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Deep Soil-Cement Mixing for CA/T Project

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Summary

In order to construct one segment of a complex cut & cover tunnel network deep into soft clays, large scale insitu ground improvement installed via deep soil-cement mixing (Deep-Mix Method or DMM) has been initiated. DMM emerged ahead of conventional cofferdam methods, based on design, construction, and economic considerations.

This project synthesized the resources of past practice and conventional wisdom with new technologies, to push the USA state of practice into accepting another viable design & construction expedient: the Deep-Mix Method. Both excavation support systems and permanent foundations for structures can be facilitated by this technology. The analytical and analogous assumptions which were crucial in advancing this application are described.

1.0 Introduction

The Central Artery/Tunnel (CA/T) Project in Boston, MA is the largest and most complex urban highway project undertaken by the United States of America (USA). The multi-billion project will replace a forty+ year old elevated Central Artery (I-93) viaduct with a modern underground expressway and extend the Massachusetts Turnpike (I-90) to Logan International Airport, all through an arrangement of tunnels which include cut & cover, immersed tube, and jacked tunnel elements.

One important aspect of the multi-billion dollar CA/T project has been the I-90 extension. A section of this extension must cross through a channel (Fort Point Channel), which is underlain by deep, soft blue clay (refer to Figure 1). The site is surrounded on three sides by 1) an active rail facility owned and operated by AMTRAK and the Massachusetts Bay Transit Authority, 2) the largest private employer in Massachusetts; Gillette, Inc., and 3) the northeast distribution center for the United States Postal Service. Disruptions in service caused by construction to any of these abutters would be troublesome. Hence, the impact of construction-induced ground movements had to be maintained within allowable limits.

This section was complex due to the alignment of the tunnels cutting deep and wide into the soft BBC, all the while bordering the land/marine interface. This siting of the tunnels precipitated major concerns on global stability and heave during excavation; net lateral-load tunnel foundation capacity; and lateral ground movements, both during and post-construction. Several construction methods received attention during conceptual design, essentially exploring conventional wisdom: 1) full and/or partial channel bypassing & backfilling, with conventional cut & cover construction; 2) full and/or partial marine cofferdam excavation. These types of systems failed to instill confidence that the major concerns cited above could be manageable.

Recognizing that delays would cause the entire project to experience significant delays, a team of experts was mobilized to study the problem area. During those discussions, the idea of utilizing DMM was conceived. B/PB and the SDC developed a preliminary design and construction strategy, including construction cost and schedules. Global stability, heave, and ground movements all appeared manageable, and confidence levels were reinvigorated. Successfully balancing all of the concerns, B/PB recommended that DMM be pursued as the solution to the myriad problems of this site. It was further recommended to divide the area into two construction contracts; the first would stabilize the ground with DMM; the second would build the cut & cover tunnels in the stabilized ground - this would make the Project's opening-day schedule once again viable.

This paper discusses aspects of design that were crucial in providing the support conditions and constructibility necessary to facilitate follow-on cut & cover tunnel construction. There are many facets to this project that pushed the state of practice beyond the limits of timid imagination. This case study represents the first-ever DMM use in the USA to serve as both an excavation support system and a permanent foundation for cut & cover tunnels.

The owner of the project is the Massachusetts Highway Department (MHD); the joint venture of Bechtel/Parsons Brinckerhoff (B/PB) serves as Management Consultant and performed preliminary design; the joint venture of Maguire/Harris was the Section Design Consultant (SDC), and prepared plans and specifications for the construction contract.

2.0 Site Conditions & History

Soil stratigraphy for the site, from bedrock to ground surface, is composed of glacial till, marine clay, organic silts, and man-made fills. The bedrock typically consists of moderately to severely weathered and kaolinized argillite, weathered to depths of 7.6m or more. The dense glacial till deposit varies from 1.5m-6.1m thickness. The most influential stratum is the deep deposit of BBC. This deposit is approximately 22.9m thick, and varies in shear strength from 38kPa to 48kPa. The organic silt deposit has a shear strength of approximately 5kPa, and varies in thickness between 3.0m to 6.0m. Significant areas of these naturally occuring sediments were backfilled during colonial times to create land suitable for human habitation and development. These fills are approximately 7.6m thick.

