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Town Brook Local Protection Project

Quincy and Braintree, Massachusetts

Design Memorandum No. 4
Tunnel, Conduits and Shafts

Site Geology, Embankments and Foundations and Detailed Design of Structures

September 1986

US Army Corps
of Engineers
New England Division

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254-9149

REPLY TO
ATTENTION OF

NEDED-G

15 October 1986

SUBJECT: Town Brook Local Protection Project, Quincy and Braintree, Massachusetts, Design Memorandum No. 4, Tunnel, Conduits and Shafts, Site Geology, Embankments and Foundations and Detailed Design of Structures

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In accordance with ER 1110-2-1150, dated November 1984, Design Memorandum No. 4, "Tunnel, Conduits and Shafts" for the Town Brook Local Protection Project in Quincy and Braintree, Massachusetts is submitted for review and approval.

FOR THE COMMANDER:

Encl (10 cys)

RICHARD D. REARDON
Chief, Engineering Division

WATER RESOURCES DEVELOPMENT PROJECT

TOWN BROOK LOCAL PROTECTION PROJECT
MASSACHUSETTS COASTAL STREAMS
QUINCY AND BRAINTREE, MASSACHUSETTS

DESIGN MEMORANDA INDEX

Number	Title	Anticipated Submission Date	Date Submitted	Date Approved
1.	General Design and Hydrology		Feb 85	April 85
2.	Quincy Reservoir Dam Embankment and Foundations, Site Geology and Detailed Design of Structures	Nov 86		
3.	Concrete Materials		Jul 85	Aug 85
4.	Tunnel, Conduits and Shafts Site Geology, Embankments and Foundations and Detailed Design of Structures		Sep 86	
5.	Hydraulics and Water Quality		Jul 85	May 86

WATER RESOURCES DEVELOPMENT PROJECT

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MASSACHUSETTS COASTAL STREAMS
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DESIGN MEMORANDUM NO. 4
TUNNEL, CONDUITS AND SHAFTS
SITE GEOLOGY, EMBANKMENTS AND FOUNDATIONS AND DETAILED DESIGN OF STRUCTURES

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASSACHUSETTS

SEPTEMBER 1986

WATER RESOURCES DEVELOPMENT PROJECT

TOWN BROOK LOCAL PROTECTION PROJECT
MASSACHUSETTS COASTAL STREAMS
QUINCY AND BRAINTREE, MASSACHUSETTS

DESIGN MEMORANDUM NO. 4

TUNNEL, CONDUITS AND SHAFTS
SITE GEOLOGY, EMBANKMENTS AND
FOUNDATIONS AND DETAILED DESIGN OF STRUCTURES

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WATER RESOURCES DEVELOPMENT PROJECT

TOWN BROOK LOCAL PROTECTION PROJECT
MASSACHUSETTS COASTAL STREAMS
QUINCY AND BRAINTREE, MASSACHUSETTS

DESIGN MEMORANDUM NO. 4

TUNNEL, CONDUITS AND SHAFTS
SITE GEOLOGY, EMBANKMENTS AND
FOUNDATIONS AND DETAILED DESIGN OF STRUCTURES

A. PERTINENT DATA

PURPOSE	Flood Control
LOCATION	
State	Massachusetts
County	Norfolk
Town	Quincy and Braintree
River	Town Brook and Town River
TOWN BROOK DRAINAGE AREAS	
Town River at Town River Bay	4.74 square miles
Town Brook at USGS Gage	4.46 square miles
Town Brook at Tunnel Inlet	4.04 square miles (1)

(1) Includes area of 1.56 square miles controlled by Old Quincy Reservoir

RECORD OF MAJOR FLOODS

Year	Month	Peak Discharge (2)
1955	August	720 cfs (3)
1968	March	420 cfs (3)

(2) At location of present USGS gage
(3) Computed discharge

DESIGN FLOOD (4)

1% Chance Flood (100-year)
Rainfall: 4.8 inches/6 hours
Discharge: 1,200 cfs

(4) At Tunnel Entrance

TUNNEL

Inside Diameter	12'-6" with lining
Length in Granite	2560'
Length in Argillite	1500'
Total Length	4060'
Average Depth	180'
Lining Material	Concrete

INLET SHAFT

Inside Diameter	12'-6"
Total Depth	145'
Depth to Rock	70' after site development
Lining Material	Concrete

OUTLET SHAFT

Inside Diameter	12'-6"
Total Depth	170'
Depth to Rock	61'
Lining Material	Concrete

OUTLET CHANNEL

Width	60'
Length	300'
Wall Height	6'-0"
Walls	Stone-filled gabions
Bottom Lining	9" stone-filled gabion mattress

TOWN RIVER CHANNEL UPSTREAM OF SOUTHERN ARTERY CULVERT

Bottom Width	52'
Average Depth	4'
Side Slopes	1 vertical on 3 horizontal
Lining	9" stone-filled gabion mattress

SOUTHERN ARTERY CULVERT

Number of Barrels	3
Barrel Size	16' x 7'
Length	280'

TOWN RIVER CHANNEL DOWNSTREAM OF SOUTHERN ARTERY CULVERT

Bottom Width	Varies from 45' min to 100' max
Average Depth	13'
Side Slopes	Right bank: vertical Left bank: 1 vertical on 3 horizontal
Lining	Right bank: none Left bank: 12" stone-filled gabions

B. INTRODUCTION

1. PURPOSE. The purpose of this memorandum is to present for review and approval the geotechnical and structural designs for the tunnel, shafts, intake and outlet structures, and the outlet channel of the Town Brook Local Protection Project, Quincy, Massachusetts.

2. SCOPE. This memorandum contains the geotechnical and structural aspects of the design and construction. Basic criteria, loading assumptions, typical computations, typical logs of explorations, laboratory data, definitive plans and other pertinent data are presented. The geotechnical aspects and designs of structures for the Quincy Reservoir dam are contained in Design Memorandum No. 2.

3. DESCRIPTION OF PROJECT. The purpose of the project is to alleviate flooding of Town Brook. The main components of the project will be the construction of a relief tunnel to convey excess stormwater flows underneath the city of Quincy, the Town River culvert and channel improvements. The project also includes improvements to Old Quincy Reservoir in the town of Braintree, Massachusetts. Plate 4-1 shows the general plan.

4. DEVIATIONS FROM PREVIOUS MEMORANDA. The tunnel design contained in this memorandum varies from the General Memorandum in several ways as summarized below.

Previously, the tunnel in profile appeared as a single path that steepened in slope between stations 32+00 to 41+00 for both conventional drill and blast excavation and tunnel boring machine (TBM) excavation (Plate 4-2). This is the most direct and economical route for the conventional excavation option. However, the most direct route for a TBM is a straight line. Although a deeper intake shaft is required, the risk and expense of changing the path of the TBM are eliminated.

A design option of tunnel boring machine excavation with a precast liner is introduced and cost presented. This option was not presented in the GDM due to the large initial investment for a TBM associated with a relative short length of tunnel.

It has been determined that discharge of anaerobic waters from the tunnel can be expected without inclusion of appropriate water quality improvement measures. A mechanical aeration/circulation scheme has been developed to provide acceptable discharge water quality (See DM #5, Hydraulics and Water Quality). Two 6-inch diameter pipes which are a part of this scheme will be attached to the liner on the inside of the tunnel. The tunnel and shaft diameter is conservatively increased from 12.0 feet to 12.5 feet to compensate for additional hydraulic losses caused by the circulation pipes. Previous design presented in DM No. 5 showed the circulation pipes embedded in the concrete liner of a 12 foot

diameter tunnel. Additional hydraulic analysis will be conducted to determine if acceptable hydraulic performance and water quality conditions can be maintained using the previous 12.0 ft diameter tunnel.

The biological (aquatic life) studies identified in Design Memorandum No. 5 were not undertaken. These studies were proposed to quantify potential impacts associated with anaerobic and dissolved supersaturation discharges from the tunnel for use in designing the water quality improvement scheme. The proposed engineering solution to the potential impacts is an aeration/circulation system which eliminates both problems. Therefore, these biological studies were no longer deemed necessary.

5. ACKNOWLEDGMENTS. This design memorandum incorporates portions of studies and detail designs done for the Metropolitan District Commission by the firm Metcalf and Eddy, Inc. Their reports to the M.D.C. included:

a. Preliminary Design Report on the Town River Culvert Design and Channel Improvements, Quincy, Massachusetts, January 1984.

b. Preliminary Design Report, Part I, Tunnels and Shafts, Site Geology, Foundations, Quincy, Massachusetts, March 1984.

c. Preliminary Design Report, Part II, Intake and Outlet Structures, Hydraulics, Functional Design, Subsurface Conditions, Design Data, May 1984.

These reports were fully utilized for this memorandum. The data and designs were checked and revised as needed to meet Corps criteria.

6. TOWN BROOK RELIEF TUNNEL.

a. General: The completed tunnel will have a minimum internal diameter of 12.5 feet. The tunnel will extend northeasterly for about 4060 feet from a point just north of School Street to a point about 300 feet north of Washington Street as seen on Plate 4-1.

b. Construction Alternates: Three tunnel lining systems will be included in the contract documents with bidding schedules for each.

The alternates will be as follows:

(1) Cast-in-place concrete liner with excavation by drill and blast.

(2) Cast-in-place concrete liner with excavation by tunnel boring machine.

(3) Precast concrete liner with excavation by tunnel boring machine.

The least expensive of the alternates will be selected.

7. TUNNEL HEADWORKS. Flows to the relief tunnel will be from the Town Brook itself and from the Burgin Parkway Culvert which has been constructed in conjunction with the Burgin Parkway. The Burgin Parkway culvert is not part of the Corps' project and has been constructed by others.

a. Town Brook. The Town Brook system will comprise the existing culvert in Brook Road and proposed Brook Road Diversion Conduit to be constructed by others (see Plate 4-1). Downstream of Fort Square, a proposed gabion-walled channel (non-Corps) and then a concrete channel (Corps) will extend to an overflow structure (Corps). See Plate 4-13. Diverted flows of the overflow structure will be directed to the tunnel intake shaft.

b. Burgin Parkway Culvert. The Burgin Parkway Culvert, a concrete box culvert extending about 4000 feet from Centre Street, is to convey to the tunnel stormwater overflows from Town Brook from areas upstream of Centre Street and runoff entering the culvert between Centre Street and School Street. At the tunnel intake site, the culvert will be connected to a forebay from which the flow will pass into two grit deposit basins. The grit basin is part of the Corps' project. In exiting the basins, flows will pass into the tunnel shaft.

8. HYDRAULIC ANALYSIS. Hydraulic analyses were performed to determine the size of the Town Brook conduit sections required to convey the design discharges. Because of the complex hydraulics of flow, involving combinations of open channel, closed channel, pipe, culvert, overland, and roadway weir flow, efforts to compute backwater profiles using available computer programs were not completely successful. Profiles were mostly computed by hand calculation using step backwater procedures generally in accordance with EM 1110-21-1409, "Backwater Curves in River Channels." Manning's "n "values ranged from 0.012 for smooth concrete to 0.075 for rock channels and overland flow.

Discharge capacity of the planned tunnel was computed by Manning's equation with an assumed Manning's "n" roughness coefficient of 0.012. Entrance and exit loss coefficients are 1.0.

Detailed hydraulic design of the tunnel and the improvements to Old Quincy Reservoir was accomplished by Metcalf and Eddy, Inc. (M&E) for the Commonwealth of Massachusetts, Metropolitan District Commission. The New England Division reviewed the final product for conformance with Corps technical and policy criteria. The hydraulic design resulting from that evaluation is presented in Design Memorandum No. 5, "Hydraulics and Water Quality."

The addition of two 6-inch diameter circulation pipes attached to the insides of the tunnel causes increased hydraulic losses due to increased friction and slightly reduced cross-sectional area. Tunnel diameter was increased to offset these losses. (Previously, a tunnel design was planned in which the 6-inch diameter pipes would be embedded in the concrete liner of a 12 ft diameter tunnel.) Friction losses for the tunnel plus circulation pipes were computed using Manning's equation and a composite Manning's "n". This composite "n" was conservatively computed by proportioning the "n" for the tunnel walls and the "n" for the flushing pipes to the size of their wetted perimeters. For the tunnel walls, the "n" of 0.012 presented in DM No. 5 was used. For the circulation pipe, an "n" of 0.018 was used to include the friction along the outside of the pipe and the effect of the brackets holding the pipes to the tunnel wall. This analysis showed that a tunnel with a 12 1/2-foot diameter and two 6-inch circulation pipes attached to the walls would have no greater hydraulic losses during the peak 100-year flow than a 12-foot diameter tunnel with circulation pipes embedded in the tunnel liner as described in DM No. 5. Further hydraulic analysis including coordination with WES to develop an accepted less conservative method for assessing the increased hydraulic losses associated with the 6-inch diameter circulation pipes and evaluation of increased tunnel head loss on upstream water surface profiles will be conducted to determine if an acceptable tunnel circulation scheme can be developed using a 12.0 ft diameter tunnel.

9. WATER QUALITY CONTROL. Town Brook and Town River have been assigned objective water quality classifications of Class B and Class SB by the Massachusetts Water Resources Commission. Both Class B and Class SB waters are suitable for the propagation and protection of fish, wildlife and aquatic life and for primary and secondary recreation; Class SB waters also support shellfish harvesting with depuration. Both streams are located in a very heavily urbanized area of eastern Massachusetts. As such, conditions in the streams are greatly influenced by man's activities particularly through nonpoint sources of pollution. Urban stormwater runoff can seriously degrade water quality by introducing a myriad of contaminants. Consequently, water quality conditions in Town Brook and Town River marginally meet the requirements for Class B and SB waters.

10. TUNNEL AERATION PLAN. The Town Brook relief tunnel has the potential to discharge anaerobic waters with possibly high hydrogen sulfide levels about three times per year. However, a combination of circulation and mechanical aeration included in the tunnel design will prevent these problems from occurring. Water Quality Controls are presented in DM No. 5, "Hydraulic Analysis and Water Quality."

The water quality improvement scheme accomplishes aeration of the tunnel outlet shaft and mechanical mixing of the tunnel's contents (Fig. 1). The required two 6-inch diameter circulation pipes will be bolted to the inside of the tunnel lining (Plate 4-9). This method will allow easier installation and maintenance than if the pipes were inside the lining. The tunnel inside diameter was conservatively enlarged from 12.0

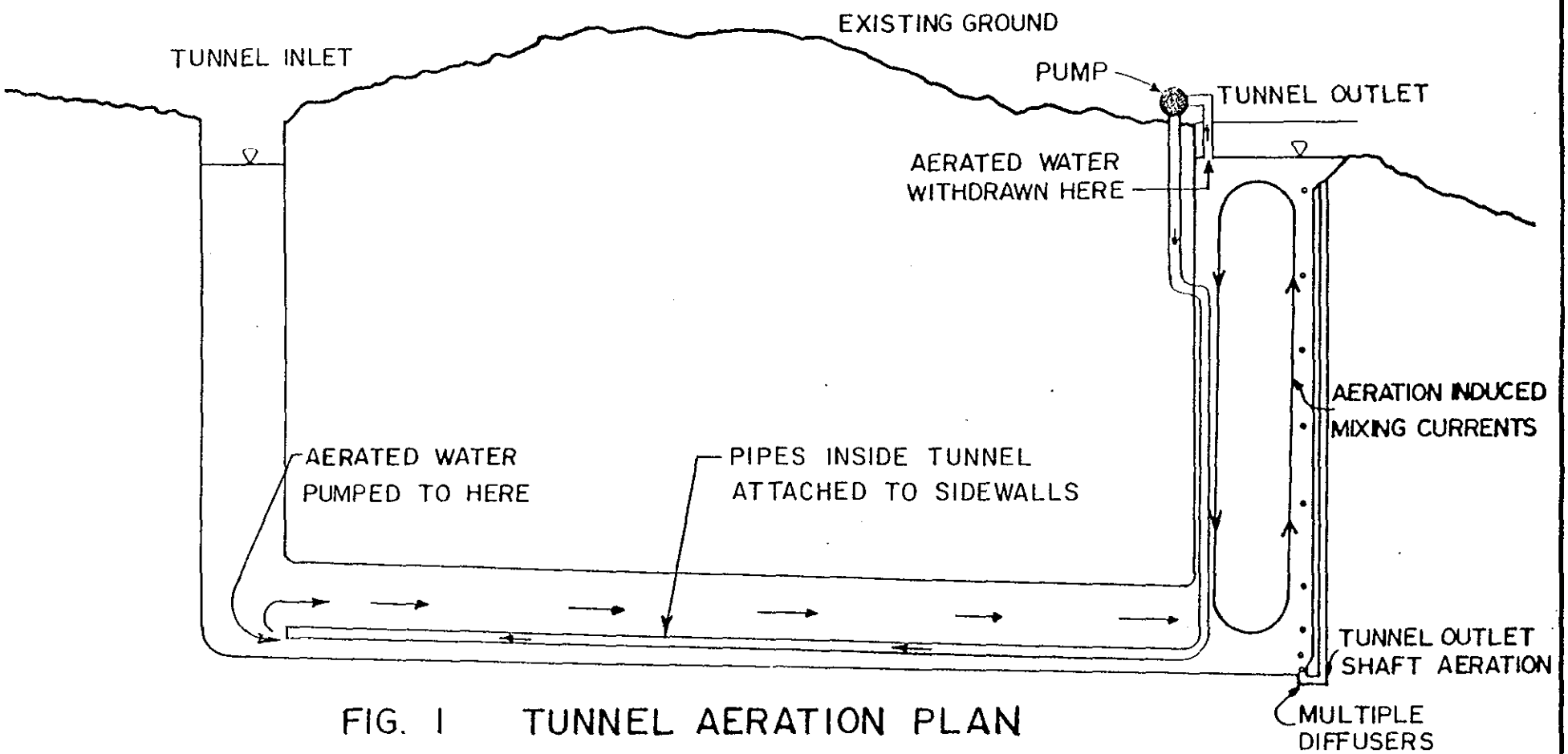


FIG. 1 TUNNEL AERATION PLAN

NO SCALE

feet to 12.5 feet to account for hydraulic losses created by the pipes. Additional hydraulic investigations will be conducted to determine if acceptable water quality conditions and hydraulic performance can be maintained using the previous 12.0 ft diameter tunnel. A 10 ft x 10 ft pre-engineered metal building will be constructed adjacent to the outlet structure to house a pump and compressor for the aeration/circulation system.

