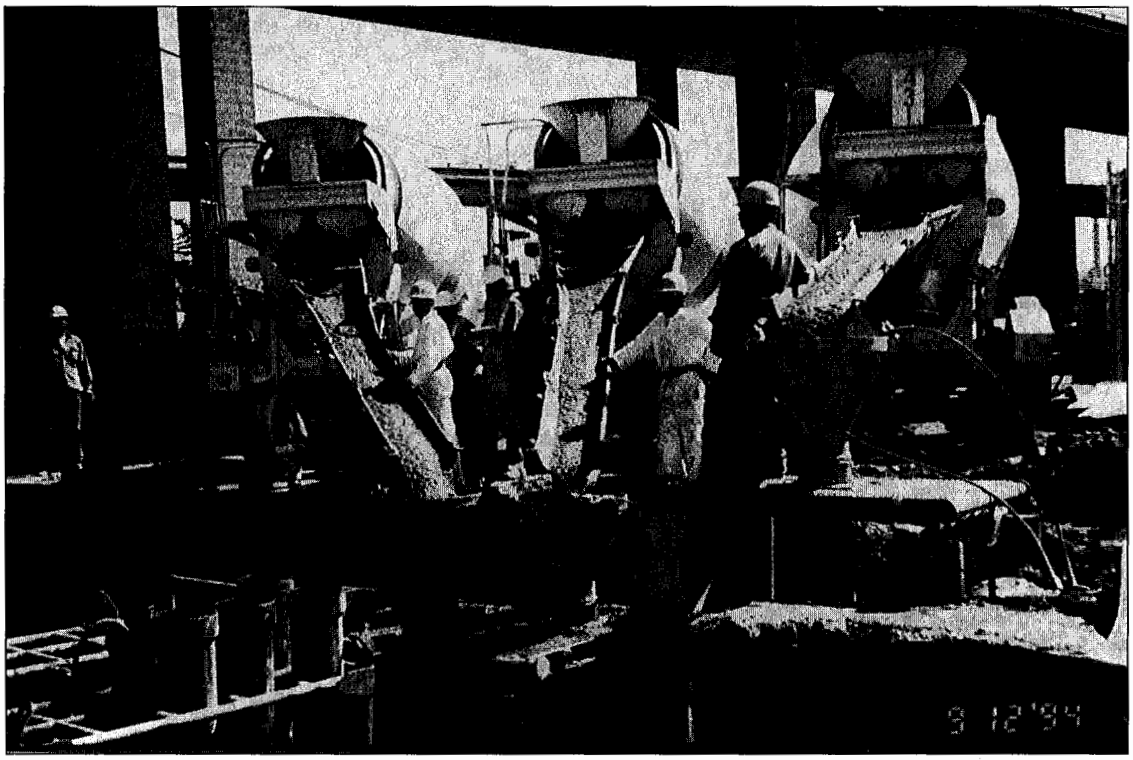

FIGURE 8. A three-tremie pour between beams.

again after the tremie concrete operation had been completed.

The results of these inclinometer surveys are summarized in Figure 9, which shows a plan view of the panel and notes the location and orientation of each soldier pile and associated inclinometer. The plot of inclinometer E-5 (the pile without the strong-back) and the plot of inclinometer E-3 (the pile with the strong-back) are both shown before and after the placement of tremie concrete. Note that both plots are absolute position readings. The results reveal that during concrete placement, the piles did experience bending about the weak axis due to the unbalanced load between the wet concrete and the weak Bay Mud.

Although the magnitude of bending was approximately halved for the pile with the strong-back installed, it was determined that this improvement did not justify the extra costs associated with changing the design. Although this phenomenon was not anticipated by the project team, calculations based on the displacements measured with the inclinometers were used to determine that the initial stresses resulting from this bending would be tolerable.

Top-Down Construction at the Selby Control Structure. Construction of the Selby Control Structure (see Figure 3) required that a section of the existing pile-supported Alemeny Sewer be removed and replaced with a weir structure for connection to the West T/S. The West T/S in this area was constructed using the top-down method after the Selby Control Structure was completed. The completed Selby Control Structure, supported vertically on the SPTC walls, provided the top level of bracing for the underlying excavation of the West T/S. Construction of the SPTC walls, the jet grout kicker slab and the new weir structure had to be performed while flow through the three compartments was maintained.

At the start of construction in this area, a system of weirs and pumps were installed to divert flow in the Alemeny Sewer. The SPTC walls on either side of the Alemeny Sewer were installed next. To accomplish that installation, the existing section of the Alemeny Sewer was demolished down to its invert slab. Then, the invert slab was removed in 46-inch (1,170-