Groundwater conditions of the site include a lower, confined aquifer in the glacial till, and an upper, unconfined aquifer in the man-made fills. There is no apparent connection between these two water sources - piezometric levels in the deep glacial till deposits are lower than the upper backfills. The Fort Point Channel (FPC) traverses these fill areas.

3.0 Soil-Cement Design

3.1 Background

Once DMM was selected to be pursued, B/PB and the SDC met frequently to partner through design criteria issues, since DMM has no established US code of practice. B/PB and the SDC advanced the analogy of utilizing DMM technology to create a series of "weak-concrete" shearwalls, to serve as buttresses capable of withstanding insitu lateral soil pressures with little or no movement, while relieving the tunnels from long-term differential ground movements. The "aggregate" of the "weak-concrete" would be the insitu soils. An unconfined compression strength increase of two-orders of magnitude relative to the unconfined compression strength of BBC would be facilitated by this process. The layout was chosen based on control of external (global) and internal (stress) limit states. This application of DMM was analyzed, designed and detailed to function as though it was a structural element, although discussion continues on whether it is a ground-improvement technique or a man-made underground structure.

Typical equipment necessary to install DMM is shown in Figure 2. The rig is outfitted with a system of inter-related shafts, each possessing discontinuous, alternating auger flights and mixing paddles. Soil-cement installations initiate from the ground surface. The penetration stroke is generally utilized to fluidize the soil-column in preparation for cement-mixing during the withdrawal stroke. The withdrawal stroke introduces the majority of a proportioned cement grout at the cutting head. The cutting head has the capacity to reduce the soils to less than 25.4mm particle sizes when RPM and advancement/withdrawal rates are optimized for the soils to be encountered. The withdrawal stroke creates the contiguous soil-cement element, mixing the fluidized soil-column with grout. Succesively advancing this multiple-shaft system along a line of progression that repenetrates the outside bores of adjacent soil-cement elements creates a continuous soil-cement shearwall. Figure 3 describes the advancement sequence necessary to create a shearwall. Excavation support may be expedited by installing steel wide flange shapes into the fluid soil-cement boreholes, which can be designed as soldier-piles. This aspect of DMM use is not covered as part of this paper.

A laboratory-study of BBC mixed homogeneously with varying amounts of grout found that an unconfined compressive strength (q_u) of 2.1MPa was reasonably achievable given the site conditions, available equipment, and preliminary treatment pattern geometric requirements. As for cement materials, Portland Cement Type II was specified, since the insitu soils possessed a moderate risk for sulfate attack. The design then proceeded based on the analytic assumption that soil-cement is an elastic, brittle material. For internal stability, limit states and their allowable stresses were then chosen to keep total material stresses within the elastic range.

The limit states and their allowable stresses received much attention during development of the project. Compression, tension, direct shear and diagonal tension were considered. A factor of safety of 2.0 for the long-term compression allowable stress, with a one-third allowable stress increase for short term (construction) cases was selected as the basic parameter, since compression-strength could be used as a primary quality assurance parameter.

Evaluation of the available literature regarding Japanese experience concluded that soil-cement installed via DMM has been designed similar to unreinforced concrete design practice in the

USA. The basic "long-term" allowable stresses for which the project was designed were as follows:

$$q_{ac} = 0.5q_u$$
$$q_{at} = q_{adt} = -0.05q_u$$
$$t_{ad} = 0.25q_u$$

where
q_u = specified minimum compressive strength of insitu soil-cement
q_{ac} = allowable compressive stress
q_{at} = allowable tensile stress
q_{adt} = allowable diagonal tension stress
t_{ad} = allowable direct shear stress

3.2 Model Development

A number of representative cross-sections of the tunnels were studied via a series of manual computations. Construction sequences for the tunnels were assumed to be similar to cut & cover construction, with the exception that the ground had been stabilized with soil-cement shearwalls. These shearwalls were studied to provide the requisite lateral and vertical stiffness to serve as permanent foundations for the tunnels.