The biological (aquatic life) studies identified in Design Memorandum No. 5 were not undertaken. These studies were proposed to quantify potential impacts associated with anaerobic and supersaturated dissolved gas discharges from the tunnel for use in designing the water quality improvement scheme. The proposed engineering solution to the potential impacts is an aeration/circulation system which eliminates both problems. Therefore, these biological studies were no longer deemed necessary.

11. OUTLET STRUCTURES. The outlet structure located on Plate 4-1 and shown on Plate 4-17 is designed to provide a transition from the 12.5-foot diameter outlet shaft to the 60-foot wide outlet channel. Because of its proximity to the city of Quincy recreational area, the channel walls will be essentially vertical, composed of gabions. The drop structure will also be constructed of gabions.

12. CHANNEL IMPROVEMENTS AND SOUTHERN ARTERY CULVERT. Downstream of the tunnel outlet channel improvements consist of widening and deepening portions of Town River upstream and downstream of the proposed culvert, some channel realignment, and provisions for slope and bottom protection. Flows from the outlet will be conveyed in a trapezoidal channel with gabion sideslopes. At the Southern Artery, the proposed improvements will consist of a new 3-barrel pre-cast concrete culvert. The culvert will replace twin 72-inch corrugated metal pipes under the parking lot of the former A&P building and the existing bridge passage under the Southern Artery. (See Plate 4-19.) From the Southern Artery conduits flows will be conveyed through an improved channel to Town River Bay.

C. GENERAL

13. PHYSIOGRAPHY. The Town Brook project is located in the Seaboard Lowland section of the New England physiographic province. The project is on the northeastern edge of the east-west trending Blue Hills Ridge which reaches its maximum height at Great Blue Hill, elevation 630 feet NGVD. This ridge forms a topographic high which separates the lower and flatter Boston and Norfolk basins to the north and south, respectively. Town Brook itself runs in a net easterly direction from an elevation of about 80 feet NGVD upstream of the Old Quincy Reservoir to sea level at the Town River Bay estuary.

14. REGIONAL GEOLOGY. The geology of the region is shown on Figure 2. The Quincy Granite and associated Blue Hill Granite Porphyry form a small yet prominent topographic high known as the Blue Hills Ridge. These rocks are in direct igneous contact with older sedimentary rocks, mostly argillites, and in faulted contact with younger sedimentary rocks in the Boston and Norfolk Basins which are structural and topographic basins. Some detritus from the Blue Hill Porphyry is found in the Boston Basin, the floor of which is older igneous rocks. The basins are in faulted contact with the surrounding older igneous and metasedimentary rocks.

The rocks in the basins were compressed during a period of tectonism in a north-south direction and were folded. The more rigid Quincy Granite mass rode up against the Boston Basin sedimentary rocks along a steep east-west reverse fault named the Blue Hills Thrust (see Fig. 2). Subsequently, north-south trending faults developed which offset the older east-west folds and faults, followed by a long period of erosion. The final major geologic shaping of the landscape occurred from glaciations which removed loose and weathered rock and deposited tills up to 40 feet thick. As the ice margin retreated, sea level rose and there ensued a period of deposition of glaciofluvial and glaciomarine sediments which are known to be more than 100 feet thick in places.

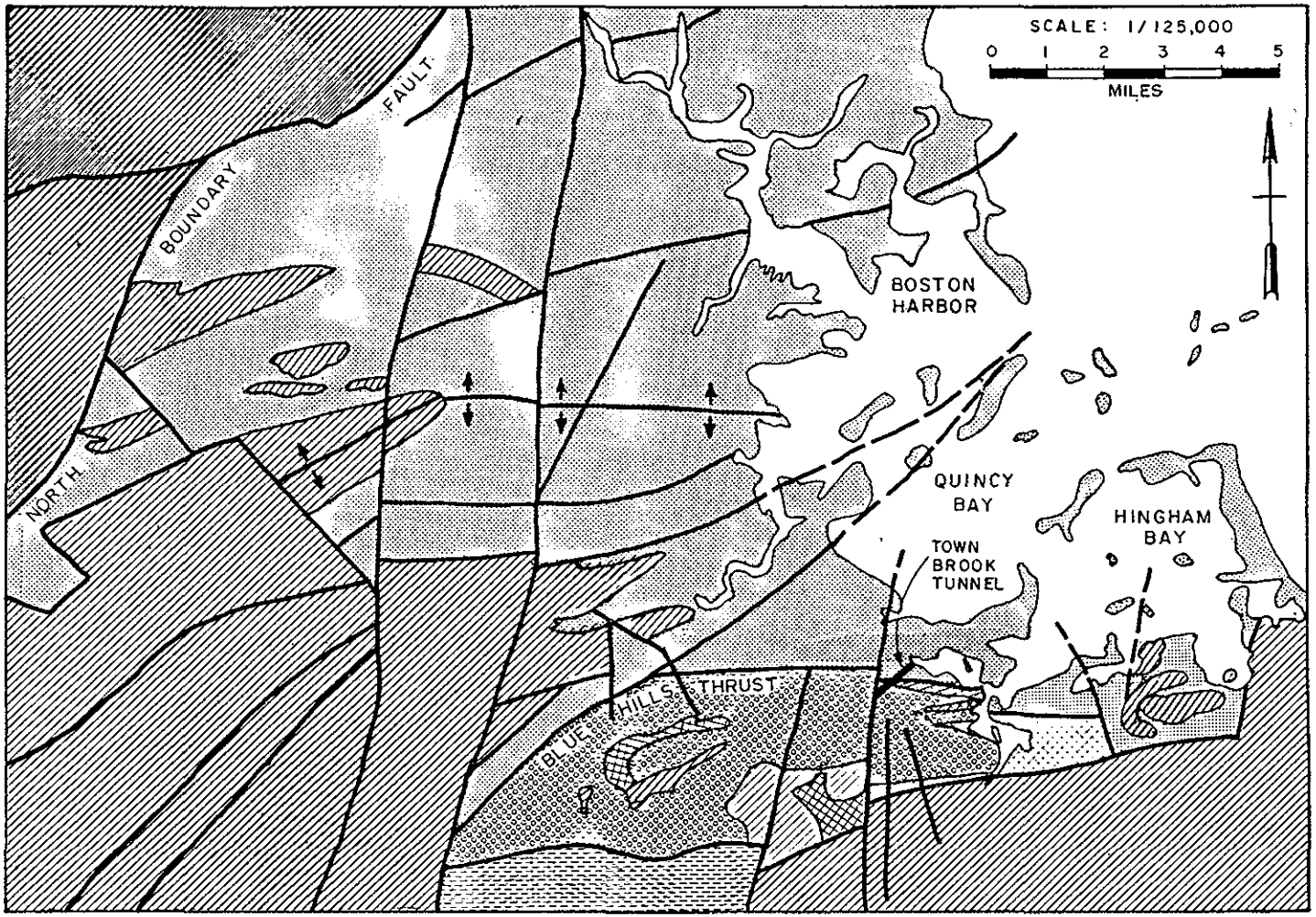
15. SITE GEOLOGY. The site lies in an area underlain by bedrock of the Blue Hills Ridge and Boston Basin (see Fig. 2). The rock in the western portion of the site is Quincy Granite, a massive, medium-grained, pink to light gray hornblende granite rock which has long been famous as building and monument stone.

The rock in the eastern portion of the site is a massive to finely bedded, dark to light gray, sometimes greenish argillite which contains thin beds of non-fissile mudrock (in part contact-metamorphosed), limestone, and sandstone, as well as calcareous lenses and nodules. The formation ranges from argillite to calcarenite.

As expected from the regional geology, the orientation of bedding is variable and dipping from 40 degrees to 75 degrees from horizontal. The argillite becomes more highly metamorphosed (hornfelsic) near the granite, and intrusive contacts are visible in the cores. Structurally, there are

LEGEND

- SEDIMENTARY ROCKS OF THE BOSTON BASIN.(SANDSTONE ARGILLITE AND CONGLOMERATE).
- QUINCY GRANITE AND BLUE HILL GRANITE PORPHYRY.(MOSTLY GRANITE).
- SEDIMENTARY ROCKS OF THE NORFOLK BASIN.(SANDSTONE SLATE AND CONGLOMERATE).
- OTHER SEDIMENTARY ROCKS. (MOSTLY ARGILLITE).
- OTHER IGNEOUS AND META-SEDIMENTARY ROCKS (MOSTLY GRANITE, SCHIST AND GNEISS).
- FAULTS.
- ANTICLINAL AXIS.



BASE MAP FROM USGS OPEN FILE REPORT 77-285 (1977)

FIGURE 2 - REGIONAL GEOLOGY

two major fracture zones within the site. The first, located within the granite east of the inlet shaft, consists of a highly fractured breccia zone rehealed with quartz and with occasional slickensides. The second zone, located within the argillite west of the outlet shaft, is several hundred feet wide and consists of very highly fractured and slickensided shattered rock. The predominant joint sets are oriented N68W, dip 50 degrees; N68E, dip 68 degrees; and S25E, dip 70 degrees. The bedrock is buried by glacial and alluvial deposits 15 to 125 feet thick.

16. SEISMICITY. According to the Seismic Zone Map of Engineering Regulation ER 1110-2-1806 dated 16 May 1983, the project lies on the border of zones 2 and 3.

A site specific evaluation of seismicity was done based upon recent studies for other projects in New England by Dr. Ellis Krinitzsky, Geotechnical Laboratory, WES. Figure 3 shows the project site and the seismic zones in southeastern New England. Using the methods in Dr. Krinitzsky's report and assuming a mean and one standard deviation values, the earthquake motions for the Town Brook site are:

Acceleration	0.15g
Velocity	27 cm/sec.
Duration	9 sec.

The result corresponds to Zone 3 of ER 1110-2-1806. Computations are in Appendix A.

17. PREVIOUS TUNNELING IN THE AREA. Numerous tunnels have been constructed in the Boston area, primarily in Boston Basin sedimentary rocks. The geotechnical literature is scanty and details of engineering significance relating to tunnel geology generally are not highlighted. The tunneling has been successful with no indication of unusual subsurface conditions.

There is one modern account of a relevant problem described by D.D. Ashenden of the Metropolitan District Commission in his paper "Geologic factors affecting failure of the Dorchester Tunnel, Boston." The Dorchester tunnel is a high pressure (200 psi) water supply tunnel for Boston. It is 10 feet in diameter and concrete lined (unreinforced). The zone of failure occurred in the Cambridge Formation argillite (also encountered in this project) about 6 miles northwest of the project site.

When the tunnel was pressurized in 1974, flooding occurred in a few basements of homes located on or close to a local high of argillite in an area of generally thick overburden. No problem would have been detected if there had not been shallow rock. Inspection revealed longitudinal cracks in 2,500 feet of lining in the argillite. The author was of the opinion that the abundant subhorizontal joints present in the argillite allowed enough movement during pressurization to crack the lining. Grouting solved the problem.

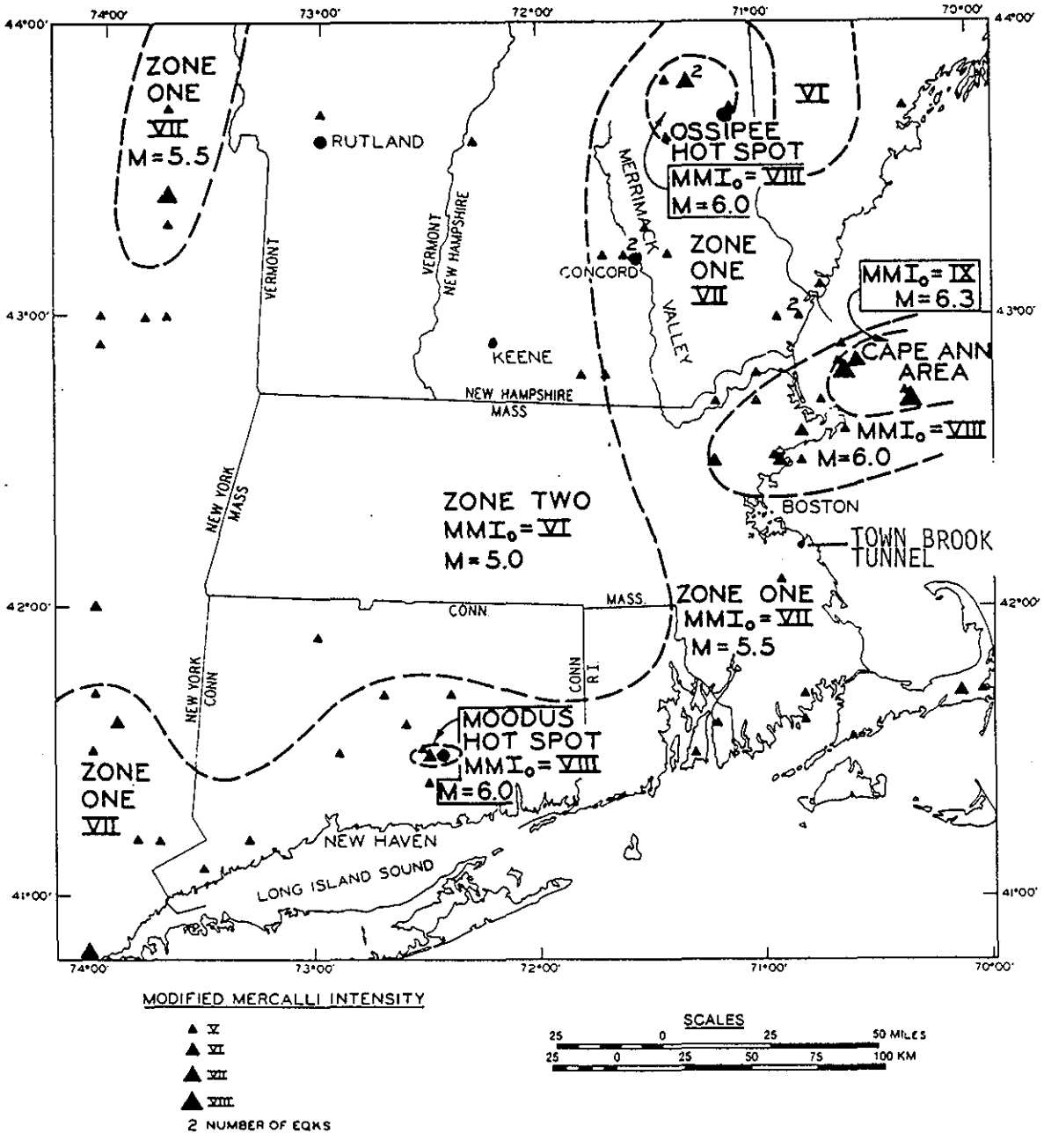


Figure 3. Seismic zones in southeastern New England

Extracted from Krinitzsky, E. L. 1984. "Geological-Seismological Evaluation of Earthquake Hazards at Surry Mountain Damsite, New Hampshire", Technical Report GL-84-7, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MISS.

18. SUBSURFACE INVESTIGATIONS.

a. General. Subsurface investigations were obtained by the Metropolitan District Commission and comprehensive design reports were prepared by the firm Metcalf and Eddy for the MDC. An early program of borings, labeled "D-Series", was done during August-September 1975 patterned throughout the area in order to determine the most suitable tunnel alignment. A seismic refraction survey was performed in 1980 for the purpose of filling in detail between the 1975 borings. Once a tunnel route was decided, an additional series of twelve borings (T-2 through T-10) was taken from March 1983; T-1 was taken in September 1980. Tests in the T series borings included oriented core, pressure testing, water levels, and test pumping.