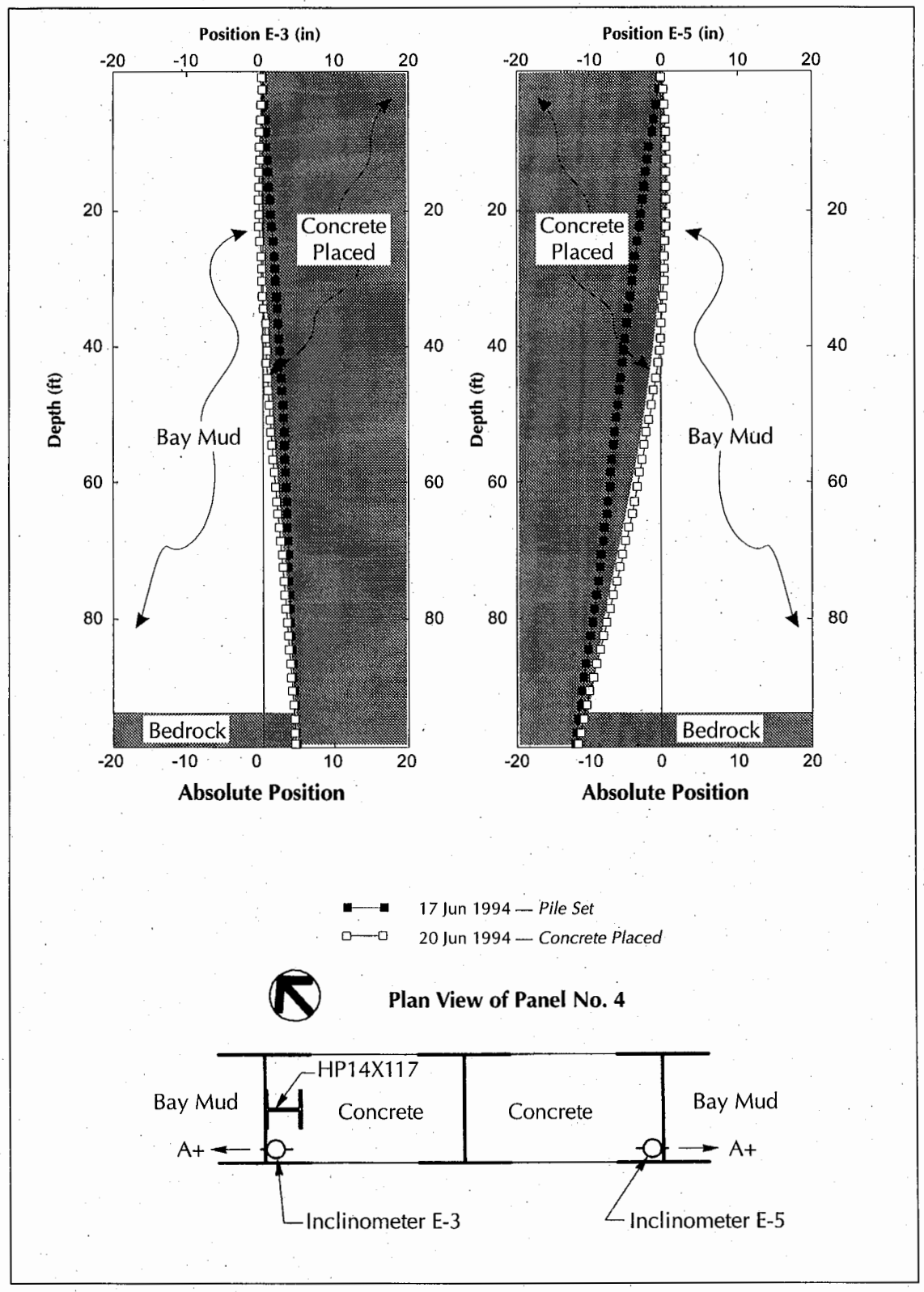

FIGURE 9. Inclinometer data showing axial bending.

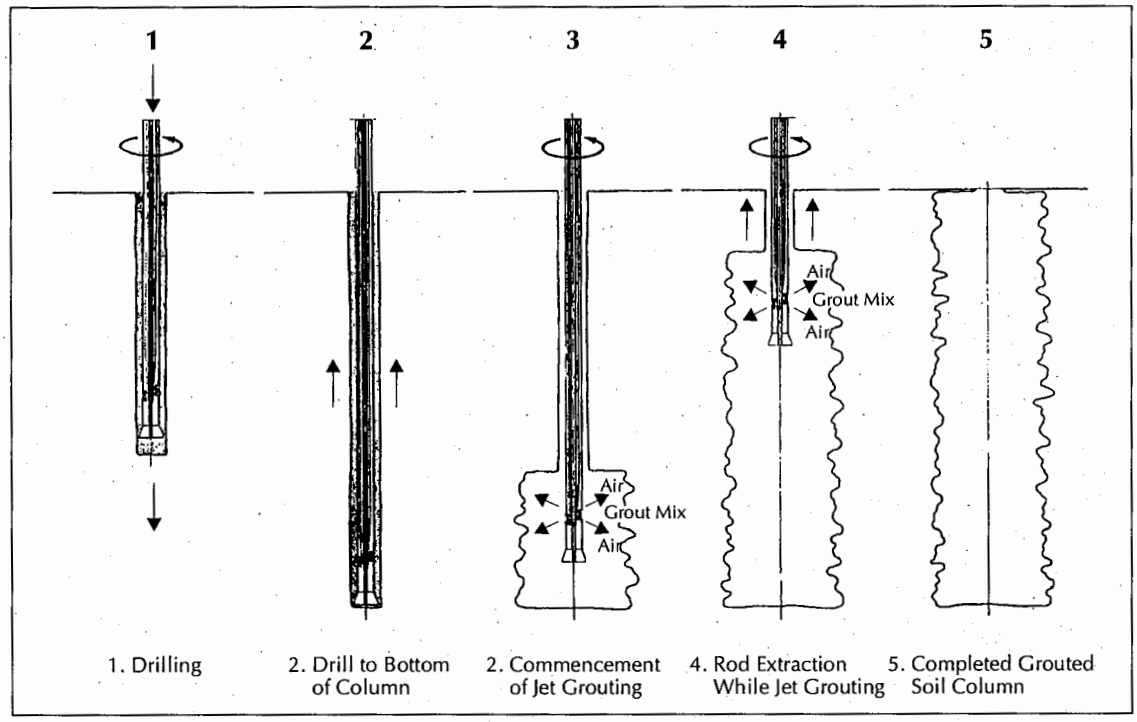

FIGURE 10. Jet grouting procedures for the double-fluid method.

millimeter) wide "slots" that corresponded to the width of the SPTC wall excavation. From within these slots, a total of forty-two 14- to 18-inch (355- to 460-millimeter) diameter, more than 100-foot (30-meter) long redwood timber piles that supported the Alemeny Sewer were extracted using a vibratory hammer suspended from the excavator crane.

Excavation of the SPTC wall panels within the Selby Control Structure was then performed around the clock from between guide walls set on the invert of the existing sewer on either side of the 46-inch (1,170-millimeter) slot. The effect of these guide walls was to lift the top of slurry level approximately 4 feet (1.2 meters) off the bottom of the invert slab. However, even with this 4-foot rise, the excavation of the panels within the Selby Control Structure was performed with slurry levels approximately 5 feet (1.5 meters) below the area groundwater table. In the event of instability or caving of the slurry panels during excavation, the contractor had the ability to flood the area above the tops of the guide walls to groundwater level. Although flooding the area would have stabilized the slurry panel excavations, it would have required excavation to proceed in the blind. As it turned out, panel instability did not occur during excavation and flooding was never required.

