JET GROUTING FOR SOLVING TUNNELLING PROBLEMS IN SOFT CLAYS

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ABSTRACT

This paper describes the use of jet grouting in clays for two major tunnelling projects. The first example is for ground treatment associated with retaining wall remediation on a huge cut and cover section of the Central Artery in Boston, Massachusetts. The second is for full face pretreatment, and structural underpinning, for an underground tunnel in south San Francisco, California, where its use obviated the traditional need for compressed air.

1. BACKGROUND

Overviews of ground treatment such as by Bruce (1993 and 1994) typically identify four basic categories of soil grouting (Figure 1).

1. Hydrofracture (or claquage).
2. Compaction.
3. Permeation.
4. Jet (or replacement).

Jet grouting is the youngest major category of ground treatment. According to Miki and Nakanishi (1984), the basic concept was propounded in Japan in 1965, but it is generally agreed that it is only since the early 1980s that the various derivatives of jet grouting have approached their full economic and operational potential to the extent that today it is arguably the fastest growing method of ground treatment worldwide. Its development was fostered by the need to thoroughly treat soils ranging from gravels to clays to random fills in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements.

Jet grouting can be executed in soils with a wide range of granulometries and permeabilities. Indeed, any limitations with regard to its applicability are imposed by other soil parameters (e.g., the shear strength of cohesive soils or the density of granular deposits) or by economic factors.

The ASCE Geotechnical Engineering Division Committee on Grouting (1980) defined jet grouting as a "technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn." Figure 2 depicts one particular type in which the soil is jetted by an upper nozzle ejecting water at up to 50 MPa inside an envelope of compressed air at up to 1.2 MPa. The debris are displaced out of the oversized drill hole by the simultaneous injection of cement-based grout through a lower nozzle (at

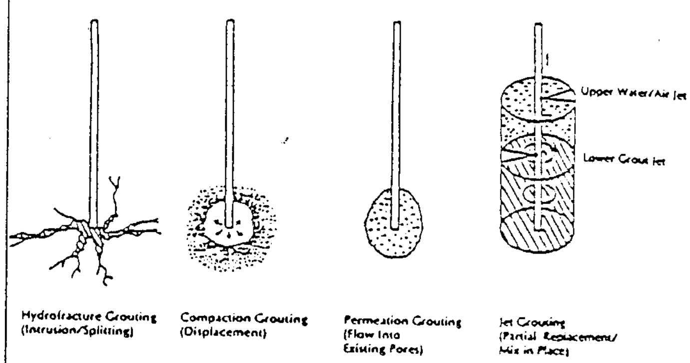

Figure 1 Basic Categories of soil grouting (Bruce, 1994).

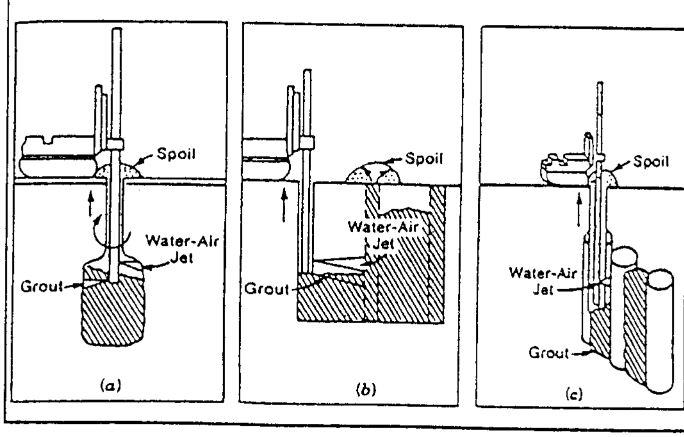

Figure 2. Shematic representation of three-fluid jet grouting method (R3) for a) columns, b) panels, or c) "wings" (ASCE, 1987).

pressures up to 8 MPa). This type is called the three-fluid system (R3). Other simpler variants (e.g., the one-fluid system R1) utilize grout jetting alone to simultaneously erode and inject, giving much more of a mix-in-place action. At the other extreme of complexity, the new Japanese Super Soil Stabilization Management (SSSMAN) system provides total (and verifiable) excavation of the soil prior to grouting or concreting. Clearly, each system has its own cost implications. It would seem that 45 m is the practical maximum depth of treatment.

In contrast to the sensitivity and sophistication of some aspects of permeation grouting, the principle of jet grouting stands as a straightforward positive solution, using only cement-based grouts across the whole range of soil types. It therefore has the potential of being "designer-driven" as a technology, unlike the case with other grouting methods. However, it must be emphasized that any system that may involve the simultaneous injection of up to three fluids at operating pressures of up to 50 MPa must be handled with extreme care and only in appropriate applications, circumstances, and ground conditions. Although applications have long been reported throughout the world, it is only in the last ten years or so that it has been used with any regularity in North America. Even then, most of the applications have been for structural underpinning or seepage cutoffs, in granular soils.

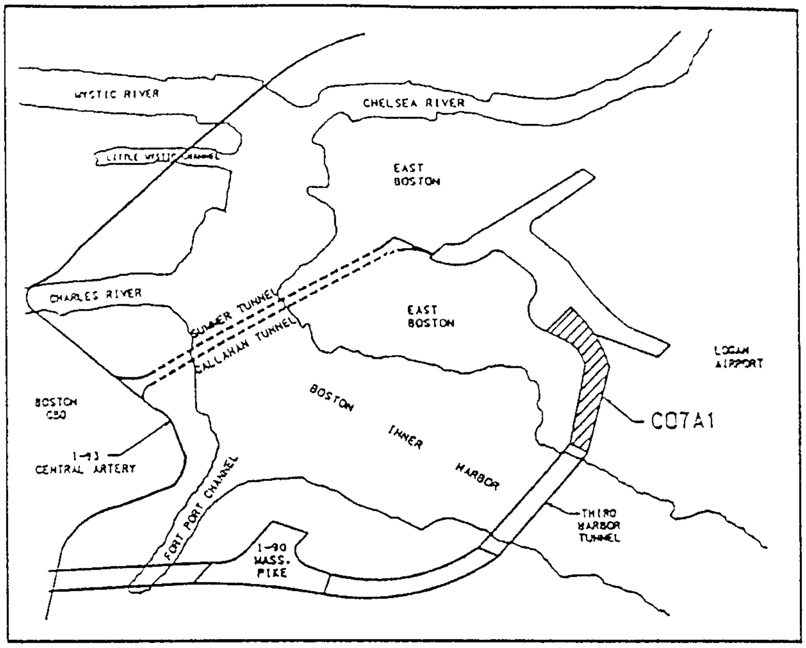

Figure 3. Location of Contract C07A1, Boston, Massachusetts (Pearlman and Himick, 1993).