Borings were also done at the intake and outlet structure sites and in the areas of channel improvements.

b. Core Drilling. The twelve borings in the T-Series along the tunnel were "NQ" (approximately 2" diameter) and were drilled with wireline equipment. Rock cores were logged, classified, tested, and photographed in the laboratory by Metcalf and Eddy. A typical driller's boring log and detailed graphic log are in Appendix A. Cores and classifications were verified by NED geologists.

c. Oriented Core. The cores from four borings were oriented within the zone of the tunnel. Rock cores were measured by a goniometer to determine joint orientation and the core orientation report logs prepared of joint orientations and depths of rock structure. Polar equal area, upper hemisphere stereo nets were then plotted from these logs. An example log is in Appendix A. Stereo nets are shown on the detailed geologic profiles of the tunnel.

d. Pressure Testing. Eleven boreholes were pressure tested to evaluate the permeability of the rock surrounding the borehole. Ten-foot zones were tested. A typical pressure test log is in Appendix B.

e. Test Pumping. Test Pumping was conducted at each of the shaft locations to determine dewatering conditions in overburden. A discussion of the test pump procedure is found in Section F, SHAFTS.

f. Piezometers. Eight boreholes contain piezometers, two in each hole. One piezometer is installed in the overburden and the other in the rock as shown on Plate 4-2 and the detail geologic profiles.

g. Seismic Survey. A seismic survey was conducted between May 29 and June 23, 1980. Results are incorporated in the detail geologic profiles. This survey consisted of approximately 14,000 ft. of refraction profiling to determine depths to top of bedrock, to detect any major shear zones or changes in lithology, and to subsequently select the borehole locations.

19. LABORATORY TESTING.

Rock core and soil samples were chosen for testing by Metcalf and Eddy primarily from zones within the tunnel at the shafts and the other structures. Laboratory testing was done to determine rock strength, the mode of failure, petrographic observations, and drillability. Testing on the rock samples taken from the cores during the logging procedures was as follows:

(1) Resource Engineering Inc. for unconfined compression, petrographic analysis by thin section, quartz content.

(2) Atlas Copco Jarva Inc. for drillability and unconfined compression.

(3) Robbins Equipment Co. for drillability and unconfined compression.

Additional unconfined compression and density tests were done in the New England Division Laboratory. Metcalf and Eddy obtained tests on selected overburden samples from various borings. Tests consisted of grain size analysis, moisture contents, and Atterberg limits. Results are in subsequent paragraphs pertaining to project structures.

D. GEOTECHNICAL DESIGN OF TUNNEL

20. DESCRIPTION OF TUNNEL. The tunnel will have a finish diameter of 12.5 feet and it will have a permanent concrete liner for the entire length. Excavation by drill and blast and by tunnel boring machine are options. The tunnel will pass through granite and argillite as shown on Plate 4-2. In the drill and blast excavation alternative, the westerly 1000 feet of the tunnel will slope in the downstream direction (northeasterly) at the rate of 50 feet per 1000 feet; then for 3100 feet at the rate of about 2.3 feet per 1000 feet. The invert at the inlet shaft will be at elevation -105, and at outlet shaft at elevation -161. It will pass beneath the low point on the bedrock surface (at an inferred shear zone) with a crown cover over the crown of about 40 feet. Rock cover elsewhere in the tunnel ranges upward to a maximum of 180 feet.

In the TBM option, the tunnel would be along a straight line and slope uniformly at 2.3 feet per 1000 feet. This alignment requires the inlet shaft to be deeper (invert el. -145) but avoids changing the direction of the TBM in a possible shear zone.

21. TUNNEL GEOLOGY.

a. Stratigraphy. As shown in detail on Plates 4-3 through 4-6, the tunnel alignment traverses about 2,560 feet of Quincy Granite and 1,500 feet of Weymouth Formation argillite. A fault is inferred in the granite. Overlying the bedrock is overburden composed mainly of stratified glacial deposits, fill, and, at the outlet shaft, a 12-foot layer of tidal swamp deposits.

b. Structural Geology.

(1) Jointing. Upper hemisphere polar equal-area stereo plots were made of strikes and dips of the joints, bedding, and contacts as measured on the oriented cores from four boreholes. These stereo plots are presented in the geologic profiles on Plates 4-3 to 4-6.

Quincy Granite - The strikes of joints measured in granite appear to have two major trends. In T-2 (Sta 40+88 on Pl. 4-6), the dominant trend is at N-NE (90° spread) with dip to the NW, the minor at N-S with dip to the E. Only six joints were observed it T-6, three of which suggest a N-S trend with dips to the E. The indication is that many joints in granite will strike nearly parallel to the tunnel. But most will have strikes that are 30° to 70° from the tunnel axis and the drive will be with the dip, a favorable situation.

Weymouth Formation argillite - Borings T-9 and T-10 (Sta. 4+68 and 0+00 respectively, Pl. 4-3) also have two dominant joint trends. Both sets strike generally within the NE quadrant; the dominant set dips to the SE and the other to the NW. Many joints in the argillite will strike

parallel to the tunnel alignment which is considered unfavorable. There is a considerable spread (90°) in the data, however. The majority of joints are not along the bedding structure.

(2) Shear Zones - Evidence of previous movement exists at borings T-5 (Sta. 29+68) and T-4A (Sta. 29+02) as shown on Plates 4-2 and 4-5. Brecciated zones exist in T-5 at elevation -174 and T-4A elevation -166 to -178. These zones contain mylonite, recemented quartz and vugs. Boring T-5 has two inclusions of argillite or slate at elevation -151 to -153 and at elevation -188 to -194. A lowpoint in the top of bedrock exists in the area of the shear zone. The extent and orientation of the shears are not known. Extrapolation of the shear zones observed in T-4A and T-5 down to tunnel elevation suggests that fractured rock may appear in the tunnel between Sta. 28+00 and 33+00. Condition of the rock would not be uniform as RQD's are seen to vary from 100 percent to 20 percent. Another area of multiple shear zones is assumed between Sta. 1+00 to 10+00 from cores in T-9, T-9A and T-9B (Pl. 4-3). Although RQD's are lower in the shear zones, pressure tests show very small takes.

(3) Weathered Zones. The rock mass is essentially sound. No evidence of any severely weathered zones was seen in the rock cores either at the contact between overburden and bedrock or in the rock itself.

(4) Alteration. Metamorphism of the Weymouth Formation argillite and calcarenite at the granite contact is the most evident alteration feature. The argillite becomes more highly metamorphosed (hornfelsic) near the granite which indicates the contact is intrusive. It has a much greater compressive strength (up to 33,360 psi and is more dense by about 2 pcf.).

c. Special Geotechnical Considerations.

(1) Shear and Fracture Zones. Shear zones, some containing clayey coatings, are found in the granite, and multiple shear zones are found in the argillite. RQD's vary from excellent to very low. Although pressure testing showed tight conditions, support requirements will be carefully considered.

(2) Granite - Argillite Contact. The contact between the granite and argillite falls between Stations 15+50 and 14+60 (Plate 4-4). It is expected to be at a high angle and probably not faulted. Changes in grain sizes of the rock in both granite and argillite are expected due to chilling of the margins at the contact. The contact is most significant to the TBM option where rock properties have a great affect on machine performance.

22. PHYSICAL ROCK PROPERTIES.

a. Mechanical Properties. Table 1 is a summary table of the mechanical properties of the rocks tested. The results are also plotted on Plates 4-3 to 4-6 along with rock quality designations (RQD's) in accordance with EM-1110-2-2901.

About 65% of the tunnel is in granite. The unit weight is about 165 pcf and the compressive strength ranges from about 4,800 psi to 29,300 psi. The lowest strength values are due to failures along joints.

The calcarenite and argillite near the granite contact had unit weights of about 176 pcf (boring T-8) and compressive strength up to 43,100 psi compared to a unit weight of 174 pcf and much lower strengths away from the contact.

The properties of the minor rock types are in Table 1. Tests done by Atlas Copco Jarva Inc. for Metcalf and Eddy, Inc. include cerchar values which showed that rocks are only medium hard and somewhat abrasive, and good for TBM excavation.

b. RQD's. Table 2 is a summary of the effective RQD's in zone of tunnel, and maximum, minimum and average RQD's in the entire boring in bedrock. The effective RQD has been taken as the weighted average RQD in the zone considered to come into play with regard to tunnel support. This zone extends upward from the springline to a point one diameter above the excavated roof, or a total length of boring extending (6+1) + (12+2) = 21 feet above the springline. Table 2 also shows the entire length of each boring in rock (except for the 20 feet immediately below the top of rock, which is considered to be meaningless to this analysis. A 20-foot disturbed and weathered zone at top of bedrock was omitted from this compilation. Table 2 indicates that rock conditions to be encountered will vary between wide limits.

23. GROUNDWATER HYDROLOGY.

a. Groundwater. Groundwater levels measured by piezometers in rock are shown in an overall view by Plate 4-2 and in detail profiles on Plates 4-3 to 4-6. Appendix B Table B-1 shows a tabulation of readings. The readings from multiple piezometers in boreholes show that, with the exception of borehole T-9 (Sta. 4+68, Pl. 4-3), water levels in rock and overburden are nearly the same indicating good hydraulic connections to the depths tested. In borehole T-9, the piezometric level was higher from the rock and slightly flowing conditions noted.

b. Permeability. Permeabilities were calculated from pressure test results. There were 74 water pressure tests made in eleven borings. The results are shown on Plates 4-3 thru 4-6. Each of these tests consisted of five individual runs at ascending gage pressures of 15, 30 and 45 psi, and descending pressures of 30 and 15 psi. The permeabilities resulting

TABLE 1. SUMMARY OF MECHANICAL ROCK PROPERTIES

BORE-HOLE	ROCK TYPE	COMP. STR. psi x 1000	ABSORP-TION(%)	UNIT WT PCF	CERCHAR FIGURES (highest value=6)
T-2	Granite	14.0	0.14	165	
		15.0			
		16.9			
		29.3			
T-3	Granite	10.2	0.32	164	
		14.5	0.33		
		15.3			
		27.8			
T-4A	Granite	5.5			
T-5	Granite	4.8			
T-6	Granite	24.1			5.4±0.4
T-7	Granite	28.9			
T-8	Calcarenite	43.1			
T-8	Argillite	17.5	0.05	176.3	
		22.6			
		28.2			
		33.4			
T-9	Calcarenite	7.1			
T-9A	Arenite	2.1	0.04	172.8	
		6.6			
		13.8			
		14.6			
		15.2			
T-9B	Limestone	24.5			
T-9B	Calcarenite				3.2±0.8
T-10	Argillite	4.6	0.10	173.8	0.9±0.3
		7.0			
		7.2			
		8.8			
T-10	Quartzite	15.0			
T-10	Limestone	8.8	0.32	163.4	
		11.7	0.35	164.9	
		15.1			

TABLE 2 - SUMMARY OF EFFECTIVE RQD's

Boring No.	Effective RQD in Zone of Tunnel	Average RQD in Entire Boring (except top 20') Max	Min	Avg	Rock Type	Avg RQD Tunnel Zone	Avg RQD Entire Hole
T-2	98 D&B*	100	60	92	Granite		
	76 Mole*						
T-3	92	100	79	91	Granite		
T-5	63	90	0	65	Granite		
T-4A	99	100	0	67	Granite	86	80
T-6	78	98	61	81	Granite		
T-7	88	100	61	84	Granite		
T-8	61	100	0	69	Argillite		
T-9B	35	97	0	52	Argillite		
T-9	43	90	0	47	Argillite	54	57
T-9A	37	62	0	27	Argillite		
T-10	94	99	60	92	Argillite		

*See Pl. 4-6 (COE Modification)

from these five runs in each test were averaged to produce a composite permeability. Permeabilities were calculated on the test data sheets shown by the example in Appendix B. In nine cases, the results were discarded because of evident leakages around the packers. In these nine cases, which were usually in rock with very high RQD's, the water losses and resulting permeabilities were several orders higher than could be expected, and it is virtually certain that the spurious results are more likely the product of a defect in the test procedure than actual leaky rock.

The permeabilities of 65 tests were plotted against RQD as there is often a strong correlation between low RQD's and high permeability. In this case, there is no evident correlation between these two parameters; indeed, a number of tests with no "take" at all occurred in formations with RQD's less than 40. The rock cores indicate that the in situ rock has mostly tight and mineral-filled joints.

The characteristic permeability was calculated simply as the arithmetic average of 65 individual tests. This average permeability is 0.000112 feet per minute, which equates to 0.00006 centimeters per second. The derivation is shown by permeability and inflow calculations in Appendix B.

c. Inflow into Tunnel.

The average permeability of 0.000112 feet/minute was used to calculate inflow into the tunnel in accordance with the method of Goodman shown in Appendix B. The calculation is made for the section of tunnel near the outlet shaft, which is deepest below groundwater table and will therefore have the greatest calculable inflow, as follows:

$$Q = 2*k*(D-d)/(2.3*\log(3.5/(D-d))$$
$$= 2*0.000112*(180-0)/(2.3*\log(3.5/(180-0))$$
$$= 0.01024 \text{ cfm per foot of tunnel}$$

This results in an anticipated total inflow for the entire tunnel of 4000*0.01024 = 41.0 cubic feet per minute. Based on this result, very low inflows are expected over most of the tunnel alignment. Actual zones of high inflow cannot be predicted.

24. GEOLOGIC FACTORS AFFECTING TUNNEL CONSTRUCTION.

a. Classification Along Tunnel. According to Corps criteria (EM 1110-2-2901), the magnitude of rock loads on tunnel supports and linings is dependent primarily upon RQD; and secondarily on number of joint sets, roughness and alteration among joints, amount of water inflow, occurrence of random material (in seams) weaker than rock mass as a whole, and possibly others. Since explorations in tunnels generally cannot be continuous, ambiguities are bound to exist between points where explorations took place. Nevertheless, judicious consideration of all

existing data permit the assignment of classifications to all reaches of the tunnel. These anticipated classifications are useful primarily for design, as rock classes and character different from those anticipated will be manifested during actual construction. For the purpose of design, however, the tunnel will be assumed to have classifications as indicated below.

Approx. Station From	To	Length (feet)	Description
41+00	33+00	800	Massive, moderately jointed granite
33+00	28+00	500	Massive, moderately jointed granite with shear zones
28+00	15+00	1,300	Massive, moderately jointed granite
15+00	10+00	500	Very blocky and seamy argillite
10+00	1+00	900	Completely crushed argillite
1+00	0+00	100	Massive, moderately jointed argillite

b. Rock Loads. Rock loads on the tunnel will be assumed to act vertically on a horizontal projection of tunnel excavation. There will be no active rock loads assumed to act horizontally. The magnitude of rock loads on the tunnel is given in Table 3.

c. Tunneling Methods. The method of tunneling will affect to a large extent the tunnel condition and permanent rock loads. As is indicated in Table 3, load factors due to rock loosened during tunneling may be reduced by as much as 50 percent in the major portion of the tunnel with machine excavation.

d. Standup Time. Standup time will also depend largely on the controls exercised during excavation; however, using an analysis outlined in a report by Bieniawski entitled "Engineering Classification of Jointed Rock Masses", the following standup times estimated for the five rock conditions shown in Table 3 for an excavated diameter of 15 feet.

Hard and intact - 10 years
Massive, moderately jointed - 1 year
Moderately, blocky & seamy - 4 days
Very blocky & seamy - 12 hours
Completely crushed - 1 hour

This condition is defined by two parameters which consider an unsupported span of rock and the time that it takes this span to fail. The active unsupported span is defined as either the width of the tunnel or the distance from the support to the face of the tunnel if this is less than the tunnel width.

e. Jointing. The high strength irregular surface joints are expected to have a minimal effect on the tunnel configuration with regard

TABLE 3 - CLASSIFICATION OF ROCK AND CORRESPONDING ROCK LOADS TO BE USED FOR DESIGN OF SUPPORT (ROCK REINFORCEMENT OR RIBS) AND OF CONCRETE LINING

		Rock Load (in feet of rock)			
		Excavation by TBM		Excavation by D&B	
RQD	Classification	For Support	For Lining	For Support	For Lining
95-100	Hard & Intact	0	0	0.25B	0.25B
85-95	Massive, mod. jointed	0.2B	0.2B	0.4B	0.3B
75-85	Mod., blocky & seamy	0.3B	0.2B	0.8B	0.5B
40-75	Very blocky & seamy	0.8B	0.4B	1.2B	0.6B
>40	Completely Crushed	1.4B	0.7B	1.8B	0.9B

B = Excavated width of tunnel

The data in Table 3 are based on guidelines contained in EM 1110-2-2901, as follows:

Table 3-1 -- Estimate of Rock Load (Proctor & White)

Table 3-2 -- Rock Loads & Classification (Deere et al)

Table 3-3 -- Support Recommendations (Deere et al)

Also recognize that if the more sophisticated methods proposed by Barton, Lien & Lunde for the Norwegian Technical Institute (NGI) or that by Bienawski for the South African Council for Scientific and Industrial Research (CSIR) had been used, then the loads would be smaller than through the use of those in Tables 3-1, 3-2 and 3-3 in EM 1110-2-2901. For comparison, a table of evaluations according to the Q-system is included in Appendix A. The results show that support is not required over much of the tunnel.