Transverse to the tunnel alignment, a unit-width design strip was assumed. Loadings on an averaged soil-cement buttress were proportioned according to the layout geometry and influence width. The initial layout reflected the geometry of the future tunnels. Soil-cement shearwalls were then located to traverse the width of the tunnels, similar to timber ties beneath a railroad track. This layout geometry was then optimized based on evaluation of the external and internal stability analyses outlined below.

3.3 External Stability Analysis

The first phase of design focused on external stability. The limit states here were two-dimensional analyses of sliding and overturning. This process attempted to close-in on the treatment pattern utilizing a series of traditional "retaining-wall" calculations to estimate the length of the buttress required to create a reaction along the soil-cement/till interface which was within the middle-third. This length was then compared with the future tunnel network width requirements. It was discovered that the width required was generally well in excess of the tunnel geometry, since shearwalls need to be wider than they are tall for efficiency. As it turned out, the length of the buttress which was required provided another construction expedient - a 100% (see section 4.6) soil-mix cofferdam to install struts against during tunnel excavation sequences, as shown in Figure 4.

Conventional Japanese practice of utilizing active/passive conditions for external stability analysis and at-rest conditions for internal stability were generally followed, with the exception that external stability analysis assumed active/at-rest rather than active/passive conditions. This was prudent considering that large ground movements to mobilize passive resistance were contrary to mitigating construction and permanent lateral ground movements.

These stability analyses were performed assuming geostatic earth pressures proportioned to the shearwall geometry. The lateral earth pressure coefficients used for analysis appear below:

Fill:	$K_{ACTIVE} = 0.33$	
Organic Silt:	$K_{ACTIVE} = 0.40$	
Marine Clay:	$K_{ACTIVE} = 0.46$(drained)	Driving side
	$K_{AT\text{-}REST} = 0.75$(oc)	Resisting side

where: oc = over consolidated

Note that some relief over the $K_{AT\text{-}REST}$ condition was recognized for the short-term (construction) condition on the buttress-driving load. It was recognized that the construction-method would partially relieve the insitu $K_{AT\text{-}REST}$ pressures. These pressures were assumed to climb back to $K_{AT\text{-}REST}$ for the long-term condition. Stability against overturning and sliding were generally governed by a construction case (open excavation - minimum weight, minimum available base shear resistance).

3.4 Internal Stability Analysis

The next phase of design development then focused on internal stability, or stress limit states. Analysis of the soil-cement shearwalls was performed using GT-STRUDL, a microcomputer-based software program. The analysis required the nonlinear and Finite Element (FE) capabilities of GT-STRUDL; the till reaction was assumed to be a nonlinear, compression-only response. Two-dimensional elastic plane-stress elements were used as the basis of evaluating the behavior of the soil-cement shearwalls.

For output, GT-STRUDL allowed the visual presentation of graphical images to generate colored plots of direct and principal stresses. The analysis was then iterated-on each time the stress plots were interpreted. The design process included both "thickening" the FE mesh where stresses were excessive, and "thinning" where stresses were low.

These stability analyses were performed assuming geostatic earth pressures proportioned to the shearwall geometry. The lateral earth pressure coefficients used for analysis appear below:

Fill:	$K_{AT\text{-}REST} = 0.50$	
Organic Silt:	$K_{AT\text{-}REST} = 0.60$	
Marine Clay:	$K_{AT\text{-}REST} = 0.60$(nc), 0.75(oc)	Driving side
	$K_{AT\text{-}REST} = 0.75$(oc)	Resisting side

where: oc = over consolidated
nc = normally consolidated

Elastic properties for the FE analyses:

Unconfined compression strength, $q_u = 2.1$MPa
Shear strength, $S_u = 1.03$MPa
Poisson's Ratio, $v = 0.25$

An important parameter necessary to evaluate the elastic behavior of the buttress designs is the material property of Elastic Modulus, E. During design, insufficient information was available to determine the actual value as soil-mixing experience with BBC was limited. Borrowing from information obtained from a DMM research-related trip to Japan, it was found that typically the Elastic Modulus is in the range of $350q_u$-$1000q_u$. An assumed value of $300q_u$ was used, with the understanding that a lower bound would provide conservative estimates of displacements. There would be no effect on stresses by this presumption, since the elastic plane-stress FE analyses would preserve the relationship between strain and displacement.

