DESIGN OF THE BMIP BOAT SECTION

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Introduction

The Boston Marine Industrial Park (BMIP) Boat Section, which is part of contract C04A2 of the Central Artery/Tunnel (CA/T) project, is currently under construction in South Boston, Massachusetts (Fig. 1). The Boat Section is approximately 900 feet long and varies in width from about 135 to 185 feet (Fig. 2). The depth to the top of the Boat Section base slab varies from about 28 to 31 feet below final ground surface and about 23 to 26 feet below normal ground water levels. The Boat Section is supported on 578 anchors drilled into the underlying bedrock each designed for a working load of 630 kips.

Design and Construction Team

The owner of the project is the Massachusetts Highway Department (MHD). The owner's representative is the Management Consultant, Bechtel/Parsons Brinckerhoff. The section design consultant (SDC) consisted of HDR Engineering, Inc. with GEI Consultants, Inc. as the geotechnical subconsultant. The general contractor was the joint venture of Kiewit, Perini, Atkinson, and Cashman. The boat anchors were installed by the subcontractor Delta Geotechnical Services, Inc.

Subsurface Conditions

The subsurface conditions were investigated by Haley & Aldrich, Inc., the Area Geotechnical Consultant. The generalized subsurface profile is presented in Fig. 3. The soils encountered at the BMIP Boat Section generally consist of the following in order of increasing depth. The ground surface prior to construction was generally flat at about El. 110 feet (CA/T datum).

Fill - The fill deposit, consisting of granular, miscellaneous, and cohesive soil, was encountered from the ground surface to a depth of about 20 to 25 feet (El. 85 to 90).

Organics - The organic soil layer, consisting of organic silt and thin layers of peat, was encountered below the fill deposit. The organic soil layer varied in thickness from about 3 to 16 feet.

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Marine Clay - Marine clay (Boston Blue Clay), consisting of lean clay with varying amounts of silt and sand, was encountered at a depth of about 35 feet (El. 76 feet). The marine clay was overconsolidated (OCR of 4 to 6) and very stiff to hard at the top. The marine clay became increasingly softer with depth. The OCR decreased to about 1.5 or less at the bottom of the layer. The undrained shear strength, as measured by undisturbed unconsolidated triaxial tests, ranged from about 1000 to 2000 psf.

Glacial Soil Deposits - Glacial soil deposits were encountered below the marine clay at a depth of about 105 feet (El. 5). The glacial soils contained varying amounts of boulders, cobbles, gravel, sand, silt, and clay.

Bedrock - Bedrock, generally consisting of Argillite, was encountered below the glacial soils at about El. -5 to -25 feet (115 to 135 feet below ground surface). The characteristics of the Argillite varied widely from very soft to hard, very slightly to very severely weathered, and sound to extremely fractured. The Argillite was generally soft, very fractured, and severely weathered at the surface and improved with depth. The logs for several borings indicated that a partial and sometimes even a complete loss of drilling fluid occurred during rock coring. The inference from this observation is that the Argillite is sometimes extremely pervious because of open joints.

Preconstruction ground water levels in the upper aquifer (perched on marine clay layer) varied from a depth of about 4 to 8 feet (El. 102 to 106). The normal piezometric pressure head below the marine clay is just below El. 100 feet (mean sea level).

Design Issues

General

The BMIP Boat Section consists of a reinforced concrete base slab with integral walls. The Boat Section is designed for various combinations of at-rest soil pressures, ground water levels up to El. 111.5 feet (1000-year flood), surcharge pressures, heave pressure, and seismic loads. Under normal conditions the Boat Section is in a buoyant condition.

The buoyant forces are resisted by 578 passive anchors drilled into the bedrock. Each anchor was designed for a working load of 630 kips, the maximum anticipated load for any of the anchors within the Boat Section. The structural capacity of the anchor exceeds 630 kips due to stiffness requirements to control deflection. The boat anchors are also designed for compression loads due to dead weight of the Boat Section. The anchors are considered passive because they are not prestressed.

Original Prestressed Anchor Design

The original design was based on 1033 prestressed strand anchors. The design consisted of 22 strand anchors with a design load of 645 kips and 12 strand anchors with a design load of 350 kips. The 22 strand anchors were located along the edge of the boat slab to resist additional forces from rotation of the slab due to lateral loads on the boat wall. This "curling effect" also produced the maximum anchor loads in the passive anchor system.