However, in the last year, two significant applications have been conducted by the authors' company for tunnelling projects in soft, saturated clays. While reflecting common practice, specially in the Far East, these applications are so far unique in the United States.

This paper outlines these projects, with special attention paid to the reasons for selecting jet grouting, and the results of the process. The projects are:

- Contract CO7A1 in Boston, Massachusetts where jet grouting was used in excavation support remediation in a massive cut and cover tunnel, and
- Contract E, at Islais Creek, San Francisco, California, where full face pre treatment of the clay permitted underground tunnelling to proceed without the need for compressed air.

2. CONTRACT CO7A1, BOSTON, MASSACHUSETTS

2.1 Background

The city of Boston, Massachusetts, is geographically constrained in its Central Business District (CBD) by Boston Harbor to the East, and the Charles River to the North. Its main international airport (Logan) is accessible only from the OBD by a pair of Tunnels (Sumner and Callahan), that cross Boston harbor to East Boston (Figure 3). These tunnel portals are to the east of the existing North-South elevated I-93 (Central Artery). About 1 mile south of the tunnel portals is the Eastern termination of the Massachusetts Turnpike (I-90). Traffic must proceed northward on the Artery in order to then exit to the airport. Massive traffic congestion in this area has led the Massachusetts Highway Department (MHD) to embark on an ambitious program which includes a new Third Harbor Tunnel (THT) and replacement of the existing Central Artery (I-93) with a cut and cover tunnel.

The third harbor tunnel is to be connected to a direct Eastward extension of I-90 across South Boston. One portion of this I-90 extension is currently under construction and connects to the Western THT portal. Other portions of I-90 through South Boston are in final design. This whole Central Artery/Tunnel projects is one of the largest highway and transit projects ever undertaken in the United States. A Joint Venture of Bechtel-Parsons Brinckerhoff was appointed as management consultants for design and construction.

The East approach to the THT consists of parallel reinforced concrete box structures built in a cut and cover excavation. Contract CO7A1, which included substantial excavation support, is the subject of this paper. Excavation support walls using a steel beam- reinforced SMW (Soil Mixed Wall) method (Taki and Yang, 1991) were selected to provide a ground water cutoff and a structural excavation support for the cut and cover excavation. Primary lateral support was by prestressed ground anchors. The SMW wall was made using overlapping and discontinuous 1 m diameter augers and mixing paddles that penetrate the ground at 610 mm centers while injecting a cement grout and mixing it with the soil. Extensive overlapping creates a continuous treated slot. In total, about 40,000 sq. m of SMW shoring was created (35000 sq. m exposed) including 3,700 tons of structural steel and 3,400 ground anchors of service loads 70-190 tonnes to stabilize excavations as deep as 25 m.

The general contractor for the 900 m long Contract CO7A1 was a Joint Venture of Modern Continental (Cambridge, Massachusetts) and Obayashi Corporation of Japan. The design-build subcontract for the excavation support was awarded to a joint venture of Nicholson Construction Company and SMW Seiko (Hayward, California).

Given the proximity of many important structures, including the airport, it was essential to avoid settlements outside the excavation, caused by lateral wall movements and/or drawdown of the water table.

2.2 Ground Conditions

The alignment passes through a complex set of ground conditions, due to distinct variations, both vertical and lateral, in the natural soils, and in the overlying fills. The area known locally as Bird Island Flats was historically exposed only at low tide. A dredged shipping channel and various bulkhead walls were created. From the early 1900s until 1973 successive stages of fills were placed to raise and level the site to its current elevation (4m NGVD).

For the purpose of excavation support, three successive distinct areas of the alignment were identified by the Area Goetechnical Consultant, from south to north:

• Zone A (330 m long): Sandy fill (5-6m) overlying 3 m of organics and marine deposits, underlain by dense glaciomarine deposits. These glaciomarines dominated the excavation face (maximum height 25 m).

• Zone B (280 m long): Variable deposits of sandy fill, cohesive fill, and man-made debris filling a 15 m deep, 180 m wide trapezoidal ship channel overlying very dense glaciomarine deposits at the approximate tunnel subgrade.

• Zone C (235 m long plus ramps): cohesive and granular fills overlying organics, marine clay and glaciomarine. The soft marine clay (Boston Blue) is much thicker than elsewhere and the tunnel subgrade was to be founded predominantly on it. (Figure 4 and 5). Typically it may have a shear strength over 70 KPa in its upper 5 m, but decreases to less than 50 KPa below. The water table was about 2 m below ground surface.

The Problem

In Zone A, no significant problems were encountered during excavation, with wall movements upon completion being less than 0.2 percent of