Construction of the Jet Grout Kicker Slab

Shortly after the award of the contract, the contractor submitted a Construction Incentive Change Proposal (CICP) to use jet grouting to pretreat the weak Bay Mud along the two tunnel alignments of the project, and to mine the tunnels open-face through jet grouted Bay Mud without using compressed air. This proposal was accepted by the city, contingent on the successful performance of a jet grout test program. The city and the contractor agreed that one jet grout test program would be acceptable for both jet grouting operations, provided that the design criteria were similar.

Jet Grout Test Program. The jet grout test program was used to show that jet grouted columns could be installed to create a continuous mass of treated soils in the weak clay formation, that the design strength of 120 psi (830 kPa) for the grouted soils could be achieved,

FIGURE 11. A view of the test excavation.

and that heave could be controlled. The test consisted of 12 jet grouted columns installed using the Rodinjet technique and equipment. Six of the test columns were installed using the single-fluid method of cement grout (Rodinjet 1), while the other six test columns were installed using the double-fluid method of cement grout and air (Rodinjet 2). Figure 10 on the previous page illustrates jet grouting procedures of the double-fluid method.

An automatic batching plant was set up and calibrated to obtain a constant water/cement ratio of the grout of 0.83 by weight at a production rate of up to 700 cubic feet per hour (20 cubic meters per hour). Type I/II low-alkali cement was used in the grout mix, which had a Marsh viscosity of 32 seconds, and a unit weight of 98 pounds per cubic foot (15 kN/m^3). The unconfined compressive strength of the cement grout averaged 2,600 psi (18 mPa) at 7 days and 4,800 psi (33 mPa) at 28 days.[3]

The 12 jet grout test columns were installed in a period of two days. Several of the grout injection parameters were varied during the course of the testing program. Grout injection pressures were either 4,400 or 5,900 psi (30.4 to 40.7 mPa); grout injection rates were either 2.0 or 2.35 liters per second; and grout volumes were either 80 or 90 liters per foot of column for Rodinjet 1 and 235, 230 or 200 liters per foot of column for Rodinjet 2.

TABLE 1
Jet Grouting Parameters

Grout Injection Pressure	5,850 psi (40 mPa)
Air Injection Pressure	32 psi (900 kPa)
Grout Volume Injected	8.28 ft^3/ft (770 l/m)
Grout Flow During Grouting	37 gallon/min (2.35 l/sec)
Bulk Cement/ Unit Grouted Soil	25 lb/ft^3 (400 kg/m^3)
Rotation Rate of Rods	10 rpm
Withdrawal Rate of Rods	0.5 ft/min (12 cm/min)
Design Replacement Ratio	38%

Following the installation of the test columns, a 30-foot (9-meter) square sheet pile supported test pit was excavated to expose the top 4 feet (1.2 meters) of each of the columns. It was found that the Rodinjet 1 columns had diameters ranging from 2.6 to 3.3 feet (0.8 to 1 meter). The Rodinjet 2 columns had diameters ranging from 7.9 to 9.5 feet (2.4 to 2.9 meters). Core samples were obtained from the test columns and unconfined compressive strength tests were performed. The strength of the Rodinjet 1 columns averaged 500 psi (3.5 mPa), while the strength of the Rodinjet 2 columns averaged 200 psi (1.4 mPa). A plan view of the test excavation in shown is Figure 11.

Production Jet Grouting. On the basis of the results of the test and the success of jet grouting already performed on the Davidson Tunnel alignment, the contractor elected to use the same plant, equipment and drilling methods to install the columns of the kicker slab. The pa-