The number of strands was based on stiffness and not strength requirements. The design required that each anchor would be prestressed to about 80% of the design load because an increase in prestress to design load would have resulted in larger heave pressures, as discussed below, and larger initial settlement of the boat slab. Each anchor was to be proof or performance tested to 133 percent of the design load.

Tiedown "blanks," consisting of trumpets and anchor plates, were to be installed midway between each anchor in the original design. The purpose of the blanks was to facilitate the future installation of replacement anchors in the event of failure or long-term deterioration of the original anchors. The design included instrumentation to monitor the long-term performance of the anchorage system.

Due to the unfavorable results of the waterproofing results of the Tension Element Testing Program (TETP), the MHD decided to entertain proposals from the contractor to use a passive anchorage system that did not require penetrations through the boat slab. The contractor's proposal for a multi-bar, passive anchorage system was subsequently adopted, and the Boat Section was redesigned by HDR and GEI.

Structural Analysis of Boat Section

The analysis of the Boat Section was performed using GTStrudl on 2-D frame models of the structure in both the transverse and longitudinal directions. The transverse analyses, which were used for the primary design of the Boat Section anchors and reinforcing, were selected at set intervals along the length of the section and where the structural configuration varied from normal (such as in the end portal areas and the storm water pump station). The longitudinal analysis was performed on the Boat Section and adjacent sections of tunnel to determine both the longitudinal concrete reinforcing requirements and the global deflection characteristics.

In the transverse analyses, the rock anchors were modeled as non-linear springs with a stiffness equal to the rock anchor stiffness in tension and the effective contributory area of soil stiffness in compression. The model was analyzed for various combinations of loading including the primary upward loads due to ground water and heave to obtain the worst possible condition. The number and stiffness of anchors were varied to handle the anticipated forces with acceptable deflections and anchor loads.

The Boat Section base slab was designed to accommodate the anchor reactions and the shearing forces these reactions caused. The steel bearing plates, which were used to transfer and distribute the anchor loads within the base slab, were encased in reinforcing cages capable of resisting the concentrated loads due to both maximum anticipated tensile loads (pull-out) and compressive loads (punching).

Heave Pressure

The heave pressure is the swelling pressure of the marine clay that reacts against the bottom of the boat slab, which is restrained by the anchors. The heave pressure occurs due to the decrease in effective soil stress in the marine clay after the structure is completed and the ground water recovers to normal level.

The effective soil stress in the marine clay was computed for each of the following construction stages:

- Stage I - Excavation and dewatering. Excavation causes a decrease in effective stress, and dewatering causes an increase in effective stress. The net effect is swelling of the marine clay due to a significant decrease in effective stress.

- Stage II - Construction of base slab and installation of anchors. The construction of the base slab causes an increase in effective stress and settlement.

- Stage III - Construction of boat walls. The addition of the walls and other loads causes further increase in effective stress and slight increase in settlement.

- Stage IV - Backfill and recovery of ground water level. The net effect of backfill and ground water recovery is a decrease in effective stress and swelling of the marine clay.

The effective soil stress is difficult to estimate because it is a function of the percent consolidation that occurs during each construction stage. Consolidation rate is highly dependent on the permeability of the marine clay. The percent consolidation is also dependent on the time to complete each construction stage that is also indeterminate at the design stage. Therefore, it is necessary to bound the potential effective stresses for each construction stage. These bounded estimates can vary significantly.

The maximum potential heave pressure is the difference between the effective pressure at Stage II and Stage IV. However, the maximum potential heave pressure would only occur if the anchors were infinitely stiff compared with the stiffness of the soil. The heave pressure is not a following load and thus decreases with increasing deflection of the boat anchors. The stiffness of the marine clay was estimated based on consolidation theory using the estimated effective stresses.

The Boat Section design was based on a linearly varying heave pressure equal to 1000 psf at zero deflection and equal to 0 psf at 2.1 inches of deflection. The Boat Section analyses were iterated until a balance existed between the heave pressure used and the deflections obtained. Generally, the heave pressure was slightly lower at the edges of the boat due to increased deflections from boat slab rotation. For the majority of the design sections, the design heave pressure was approximately equal to 500 psf.