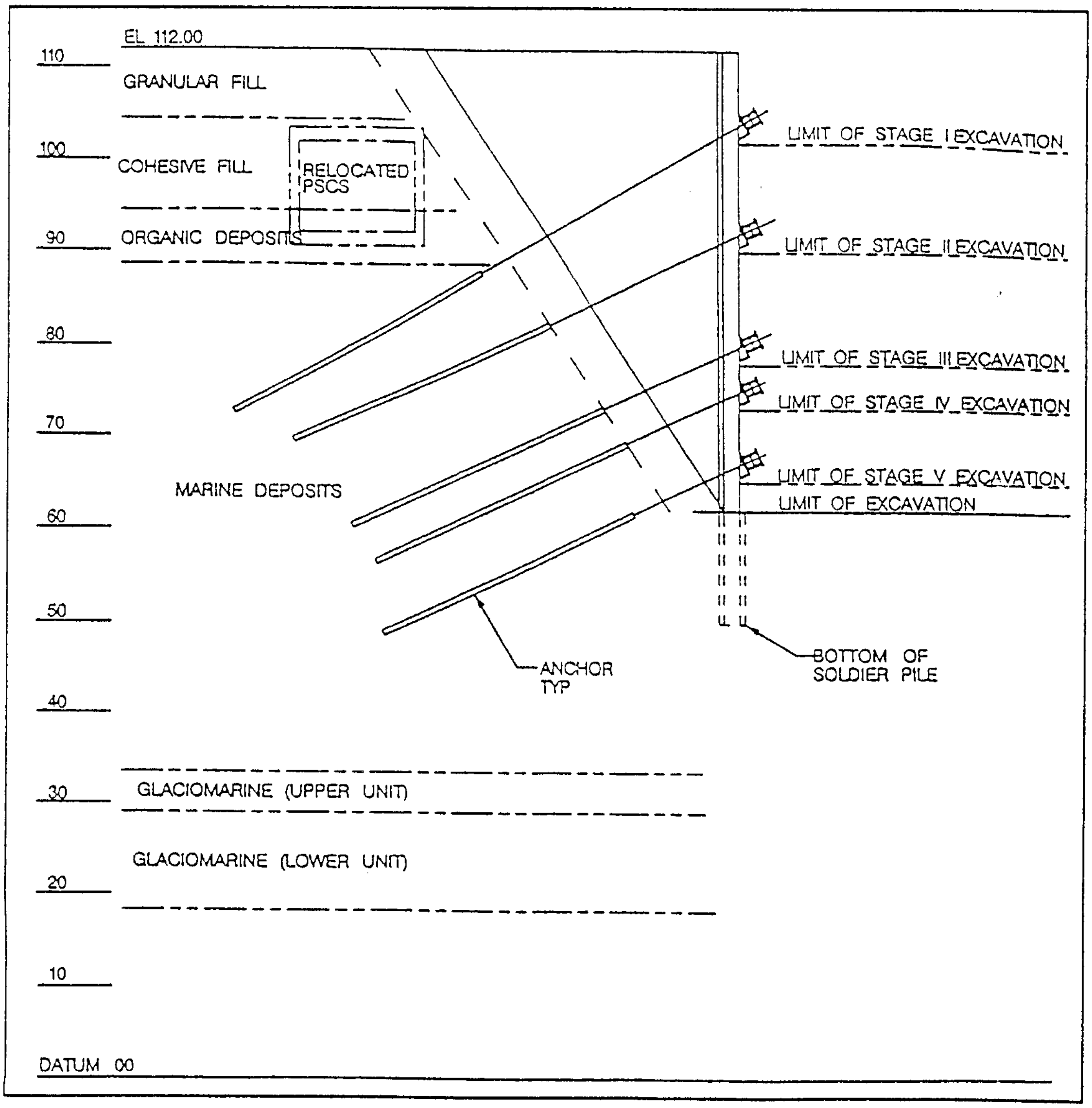

Figure 4. Typical foreseen East Wall section (STA157+70) in Zone C. (Dimensions in feet: 1 foot = 0.3048m)

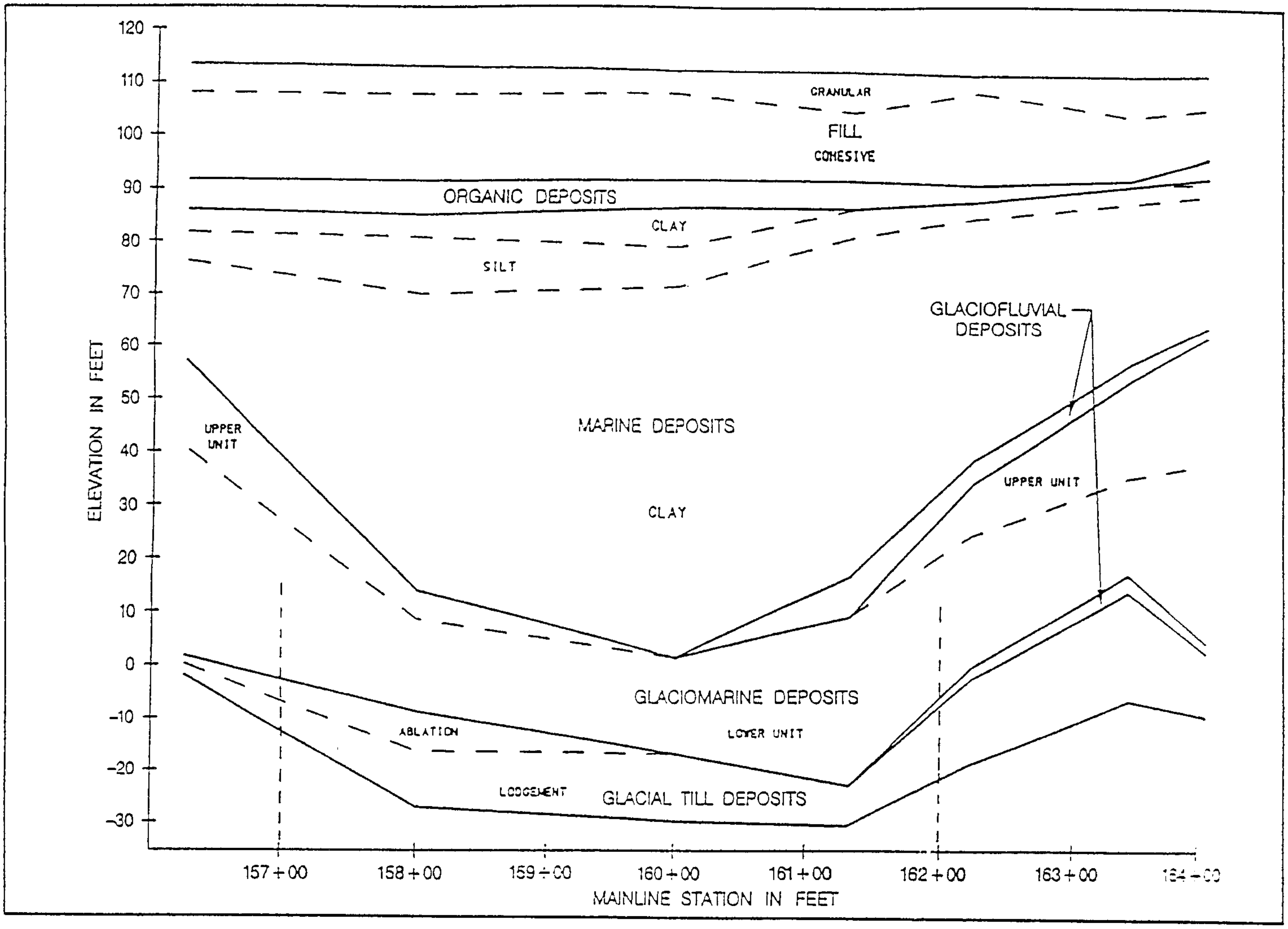

Figure 5. Simplified geological cross section at East Wall in Zone C (Dimensions) in feet: 1 foot = 0.3048m). (Nicholson and Chu, 1994)

wall exposed height. Likewise in Zone B, despite the need to amend certain construction methods for the installation of the SMW wall in the ship channel, the final excavation performed well, and within the specifications.