Values in table 3 are generally lower than the maximum values suggested by EM 1110-2-2901 because of the following facts:

1. Joint surfaces are moderately rough and free of filling materials;

2. Water inflows as determined by packer test calculations are expected to be moderate or nil; and

3. There are very few seams of weak material interbedded within stronger material.

Loads shown are final -- that is, loads that would have to be taken by the support after all relaxation of rock stress.

to either load or shape. The frequency and orientation of these joints have been indicated by the point diagrams for borings T-2, T-6, T-9 and T-10 as shown on Plates 4-3 to 4-6 and have been given consideration as suggested by the Barton, Lien and Lund method of analysis.

25. TUNNEL DESIGN FEATURES.

a. General. The rock loads for which support and lining will be designed are given for tunnels in Table 3. Hydrostatic water loads for design of the lining are discussed in Section 29 - BASIC DATA AND ASSUMPTIONS.

b. Excavation Conditions. Tunnel excavation conditions will be largely determined by the construction method elected. The direction of excavation will be a significant factor in the ease with which water is cared for. The normal direction is upgrade to allow a dry heading; this direction will be enforced for this reason, and for the additional reason that room exists for the construction plant at the outlet shaft.

c. Support. Temporary rock support consisting of rock bolts or steel ring beams will allow for maximum flexibility within short sections where they may be modified to treat conditions as they exist on a per shift basis. Design of temporary support systems is discussed in paragraph 30, ANALYSIS AND DESIGN.

d. Lining. The lining cross-section is determined by rock loads and hydrostatic water pressures. Grouting of the lining will be done to close any voids at the tunnel crown. Results from drilling show that large sections of granite are probably very sound and, from a structural standpoint, may not require a lining. However, because this tunnel is in an urban area and there may be unpredictable hydrologic and environmental effects and considerations, it is concluded that the entire length of the tunnel should be lined. Three options are considered for this project; drill and blast excavation with a cast-in-place liner, mole excavation with a cast-in-place liner, mole excavation with a precast liner. Hydraulicly smooth joints will be provided in the precast liner as discussed in DM No. 5.

e. Circulation Pipes. Two 6-inch diameter fiberglass pipes will be located inside the finished tunnel diameter as part of the aeration/circulation system discussed in Design Memorandum No. 5. These pipes will be attached below the springline to the permanent concrete lining with corrosion resistant fittings (Pl. 4-9).

f. Treatment of Water. Although the areas of rock that were pressure tested have low permeabilities, broken zones may be encountered where high water inflows will occur requiring control and diversion of water prior to lining. Fractured argillite and the granite around Sta. 30+00 are such zones. Monitoring of the water drawdown table at critical structures and providing alternate methods of recharging the water table

as necessary to prevent unnecessary settlement will be required. Control of water inflow at critical areas by grouting ahead of and during the tunnel construction will allow further reduction of any localized areas in high water inflow. Drilling ahead through potential zones of high permeability will be considered to detect inflows.

g. Vibration Controls. Controls will be exercised throughout the work to prevent overstressing any of the surface structures. Some urban structurers may require a preconstruction survey to determine safe vibration levels for that particular structure. Construction controls would then be exercised to prevent safe vibration levels from being exceeded during the prosecution of the work.

h. Tunnel Instrumentation. This phase of the project is primarily divided into two phases. Phase I will cover the monitoring of the excavation during construction for the purpose of verification that all strains in the mass of the rock surrounding the tunnel have been arrested prior to installation of lining. Phase II will consist of post-construction monitoring of the completed project. The work anticipated to be accomplished in each phase is as follows:

(1) Phase I. This work will be carried out under the construction contract and will consist of instrumented stations containing borehole extensometers, convergence anchors and strain gages (Pl. 4-7). Extensometers will be located at 4 stations that represent typical rock conditions: 1) sheared argillite, 2) argillite near granite contact, 3) massive granite, and 4) sheared granite near the inferred fault zone. Convergence anchors will be located at stations approximately 200 feet apart in the Weymouth Formation and 400 feet apart in the Quincy Granite. Strain gages will be placed on steel ribs where installed and spaced at an average of 50 feet apart. In addition, similar instrumentation will be required to be installed wherever there is visual evidence or suspicion of ground movements. The purpose of this phase would be to evaluate the reliability of the predesign data and assumptions and to modify the tunnel construction and support as necessary to arrest strain in the remaining portion to be constructed.

(2) Phase II. This monitoring work would be of limited extent and conducted on the concrete lining. The intent is to conduct the studies for 3 to 6 months prior to filling of the tunnel to determine the degree to which ground stresses are transferred to the lining.

26. CONDITIONS OF SPECIAL ENGINEERING SIGNIFICANCE.

a. Lowering of Groundwater Levels During Tunnel Construction. Piezometers installed in all borings indicate a good hydraulic connection between the bedrock and the overburden in all borings except one, T-9 (Sta. 4+68); in this boring the piezometer level was higher in the bedrock. It is anticipated that there will not be a substantial lowering of groundwater levels in the overburden beneath the city because of

construction of the tunnel. This conclusion is based upon the fact that permeability of the overburden is several orders of magnitude greater than that of the rock. However, a preconstruction survey and monitoring during construction of groundwater levels will be required of the contractor.

b. Disturbance to the Public. Geologic conditions at tunnel grade appear suitable for the use of mechanical excavation equipment (TBM). The utilization of such systems would avoid much of the noise and vibrations which would occur during conventional drill and blast. However, construction controls would be exercised with any option to maintain safe vibration levels during prosecution of the work. Some urban structures may require a preconstruction survey to determine safe vibration levels.

E. STRUCTURAL DESIGN OF TUNNEL

27. PURPOSE AND SCOPE. This section presents the basic data, design criteria, and assumptions for the structural design of the tunnel.

28. DESIGN CRITERIA.

a. General. Allowable stresses, loading conditions, and design assumptions are based on applicable parts of the following references.

(1) EM 1110-1-2101, Working Stresses for Structural Design, 1 November 1963 (with Change 3, 17 January 1972).

(2) EM 1110-2-2000, Standard Practice for Concrete, 1 November 1971.

(3) EM 1110-2-2103, Details of Reinforcement - Hydraulic Structures, 21 May 1971.

(4) EM 1110-2-2901, Tunnels and Shafts in Rock, 15 September 1978 (with Change 1, 19 February 1982).

(5) EM 1110-2-2902, Conduits, Culverts and Pipes, 3 March 1969 (with Change 3, 30 November 1978).

b. Concrete. Concrete with a minimum ultimate compressive strength of 4,000 pounds per square inch is selected for design of the cast-in-place liners as recommended in EM 1110-2-2902. Concrete with a minimum ultimate compressive strength of 5,000 pounds per square inch is selected for design of the precast liner. Working stresses are generally in accordance with "ACI Building Code Requirements for Reinforced Concrete" as modified by EM 1110-1-2101. The maximum working stresses used for the design of the tunnel liner are as follows:

	f'_c = 4000 psi	f'_c = 5000 psi
Flexure (extreme fiber in compression) $0.45\ f'_c$	1800 psi	2250 psi
Flexure (extreme fiber in tension for unreinforced liner) $\sqrt{1.6\ f'_c}$	101 psi	113 psi
Principal Diagonal Tensile Stress $\sqrt{2\ f'_c}$	126 psi	141 psi

c. Steel Reinforcement.

(1) General. Steel reinforcement will be deformed bars conforming to ASTM A 615, Grade 60. Allowable working stress for tension in reinforcement is 20,000 pounds per square inch, as specified in EM 1110-1-2101.

(2) Details. Detailing of reinforcement will be in accordance with EM 1110-2-2103.

(3) Minimum Cover. A minimum clear cover of 4 inches is required between the inside face of the liner and the inside ring of main transverse reinforcement for the cast-in-place option, as recommended in EM 1110-2-2901. This cover requirement will be reduced to 3 inches for the precast option. A minimum clear distance of one inch will be specified for the outside transverse reinforcement in the precast liner.

(4) Minimum Flexural Reinforcement. As presented in EM 1110-2-2902, the minimum ratio of transverse reinforcement area to gross concrete area is 0.002 in each face.

(5) Temperature and Shrinkage Reinforcement (Precast Option). The minimum area of longitudinal reinforcement is 0.001 times the gross area of concrete in each face.

d. Structural Steel. Where steel sets are required for temporary support, full circle ring beams conforming to ASTM A36 will be used. Maximum allowable fiber stress in the ring beams is 26,300 psi (0.73 fy), as listed in EM 1110-2-2901 for temporary construction.

e. Shotcrete. Shotcrete will be specified for use in combination with both rock bolt and steel ring beam rock support systems. Shotcrete will be used essentially in a non-structural capacity for limited reaches to seal rock surfaces with large inflows. In addition, it may be used where steel sets are installed in reaches of completely crushed rock to provide lateral stiffness to the sets and protect against fallout. The shotcrete will be specified to have a minimum thickness of 2 inches; the use of wire mesh reinforcement in the shotcrete is not considered necessary for this application. Prior to application, the rock surface will be cleaned by water jet. Shotcrete strengths of 500 psi and 3,000 psi at 8 hours and 28 days, respectively, will be specified.

f. Rock Bolts. The rock bolt system which will be specified is considered to be the industry standard and consists of the following:

(1) Bolts made of high-strength steel (60,000 psi minimum yield strength, with continuous rolled-in threadlike deformations capable of developing the full strength of the bar with a threaded nut (Threadbars);

(2) Anchorage of polyester or epoxy resin for a length sufficient to develop fully the strength of the bolt;

(3) Grouting with the same material used for anchorage (but with longer set to permit prestressing) for the full length of the bolt to inhibit corrosion and to limit elastic elongation;

(4) Bearing Pad, consisting of a smooth area of about 36 square inches for bars less than 1 inch diameter;

(5) Prestressing of the bolt after the set of the anchorage but before the set of the grout along the shaft of the bolt;

(6) Chain Link Fabric placed over the surface of the rock to prevent the fall of shallow pieces of rock.

g. Monolith Joints. The cast-in-place tunnel lining, if selected, will be placed in 30-foot long monoliths. The joints between placements will have continuous plastic or rubber waterstops. The longitudinal reinforcement in the liner will be continuous through the joints.

h. Precast Liner. Precast liners, if selected, will be constructed of reinforced concrete with a minimum ultimate compressive strength of 5,000 psi. The liner will be cast in four segments, transported to the site, and erected. The segments will be approximately 11 feet in length circumferentially and 6 feet in length longitudinally. The transverse and longitudinal joints between the precast segments will contain joint material which will be compressed during installation and will remain in compression under all loading conditions. As discussed in DM No. 5, limits on joint offsets will be established in the specifications so as to ensure hydraulic smoothness. Specifications will be written to allow the Contractor the latitude to propose alternate precast details for consideration subject to Corps approval.

i. Grouting. Grouting of the interface between the concrete liner and the rock surface will be required with either the precast or cast-in-place alternatives. Grout pipes and relief ports will be provided in the crown of the cast-in-place liner and along the entire circumference of the precast liner. A contact grout pressure of 15 psi will be specified.

29. BASIC DATA AND ASSUMPTIONS

a. Controlling Elevations.

Tunnel invert at intake end	-105.0 feet NGVD (D&B)
	-145.0 feet NGVD (TBM)
Tunnel invert at outlet end	-161.0 feet NGVD
Maximum River Stage at Outlet	7.9 feet NGVD
Maximum hydraulic gradient at inlet	19.3 feet NGVD
Finished ground at inlet end	23.0 feet NGVD
Ground water at inlet end	14.0 feet NGVD
Finished ground at outlet end	11.0 feet NGVD
Minimum rock cover over tunnel	40 feet

b. Dead Loads.

Concrete	150 P.C.F.
Granite	165 P.C.F.
Argillite	175 P.C.F.

c. Live Loads.

Water 62.5 P.C.F.	
Contact Grout Pressure (at nozzle)	15.0 P.S.I.

d. External Water Pressure. The level of groundwater will vary between the approximate limits of Elevation 5 and 20 feet NGVD along the length of the tunnel. It is assumed that pressure on the tunnel liner from external water will always exist after construction regardless of the permeability of the rock. Measurements of hydrostatic pressure in the rock will be made both prior to and during construction. The buoyancy effect resulting from the difference in external pressure between the crown and invert induces both bending and thrust on the liner cross-section. The maximum external water pressure will occur in the vicinity of the outlet shaft where the tunnel is at its deepest point and groundwater level is assumed to coincide with ground level.

e. Internal Water Pressure. The relief tunnel will be completely full when in operation. The internal pressure on the tunnel liner will be determined by the level of water in the intake and outlet shafts. Three conditions are presented for discussion of internal water pressures:

(1) Non-Flood Condition. During the non-flood condition, the level of water in the inlet and outlet shafts will be Elevation 2.0 feet NGVD. Observed groundwater levels along the length of the tunnel vary between the limits of Elevation 5 and 20 feet NGVD. Thus, the external water pressure will exceed internal pressure for the entire length of tunnel during this condition.

(2) Flood Condition. During the flood condition, the hydraulic gradient at the inlet shaft and the river stage at the outlet shaft will be Elevation 19.3 and 7.9 feet NGVD, respectively. Observed groundwater levels at these points are at Elevation 14± and 5± feet NGVD, respectively, and a rise in groundwater levels would be expected during flood events. Therefore, a small net internal pressure may be exerted on the liner during the flood condition.

(3) Low Permeability Rock. A significant consideration is the possibility of hydrostatic pressures in the rock being substantially below the pressures which would result from observed groundwater levels. Such an occurrence would result in higher net internal pressures than discussed above. However, it is not likely that long reaches of the rock will exist without intercepting water bearing joints. If low permeability rock is encountered, it would most probably occur in reaches of high quality rock

which would provide restraint to the liner. Further, it is assumed that once water pressure is introduced, it will always exist between the rock face and concrete liner at least to the level of mean water (Elevation 0.2 feet NGVD) in Town River Bay. It is concluded that a condition with the hydraulic gradient at the inlet shaft at Elevation 19.3 feet NGVD and external groundwater level at Elevation 0.2 feet NGVD represents the maximum net internal pressure which will be experienced. A net internal pressure of 20 feet is adopted for the design of all reaches of the tunnel liner.

f. Rock Loads. Rock loading on the roof of the tunnel is determined from the geologic investigations discussed earlier. The temporary support system and the permanent lining are designed for the rock loads which are described qualitatively in Table 3 and listed quantitatively in Appendix C. These values reflect a reduction in rock load if excavation is by boring machine as opposed to the conventional drill and blast method. Explorations indicate that the worst rock conditions will be encountered between approximately Station 10+00 to 1+00 where excavation will be in completely crushed argillite.

g. Seismic Considerations. The proposed tunnel alignment has no reaches of shallow rock cover and does not cross any capable faults. For these reasons, no dynamic stability analysis or special design features are considered necessary to account for earthquake effects. Accordingly, it is not considered necessary to prescribe any minimum amount of reinforcement for seismic consideration in the design of the permanent concrete tunnel lining.

h. Grouting Pressure. A pressure of 15 psi will be specified for contact grouting the tunnel liner for both the cast-in-place and precast liners. For the precast liner, it is assumed that the grout pressure will insure that the liner remains in compression under net internal pressure conditions.

i. Excavation. Excavation for the tunnel will be either by conventional drill and blast or by tunnel boring machine. For the cast-in-place liner excavated by drill and blast, a minimum line of excavation and a payment line for excavation will be established. The minimum line of excavation, designated the "A" line, is that line beyond which no unexcavated material shall remain. The "B" line is established as that line within which measurement for payment of excavation will be made. The distance between the "A" and "B" lines will be varied with rock quality and represent a reasonable estimate of the average overbreak expected for a specific rock condition. For the cast-in-place liner with excavation by tunnel boring machine, excavation will be required within a minimum diameter equal to the inside diameter plus twice the specified liner thickness. For the tunnel boring machine with precast liner alternative, excavation will be required within a minimum diameter which will provide an annular space of 2 inches between the outside of the liner and the excavated rock surface. The annular space will allow for back packing and

grouting. No "A" or "B" lines will be established for the tunnel boring machine alternatives, and excavation will be paid on the basis of price bid per linear foot of tunnel.

j. Rock Modulus. For stress analysis of permanent concrete liners, a foundation modulus for the rock is calculated based on an assumed modulus of deformation by the method presented in EM 1110-2-2901. A modulus of deformation of 500,000 psi, assumed to be a minimum value for completely crushed argillite, is used in the analysis.

30. ANALYSIS AND DESIGN.

a. General. The permanent tunnel lining is analyzed using the computer program "Tunnel," as described in Technical Report C-73-2, "Computer Study of Steel Tunnel Supports," prepared by the U.S. Army Engineers Waterways Experiment Station. This program uses the stiffness matrix method and accounts for the interaction of the lining and surrounding rock. Analysis of the temporary support systems, steel ring beams and rock bolts, is performed by manual calculations in accordance with the methods presented in EM 1110-2-2901.

b. Temporary Support Systems.

(1) General. Temporary rock supports will be required only if a cast-in-place alternate is selected and will provide the primary rock support for periods of up to one year prior to placement of the permanent lining. The tunnel excavated by drill and blast will require continuous rock support for its entire length. Rock support will be less if machine excavation is selected and will be omitted entirely in reaches of "hard and intact" rock. Precast linings, if selected, will be installed immediately behind the machine shield and rock support will be required only in areas of short stand-up time or when delays in lining operations are encountered.