Details of Boat Anchors

The main components of the boat anchors are shown in Fig. 4. Each boat anchor consists of seven 1-3/8-inch-diameter, high strength steel (150 ksi), deformed threadbars. The bars and anchor hardware were epoxy-coated for corrosion protection to a depth of 10 feet below the boat slab. The bars were splayed apart and attached to a bearing plate embedded 4 feet into the base slab. These anchors were not prestressed. Load testing was performed on six test anchors as discussed below.

The anchor capacity was achieved in rock. The design was based on a side friction between the rock and anchor grout of 55 psi. Based on a 12-inch-diameter rock socket, the minimum bond length into sound rock was 25.3 feet.

The double corrosion protection consisted of a watertight, corrugated PVC encapsulation and the interior grout. Epoxy coating at the top of the anchor provided additional protection. The PVC encapsulation extended 12 inches into the base slab. A waterproofing plug, consisting of hydrophilic sealant, was installed in the top of the PVC encapsulation. Special details were developed to provide a waterproof seal between the PVC encapsulation and the boat slab membrane waterproofing.

Construction Issues

Load Testing Program

A total of six load tests were performed on test anchors installed from the ground surface around the perimeter of the Boat Section (refer to Fig. 2). The purpose of the load tests was to confirm the design side friction (rock to grout bond stress).

The test anchors were incrementally loaded and unloaded up to 200% of the design load. The criteria for a successful load test were:

1. Successfully hold 200% of design load.

2. Creep rate of less than 0.08 inch per log cycle of time at 150% of the design load.

3. Elastic elongation of at least 90% of the theoretical elastic elongation of the unbonded length. The contract documents required the unbonded length of the test anchors to extend to the top of rock to assure that the test load was applied to the rock socket.

The results of the load tests were as follows:

- For test anchors S-1, S-2, and S-3, the elastic elongation of the anchor was less than 90% of the theoretical elastic elongation of the unbonded length. Although the anchors did not fail, the results indicated that the full test load was not applied to the rock bond zone.

- Grout failure occurred in test anchors S-4 and N-1 due to dilution of the grout in the bond zone during flushing. The contractor attempted to flush away the internal grout above the rock socket to debond the bars to the top of rock. Apparently, dilution of the internal grout below the flush tube occurred because of holes in the corrugated encapsulation. The integrity of the corrugated encapsulation was not considered important for the test anchors prior to identifying this problem.

- Test anchor N-2 successfully passed the load test.

- Test anchors S-4 and N-1 were drilled out, reinstalled, and retested. The unbonded length was achieved by encapsulating the individual bars in a plastic sheath to the rock socket. Both test anchors then met the acceptance criteria.

- Anchor S-1 was overcored with a 12-inch-diameter core barrel to a depth of 120 feet (20 feet above the top of rock). The anchor was retested and successfully met the acceptance criteria.

- The contractor attempted to overcore test anchor S-3 but was not successful.

The complete load test results are summarized in the accompanying paper entitled "Tied-down Boat Section Construction," by R. Micciche.