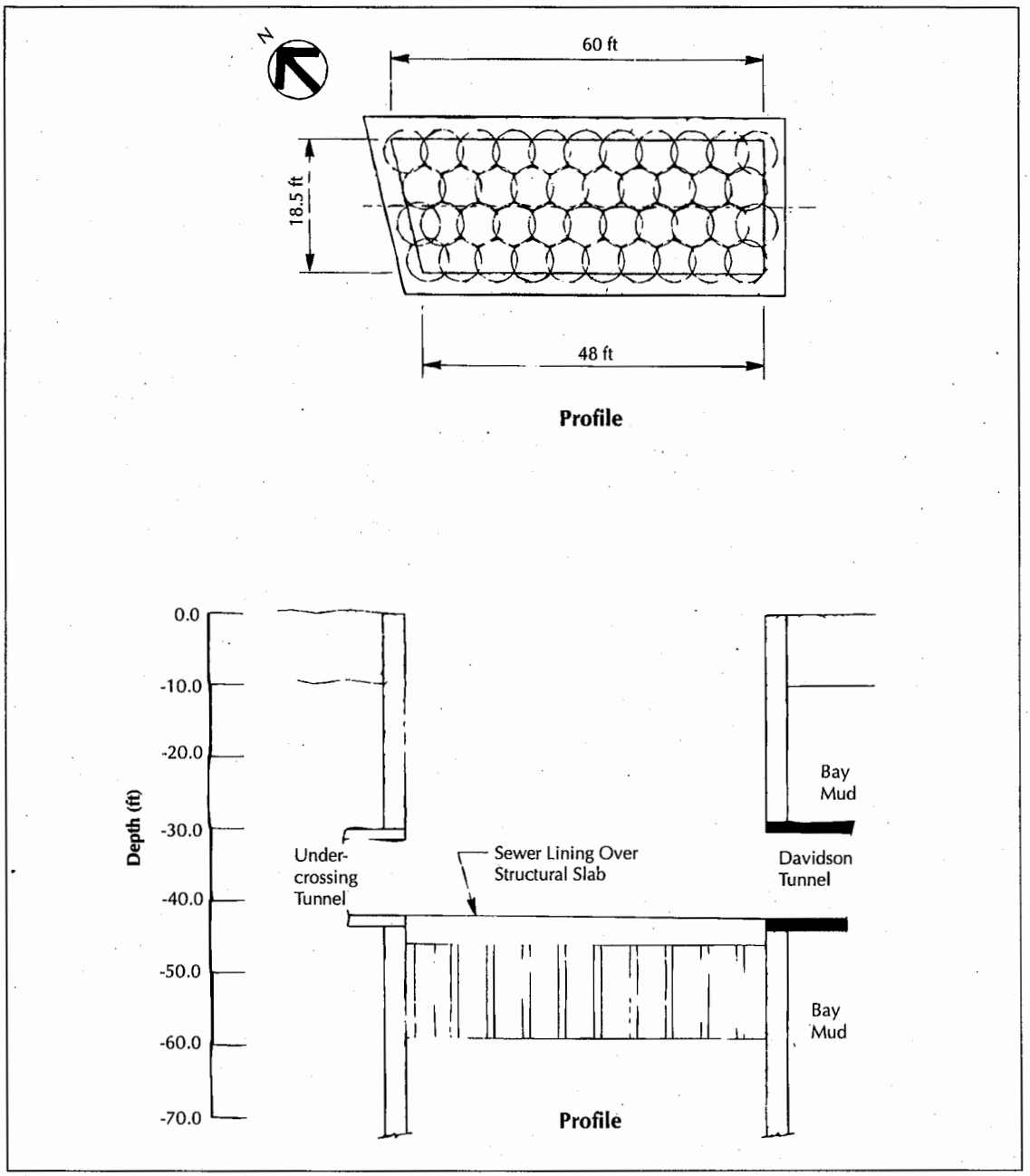

FIGURE 12. The East T/S kicker slab plan and profile.

rameters selected for jet grouting project-wide are listed in Table 1. Throughout the kicker slab operations, the water/cement ratio was held at a constant 0.85.

The jet grout columns for the East T/S structure are shown in plan and profile in Figure 12. The layout for the West T/S used the same spacing in a similar row pattern. As illustrated in Figure 12, the columns were laid out in rows, split-spaced on 5-foot (1.5-meter) centers, with spacing of 5 feet (1.5 meters) between jet grout columns in each row. To develop the required capacity of 100 tons per linear foot of wall, the height of the jet grout columns was chosen as 13 feet (4.0 meters). Using an average jet grout strength of 160 psi (1,100 kPa), the load carry-

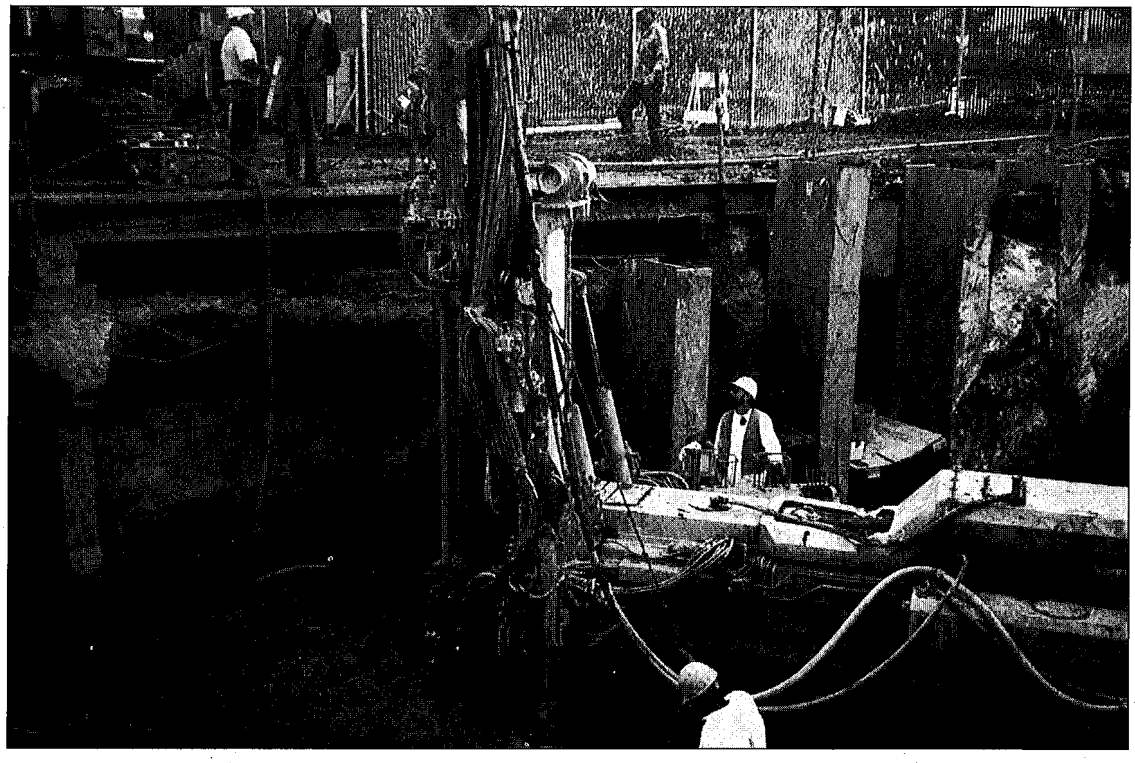

FIGURE 13. Jet grouting operations from within the three-compartment box.

ing capacity of the kicker slab was 300 kips per linear foot (4,400 kN per meter) of wall. Although this resulted in a factor of safety of 3 as opposed to the required value of 4, the city accepted the reduced factor of safety due to the performance to date of the jet grouting operations.

Like the slurry wall operations, completion of the jet grouting operations around the existing structures required several mobilizations between the East and West T/S structures. On the initial mobilization to the West T/S, all columns located alongside the three-compartment box were installed. Following the diversion, demolition and SPTC wall construction for the Selby Control Structure, the jet grouting operation was remobilized to complete the columns beneath the existing sewer.