3.5 Boundary Conditions

The glacial till layer was modeled in two directions: a) Vertically, bilinear-stiffness compression only springs based on subgrade modulus values proportioned to a load/displacement relationship were utilized. This bilinear stiffness approach allowed the soil-cement vertical base reactions to be redistributed as the till gets "softer" with increasing bearing pressure; b) Horizontally, the reaction of the soil-cement buttresses engaging the till was proportioned with elastic springs, at locations where the vertical springs demonstrated compression reactions. Since the overall loadings on the soil-cement buttresses have a calculated net overturning force, the buttresses analytically rotate, theoretically disengaging "tensile" springs, and activating "compression" springs, as in the design case of a rigid footing losing contact with soil due to overturning loads. This principal was extended to similarly release the horizontal reaction, given the lack of "confinement" by a bearing pressure. This is shown conceptually in the "base reaction" in Figure 4.

3.6 Interpretation and Optimization

In order to optimize the DMM treatment pattern, both an "open-cut" construction model and a "final-grade" FE model was necessary to study the distribution of material stresses. As patterns were changed to optimize stress distributions, external stability analyses were updated to assure that global stability and heave between shearwalls remained acceptable.

For construction, a unit buttress was assumed to consist of three (3) soil-cement shearwalls, or 3 rows of interlocked, ground treatment. This initial assumption was then called "38% DMM" (percent DMM is the amount of treated ground strips in proportion to the total area treated and untreated). The actual physical spacing of the buttresses (shearwalls) are governed by the auger-diameter of the DMM equipment. For the tunnel geometry and presumed lateral loadings, a shearwall layout of 38% (3 rows treated out of 8 rows possible) was necessary for "equivalent timber ties" vertically under the tunnels. Due to the analytical "rotation" of the buttresses, the shearwall base reaction resembles a classic retaining wall footing contact-pressure diagram. The pattern of ground treatment responds by increasing the coverage at the analogous "toe" to 100% (8 rows out of 8 rows), via a 63% (5 rows treated out of 8 rows possible) "reinforcing" between 38% and 100% coverage areas. The 100% zone serves both to dissipate stresses and provides a "heave" cut-off barrier.

4.0 Quality Assurance

Coupled with design, A Quality Assurance/Quality Control program was necessary to assure compliance with the intent of the design. Three important checks on construction are measured:

1) Unconfined compressive strength (q_u): A minimum average compressive strength was specified, and fluid samples of the soil-cement elements are taken at various depths within the element. A 56-day strength was specified rather than the typical 28-days, in order to reduce the cement dosages required to achieve the minimum strength. Similar to concrete practice, there is a 10% failure criterion assuming strengths lie in a normal distribution. Also, the cylinder strengths are tested on a stress-controlled device, on 152.4mmx304.8mm soil-cement cylinders, which have been moist-cured at 38 degrees Celsius. These curing conditions were assumed to be typical of insitu conditions, and are mirrored in the laboratory.

2) Verticality: The specifications require a 2.0% tolerance on verticality for each soil-cement element. Inclinometer readings are taken in one of the outside bores of the auger-shaft equipment at regular intervals. Verticality is used as a convenient measure of continuity in the shearwall. The real concern is loss-of-contact with depth in the ground of the shearwall, as boreholes may drift apart with depth. The auger-systems are very-stiff in the in-plane direction, but weaker in the out-of-plane direction. Repenetrating the outside auger-boreholes effectively eliminates in-plane drift from becoming a problem, but out-of-plane drift can lead to loss of continuity with depth.

3) Engagement of Till: In order to develop the lateral-load capacity of the shearwalls, they must engage the firm till stratum. The depth to the till layer was compiled into contour maps from the multitude of available boring logs. The plans call for a minimum depth drilled & mixed into the till of 0.3m, determined by drilling an additional 0.3m after a significant increase in penetration resistance is noticed by the rig operator on his instruments. Knowing the anticipated depth to the stratum was a necessary protocol since an obstruction encountered during the penetration stroke might falsely suggest encountering the firm stratum.