Criteria for Production Anchors

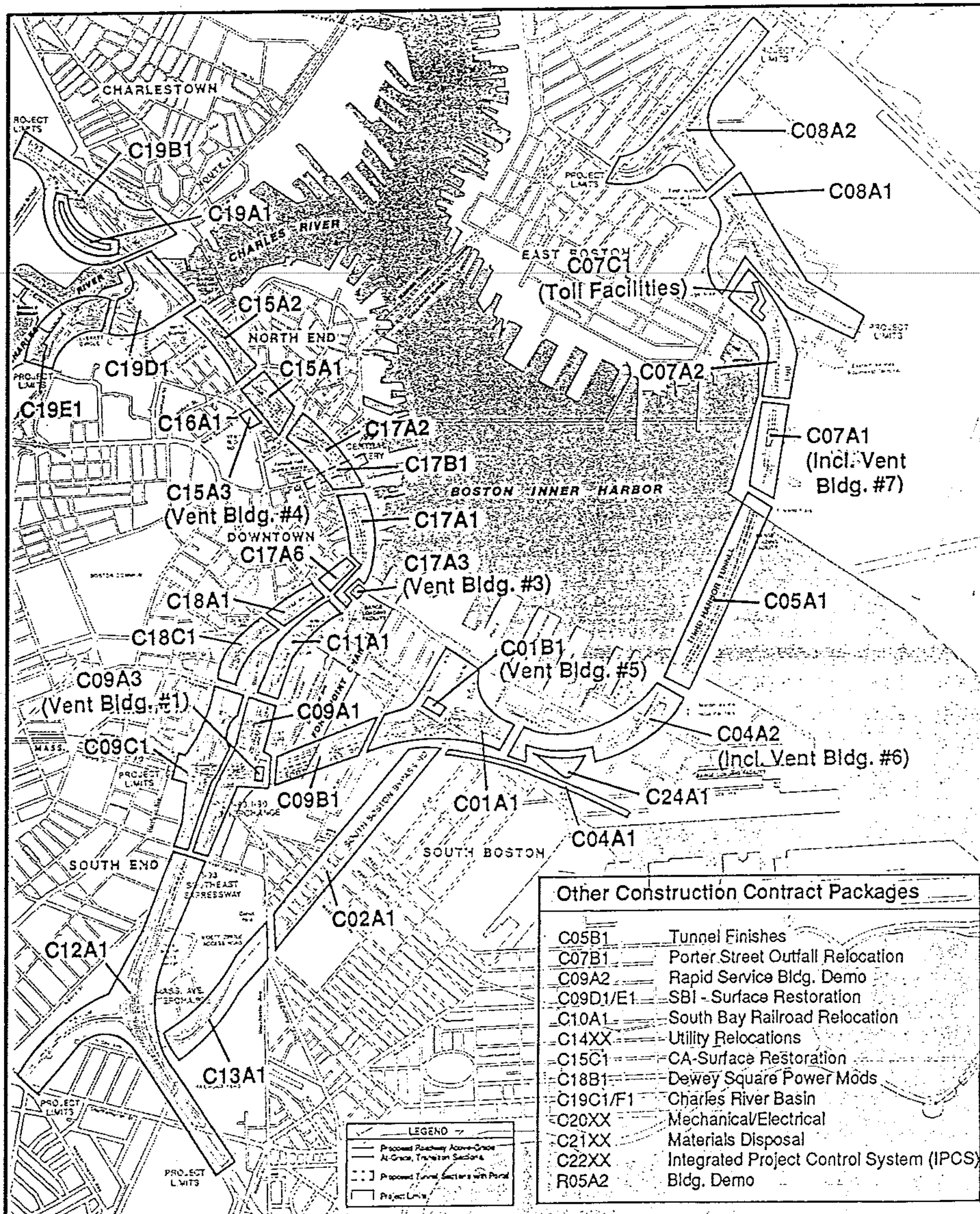
Standard practice regarding installation of production anchors is to use identical procedures and criteria as used on the test anchors. However, unlike the test anchors, the production anchors were installed from inside of the excavation under artesian conditions.

The net artesian head was up to 30 feet based on the deepest excavation subgrade at El. 68 feet and the preconstruction piezometric pressure in the rock at about El. 98 feet. The contract documents included strict limits on lowering of the ground water level to protect adjacent structures and utilities; therefore, opportunity to lower the piezometric pressure was limited. However, an adequate level of grout head could be developed from the excavation subgrade to overcome the artesian pressure.

To verify that the grout in the rock bond zone was not eroded by artesian flow, the following criteria were included in the specifications:

- Provide a temporary casing seated tightly into rock.

- Pump grout through tremie tube located at the bottom of the drill hole.
- Tremie grout until good quality grout (no dilution from ground water or soil) is returned at the ground surface.
- Maintain grout level at the cut-off elevation until the grout level stabilizes.