The jet grout columns for the kicker slabs were installed in a systematic manner and were designed to allow for at least one day of set time on a column before an adjacent, overlapping column would be installed. The East T/S structure had a total of 42 columns installed between the SPTC walls. The West T/S structure was more complex because it was narrower, and because of the requirement for grouting to occur from within the three-compartment box sewer at the Selby Control Structure. The West T/S structure had a total of 194 jet grout columns. At both structures, jet grouting required drilling to a depth of 60 feet (18 meters). The average production rate during the installation of the kicker slab jet grouting was 10 columns per shift. Figure 13 shows jet grouting operations underway from within the three-compartment box structure.

Spoils from the jet grouting operations were semi-liquid and tended to flow out and away from above the jet grout column at the ground surface. In order to make the spoils acceptable for landfill disposal, they had to be dried prior to transport. The contractor established a drying area on an unused portion of the site. During the course of the jet grouting shift, spoils were moved from adjacent to the operations into the drying area. When excavation for other structures on the project was occurring, the jet grout spoils would be mixed with the excavated materials to reduce the moisture content

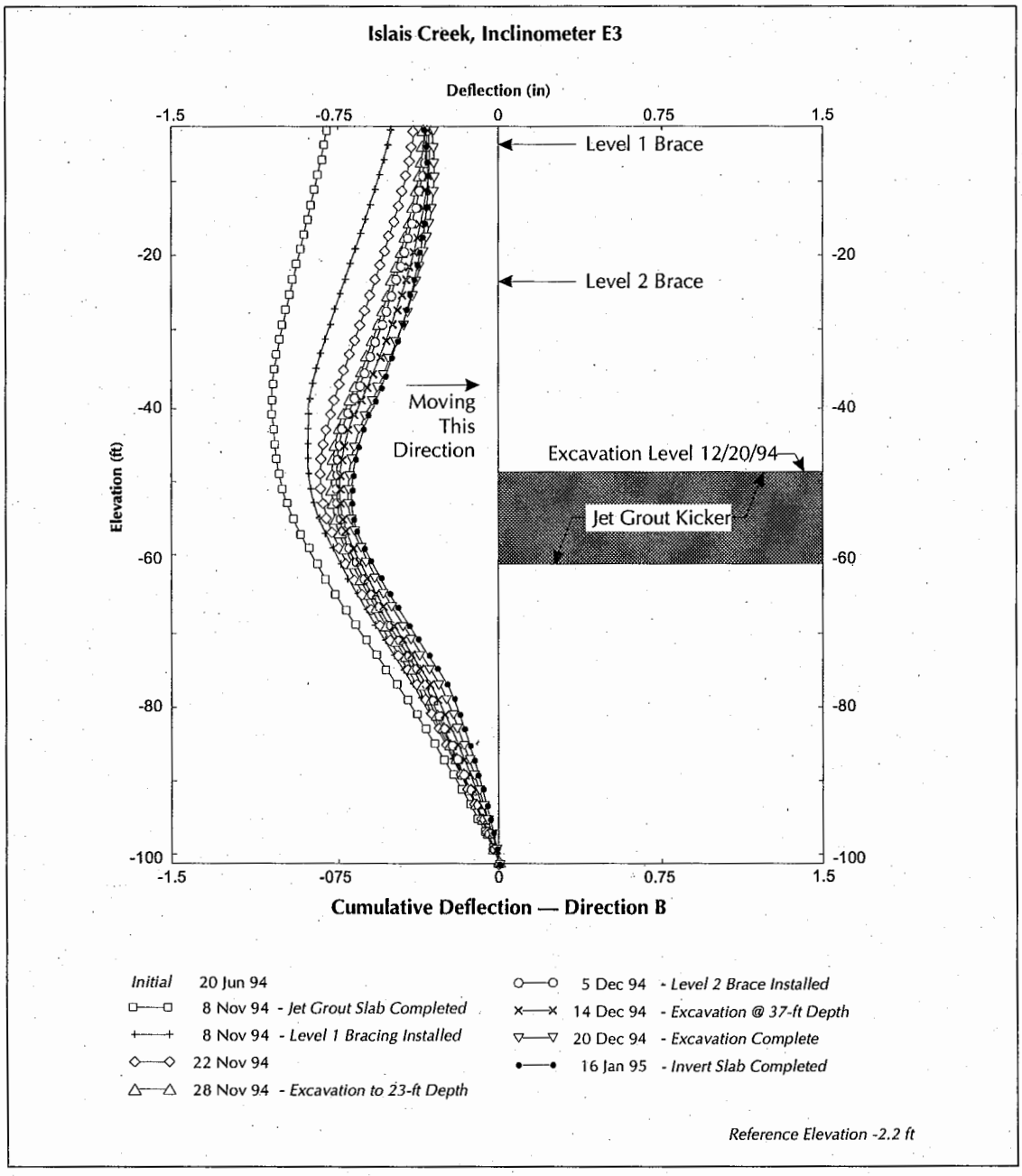

FIGURE 14. Inclinometer data for the East T/S structure.

and make them suitable for transport and disposal.

Quality control during the installation of the jet grout kicker slabs consisted primarily of visual inspection. By the time the kicker slabs were installed, the contractor already had performed a significant amount of jet grouting for the Davidson Tunnel. The grouting procedures had been well established and strengths had been confirmed by performing unconfined compressive strength tests on random core samples of the jet grouted Bay Mud. One of the most important tasks associated with visual inspection was to make sure that each column location was grouted and that adjacent columns were not grouted on the same day.