5.0 Design Implementation

Auger diameters were given a range on the drawings, so that a proprietary system would not be specified. This allowed maximum competition between bidders. The construction contract was estimated to involve the installation of 17,000 cubic meters of soil-cement, with land-based and water-based operations. The Engineers' Estimate was $152 million, and the contract was awarded to the low-bidder at $132 million. The second-lowest bid arrived at $165 million. Thus, the results of competition were readily apparent. Construction has been scheduled to span a total of 42 months. Installation of the DMM accounts for approximately 26 months of the contract work.

6.0 Conclusion

This project synthesized the resources of past practice and conventional wisdom with new technologies, to push the USA state of practice into accepting another viable design & construction expedient: Deep Soil-Cement Mixing. This technology can serve as both an excavation support system and as a permanent foundation for structures. The analytical and analogous assumptions which were crucial in advancing this application have been described.

Acknowledgements

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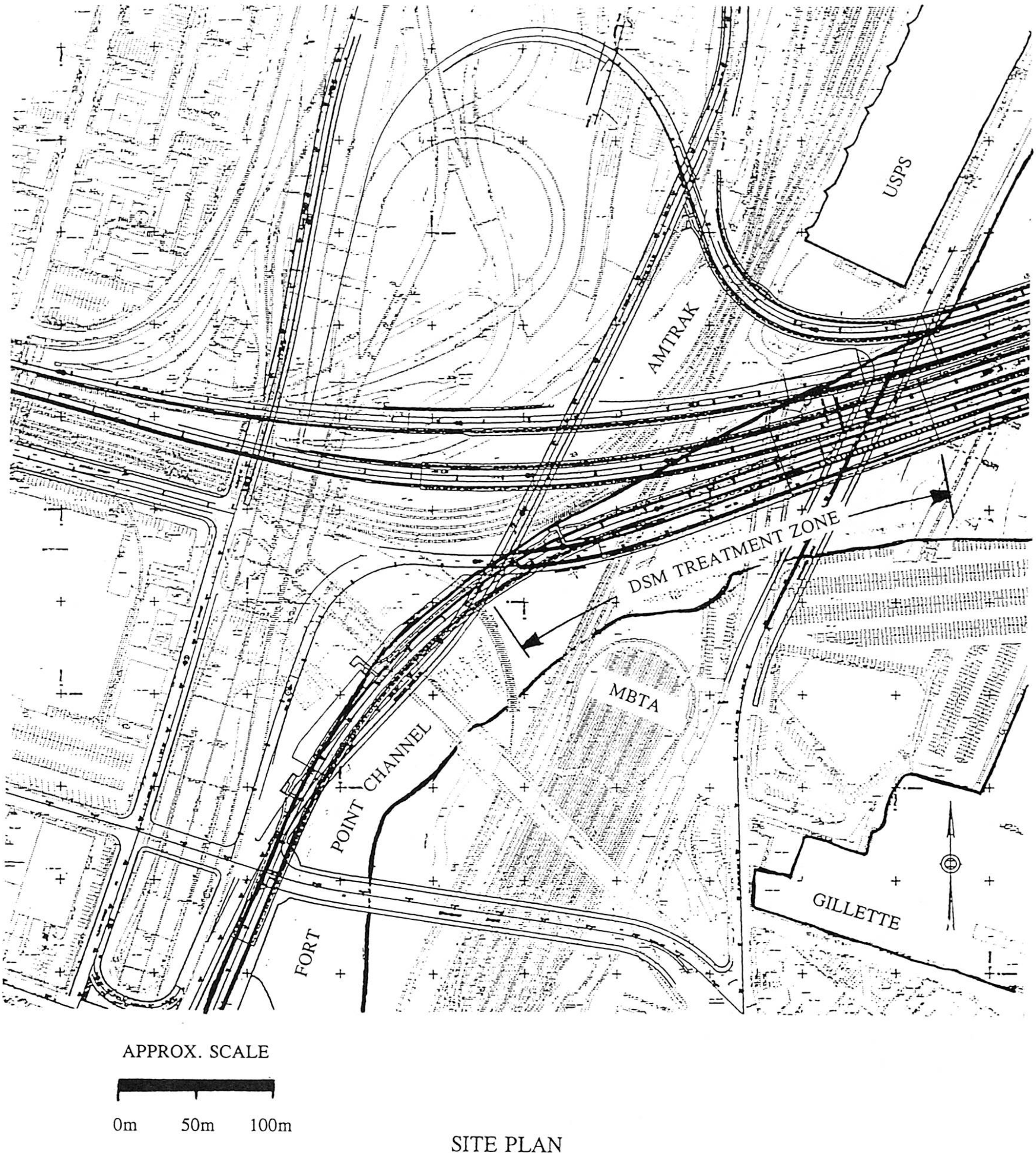
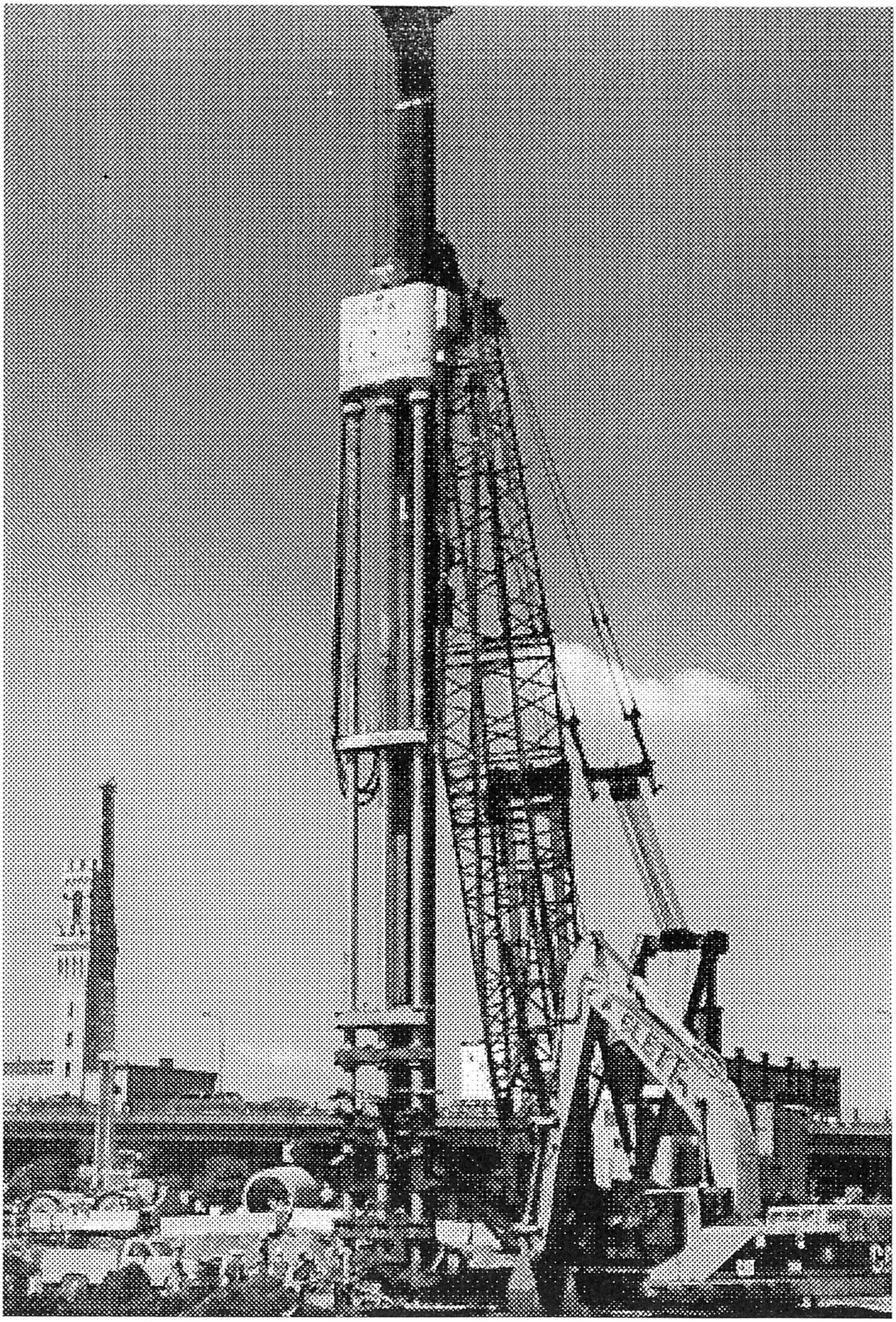
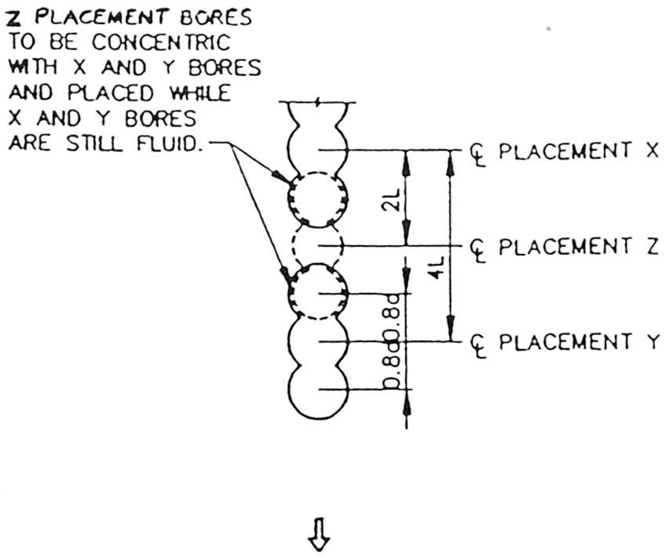