(2) Steel Ring Beams. Steel ring beams will be specified for temporary rock support in reaches of "completely crushed" rock when excavated by drill and blast method. The beams are designed to support a vertical rock load of 2.284 tons per square foot at a longitudinal spacing of 2 feet. Full circular beams will be fabricated in segments which will be spliced using butt plates requiring field bolting. Longitudinal bracing between ring beams will be required at a maximum spacing of 5 feet. Shotcrete, if used, will provide additional lateral support to the beams. Timber blocking inserted between the rock surface and the outside of the ring beam is assumed to be at 24 inches on center. This blocking arrangement induces both bending and thrust in the ring beam. As determined by the calculations presented in Appendix D, a W6x15 beam is selected. Variation in actual rock conditions encountered during construction will be accommodated by varying the longitudinal spacing of ring beams.

(3) Rock Bolts. Rock bolts will be specified for the temporary rock support system, except where steel ring beams are required. An initial tension of 40,000 psi will be specified. The rock bolts are designed to resist the rock loads previously discussed. Size, spacing, and length of the rock bolts are based on the empirical rules contained in EM 1110-2-2901. Bolt patterns, dependent on the method of excavation and quality of the rock, are derived using 3/4-inch diameter bolts stressed to 100 percent of yield strength. The adopted rock bolt patterns and details are shown on Plate 4-8. Calculations for rock bolts are presented in Appendix D.

c. Permanent Lining.

(1) General. Design calculations for permanent lining, cast-in-place and precast, are presented in Appendix E. Loading, moment, and thrust diagrams are shown on Plate 4-11. A section representative of conditions in the vicinity of the outlet shaft is adopted for design. This section is based on the lowest quality rock anticipated - completely crushed argillite. Minimum design thickness of 12 inches for cast-in-place linings and 9 inches for precast liners are adopted.

(2) Cast-in-Place Liner. The load cases considered for design of the cast-in-place liner are as follows:

Case 1 - Dead load plus rock load
Case 2 - Dead load plus full external water pressure (172-foot head above invert)
Case 3 - Dead load plus rock load plus full external water pressure
Case 4 - Dead load plus net internal water pressure (20-foot net internal head)
Case 5 - Dead load plus grout pressure

These load cases are analyzed using a computer program which assumes the cast-in-place liner to be a 12-inch thick continuous ring interacting with the surrounding rock. From computer output, stresses in the liner are manually calculated at all nodes for the combinations of moment and thrust assuming the liner to be unreinforced concrete. The rock loads used for this analysis are representative of completely crushed argillite (RQD<40) excavated by drill and blast.

The calculations indicate that except for Case 1, all computed stresses are below allowables for unreinforced concrete. For Case 1, tensile stresses at the crown exceed allowables, and reinforcement is designed to resist the tensile stresses. Also, Case I is re-analyzed with reduced rock loads indicative of rock quality better than completely crushed argillite excavated by drill and blast. This re-analysis indicates that crown reinforcement is not necessary for reaches where rock with RQD greater than 85 is encountered.

It is concluded that the design for the cast-in-place alternates is as follows: 12-inch thick unreinforced concrete liner for tunnel reaches with RQD greater than 85, if excavated is by drill and blast, or 75, if excavation is by tunnel boring machine; and 12-inch thick concrete liner with tensile reinforcement in the crown for all other tunnel reaches. Details of the cast-in-place liner are shown on Plate 4-9.

(3) Precast Liner. The load cases for the precast liner are the same as those outlined for the cast-in-place liner. The rock loads for this analysis are representative of completely crushed argillite (RQD<40) with excavation by boring machine. The net internal pressure condition, Case 4, is not valid for analysis of the precast liner due to its inability to carry axial tensile forces. Grout pressures around the circumference of the liner will be utilized to insure the liner segments remain in compression under this condition.

The precast liner is analyzed using a computer program which models a 4 segment circular member connected by pins incapable of transmitting moment between segments. The liner is assumed to interact with the surrounding rock. Critical combinations of moment and thrust are obtained from computer output, and reinforcement is designed by manual calculations.

For the precast liner alternate, a 9-inch thick concrete liner with transverse No. 5 bars at 12 inches and longitudinal No. 4 bars at 18 inches (both faces) is adopted for the entire length of tunnel. Details of the precast liner are shown on Plate 4-10.

F. SHAFTS

31. DESCRIPTION OF SHAFTS. The completed shafts will have an internal diameter of 12.5 feet. The inlet shaft will pass through about 70 feet of man-made fill and glacial drift and then through granite. The outlet shaft will pass through about 60 feet of recent alluvium and glacial drift and then 110 feet of argillite. The inverts at the inlet and outlet shafts will be at elevations -105 (-145 TBM tunnel option) and -161, respectively. The shafts will be lined with unreinforced concrete in the rock portions, while reinforcement will be required for the overburden portions and rock portion with RQD<50. Plate 4-2 is a plan and geologic profile along the proposed tunnel alignment. Plates 4-3 and 4-6 are detailed plans and geologic log profiles of the proposed shafts.

32. SHAFT GEOLOGY.

a. Inlet Shaft. Boring T-2 is 13 feet from the inlet shaft location. The completed shaft will pass through about 70 feet of overburden after site development is completed. The upper portion of the inlet shaft as shown on Plate 4-6 is located in overburden comprised of a 27-ft surficial layer of medium dense to very dense granular fill materials (SM) overlying 45 ft of medium dense to very dense glacial outwash deposits of stratified gravelly silty sands (SM), gravelly sands (SP), and sandy silts (ML). A 14-ft layer of glacial till consisting of a very dense gravelly sandy silt (ML) is located below the glacial outwash deposits on top of the bedrock surface. About 16 feet of the existing 27-ft surficial fill will be removed during site development at the inlet area, reducing the overburden thickness at the shaft location from 86 ft to about 70 ft.

In the drill and blast tunnel option, the invert will be about elevation -105 (59 feet into granite). In the TBM tunnel option, the invert will be at about elevation -145 (100 feet in granite). The first 22 feet of granite is very blocky and seamy (RQD's less than 50), and is considered poor rock quality. The rock below has RQD's greater than 90 and is considered excellent rock quality overall.

b. Outlet Shaft. Boring T-10 is 26 feet from the outlet shaft location. The bottom of the shaft is at elevation -161. About 61 feet is in overburden and 109 feet in argillite.

The upper portion of the outlet shaft as shown on Plate 4-3 is located in overburden comprised of a 3 ft. surficial granular fill (SM) over a 9-ft. tidal marsh deposit of very loose organic silt (OL) and peat (PT) overlying 23 ft. of medium dense to very dense glacial outwash deposits of stratified gravelly silty sands (SP-SM), silty sands (SM) and sandy silts (ML). A 25-ft. layer of glacial till consisting of very dense gravelly silty sand (SM) and gravelly sandy silt (ML) with phases of sandy clay (CL) is located below the glacial outwash deposits on top of the bedrock surface.

The glacial till materials have fine contents between 40 and 68 percent and range from nonplastic to slightly plastic with liquid limits of less than 28 and plasticity indices of less than 10.

The first 20 feet of argillite is completely crushed (RQD's less than 50%) and is poor quality. The next 10 feet are very blocky and seamy. The rock below becomes more massive and moderately jointed. At tunnel grade it is good to excellent.

33. SOIL PROPERTIES. The boring logs at each shaft depict stratification in greater detail than is necessary for design of temporary and permanent support. Therefore adjacent strata that are expected to exert similar loads on the support will be grouped together. These stata, along with their moist, saturated and submerged unit weights and their strength parameters, are shown in Table 4. Strengths and unit weights have been inferred from the standard penetration tests, drillers' and laboratory descriptions, limited laboratory testing, and past experience with similar materials.

34. GROUND WATER HYDROLOGY. Two pumping tests were conducted during the field investigations to determine the aquifer characteristics at the inlet and outlet shaft locations. Each test consisted of pumping an 8-inch diameter test well and observing water level drawdowns and recoveries in 2-1/2-inch diameter observation wells and piezometers.

a. Inlet Shaft. A 5-day pumping test was conducted at the inlet shaft location. The test well was pumped at a rate of 150 gallons per minute (gpm). Water level measurements were made in seven observation wells and two piezometers. Analysis of the drawdown data indicates a transmissivity of about 1,400 ft^2/day and a storage coefficient of 0.14. If an aquifer thickness of 28 feet is assumed, then the resulting average permeability is 50 ft/day or 1.8×10^{-2} cm/sec.

Approximately 12 feet of drawdown had occurred at a radial distance of 21 feet from the pumping well at the end of the test. The distance-drawdown curve indicates that the drawdown was about 16 feet at a radial distance of 8 feet and was 0.1 feet at a distance of about 650 feet. These data suggest that the groundwater inflow from the tested interval into the shaft excavation, if uncontrolled, could be on the order of several thousand gallons per minute. This estimate is based on a short-term test. The actual long-term groundwater conditions that will occur during construction cannot be precisely determined beforehand. The inflows from the depth intervals that were not tested during this program, including the soil-rock contact, are not known, although they are estimated to be less than those from the tested intervals.

b. Outlet Shaft. An 8 hour pumping test was conducted at the outlet shaft location. The test well was pumped at a rate of 55 gpm. Water level measurements were made in six observation wells and two piezometers. Analyses of the drawdown data indicate a transmissivity of about

TABLE 4. DISTRIBUTION OF SOIL TYPES, WITH STRENGTH PARAMETERS AND UNIT WEIGHTS TO BE USED IN DESIGN

Depth	Material	N Range	Avg. N Valve	Adopted Cohesion (psf)	Adopted Friction Angle (deg.)	Adopted Unit Weights (pcf) Moist	Saturated	Submerged
INLET SHAFT								
0-27'	Br. & Gr. Med to V. dense gravelly silty sand (SP-SM, FILL)	16-88	41	0	30	120	130	70
27'-56'	Stratified, Br. Med. to V. dense, gravelly silty sand (SM) and gravelly sand (SP)	19-54	30	0	30	120	130	70
56'-72'	Br. Med. to V. dense, sandy silt (ML)	25-61	49	0	30	120	130	70
72'-86'	Gr. V. dense gravelly sandy silt (ML, TILL)	68-115	80	0	35	120	145	85
OUTLET SHAFT								
0-3'	Br., Med. to dense, gravelly silty sand (SM, FILL)	36		0	-	120	130	70
3'-12'	Dk. Br., V. loose organic silt (OL) and Peat (PT)	1-2	2	100	0	100	110	55
12'-21'	Statified, Gr., Med. to dense gravelly silty sand (SP-SM); silty sand (SM); and sandy silt (ML)	12-31	22	0	30	120	130	70
21'-35'	Br., dense to V. dense, gravelly silty sand (SM)	31-108	62	0	30	120	130	70
35'-60'	Gr., V. dense, gravelly silty sand (SM, TILL) and gravelly sandy silt (ML-CL, TILL)	95-215	120	0	35	120	145	85

550 ft^2/day and a storage coefficient of 4×10^{-4}. If an aquifer thickness of 14 feet is assumed, then the resulting average permeability is 39 ft/day or 1.4×10^{-2} cm/sec.

Approximately 12 feet of drawdown had occurred at a radial distance of 15 feet from the pumping well at the end of the test. The distance-drawdown curve indicates that the drawdown was about 12.5 feet at a radial distance of 12 feet and was 0.1 feet at a distance of about 1300 feet. This distance is hypothetical, since the most distant observation well was only 308 feet from the pumping well. These data suggest that the groundwater inflow from the tested interval, into the shaft excavation, if uncontrolled, could be on the order of several hundred gallons per minute. This estimate is based on a short-term test. The actual long-term groundwater conditions that will occur during construction cannot be determined beforehand. The inflows from the depth intervals that were not tested during this program, including the soil-rock contact, are not known, although they are estimated to be less than those from the tested intervals.

35. DESIGN COMPUTATIONS.

a. Temporary Rock Support. For rock better than "completely crushed", bolts will be used for temporary support. Rock bolts in shafts would be similar to the tunnel support shown by Plate 4-8.

For "completely crushed" rock, the temporary support of shafts can be accomplished by the use of W8x31 ring beams on 2 foot spacing. Plate 4-8 shows the ring beam in a drilled and blasted tunnel; a cross-section of a shaft in completely crushed rock would be similar.

b. Lining of Shafts in Rock. Design calculations for the permanent concrete lining are presented in Appendix E.

Loading on which the lining design was based are for the lowest quality of rock (completely crushed) and for full exterior hydrostatic head. The resulting design calls for an unreinforced concrete lining, 12 inches thick, and the use of concrete with a 28-day strength of 4,000 pounds per square inch. These linings are shown on Plate 4-12.

c. Temporary Overburden Support. Shaft construction will require excavation and support of overburden, soils and fills existing above bedrock and control of groundwater inflow. The contractor will be responsible for final design of the earth support systems and groundwater control methods. Geotechnical information including boring logs, pumping test data, lab test results and piezometer readings will be available to the contractor during bidding. It is proposed that the excavation, earth support systems and groundwater control for each shaft be bid as a lump sum, thereby allowing the contractor to select his own combination of methods or alternatives subject to Corps approval.

Earth support systems are envisioned to consist of steel sheet piling or liner plates and ring beams with groundwater controlled by a properly designed interior pumping and sump system. This scheme would also require installation of a conventionally braced steel sheet pile cofferdam around the proposed intake and outlet structures. Steel sheet piling would be driven through the surficial fill and glacial outwash deposits to refusal in the underlying till layer. As excavation and dewatering proceed within the sheet pile cofferdam, ring beams would be installed for internal bracing. Liner plates and ring beams would be installed below the bottom of the steel sheet piling through the glacial till deposit to the bedrock surface. Additional liner plates and ring beams would be installed within the completely crushed upper bedrock zone for temporary rock reinforcement. Upon completion of the tunnel, the inlet and outlet shaft structures would be constructed with the temporary cofferdams left in place.

d. Linings of Shafts in Overburden. Preliminary design of the lining of shafts in overburden was based on conditions that exist at the inlet shaft. These conditions are more severe here than at the outlet shaft.

The shafts were designed for two loading conditions as follows:

(1) 23-foot differential soil load
(2) 70-foot external groundwater head

Calculations for the design of shaft lining are presented in Appendix F. These calculations show that a 15-inch reinforced concrete lining is required with one set of circumferential reinforcement on the outside face and another on the inside face. Each set would consist of No. 9 bars on 12 inch centers.

G. TUNNEL HEADWORKS, INTAKE AND OUTLET STRUCTURES

36. DESCRIPTION OF TUNNEL HEADWORKS. There are two proposed conduit systems which will provide flows to the relief tunnel. One system is Town Brook itself. Rates of flow, however, will be increased by construction of the proposed Brook Road Diversion Conduit by the Metropolitan District Commission (MDC). The other system is the existing Burgin Parkway Culvert constructed in conjunction with the Burgin Parkway.

A site plan of the intake area is shown on Plate 4-13 and plans, sections and details of the structures are shown on Plates 4-13 thru 4-16.

a. Town Brook. The Town Brook system will comprise the existing 8.5 ft by 5 ft culvert in Brook Road (non-Corps) and the proposed 12 ft by 8 ft Brook Road Diversion Conduit (non-Corps). Downstream of the junction of these two conduits in Fort Square (see Plate 4-1), a proposed gabion-walled channel, 14 feet wide, will extend to about the western boundary of the tunnel intake area site (non-Corps). From this point, a 14-foot-wide concrete channel, about 200 feet long, will extend to an overflow structure (Corps). At the overflow structure, the channel will be widened and deepened to reduce flow velocities and thus facilitate grit deposition in the channel and discharge over the 51-foot long side overflow weir. Downstream of the overflow structure a 10 ft by 2 ft restricted opening will be provided in the existing 10-ft by 5-ft box conduit. This restriction will reduce flow amounts discharged to the downstream Town Brook system. The maximum velocity to be experienced at the restriction will be about 6 fps. Diverted flows of the overflow structure will pass through a bar rack and then be directed to the tunnel shaft.

b. Burgin Parkway Culvert. The existing Burgin Parkway Culvert is a 10-ft by 8-ft concrete box culvert extending about 4,000 feet from Centre Street. Its purpose is to convey to the tunnel stormwater overflows from Town Brook from areas upstream of Centre Street and runoff entering the culvert between Centre Street and School Street. At the intake, the Burgin Parkway Culvert will be connected to a forebay from which the flow will pass through four 8-ft by 6-ft openings into two grit deposit basins, each 22 feet wide by 50 feet long. Flow levels in the grit basins will normally be controlled by a broad crested weir. In exiting the basins, flows will pass through a bar rack mounted on the weir and then into the tunnel shaft.