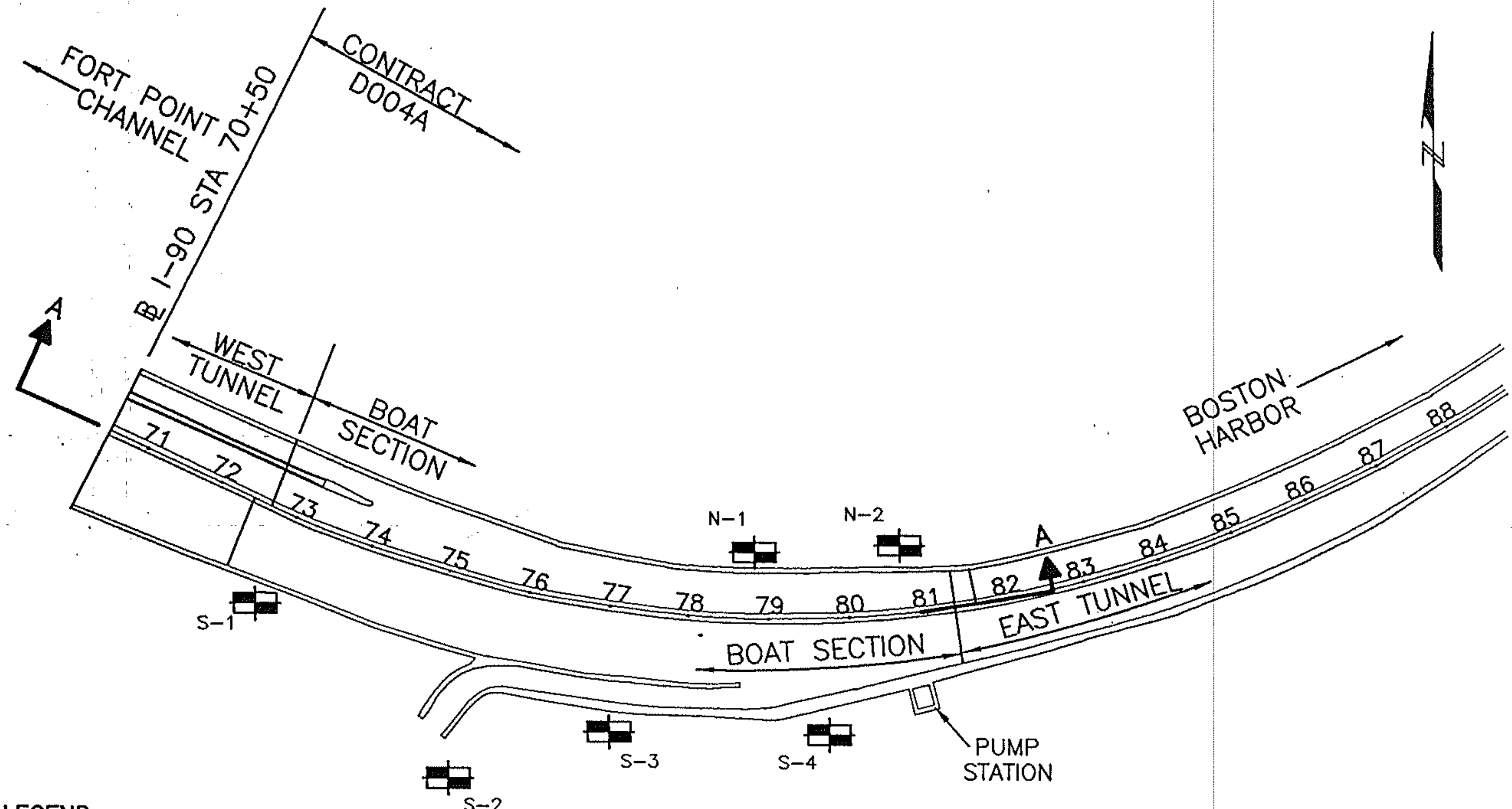
Other Construction Contract Packages	
C05B1	Tunnel Finishes
C07B1	Porter Street Outfall Relocation
C09A2	Rapid Service Bldg. Demo
C09D1/E1	SBI - Surface Restoration
C10A1	South Bay Railroad Relocation
C14XX	Utility Relocations
C15C1	CA-Surface Restoration
C18B1	Dewey Square Power Mods
C19C1/F1	Charles River Basin
C20XX	Mechanical/Electrical
C21XX	Materials Disposal
C22XX	Integrated Project Control System (IPCS)
R05A2	Bldg. Demo