SITE PLAN
FIGURE 1



TYPICAL DSM EQUIPMENT
FIGURE 2



LONGITUDINAL PROGRESSION
MULTIPLE AUGER
(3 AUGER SYSTEM SHOWN)

REPENETRATION SEQUENCE

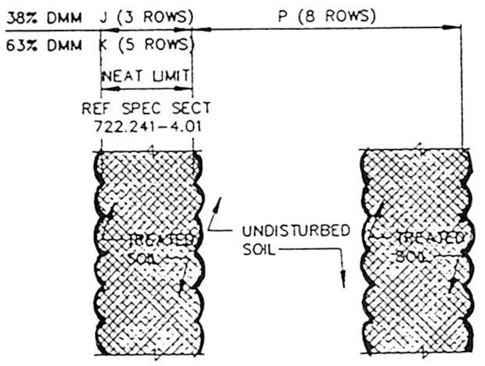


TABLE 1

AUGER CONFIGURATION	DIAMETER RANGE (FT–IN)	J	K	P	T	L	W1	W2
SINGLE	4'–0" ≤ d ≤ 6'–0"	2.3d	3.7d	5.8d	0.7d	0.5d	3.2d	2.8d
SINGLE	6'–0" < d ≤ 8'–0"	2.3d	3.9d	6.4d	0.8d	0.7d	3.8d	0.0d
DOUBLE	4'–0" ≤ d ≤ 6'–0"	2.3d	3.7d	5.8d	0.7d	0.5d	3.2d	2.8d
DOUBLE	6'–0" < d ≤ 8'–0"	2.3d	3.9d	6.4d	0.8d	0.7d	3.8d	0.0d
MULTIPLE •	2'–4" ≤ d ≤ 3'–6"	2.2d	3.8d	6.4d	0.8d	0.8d	2.8d	24'–2.8d

LEGEND:

d = DIAMETER OF THE SINGLE AUGER/MIXING PADDLE ASSEMBLY.

J = EFFECTIVE WIDTH FOR 38% COVERAGE AREAS.

K = EFFECTIVE WIDTH FOR 63% COVERAGE AREAS.

P = SPACING OF TREATED SOIL ROWS.

T = TRANSVERSE SPACING BETWEEN PRIMARY AND SECONDARY ROWS.

L = LONGITUDINAL SPACING BETWEEN ADJACENT AUGERS IN A ROW.