37. DESCRIPTION OF OUTLET STRUCTURES. A site plan, channel profile and plans and sections of the outlet structure and outlet channel are shown on Plate 4-17, and structural details are on Plate 4-18.

a. Outlet Structure. The outlet structure is designed to provide a transition from the 12.5-foot diameter outlet shaft to the 60-foot wide outlet channel. In plan, this transition is made in a distance of 40 feet from the shaft centerline. In the vertical, the invert transition is made between elevations -20.0 feet msl and 2.0 feet msl. At the shaft, a

warped curved lip will transition into a 30 degree plane, at the top of which a 20-ft radius curve will transition the invert to the horizontal. Five 10-ft wide exit bays will be provided. Provision is made in each bay for stop logs and a bar rack. The main purpose of the latter is to provide a barrier against human entry into the outlet structure. It would also keep out large debris which could be carried up the channel by strong winds and incoming tides.

b. Outlet Channel. The proposed outlet channel will be 60 feet wide and will extend from the outlet structure for about 300 feet to a junction with Town River. Because of its proximity to the city of Quincy recreational area and, in particular, a baseball field, the channel walls will be essentially vertical, composed of gabions in lieu of sloped banks which would further restrict the available playing field. The wall height will be 6 feet except where reduced heights will suffice at the junction with Town River. The proposed invert of the outlet channel is elevation 2.0 feet msl at the outlet structure and elevation 1.6 feet msl at a point 250 feet downstream where the channel will drop vertically 1.5 feet to elevation 0.10 feet msl. At this junction, a drop structure is required to protect the gabion walls from unravelling at the ends from erosion. The drop structure will be constructed of gabions with a concrete cap at the lip of the drop. A 9-inch gabion mattress is proposed for the channel bottom.

38. SITE DEVELOPMENT.

a. Inlet Area. In addition to the inlet area structures previously mentioned, the indicated development of the site includes provision for an access road, retaining walls and fencing.

An approximately 220-ft long, 14-ft wide access road with a 3-inch - 2 course bituminous pavement on a 12-inch gravel base is proposed. A plan, profile and typical sections are shown on Plate 4-13. Site limitations (i.e., steep slopes, limited area, pump station) require locating the entrance of the road between the west property line and the city of Quincy sewage pumping station. The roadway design requires the construction of a retaining wall approximately 42 feet long along the west property line with fill slopes of 1 on 2. After curving around the proposed pumping station the access road, with fill slopes approximately 1 on 2, will parallel the Town Brook channel in extending to the paved area surrounding the inlet structures. This paved area provides space for maneuvering trucks or heavy equipment required for grit removal and maintenance.

A retaining wall about 190 feet long is required to support the bank between School Street and the grit basins. A typical section of the wall is shown on Plate 4-14. The top of the retaining wall will be elevation 34.0 feet msl for most of its length. At its easterly end, the wall will abut a retaining wall along Burgin Parkway. The proposed final grades at the retaining wall are elevation 23.5 feet msl on the grit basin side and

elevation 33.0 feet msl on the School Street side with a cut slope of 1 on 3 behind the wall intersecting the existing grade.

At the eastern portion of the site between the grit basin and the wall along Burgin Parkway, a space, similar to that now existing, will be made available for maneuvering of large trucks servicing the Star Market.

A 5-ft high chain-link fence surrounding the entire inlet area is proposed primarily for safety reasons. Two 16-ft wide double gates will be provided, one located at the access road entrance and the other at the Star Market truck turnaround.

In addition, the final design will include provisions for drainage from the paved surfaces and stop log storage.

b. Outlet Area. A site plan of the tunnel outlet area is shown on Plate 4-17. Access to the outlet area structures will be provided by constructing a 30-ft wide paved roadway for a length of approximately 245 feet along presently undeveloped Vining Street. In addition, a large area surrounding the outlet structure will be paved to provide maneuverability for trucks. All of the proposed pavement is 3-inch - 2 course bituminous concrete with a 12-inch gravel base.

The site will be surrounded on three sides by a 6-ft high chain-link fence. The fence will extend from along the west property line from the proposed improved Town River channel to Vining Street. It will also continue along Vining Street for about 120 feet and along the southern side of the outlet channel and the proposed improved Town River channel. A 30-ft wide double gate will be provided at the entrance to the site from Vining Street.

39. STRUCTURAL DATA. The inlet and outlet areas for the Town Brook tunnel include the following structures:

a. Inlet Structures. The tunnel inlet receives stormwater from the Burgin Parkway culvert and overflow stormwater from Town Brook. The water from the Burgin Parkway culvert flows through twin inlet grit basins before being discharged into the tunnel shaft. During normal flow, the water in Town Brook passes through a brook grit basin and continues downstream via the existing conduits. During periods of high stormwater runoff, however, excess flow is diverted over a side weir and into an overflow channel which discharges directly into the tunnel shaft.

The Burgin Parkway grit basins are the deepest of the inlet structures. The basins form a monolithic structure comprised of a bottom concrete slab and 18-foot tall walls. The top of the slab is at el. 6 (elevation 6.0 feet msl), and the weir at the end of the basin is at el. 13.3. The walls reach el. 24. The grit basin measures approximately 50 feet by 80 feet in plan.

A narrow grit basin is to be provided for the regular brook flow through the Town Brook culvert. The top of the brook basin slab is at el. 12.0 and the adjacent ground is at el. 22±.

The weir crest in the brook overflow channel is at el. 18.0. The bottom of the channel is at the same elevation as that in the parallel brook grit basin (el. 12.0). The overflow channel discharges directly into the tunnel shaft.

The vertical shaft for the diversion tunnel extends down from el. 2.0.

b. Outlet Structure. The vertical shaft at the outlet of the diversion tunnel rises to about el. -20, above which it widens into the outlet structure. The outlet structure is a monolithic, funnel-shaped structure with a flat roof slightly above the ground surface (top of slab el. 12). Water leaves the outlet structure at el. 2.0 and continues through an outlet channel to the Town River about 300 feet from the structure.

40. SUBSURFACE CONDITIONS AT INLET. The following generalized subsurface stratigraphy was encountered at the relief tunnel inlet in order of increasing depth:

Layer 1 - Miscellaneous, but primarily granular fill consisting of sand, gravel, little silt and occasional boulders, cinders and ash. Most of the fill varies from loose to dense with occasionally high blow counts presumably caused by gravel and boulders. The entire site is covered by fill ranging in thickness from about 3 to 27 feet, but averaging approximately 20 feet. The bottom of the fill varies between el. 13 and el. 26, but is close to el. 20 in most of the borings.

Layer 2 - A 5-ft layer of organic silt and sandy peat were found in two borings (GC-1 and D-13) located within 40 feet of the Town Brook Channel.

Layer 3 - At the boring location closest to the channel, (D-13), the organic materials are underlain by stream deposits consisting of loose to medium dense fine sand and silt to el. 0.

Layer 4 - Medium dense to very dense, poorly graded silty sands and gravels were encountered in all the borings, and most of the borings were terminated in this stratified glacial outwash material. This layer was found to be 28 and 45 feet thick in borings D-13 and T-2, respectively.

Layer 5 - Dense to very dense silty sand, glacial till, overlies the bedrock. This layer was found to be 9 feet thick in boring D-13, and 14.5 feet in boring T-2.

Layer 6 - The bedrock consists of Quincy granite. The top of the bedrock was encountered at el. -37 and -47 in borings D-13 and T-2, respectively. The quality of the granite was poor to fair (Rock Quality Designation, RQD from less than 50 to 75 percent) above el. -40 and -37 in the two borings, but good to excellent (RQD 85 to 100 percent) within the first 20 feet below el. -40 and -37.

Groundwater levels observed in the permanent observation well, T-2, at the inlet site in the months of March through July 1983, varied between el. 16.9 and el. 19.1. Groundwater levels observed in the five adjacent borings at their completion in the months of August 1979, September 1975 and November 1982 ranged from el. 14.1 to 17.6. Simultaneous readings in three wells prior to the pumping test in July 1983 show a 0.2 foot variation across the site and a groundwater gradient from southwest towards northeast and the brook.

A 20-year record of observations exists for USGS well XGW-4 installed in glacial sand and gravel deposits in Weymouth about 4 miles south of the site. Using the maximum fluctuations in this well, and the statistical extreme for this type of deposit in Massachusetts, the probable high groundwater level at the site was found to be at el. 21 by a procedure outlined in USGS Open - File Report 80-1205. This probable high groundwater level is up to 6 feet above the bottom of the present fill at the site.

The 50-year design high water level in Town Brook is at el. 19.8.

41. SUBSURFACE CONDITIONS AT OUTLET. The generalized subsurface stratigraphy listed below in order of increasing depth was found at the site of the relief tunnel outlet and the outlet channel:

Layer 1 - Silty sand and gravel fill. At the outlet structure, the fill occurred only in boring T-10 and was placed to provide access for the drill rig. Four feet of fill was also found in CH-2 and 8 feet in D-18 at the outlet channel.

Layer 2 - Soft organic silt and peat were encountered in all of the seven borings. The organic materials were covered by fill only in borings T-10, CH-2 and D-18. The bottom elevation of these tidal marsh deposits ranged from el. -0.5 to el. -3.2 at the outlet structure. In the outlet channel, it was found at el. -7, -5.4 and 1.5 in borings CH-2, D-18 and CH-3, respectively.

Layer 3 - Medium dense to very dense silty sands and gravels, frequently poorly graded, and occasionally interbedded with layers of medium dense, non-plastic silt. Five of the borings were terminated in this stratified glacial outwash deposit which extended to el. -24 in boring T-10, the only boring which penetrated Layer 3.

Layer 4 - Very dense silty sands and hard non-plastic silt, glacial till, underlie the outwash materials in boring T-10.

Layer 5 - The bedrock consists of Weymouth argillite, the top of which was encountered at el. -50. The quality of the first 15 feet of the argillite was poor (RQD less than 50), but below el. -65 the quality was fair (RQD higher than 50) and gradually increasing.

The groundwater levels observed in the months of March through July 1983 in the permanent observation well in boring T-10 at the site were at el. 4 to 6. The 25-year design high runoff and tide water level is at el. 7.0 and is recommended as the probable high groundwater level.

42. SEISMIC DESIGN FACTORS.

The project is located on the border between zones 2 and 3 on the *seismic zone map of the Department of the Army, U.S. Army Corps of* Engineers, ER 1110-2-1806. Seismic activity primarily affects the walls of the proposed structures, and U.S. Army Corps of Engineers Regulation EM 1110-2-2502 specifies that retaining walls should be designed against earthquake accelerations of 0.10g and 0.15g (where g is gravity) in zones 2 and 3, respectively. The Massachusetts State Building Code specifies that structures in Massachusetts should be designed against an earthquake acceleration of 0.12g, and a horizontal seismic coefficient of 0.12 had initially been used in the design of the inlet and outlet structures. The structures are presently being redesigned using a horizontal seismic coefficient of 0.15.

43. DISCUSSION AND RECOMMENDATIONS FOR INLET STRUCTURES.

a. General. The potentially very high groundwater level at the inlet site is the most important foundation design consideration and, possibly, also the greatest concern during construction.

b. Design Against Uplift. The probable high groundwater level recommended at the inlet site is at el. 21. The bottom of the inlet grit basin (bottom of slab) is at el. 3. An extensive drainage system beneath and around the grit basin will be used to lower the groundwater table around and below the basin and thereby permanently reduce the hydrostatic uplift pressure. The drainage layer employed consists of a minimum 3-foot-thick layer of screened gravel placed beneath the slab and continued up behind the walls. Porous drain pipes of minimum 8-inch diameter are located in the drainage layer and discharge into the inlet shaft at el. 3. Filter cloth will be placed prior to the screened gravel between the screened gravel and the natural ground.

The factor of safety against uplift for the grit basin was computed assuming groundwater level at elevation +15.0 with weep holes located at elevation +14.0. The result was a safety factor of 1.1. This represents an effective drawdown of the assumed water table elevation (+21.0) of approximately 33% due to the 8" underdrain system and weep hole arrangement. This is considered a conservative assumption.

c. Foundation Subgrade. The foundation subgrade of all portions of the proposed three inlet structures except the free-standing retaining wall extends 5 to 15 feet into the outwash deposit of medium dense to dense sand (layer 4). In its undisturbed state, this material should provide an adequate foundation if properly protected against piping due to groundwater seepage during excavation below the natural groundwater table. For that reason, it is recommended to overexcavate at least 1 foot and replace the outwash material with a working mat of screened gravel. About half of the outwash materials contain more than 10 percent fines and are, for that reason, frost susceptible. It is recommended that they be removed to a depth of 4 feet below final grade and be replaced with clean granular material (processed gravel), if they are not to be replaced with screened gravel for other reasons. Organic silt and stream deposits including loose silt were found within 40 feet of the brook. These soft materials will be removed and replaced with compacted processed gravel. An allowable bearing pressure of 2.0 TSF and a subgrade reaction of 100 tons per cubic foot (TCF) is recommended.

d. Lateral Earth Pressures. The three inlet structures include several walls which are integral parts of basins and channels, and one free-standing retaining wall. The retaining wall is designed against the active pressure while the remaining walls, because of their restricted movements, are designed against the higher at-rest earth pressure. All of the walls are designed against earthquakes. The seismic pressures were initially determined based on a horizontal seismic coefficient of 0.12 and no vertical acceleration. They will be redesigned with a horizontal seismic coefficient of 0.15. The static and seismic earth pressures presently applied on the two types of walls are shown in Figures 4 and 5 and Table 5. These earth pressure diagrams are based on the following assumptions:

- A 2-foot thick vertical drain of screened gravel is installed behind the walls. The drain is connected to the underdrain and discharges into the tunnel shaft. This drain will reduce water pressures on the walls.

- Backfill behind all walls is a clean granular material (processed gravel). This material should extend to a plane rising 1:1 from the heel of the wall (footing).

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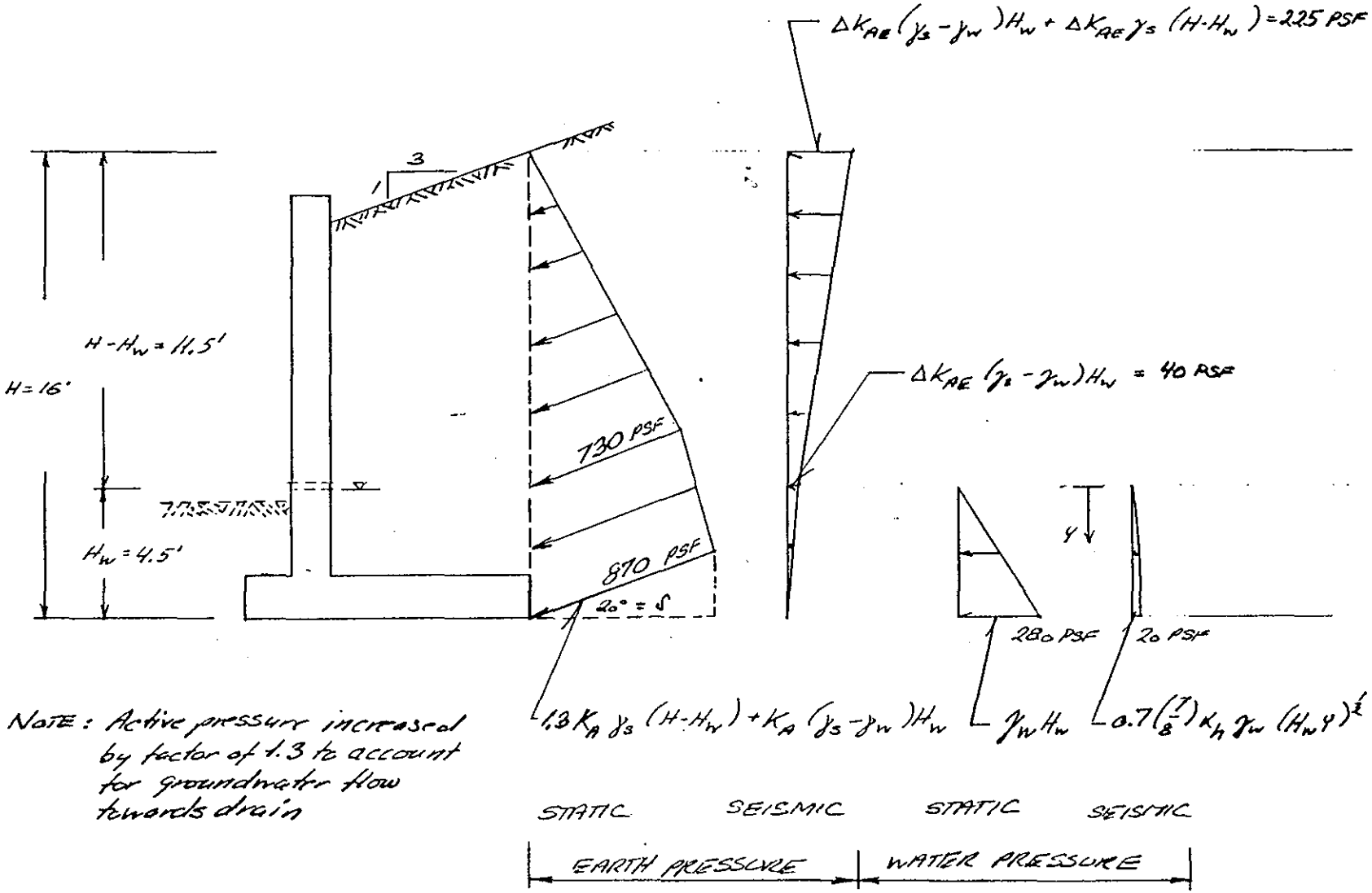
Detail ___ Ck'd. By MLL Date 5-2-84