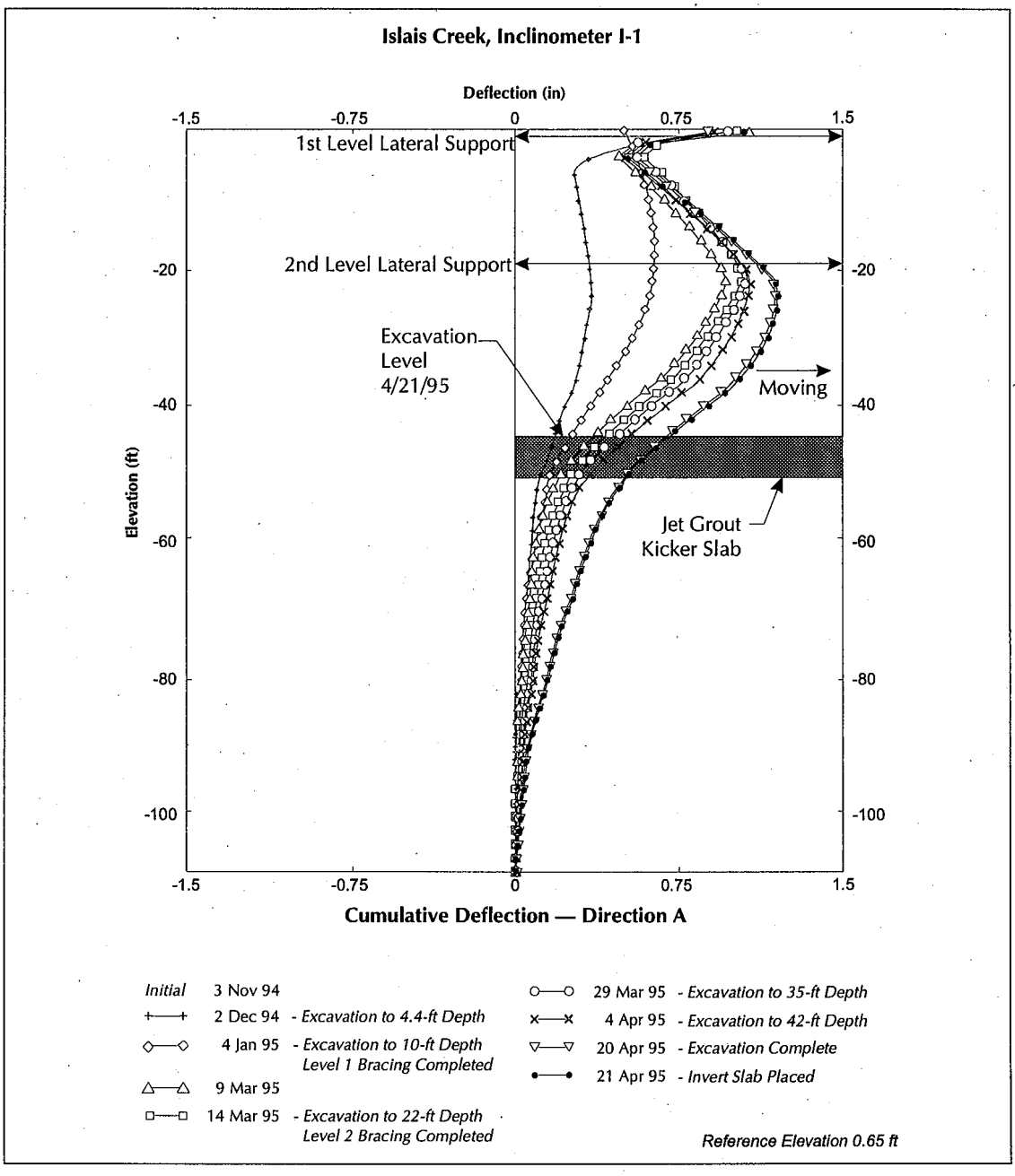

FIGURE 15. Inclinometer data for the West T/S structure.

Performance of the SPTC Wall/Jet Grout Kicker Slab System

Overall, the excavation support system performed very well. The use of the jet grout kicker slab in conjunction with only two levels of bracing was very effective in controlling lateral deflection and the resulting bending stresses in the soldier piles within the SPTC walls. The jet grout kicker slab allowed for excavation to be performed with two levels of bracing over the entire 50-foot (15-meter) cut. The jet grouting also provided a clean, stable invert surface, which greatly simplified excavation within the very sticky and unstable clays. It allowed for a