BMIP Tunnel
Central Artery/Tunnel
Contract No. C04A2
South Boston, Massachusetts

CA/T CONSTRUCTION CONTRACT PACKAGES

FIG. 1

CATFIG3 8/23/94 EVL



LEGEND

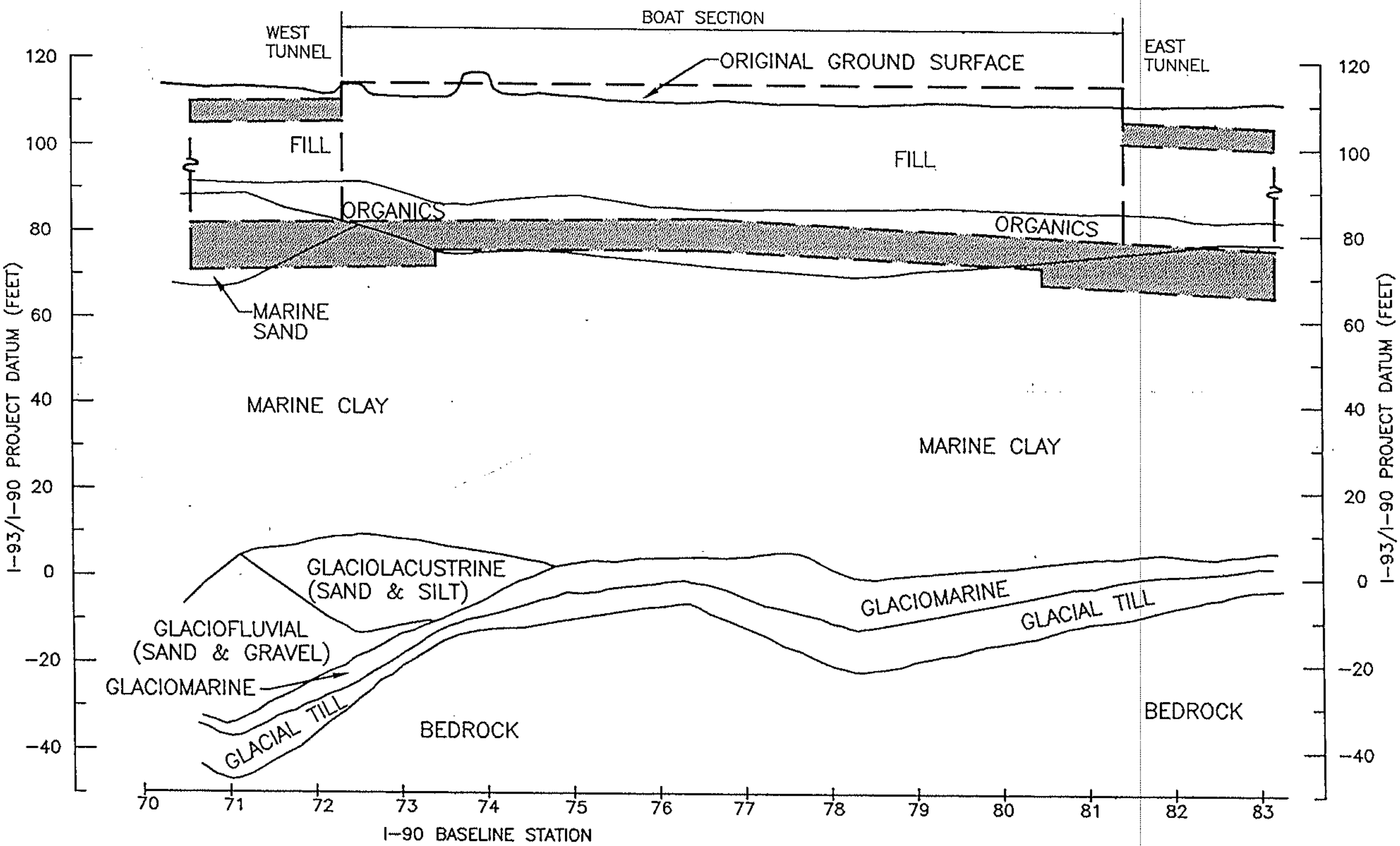
TEST ANCHOR LOCATION

NOTES

1. ALL STATIONS REFER I-90 BASELINE
2. REFER TO FIG. No. 3 FOR SECTION A-A

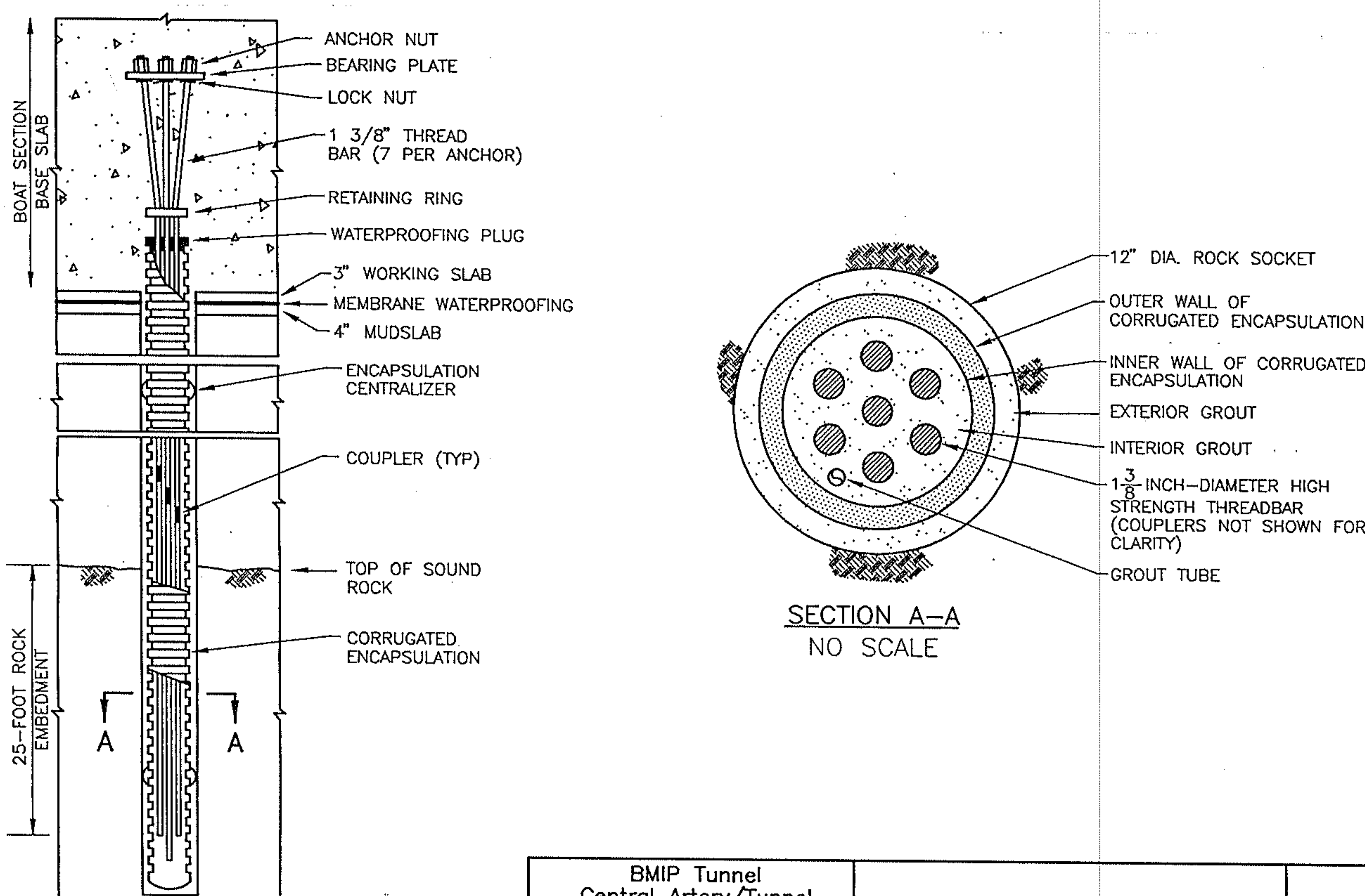
BMIP Tunnel Central Artery/Tunnel Contract No. C04A2 South Boston, Massachusetts	SITE PLAN	FIG. 2

CATFIG3 9/1/94 CAR



BMIP Tunnel Central Artery/Tunnel Contract No. C04A2 South Boston, Massachusetts	BOAT SECTION SUBSURFACE PROFILE	FIG. 3

CATFIG3 9/1/94 CAR



BMIP Tunnel Central Artery/Tunnel Contract No. C04A2 South Boston, Massachusetts	BOAT ANCHOR DETAILS	FIG. 4

CATFIG3 9/1/94 CAR