However, in Zone C, Figures 4 and 5 had indicated that for a 150 m long section, the marine clay and organics were as much as 26m deep. When excavation on a 60 m long section of the East Wall had reached the third row of anchors, about 13 m down, ongoing lateral movements of over 220 mm occurred over a period of days in the pattern shown in Figure 6. Minor surface effects were noted 12-30 m back from this wall. Load monitoring of the anchors confirmed little or no loss of load, implying they were moving along with the entire soil mass, even though they were as long as 27 m. The excavation was locally backfilled and the movements ceased.

Intense studies by all the parties, in Partnership, concluded that these movements were the result of:

- lack of passive resistance at the toe of the wall.
- basal heave occurring at subgrade inside the excavation.
- low actual factor of safety with respect to global stability.

According to Cheney (1994), "the managing consultant concluded that the clay soil strengths shown in the contract documents were an upper bound value and probably not representative of conditions at the site of the wall movement."

Various remediation schemes were proposed and judged on the basis of practicality, likely performances cost, and schedule impacts. The adopted solution was to improve the engineering properties of the clay to increase its shear strength, provide resistance across the global circular failure plane and improve passive resistance to the toe. A target treated ground strength of about 3 MPa was considered adequate.

The scheme involved the formation of 1 m wide treated soil buttresses with a clear separation of 1.5 m, in the base of the excavation (Figure 7) over a length of about 250 m on each wall. Each buttress was widened near the wall to a 2 m wide "hammer-head." It proved most economical to generally create these by the (unreinforced) SMW method (giving unconfined strengths of 2 MPa. These were keyed into the glaciomarine soils along the East Wall (to guard against global failure), but "floated" in the clay at the West Wall where geotechnical and structural conditions were better (Figure 8). Details of the design are summarized by Cheney (1994). However, the large size and configuration of the SMW machine, and its ability to drill only vertically, meant that 2-3 m square gaps would exist between the wall and the buttress hammerheads. Jet grouting was therefore selected to provide these connections, install buttresses in restricted areas, and also to underpin the base of the West Wall to minimize lateral or vertical movements during excavation and anchoring. This was the first example of large scale mass jet grouting treatment in clays in the United States.

2.4 The Jet Grouting

A design based on a conservative column diameter of 0.8m, and a minimum strength of 1.4 MPa at 28 days was prepared. This design featured six overlapping columns (in two rows of three columns) per hammerhead inclined at angles up to 8° off vertical. A full scale field test was organized to verify the actual insitu effects of the jet grouting.

On technical and economic grounds, the R2 (Figure 9) method was selected for test. Eight columns, each 8 m deep, were installed from the base of the excavation, in the clay near the East Wall. Each column was installed with a different combination of drilling and grouting parameters, with special attention paid to the potential benefits of pre-washing during drilling. Air bubbles were frequently noted to escape at the surface several meters away from the 100 or 127 mm diameter drill holes. This observation was also recorded during the subsequent production works and was ascribed to the influence of the very soft and reworked clay in the much trafficked base of the excavation, as opposed to any deep seated phenomenon.

Excavation exposure and coring of the columns followed. Columns of regular composition and shape, from 1.5 to 2.4 m in diameter were recorded, with strengths typically well over 2 MPa at 28 days.