NOTE: Active pressure increased by factor of 1.3 to account for groundwater flow towards drain

FIG 4. LATERAL EARTH PRESSURES ON FREE-STANDING WALL AT INLET AREA

NONREPRODUCIBLE GRID FORM 145

METCALF & EDDY, ENGINEERS

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Subject: INLET & OUTLET STRUCTURES
Detail:
Acct. No. 6041
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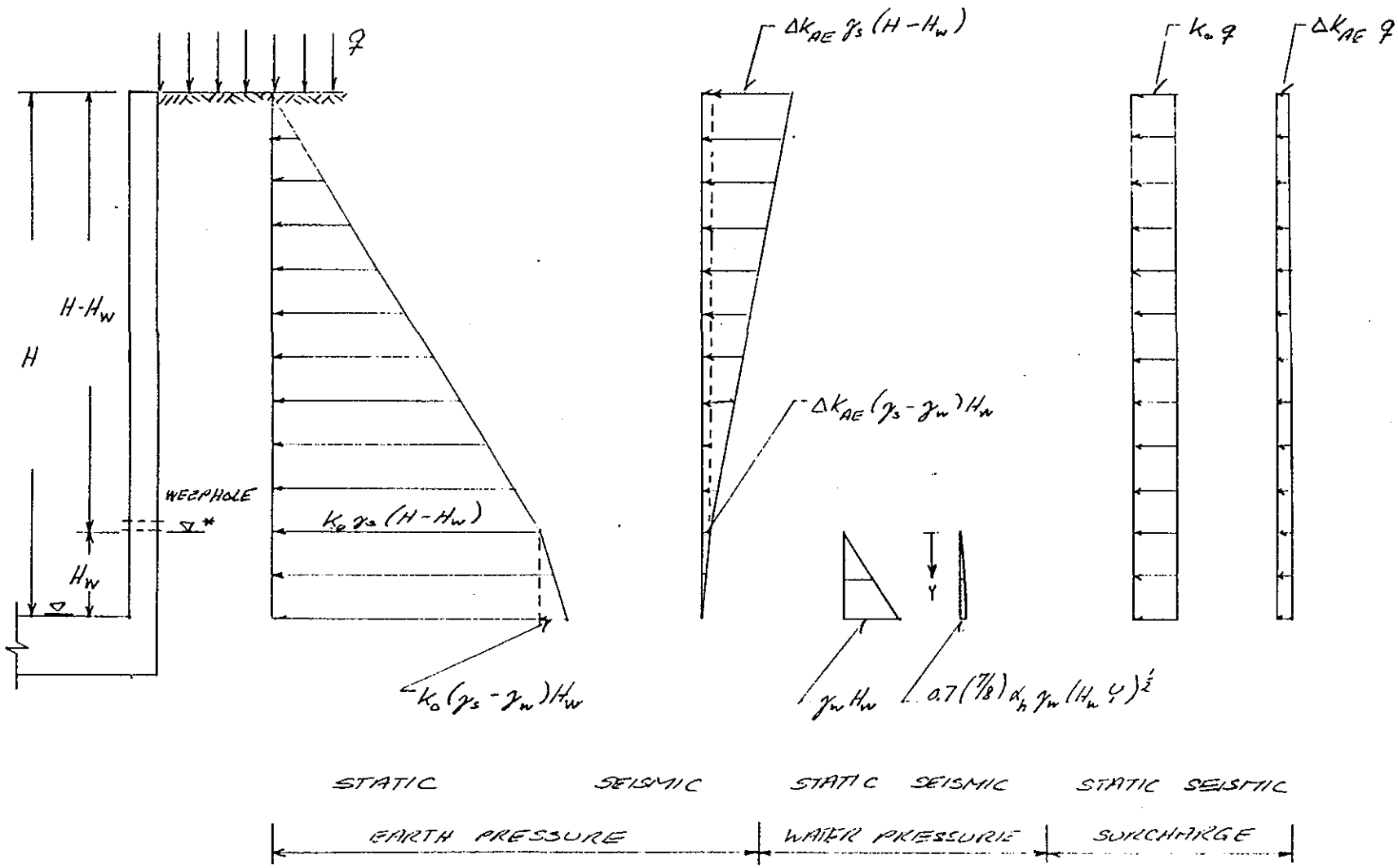


FIG 5 LATERAL EARTH PRESSURES ON RIGID WALLS AT INLET & OUTLET AREA STRUCTURES
(See Table 5)

TABLE 5. LATERAL EARTH PRESSURES ON RIGID WALLS ON INLET AND OUTLET AREA STRUCTURES

		Inlet structures			Outlet structure	
	Unit	Inlet grit basin	Brook overflow channel	Brook grit basin	Min	Max
H	ft	18	11	11	9	31
H_w	ft	8	2	4	9	31
γ_s (saturated density)	pcf	125	125	125	125	125
γ_w (water density)	pcf	62.4	62.4	62.4	62.4	62.4
q	psf	300	300	300	300	300
α_h	-	0.12	0.12	0.12	0.12	0.12
K_o	-	0.5	0.5	0.5	0.5	0.5
ΔK_{AE}	-	0.08	0.08	0.08	0.08	0.08
$K_o \gamma_s (H-H_w)$	psf	625	560	440	0	0
$K_o(\gamma_s-\gamma_w)H_w$	psf	255	60	125	280	970
$K_o[\gamma_s(H-H_w)+(\gamma_s-\gamma_w)H_w]$	psf	880	620	570	280	970
$\Delta K_{AE}(\gamma_s-\gamma_w)H_w$	psf	40	10	20	45	155
$\Delta K_{AE}\gamma_s(H-H_w)$	psf	100	90	70	0	0
$\Delta K_{AE}[(\gamma_s-\gamma_w)H_w+\gamma_s(H-H_w)]$	psf	140	100	90	45	155
$\gamma_w H_w$	psf	500	125	250	565	1940
$0.7(7/8)\alpha_h\gamma_w(H_w^2)^{1/2}$	psf	40	10	20	40	145
$K_o q$	psf	150	150	150	150	150
$\Delta K_{AE} q$	psf	25	25	25	25	25

NOTE: Quantities above are identified on Figure 5.

- A 20-ton truck may be parked immediately behind the walls of basins and channels during cleaning and grit removal. However, a uniform surcharge of 300 psf acting on the surface behind the walls results in an additional, uniform pressure on the walls equal to the maximum pressure from the truck, and only a uniform surcharge has been included.

e. Construction Provisions. The inlet structures will be constructed in the dry and the groundwater table will be lowered to at least 2 feet below the bottom of the excavation prior to construction. The groundwater table must be lowered as much as 17 feet or more and the estimated discharge rate will exceed 1500 gallons per minute. If dewatering by pumping from sumps at the bottom of the excavation is attempted, wet conditions may be encountered and excavation slopes may not be stable. Consequently, construction dewatering by pumping from sumps alone is not recommended. Wellpoints connected to a header are capable of lowering the water table by about 15 feet and they must be used in stages to provide additional lowering. The wellpoints provide a better control of the groundwater flow conditions during excavation, construction of the structures, and during backfilling, and wellpoints in stages or deep wells are recommended. Construction dewatering will be provided for in the specifications. Temporary observation wells during construction will be installed and monitored to verify that groundwater is lowered below the bottom of excavation.

44. DISCUSSION AND RECOMMENDATIONS FOR OUTLET STRUCTURE.

a. General. The foundation subgrade of the proposed outlet structure is at the most 1 foot above the surface of the outwash deposits of medium dense to dense sand and silt. These deposits should provide an adequate foundation, if they are properly protected during excavation below the natural groundwater table.

b. Design Against Uplift. The probable high groundwater level at the outlet site is at el. 7.0, i.e. approximately at the existing ground surface. The bottom of the outlet channel is el. 2.0. Most of the outlet structure extends below el. 2.0, but the normal water level inside the structure is at el. 2 and would only drop significantly below el. 2, if the outlet shaft were to be dewatered. Thus, the maximum uplift against the outlet structure corresponds to a differential head of 5 feet of water. The uplift pressure will be lower during high tide and flood when the water level in the channel is above el. 2.0.

In lieu of providing a drainage scheme similar to the one proposed for the inlet structure, the outlet structure is designed to resist the full uplift pressure. Design against the uplift pressure has the additional advantage that substantial long-term settlements of the ground surface in the vicinity of the structure are avoided. Such settlements would have resulted from permanently lowering the groundwater table in the organic deposits in the surrounding tidal marsh.

The structure terminates with an uncovered apron over which the water is discharged into the outlet channel. The apron is on level ground and it is easy to install a drainage layer beneath it to relieve any uplift pressure. The drainage layer beneath the apron will also serve as a transition and filter zone in which to lower the water pressures beneath the apron before the water emerges at the bottom of the outlet channel in front of the outlet structure. Consequently, a drainage layer of screened gravel beneath the apron is recommended.

c. Foundation Subgrade. The subgrade elevation is about el. 0 for the apron area whereas the remainder of the structure varies from about el. 0 to el. -20. The materials at subgrade for the apron are very soft silts and peat deposits which extend to depths of about 1 foot below the bottom of the slab. The materials at foundation subgrade below the funnel shaped portion of the outlet are medium dense to dense sands and occasional nonplastic silts. It is recommended that all organic silt and peat deposits be overexcavated below the bottom of the structure and replaced with clean processed gravel or screened gravel compacted to 95 percent compaction. In any event, a 1-foot thick working mat of screened gravel is recommended. Frost protection requirements are as recommended for the inlet structures. An allowable bearing pressure of 1.5 TSF and a subgrade reaction of 80 TCF are recommended.

d. Lateral Earth Pressures. The walls of the outlet structure are integral parts of the structure and are, due to the concrete cover, even more restricted in their movements than the basin walls at the inlet structure. The walls will be backfilled with pervious granular material extending a minimum distance of 10 feet behind the walls. No organic materials shall be used as backfill or left in place beneath the granular backfill. The walls will be designed for the same loading conditions as the inlet structure, except that they will be designed against the full water pressure corresponding to the design groundwater table elevation of 7.0 feet.

e. Outlet Channel. An outlet channel leads from the outlet structure through the tidal marsh to the Town River located about 300 feet from the structure. The bottom of the channel varies between el. 2 and 1.6. The bottom of the organic deposits is at el. -2 at the outlet structure and at el. 1.5 in CH-3 at the end of the channel. However, approximately halfway between the two ends of the channel the bottom of the organics was observed at el. -7 in CH-2. The organic deposits will be removed and replaced with clean granular material (processed gravel) below any retaining structures along the channel. It is recommended to over-excavate without lowering the water table and backfill with clean processed gravel end dumped. The granular fill will be loose and the walls will settle an estimated maximum amount of 2 to 3 inches. Considering the nature of the walls, however, it is believed that such settlements can be accommodated, particularly by gabions.

A portion of the gabion walls will be situated on medium dense granular outwash materials. For foundations situated on either the granular fill or granular outwash materials, an allowable bearing pressure of 1 TSF is recommended. Channel retaining walls are favored over the alternative, extremely flat side slopes required in the organic deposits. In addition, gabion walls are recommended because they act as a weighted filter and will eliminate potential piping problems caused by the emerging seepage at the bottom and lower sides of the channel. The gabions will be wrapped in filter fabric to serve as a complete filter. The bottom of the channel will also be built-up as a weighted filter composed of filter fabric, a minimum 12-inch thick layer of gravel, under a 9-inch thick gabion mattress.

f. Construction Provisions. The outlet structure will be constructed in the dry. A temporary cofferdam will be required around the proposed outlet structure site in order to control groundwater inflows and limit the amount of organic materials required to be removed. Groundwater inflows will be controlled by pumping and sumping from within the cofferdam during excavation. The lowered groundwater table within the cofferdam will be maintained until the tunnel is filled with water in order to avoid uplift on the structure in excess of the assumed design uplift value.

H. TOWN RIVER CULVERT AND CHANNEL IMPROVEMENTS

45. CULVERT ALIGNMENT ALTERNATIVES. The 1978 Metcalf & Eddy report to the MDC and the 1980 NED Corps of Engineers Feasibility Report recommend that a new and enlarged culvert be provided across the Southern Artery highway and adjacent parking lot with sufficient capacity to pass the Project Design Discharge from the Town Brook Relief Tunnel plus the coincident flow in Town River without adverse localized flooding upstream of the Southern Artery. These reports also recommend that channel improvements be made both upstream and downstream of the proposed culvert in order to alleviate existing bottlenecks and channel inadequacies. Two alternatives were reviewed and investigated to determine the most feasible location for the proposed culvert. After the location was selected, a detailed hydraulic analysis was performed to establish the size of the culvert as well as the magnitude of the channel improvements.

a. First Alternative. The first alternative considered involved aligning the culvert such that the existing bridge would remain intact. Situated just west of the bridge, the proposed culvert would run parallel to the bridge for about 100 feet then curve slightly southeasterly in order to intersect the existing inlet roughly in line with the existing river channel. This alternative, although feasible, would involve bulkheading and backfilling the present river channel at the existing bridge. It would, therefore, be necessary to maintain the structural integrity of the bulkhead as well as the new culvert. Since the proposed culvert would also act as a bridge, this alignment would, in effect, necessitate maintaining two bridges side by side. This alternative would also unnecessarily spread the construction site and increase the required land acquisition. As a result, this will be an inefficient setup.

b. Selected Alternative. The second alternative, and chosen alternative, utilizes the existing bridge opening for construction of the culvert. The existing bridge would have its deck and west abutment wall removed and the culvert would run parallel to the remaining wall, approximately 18 feet away. Similar to the first alternative, the culvert would extend about 100 feet parallel to the existing bridge. It would curve slightly southwesterly and then proceed straight to bracket the existing inlet. This selected alignment is shown on Plate 4-19. In addition to eliminating the existing bridge, the second alternative provides for easier and less costly utility relocations.

46. UTILITIES. There are several major utilities that cross the proposed culvert location in the Southern Artery as shown on Plate 4-19. These utilities generally follow an inverted U shape path in order that they could be cast in the deck slab or set between the beams of the existing bridge. The preliminary design of the culvert alleviates the need for this change in elevation to accommodate the bridge. Because the culvert is set about 5 feet below the deck slab of the existing bridge, any replacement of utilities could incorporate a straight run above the culvert.

Each of the three major private utility companies, Boston Gas, Boston Edison and Massachusetts Electric, and the Quincy Water Department will need to have their pipeline or ducts realigned after the construction across the Southern Artery. Each utility company has been consulted as to their requirements during construction and only Massachusetts Electric cannot totally remove their 16-inch line from service for the several months that this work will encompass. The other utility lines will be removed during construction and replaced on new alignments after the culvert is in place.

Liaison between the utility companies and the general contractor will be required to facilitate an efficient replacement procedure. Although the contractor will be responsible for support of each line during construction, Boston Gas, Boston Edison and Massachusetts Electric, and the Quincy Water Department will be responsible for the work associated with the actual replacement of their lines. The arrangements thereof regarding the timing of utility relocations and responsibilities for the work will be specified in the contract documents. All costs associated with support and replacement of utilities during construction are presented in the cost estimates.

47. GEOTECHNICAL ENGINEERING.

a. Original Construction and Conditions. The original bridge over the Town River was built by the Massachusetts Department of Public Works (MDPW) in 1927 to provide a river crossing for the new Southern Artery. The end walls of the bridge are essentially two stone masonry and concrete retaining walls set on timber piles with a reinforced concrete beam supported concrete deck slab and bituminous base roadway laid across them. The existing span of the river channel under the bridge is about 35 ft., however, riprap slope protection reduces the actual channel bottom to 10-1/2 feet wide.

At the outlet end of the bridge, massive stone masonry blocks were laid atop each other to form two retaining wall structures that flared out from the bridge into the channel. The river channel was quite wide (100 feet) at the immediate outlet, and then flowed into a narrow wooden lock system for a short stretch. The Quincy Lumber Company is located along the south bank adjacent to a granite wall that runs for most of the length of the channel. Although the north bank was fairly evenly graded, this wall angles into the channel to form a bottleneck about 300 feet downstream of the bridge. The channel widens out again to about 70 feet for the remainder of the reach extending to the Town River Bay. In 1960, an A&P Supermarket was built adjacent to the east abutment of the bridge and the parking lot extended from the existing bridge about 150 feet upstream where a new inlet was formed. Two 72-inch-diameter corrugated metal pipes (CMP) were buried under the parking lot and connect the new inlet headwall to the bridge channel. Upstream of the headwall, the river channel was roughly 10 feet wide with sloped banks and meandered back through the surrounding tidewater marsh.

b. Existing Conditions. The inlet and outlet headwall structures have deteriorated badly in the past several years. The stone masonry blocks on both ends have been disturbed and are now strewn about the side banks. The two 72-inch CMP's have started to corrode and there are heavy silt deposits built up at this inlet. The upstream channel has widened to 20 feet across with now vertical banks. The downstream channel has eroded in several areas near the Quincy lumber yard and the historic antique lock system has fallen into disrepair with pieces of broken off timber choking the channel flow during low flow periods.