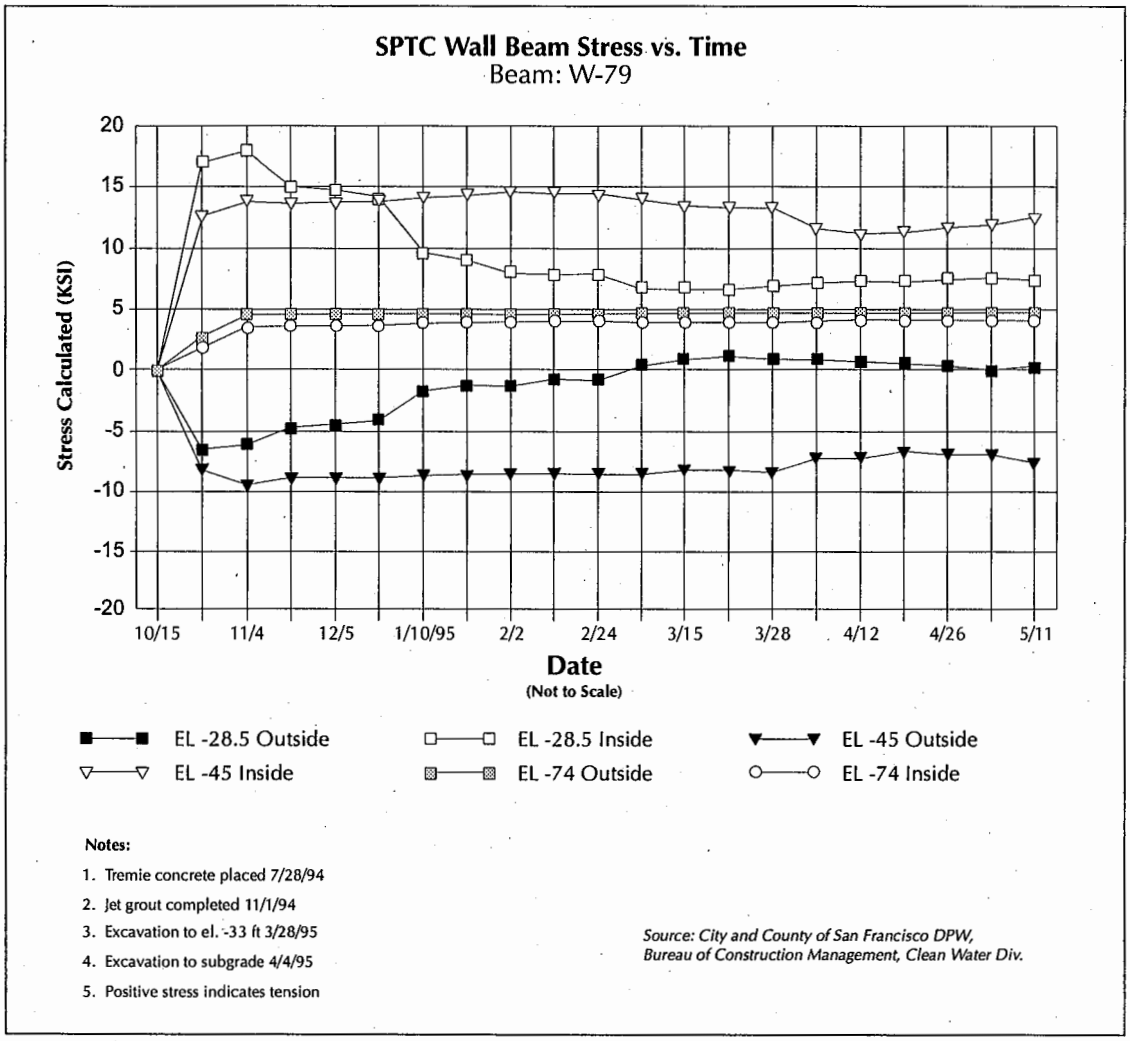

FIGURE 16. Strain gage data.

safe excavation to depth and quick construction of the invert slab. The overall lateral movement of the walls, as determined by inclinometers and lateral deflection points, was limited to approximately 0.5 to 1.0 inch (12 to 25 millimeters) into the excavation. Strain gage data indicated the stresses in the flanges of the beams were on the order of 10 to 20 ksi (69 to 138 mPa).

Lateral Support During Excavation — Inclinometer Results. Both the jet grout's ability to restrain wall movement during excavation and its impact on the walls prior to excavation are graphically presented in the actual inclinometer data obtained during the work. Figure 14 on page 29 presents a plot of data taken from an inclinometer casing installed within the East T/S structure. The centerline of the plot (zero) represents the initial readings obtained after the placement of concrete within the SPTC walls. Deflections represented as negative are movements away from the excavation, while positive deflections are movements into the excavation. The line dated 8 Nov 94 was taken after completion of the jet grout kicker slab. The subsequent plots were taken during the indicated stages of construction. This plot shows that installation of the jet grout kicker slab resulted in outward wall movement of approximately 0.8 inches (20 millimeters). This plot also show that the wall moved inward approximately 0.5 inches (12 millimeters) during the excavation of the box structure.

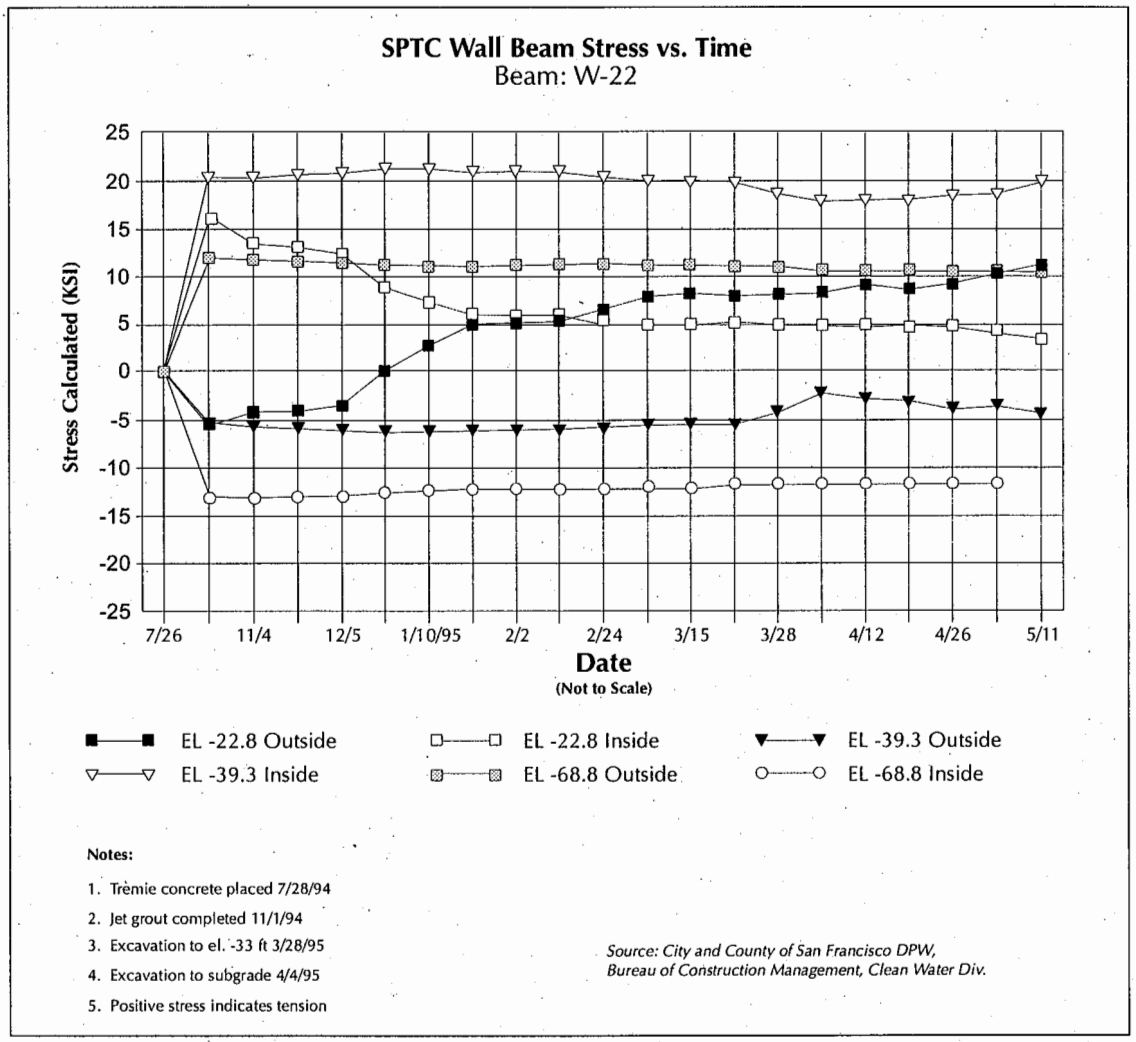

FIGURE 17. Strain gage data.