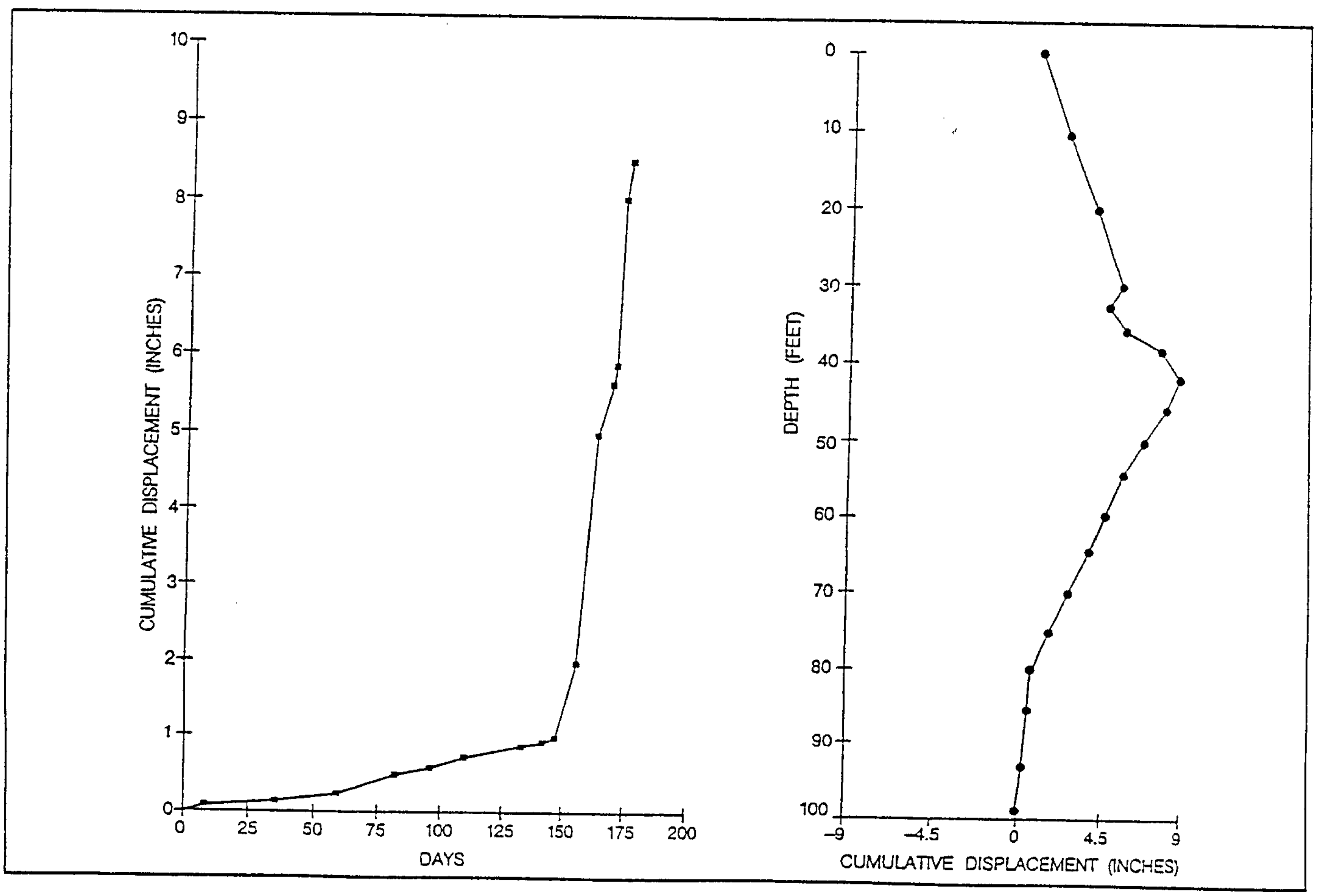

Figure 6. East Wall displacements (STA157+50) measured prior to emergency backfilling. Excavation depth approximately 13m (Dimensions in feet and inches: 1 foot = 0.3048m; 1 inch = 25.4 mm) (Nicholson and Chu, 1994)

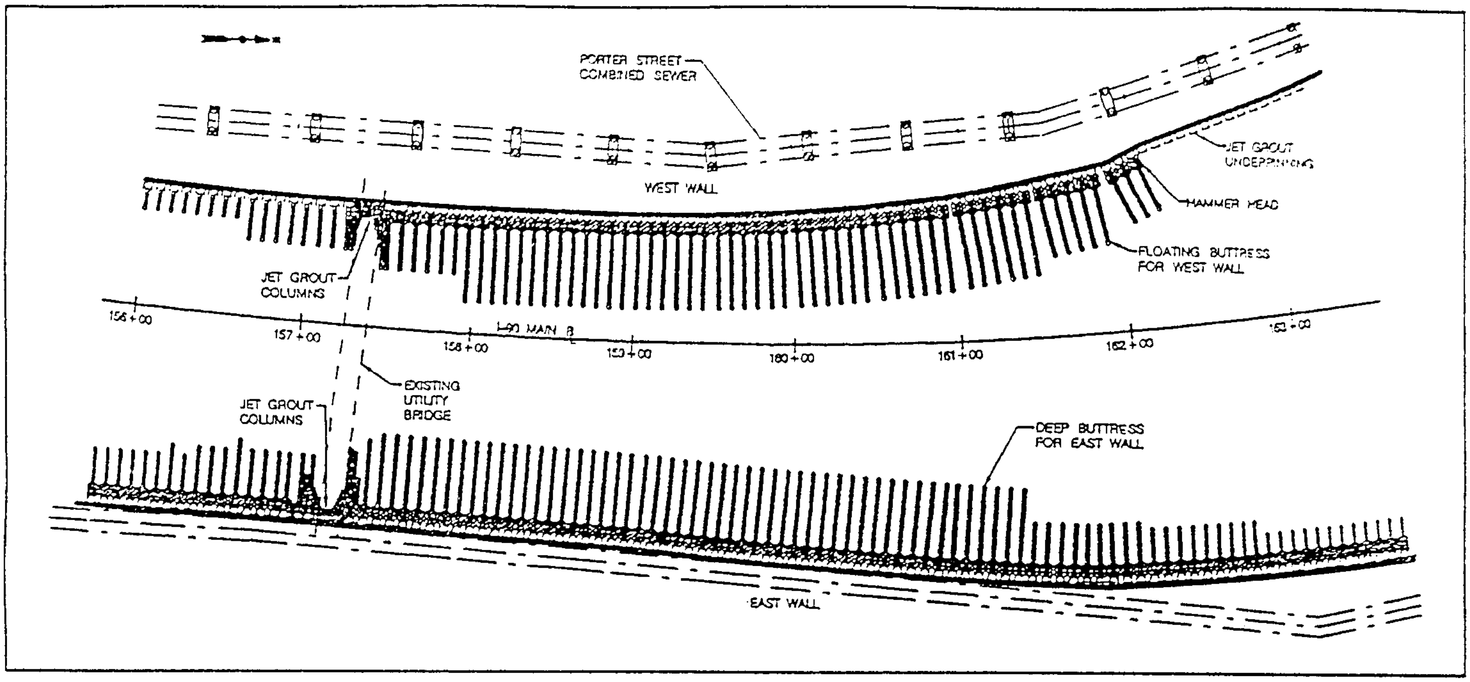

Figure 7. Location of remedial SMW buttresses and associated jet grouting in Zone C. (Stations in feet: 1 foot = 0.3048 m). (Nicholson and Chu, 1994).

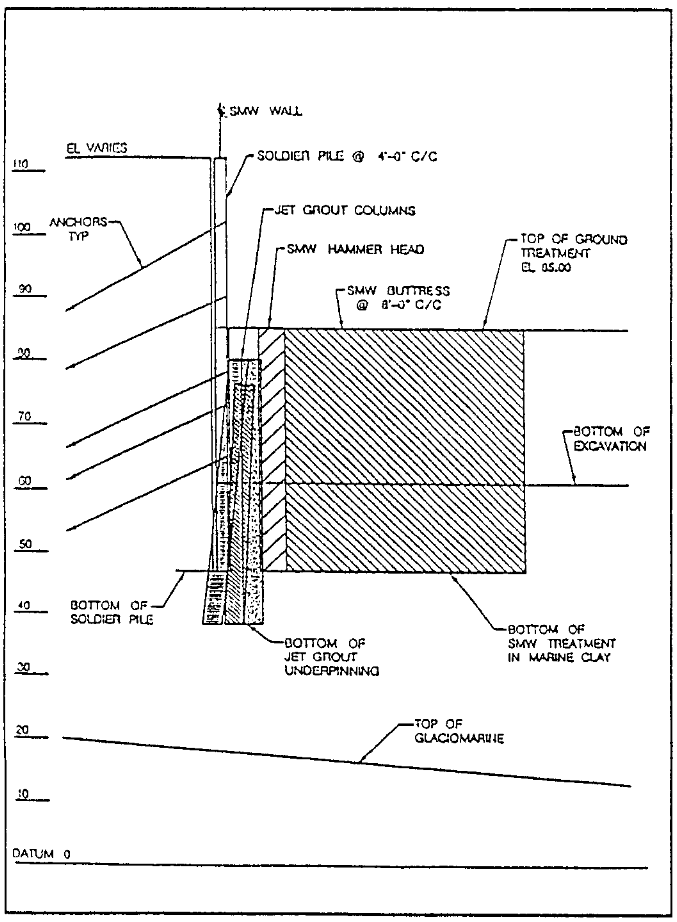

Figure 8. Typical "Floating" SMW buttress, West Wall, Zone C. (Dimensions in feet; 1 foot = 0.3048 m). (Nicholson and Chu, 1994).