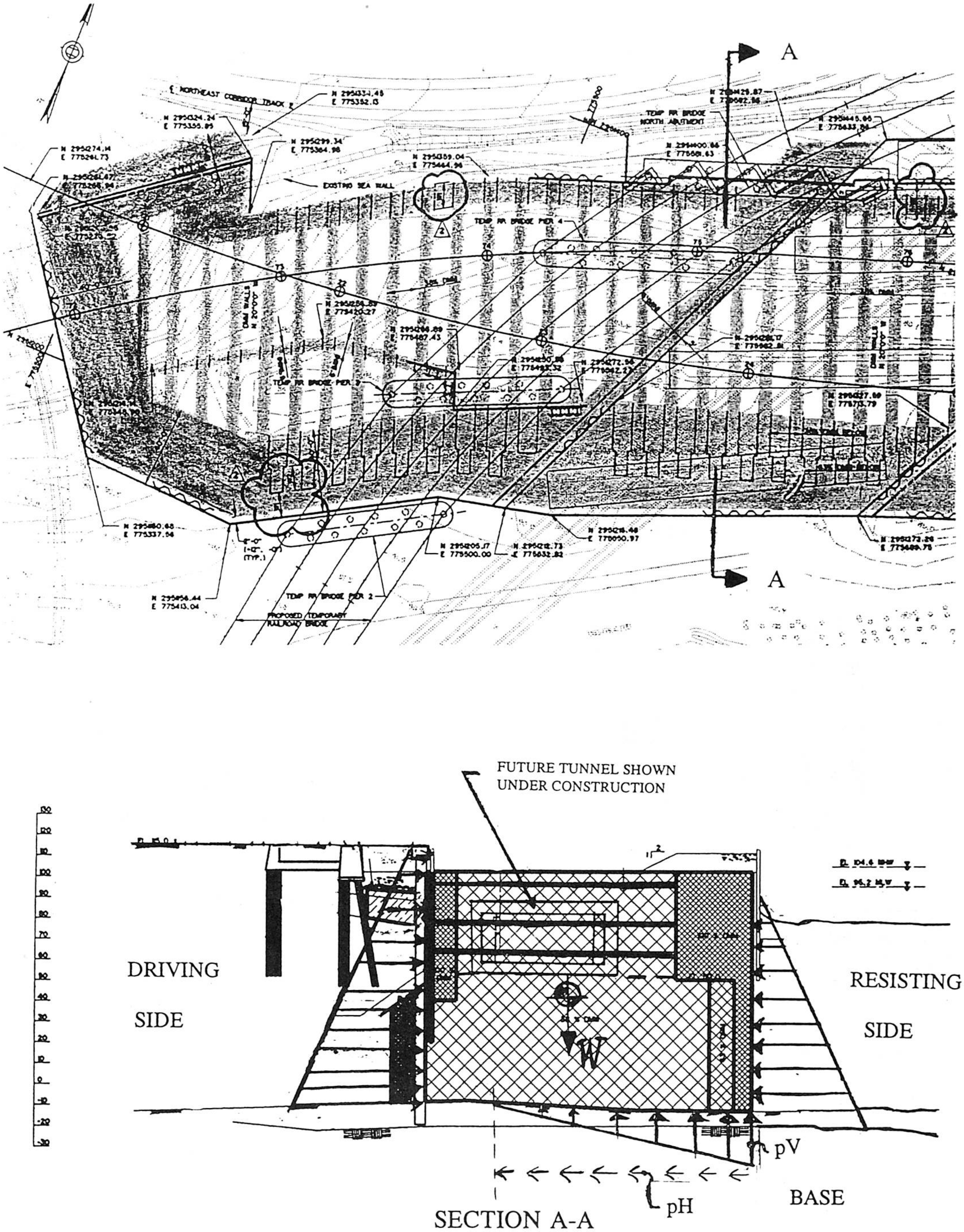
W1 = WIDTH OF 100% DMM TREATMENT AREA.

W2 = WIDTH OF 63% DMM TREATMENT AREA.

DETAIL 4
TRANSVERSE TREATMENT PROGRESSION FOR 38% AND 63% DMM COVERAGE AREAS

SHEARWALL LAYOUT DETAILS

DSM INSTALLATION DETAILS
FIGURE 3



TYPICAL PLAN & SECTION OF DSM
FIGURE 4