The actual inward deflection due to increasing lateral loading as the excavation proceeded is shown in Figure 15 on page 30, which is a summary plot of data collected from an inclinometer casing installed within the West T/S structure walls. In that figure, the initial reading was taken after the completion of the jet grout kicker slab, so the outward deflection is not shown. The maximum movement into the excavation is about 1 inch (25 millimeters) and occurs mostly at the very start of the excavation. The smaller and smaller magnitudes of movement with time indicate that the jet grout kicker slab was performing very well.

Stresses in Beams. The strain gage data dramatically reflect the lateral ground movements that occurred during installation of the kicker slab. Both Figures 16 and 17 are typical of all the strain gage data and indicate that an increase in stress developed in the flanges during the installation of the jet grout kicker slab. Interestingly, at locations at and above the bottom of the excavation they show that tensile stress generally developed in the inside flanges and compressive stresses in the outside flanges after the completion of the kicker slab. As excavation mobilized lateral load onto the wall, the tensile and compressive stresses that developed in the wall above the base of the jet grout kicker slab were reduced significantly. However, the tensile and compressive stresses that developed in the wall at

the base of the jet grout kicker slab were reduced only slightly.

Lessons Learned. Use of the jet grout kicker slab did provide adequate lateral support of the walls during excavation. However, use of the kicker slab resulted in some difficulties and some lessons were learned from observations.

The largest difficulty was the outward movement of the SPTC walls during jet grouting. These outward movements resulted in wall cracks that led to leaks during excavation. The East T/S, which was nearly rectangular, had half- to three-quarter-inch cracks between the soldier piles and concrete at all four corners. The rest of the walls were dry. These leaks were controlled by dry packing the cracks with water plug and other hydraulic cements during excavation. The walls of the West T/S experienced similar distortions — three corners had similar cracks.

Another difficulty resulting from the jet grouting was the heave of an existing temporary sewer that was installed within the SPTC walls. The Napoleon Diversion Sewer is a 48-inch (1,200-millimeter) cast-in-place concrete box sewer that, as per design, crossed through the West T/S approximately 12 feet (3.7 meters) below the ground surface, and connected with the existing Alemeny Sewer (see Figure 3). Due to the sequence of the work, this sewer was temporarily crossed through the West T/S box in a 48-inch (1,200-millimeter) steel carrier pipe, which was "blocked out" of the SPTC walls to facilitate their construction. The jet grout kicker slab was installed beneath this steel pipe by the use of angled drilling. During mass excavation between the walls of the West T/S in this area, groundwater was unexplainably high and could not be lowered within the walls. It was soon discovered that the 48-inch (1,200-millimeter) steel carrier pipe had been "heaved" out of its connection at both completed SPTC walls. This problem was solved by diverting the sewage around the carrier pipe with a pumping scheme so the cast-in-place concrete structure could be built.

Summary & Conclusions

The data presented above, as well as observations made during construction, confirm that the jet grout kicker slab did provide adequate lateral support of the SPTC walls during excavation. Maximum wall movements into the excavation ranged between 0.5 and 1.0 inch (12.5 and 25 millimeters), well within acceptable performance values for a cut of 45 feet (13.7 meters) in the weak Bay Mud strata. During jet grouting, the SPTC walls were forced out laterally, away from the excavation. This action created stresses in the SPTC piles on the order of 20 ksi (1,400 mPa), which reduced significantly as excavation proceeded.

The jet grout kicker slab, in addition to providing lateral support for the SPTC walls, provided a clean, stable invert surface. The Bay Mud is very sticky and difficult to both excavate and move around in. The kicker slab simplified bottom excavation since it enabled large excavating equipment to work off a stable invert surface. Figure 18 shows the deep excavation within the completed SPTC wall jet grout kicker slab system.

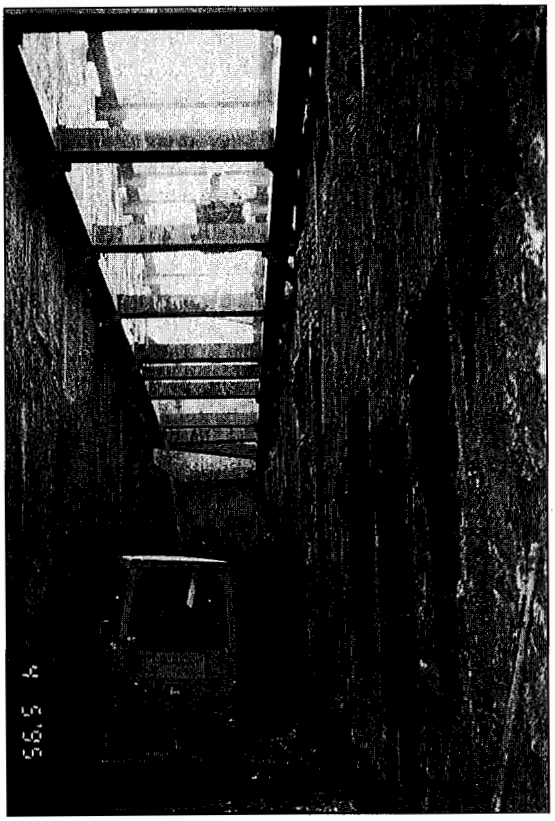

FIGURE 18. A view of the excavation within the completed SPTC wall jet grout kicker slab.

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