Thereafter the production parameters and methods were determined including grout pressures of over 40 MPa, air pressures of 1 MPa, rod rotational speed of 10 rpm, and grout of water/cement ratio 0.8 (by weight).

Production work then progressed. Special care was taken with the sequencing of column installation so that excessive volumes of the very sensitive clay would not be fluidized by jet grouting at the same time: early experience with the floating buttresses of the West Wall showed that when the jet grouting was concentrated in one area, the wall moved towards the excavation as much as 75 mm during the grout setting period, although this figure also included excavation induced movements during the same period.

A total of 14,500 lin. m. of jet grouting (in 1500 columns) was installed for the different purposes of buttress gap sealing and wall underpinning.

On the East Wall, small toe movements away from the excavation were recorded during hammerhead gap grouting. These were ascribed to the fact that the buttresses, being fixed in the competent glaciomarine, acted as immobile reaction blocks during grouting and so any volumetric expansion of the gap during grouting resulted in back movement of the (relatively flexible) wall.

2.5 Performance

Upon the conclusion of the SMW and jet grouting work, and after the treated soil was judged to have reached the target strength, carefully staged excavation and re-excavation pro-

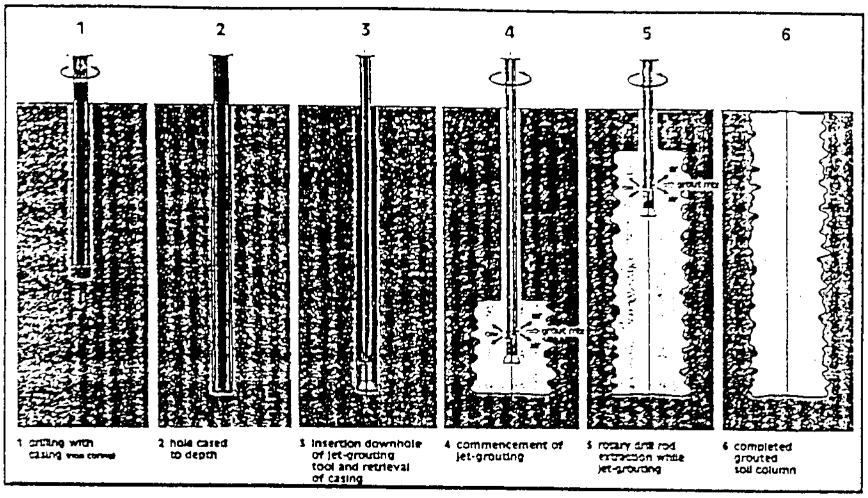

Figure 9. Construction sequence of two-fluid (R2) jet grouting.

losses of air pressure. The final lining, designed to withstand full earth pressure, was to consist of a steel pipe at least 32 mm thick. During excavation of the shield access shaft, a Cost Incentive Change Proposal was offered to pretreat the Bay Mud by jet grouting, and so eliminate the need for compressed air. With a treated soil strength in excess of 0.8 MPa, conventional open face shield and road header methods could be used, together with wood lagging and steel ribs as temporary lining.

Following a successful field test, and subsequent discussions with all interested parties, the progressed, with the introduction of additional ground anchors. Excavation was not at all hindered by the presence of the treated soil, which proved easy to remove and to dispose of (being a solid waste product). All wall movements have been within the owner's tolerable limits, and no excessive, sudden or ongoing deflections or settlements were recorded.

Prompt and decisive action by all the parties, acting in partnership, ensured that the unexpected extent of the problem posed by the deep and sensitive clay in Zone C has had a much less impact on project completion date than was at one point feared.

3. ISLAIS CREEK TUNNELS, SAN FRANCISCO, CALIFORNIA

3.1 Background

The Islais Creek Transport/Storage Project Contract E consists of upgrading the existing sewer system by construction of underground reinforced concrete box structures designed to store storm flows, and by connecting various existing sewer lines by new tunnels. The owner is the City and County of San Francisco Department of Public Works, and the general contractor is Kajima Engineering and Construction.

The two bored tunnel sections (used where the cut and cover method was precluded by the presence of existing structures) were originally foreseen to be mined through the very soft Bay Mud under compressed air, to reduce squeezing and related ground movements, and to resist flow of water into the tunnel. The temporary support was foreseen to be provided by bolted steel liner plates erected within the compartmentalized breast boarding shield, and back-grouted to prevent subsidence in the tail void. The lining segments were to be fitted with gaskets to prevent inflow of water, and

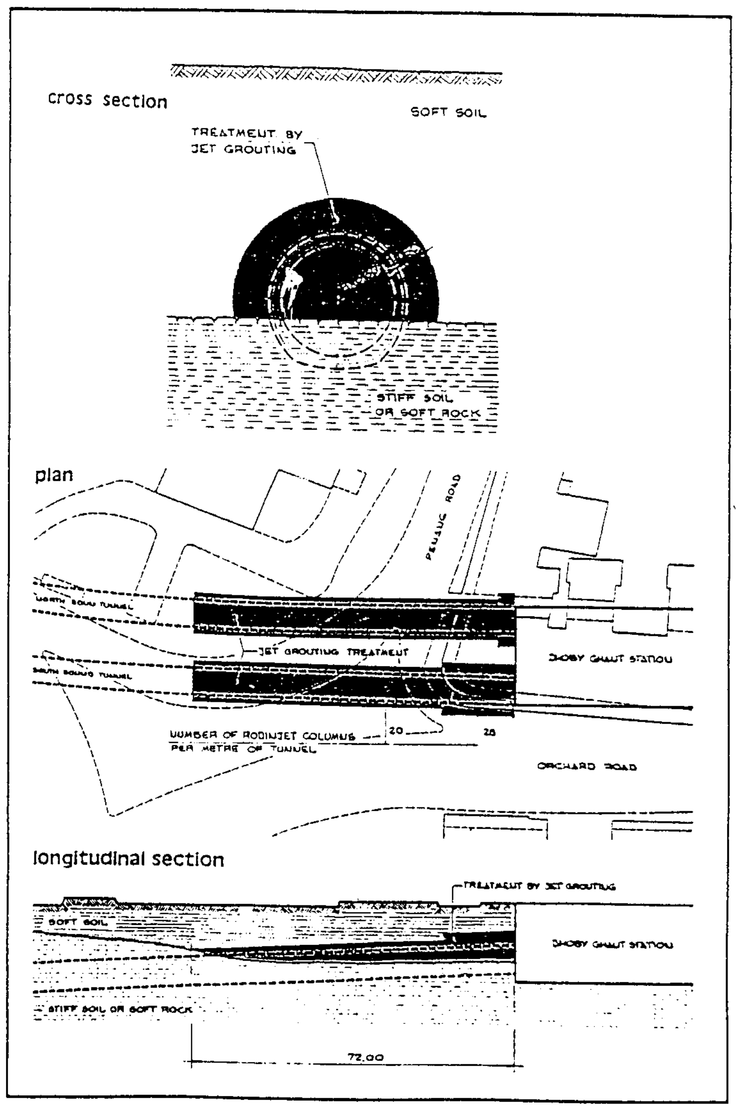

Figure 10. Example of full-face jet grouting. MRT 106, Singapore (Mongilardi and Tornaghi, 1986).

posal was accepted. This proposal covered the 4 m diameter drive approximately 130 m long crossing under Interstate 280 and the Joint Powers Board's main railroad tracks (on a 6 m high embankment) and a 155 m long, 4.5 m diameter drive parallel to Davidson Avenue. This is the first example of full face tunnel pretreatment in soft cohesive soil in North America, although such practice is relatively common in the Far East (e.g. Mongilardi and Tornaghi, 1986) (Figure 10).

This large and complex project totals 330 m in length and features many other specialty geotechnical and tunnelling processes (Burke, 1995). Given the adaptability, and effectiveness of jet grouting generally in soft ground and in such projects, other applications were anticipated for the technique as the project unfolded.

3.2 Ground Conditions

The project is within the Islais Creek basin, located on the south eastern side of the San Francisco urban area, north of the airport. Successive early survey maps show that most of the area was reclaimed from tidal marsh lands by filling with 8-12 m of dune sand, rock fill, miscellaneous debris and organic waste, resulting in a ground surface 3-6 m above mean sea level. The fill also often contains old piles, and even sunken ships.

The marine and continental deposits overlying Jurassic-Cretaceous bedrock include the Holocene Bay Mud (which controls most of the design and construction issues), Pleistocene Colma sands, and older clays, alluvium and colluvium. The Bay Mud is a plastic silty clay or clayey silt with trace amounts of organics, shells and sand. It is generally classified as CH or MH. The consistency ranges from very soft to soft in the upper portions of the deposit to soft to medium soft in the lower portions. It is easy to push the fingers of a hand into the in situ material which often has an undrained shear strength as low as 15 KPa. It ranges from normally consolidated to slightly over consolidated, with OCR values ranging from 1.05 to 1.62. Consolidation is ongoing and is therefore causing considerable movements: the maximum anticipated long term settlement below the Davidson Avenue Tunnel is about 0.25 m, related to a Mud thickness of about 30 m. At the railway embankment, the load itself is causing consolidation of the underlying Mud, resulting in some increase in shear strength from 30 to 54 KPa.

The water table is about 1.5 m below the surface, and since dewatering would result in general consolidation and accelerated surface settlement, it was not permitted as a construction option.

The tunnel inverts were as much as 12 m below the water table.

3.3 The Jet Grouting

It was foreseen at the proposal stage that, depending on the jet grouting methods and parameters selected, adequately strong columns of 1.2 - 1.5 m diameter could possibly be formed. These would be installed so as to completely treat the Bay Mud across the full face of each tunnel and for an annulus of about 1.5 m thickness all around. Most holes could be installed vertically, but surface conditions and a network of near surface services would often dictate the need for inclining holes. This was particularly necessary for the drive under the railway tracks (Figure 11).

The purpose of the jet grouting test program was to demonstrate that:

- regularly shaped columns of certain minimum diameters could be created, thereby allowing the treatment of a continuous soil mass in the production works.
- the design minimum strength of 0.8 MPa could be achieved for the jet grouted soil.
- heave or settlement of surface and services could be controlled and minimized.

A total of 12 jet grouted columns were installed, six R1 and six R2, under the fill between depths of 5 and 9 m below the surface. Drill rods of 89 mm diameter were used. The exposure,

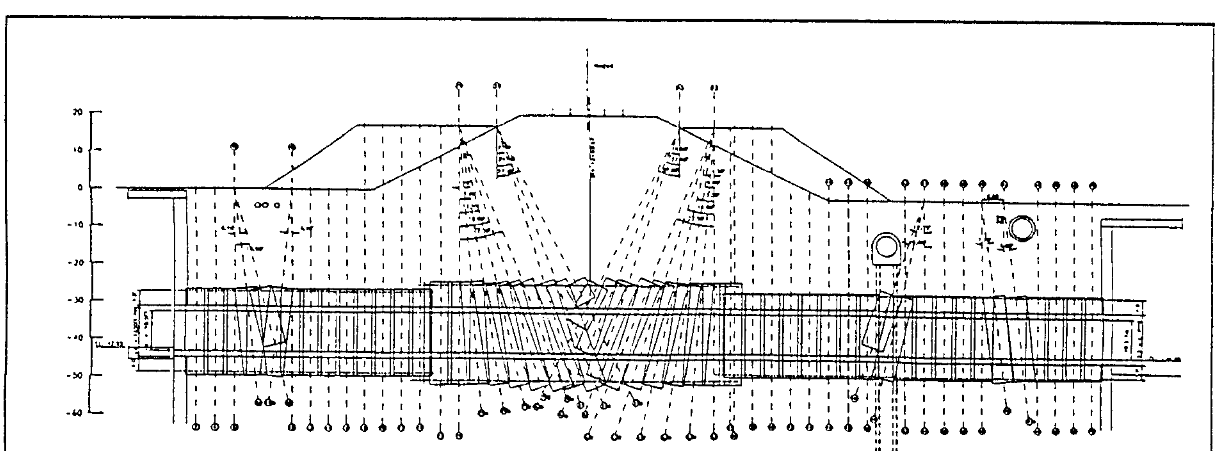

Figure 11. Cross section at Row C of jet grout columns under railroad tracks (Dimensions in feet; 1 foot = 0.3048m).

	Railway Undercrossing	Davidson Avenue
# Columns	280 ea	600 ea
Av. depth	16m	15 m
Total Drilling Length	5400m	9000 m
Total Jet Grouted Length	2400m	4200 m

examination and sampling of the test columns were accomplished within a braced sheet pile "box." Inclinometers and vertical-horizontal reference points were also installed before grouting began, to check the magnitude of ground movements associated with the jet grouting.

Using a grout of water/cement ratio 0.83 (by weight), the columns were installed with a variety of grout pressures, flow rates, target volumes, and rod rotational speeds. Two grout samples were taken from each column for strength testing at 7 and 28 days. Results averaged 18 and 33MPa respectively.

Excavation revealed R1 columns to be distinctly circular, ranging up to 0.9 m in diameter. R2 columns were similarly regular and ranged up to 2.5 m in diameter. Internal consistency appeared to be uniform and homogeneous. Cores and samples cut from the columns at various distances from their centers, showed R1 column strengths to average 3.4 MPa and R2 column strengths to average 1.4 MPa.

Subsequently, grout pressures of up to 40 MPa, air pressures of 0.8 MPa and rotational speeds of 10-20 rpm were selected for the production work. These were judged adequate to provide R2 columns of 1.8 m in minimum diameter and requisite strength.These were generally installed at 1.5 m centers.

The test results were accepted, and work commenced on the production columns as quantified below.

Special precautions had to be observed to limit movements, especially when treating under the railway where voids were likely in the fill as there was an old wooden railroad trestle buried beneath the existing tracks. By the end of the jet grouting, none of the utilities or facilities that crossed or lay adjacent to the alignment suffered damage. In addition to the railroad and I-280, pile supported sewers, high voltage lines, a gas line, and telephone and fiber optic lines had all been encountered. Throughout, mo-vements were very intensely monitored, and changes made to operational parameters to combat them. Control of movements is a critical feature of such work.

The success of the test program also encouraged the use of jet grouting for other applications on this site. These included pre-support of sheet pile and caisson walls for open cut excavations, by grouting 2.4 m thick horizontal "kicker slabs" below the future cast invert slabs (Figure 12). These kicker slabs also reduced deflections of these walls and reduced temporary bracing requirements. Jet grouting was also used as deep as 24 m to underpin existing structures which could have been impacted by the tunnelling, such as major sewers along Davidson Avenue and at the railroad undercrossing.

Routine tests on samples from production columns showed unconfined compressive strengths typically in excess of those recorded in the test program, with a maximum of 7.6 MPa.

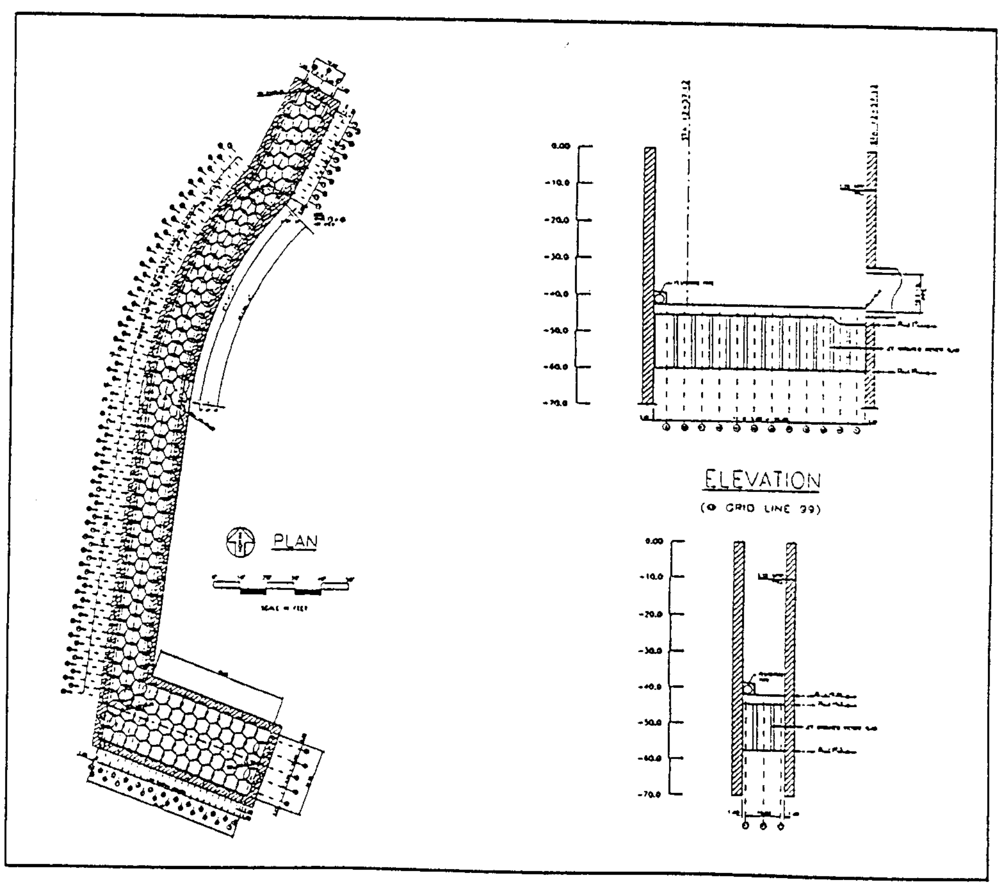

Figure 12. Jet "kicker slabs" for the West Side T/S structure. (Dimensions in feet; 1 foot = 0.3048.

3.4 Performance

Routine advances by open face shield and road header averaged 6 m per shift. Burke (1995) describes the ground as having similar excavation characteristics to the Marl encountered in the Channel Tunnel. The tunnels were excavated virtually in dry conditions without ground squeezing or surface settlement. The grouted ground proved uniform and competent and has, months after mining, shown no tendency to "collapse" against the temporary lining, even thought that small annulus remains unfilled. The muck is hard, dry, and easily handled and disposed of. The system "has performed to everyone's complete satisfaction" and has provided a fast, economical, troublefree, but above all, safe, tunnelling project in very difficult soil.

4. FINAL REMARKS

These two examples clearly illustrate the potential and performance of jet grouting for the solution of tunnelling problems in soft clays. Jet grouting has been used traditionally in North America for underpinning and seepage cut off in more granular soils: these case histories will hopefully be a source of interest and encouragement for geotechnical and tunnelling engineers alike, faced with the need to engineer and excavate in less competent materials.

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