The bridge itself has a good solid foundation with minor chipping of the stone masonry and some wearing of the concrete. The bridge decking is in acceptable shape, however, the first signs of wearing are becoming apparent. The MDPW has said that this bridge is scheduled for a roadway resurfacing within 5 years and that major rehabilitation required on the superstructure is such that it is number 1,421 (first 600+ are deemed critical) on their priority list of state bridges to be repaired. This puts rehabilitation of this bridge at around the year 2069.

The former A&P building, a brick and cinder block structure, is supported by two types of foundation elements, caissons founded at depth on the dense silty sand and slabs on grade placed on fill at shallow depths. The caissons are both belled and straight shaft. The base elevation of the caissons are not known.

c. Field Exploration Program. A subsurface investigation of the study area was carried out by Metcalf & Eddy for the MDC in November 1982. The investigation included eight test borings at the structure location, ten test borings in the upstream channel section, eight test borings in the downstream channel section and three test borings in the north bank of the downstream channel section. The boring locations are shown and the logs are presented in Appendix I.

The borings ranged in depth from 10.0 feet to 42.5 feet. Surface elevations at each boring location were obtained by the Metropolitan District Commission. Groundwater elevation data was obtained from the observation wells at the completion of drilling and at selected intervals thereafter.

Soil samples and rock core were taken from these borings, and observation wells were installed in three of the borings. Laboratory tests were performed on selected samples for gradation, pH, conductivity, moisture content, and organic content.

d. Results of Subsurface Investigation. The soil profile components of the culvert area as shown in Appendix I can be described as follows:

(1) Overburden. The soil profile as encountered in the most current borings at the culvert location varies in thickness from 19.5 feet to 37.5 feet. Rock was encountered below these depths.

The overburden in general consists of gravel and cinder fill materials, soft organic silts and dense silty sands.

(a) Fill. The surficial fills consist of a heterogenouis mixture of loose to dense sand, silt and gravel with wood and cinders. This fill mixture varies in thickness from 4 feet to 15 feet and is considered unsuitable for reuse as backfill against structures and under foundation elements.

(b) Organic Silts. This soil zone consists of soft highly compressible organic silts with traces of fibers and shells and varies in thickness from 4.5 feet to 12 feet. This material is considered as unsuitable for reuse as backfill against structures and beneath foundation elements.

(c) Silty Sands. This soil zone consists of a very dense silty sand overlying the bedrock and varies in thickness form 10 feet to 18 feet.

(2) Bedrock. Bedrock at this location consists of Braintree Formation argillite and Quincy granite. The argillite is a medium hard rock and the granite is a very hard rock. Bedrock elevations at this location varies form El-8.7 to El-27.0. The Quincy granite was encountered only in one boring. One boring was carried to refusal and the remaining borings penetrated argillite. RQD values for the argillite varied from zero to 48 percent indicating a very poor to poor quality rock. An RQD of 26 percent for the Quincy granite was obtained indicating a poor quality rock for the depth cored.

e. Geotechnical Conclusions and Recommendations The structure site is overlain by a heterogenous fill and soft highly compressible organic silt. The design grade for the structure is located within the underlying soft organic silt layer.

The soft organic silts are not capable of carrying the anticipated loads that would be transmitted to this soil by normal design foundation elements. The site will require excavation, dewatering, the diversion or stream channel and tidal flows, removal of existing hydraulic structures and the protection of adjacent existing structure foundations.

The existing A&P structure is supported by caissons founded on the underlying dense silty sand layer and by foundation slabs constructed in the surficial fill deposit. In order to protect the adjacent A&P structure and roadway from detrimental settlements, and to permit excavation of the surficial fill and soft organic silt layers, a conventially braced steel sheet pile cofferdam will be required. Permanent steel sheeting

will be driven through the surficial fill and organic silt deposits and embedded in the underlying dense silty sands. The fill and organic silt deposits will be excavated within the cofferdam in the wet and tremie concrete fill will be placed up to the base of the new box culvert (elev. - 3ft. MSL). After the concrete fill has been placed, the remainder of the excavation can be dewatered to provide a dry environment for construction of the box culvert.

The tremie concrete fill below the bottom of the box culvert precludes the requirement for major excavation dewatering and reduces the risk of detrimental settlements to adjacent structures due to lowering of the surrounding groundwater table. With the concrete fill in place, dewatering is only necessary to remove water inflows occurring during tidal fluctuations. (elev. +6 ft. MSL to -3 ft. MSL).

Increased stream flows and associated higher flow velocities necessitate channel modifications within the upstream and downstream reaches of Town River. Channel modifications associated with the box culvert include channel widening, flattening of the channel banks and placement of gabion bank protection.

The upstream north and south bank will be flattened to a 1:3 (one vertical to three horizontal) slope and lined with 9-inch thick stone-filled gabion revet mattresses for erosion control and channelization. The downstream north embankment will also be excavated to a 1:3 slope and covered with 12-inch thick stone-filled gabion mattresses for erosion control. Placement of the gravel bedding, filter fabric and gabion mattress may be performed partially in the wet and during tidal fluctuations without dewatering the channel bottom.

48. CULVERT DESIGN. From hydraulic analysis, it was determined that the desired stage of el. 7.0 MSL for the 100-year project design flow in Town River could be achieved with a 3-barrel culvert each 16 ft wide x 7 ft high and the proposed channel modifications. It is proposed that the culvert will consist of three precast reinforced concrete box sections as shown in Plate 4-19. The use of precast sections will reduce construction time and hence reduce overall costs for the culvert installations. The time required for the construction across the Southern Artery is a most important factor because of the expected significant disruption of traffic along the Southern Artery during the culvert construction. The slight curve in the culvert alignment was derived to make a smooth intersection between the centerline of the proposed modified upstream channel and the centerline of the culvert parallel to the existing bridge.

The length of each 16 ft x 7 ft precast box section will generally be 6 feet, with shorter lengths used as required. Three special identical skew sections will be used at the outlet end.

The centerline length of the culvert is 275 feet, and the cross-sectional width, as shown on Plate 4-19, is 54 feet-4 inches.

At both the inlet and outlet ends of the proposed culvert a headwall structure will be required. The final grades over the culvert at the inlet will be quite similar to the existing conditions. A cast-in-place concrete headwall placed on top of the precast culvert sections is proposed to retain the backfill. The concrete wall would extend for the full width of the culvert, would be approximately two and a half feet thick and would have a top elevation of 11.7. A handrail would be provided here. The same back configuration is proposed for the outlet headwall, except that a curb wall would extend 12 inches above the sidewalk with a highway guardrail set on top for traffic and pedestrial protection.

Retaining walls would also be required on each side of the culvert at the inlet and outlet ends. They would be constructed of cast-in-place reinforced concrete.

The proposed inlet retaining walls are symmetrical and are joined at the channel level by a concrete apron that would extend from the culvert out into the channel for about 10 feet. The retaining walls would flare out at 45 degrees from the outer walls of the culvert for a length of 30 feet. An expansion joint would be located between the headwall and the retaining walls.

The outlet retaining walls are also jointed at the channel level by a concrete apron which extends 20 feet into the channel. The walls extend out into the channel on a line with the outer culvert walls for a distance of 20 feet and then flare out at a 45 degree angle for 30 feet. An expansion joint would also be provided between the headwall and retaining walls.

Plates 4-19 through 4-22 show proposed plan layouts, sections and details of the culvert inlet and outlet headwall/retaining wall system. Details of areas of special concern are discussed in the Section 51, Structural Design.

49. UPSTREAM CHANNEL IMPROVEMENTS. The channel at the culvert inlet will be widened to 52'-4" which is the width of the culvert minus the outside walls. The bottom will be the concrete apron discussed in the previous section and the sides will be the concrete retaining walls. Upstream of the concrete apron the channel will have a fairly uniform bottom width of 52 feet. The channel banks will slope at a 1:3 grade up to the existing top of bank beyond which the existing grade will be kept. Slope and bottom protection will consist of a continuous 9-inch stone filled gabion revet mattress laid from top of bank to top of bank. At Station 6+00 the channel will start to widen to accommodate the proposed Town Brook Relief Tunnel outlet channel. The bottom of the channel will be about 60 feet wide at the intersection between the Town River channel and the incoming

tunnel outlet channel with the same 1:3 banks and the same slope and bottom protection. Above the intersection with the tunnel outlet channel the river channel will be widened to a 52-foot channel bottom with 1:3 slope banks. The bottom width will taper to match the existing channel at station 11+40. In addition to widening the channel sections, dredging of the bottom is also necessary. This is required to provide sufficient cross-section for the expected flow and to provide a smooth transition for gradual velocity changes under various flow conditions. A centerline profile and plan of the existing channel and the proposed channel is shown on Plates 4-23 and 4-24. A typical cross section of the improved channel is shown on Plate 4-20.

50. DOWNSTREAM CHANNEL IMPROVEMENTS. The channel at the culvert outlet will be tailored to accommodate the flaired retaining walls. The outlet elevation of -2.0 is maintained by a concrete apron that will extend 20 feet into the channel; this will prevent scouring of the channel bottom during high velocity flow. The channel bottom from Station 0+00 to -1+00 will also have to be dredged to conform to the -2.0 ft. elevation. The only other major modification to the channel is at Station -3+00 where the wall for the Quincy Lumber Yard and the existing bank form a bottleneck in the channel. The bottom of the channel will be widened at this location and the entire slope of the channel cut back on a 1:3 slope.

Slope and bottom protection will be similar to the upstream improvement recommendations as shown on Plate 4-20. Station 0+00 through -5+00 will be protected by a 12" stone-filled gabion mattress placed on the northern bank and along the toe of the slope. From Station 0+00 to 1+00 the gabion mattress will be placed across the entire width of the outlet channel (about 52 feet) between the concrete apron and the wooden lock system.

A centerline profile and plan of the existing channel and the proposed channel improvements down stream of the culvert are shown on Plates 4-23.

51. STRUCTURAL DESIGN. The headwall/retaining wall system for the culvert inlet and outlet will interact as one unit. For stability and to keep the mass of the walls to an aesthetic minimum, the retaining walls are joined together at the footing, and they tie into the headwall above the culvert with an expansion joint. This U-shaped footing arrangement will act as an apron that will be a deterrent against river bed scouring in addition to providing added structural strength.

The headwalls will be cast-in-place concrete that sets on top of the culvert. The raised curbs will be about 20-in. wide and will provide a base for protective railings at the Southern Artery and at the edge of the parking lot. The lower step will be about 12-in. wide and act as a base for the pavement. The headwall will be keyed into the culvert for stability and it will extend the entire width of the 3 barrel culvert to join with each retaining wall. As discussed, expansion joints should be provided between the headwalls and the retaining walls in both cases.

The retaining walls will be 1'-10" thick with vertical sides. These cantilever walls are considered more suitable for this installation than the more massive gravity walls. Typical details of the wall system are shown on Plates 4-21 and 4-22. The angle of the culvert will vary slightly between the inlet and the outlet, but the structural concepts are the same for both the inlet and the outlet.

A rounded nosing will be provided between the culvert sections on both inlet and outlet ends. These will be supported by footing that is common to the culvert section that the nosing is attached to.

I. CONCRETE MATERIALS

52. CONCRETE MATERIALS. Concrete will be obtained from existing commercial sources which meet all ACI and COE requirements. Concrete will be centrally mixed and delivered to the project site and placed in hoppers connected to a drop pipe connecting from ground elevation to tunnel elevation. Drop pipes will be located at the inlet and outlet shafts. Using these two locations the maximum placing distance from a drop shaft would be approximately 2000 feet. Concrete will be collected from the drop pipe by an agitating hopper at tunnel elevation and then transferred and placed by pump, or conveyed, to a pump for placement. Concrete materials are covered fully in DM 3, CONCRETE MATERIALS.

J. CONSTRUCTION SEQUENCE

53. CONSTRUCTION SEQUENCE The construction of the inlet and outlet area structures will be coordinated with tunnel and shaft construction. Excavation of the outlet shaft and the major portion of the tunnel excavation would be completed prior to commencing work on the inlet shaft. During this time the inlet site general excavation would be done to bring the site to the general elevation of the top of the inlet shaft. This would necessitate the construction of the retaining wall south of the intake structure. Also, the temporary diversion of flow in the existing Town Brook channel and construction of the new channel and overflow construction at Town Brook would be accomplished. This will reduce the potential for dewatering problems and provide increased protection from high Town Brook flows during construction. Roadway fill would be substantially completed before the inlet shaft excavation is initiated in order to provide access to the work area. After the shafts are completed, the inlet and outlet structures would be constructed.

The construction of the inlet and outlet systems are sufficiently independent so that construction can be undertaken concurrently at both sites. On this basis an approximate overall construction time of 15 months should be scheduled. If construction is done in conjunction with the tunnel and shaft work, it is expected that the work described herein would be completed within 10 months following completion of the lining of the shafts.

54. TUNNEL AND SHAFT CONSTRUCTION SCHEDULE. Construction of the project will be essentially linear insofar as the tunnel is concerned. The primary unknown factor is whether the contractor will elect to excavate by TBM or not.

Construction rates, assuming 2 shifts per day, used in development of the schedule are as follows;

Excavation of outlet shaft	5 feet per day
Excavation of inlet shaft	5 feet per day
Delivery time for TBM	6 months
Tunnel excavation by TBM	50 feet per day
Tunnel excavation by D&B	20 feet per day
Lining of outlet shaft	10 feet per day
Lining of inlet shaft	10 feet per day
Lining of Tunnel	50 feet per day
Mobilization	1 month

Secondary operations, such as grouting, installation of rock support, etc. are included in the above rates.

Following are approximate overall construction schedules for the tunnel and shafts, excluding surface structures. These schedules are based on 20 two-shift working days per month. Each asterisk represents one month, or major part thereof.

a. Tunneling by TBM with Cast-in-Place Liner

Months

	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1
										10										20	
Mobilization	*																				
Manufacture of TBM	*	*	*	*	*	*															
Excav. of outlet shaft		*	*																		
Excav. 200' tunnel for TBM				*																	
Excav. inlet shaft											*	*									
Set up TBM							*														
Excavate tunnel								*	*	*	*	*									
Line tunnel													*	*	*						
Line outlet shaft																*					
Line inlet shaft													*								

b. Tunneling by TBM with Precast Liner

Mobilization	*																				
Manufacture of TBM	*	*	*	*	*	*															
Excav. of outlet shaft		*	*																		
Excav. 200' tunnel for TBM				*																	
Excav. inlet shaft											*	*									
Set up TBM							*														
Excavate tunnel								*	*	*	*	*									
Line tunnel								*	*	*	*	*									
Line outlet shaft													*								
Line inlet shaft													*								

c. Tunneling by D&B

Mobilization	*																				
Excav. outlet shaft		*	*																		
Excav. inlet shaft													*	*							
Excavate tunnel				*	*	*	*	*	*	*	*	*	*	*							
Line Tunnel															*	*	*	*	*		
Line outlet shaft																				*	
Line inlet shaft															*						

55. CULVERT AND CHANNEL. Construction of the new Southern Artery culvert and Town River channel improvements can be undertaken independent of the tunnel and shaft work. It is envisioned that construction of the culvert and channel work will be completed by the time the tunnel headworks are put in operation.

Construction of the triple box culverts will be accomplished in three stages, as suggested on Plate 4-19, in order to accommodate the traffic patterns, stream diversion, utility relocations, and demolition of the existing bridge. Upstream and downstream channel improvements can be performed concurrently with the new culvert construction. Excavation to the new channel limits can be performed in the wet. Placement of gravel bedding, filter fabric, and the gabion mattress will be performed in the dry and partially in the wet during normal tidal fluctuation.

K. COST ESTIMATES

56. GENERAL. Cost estimates are shown in Appendix J. The detailed estimates presented in the GDM were primarily derived from figure developed by Metcalf and Eddy, the consultant to the local sponsor. Only the drill and blast tunnel excavation scheme was included in the GDM. Quantities and costs assumed in the GDM have been extensively revised and updated. Cost increases are explained below. In addition, two tunnel boring machine (TBM) options have been developed and priced. Improvements to the Quincy Reservoir dam shown in the GDM are being redesigned and will be presented in DM No. 2.

57. CHANNEL IMPROVEMENTS AND TOWN RIVER CULVERT. Table J-1 is the cost estimate carried in the GDM. Table J-2 shows an updated and revised detailed cost estimate for this portion of the project. The estimated cost has increased from $2,300,000 to $2,929,650. The difference is due mainly to revised quantity estimates.

58. RELIEF TUNNEL, SHAFTS, INLET AND OUTLET WORKS.

a. Cost Comparison of Options. Three tunneling options are considered: drill and blast excavation with a cast-in-place lining, TBM excavation with a cast-in-place lining, and TBM excavation with a precast lining. Detailed cost estimates are in Tables J-3 thru J-6 in Appendix J. The costs are as follows:

D&B w/cast-in-place liner	$23,205,645
TBM w/cast-in-place liner	$22,573,065
TBM w/precast liner	$20,276,590

b. Comparison with GDM Costs. Only the drill and blast tunnel scheme was presented in the GDM. Tables J-3 and J-4 in Appendix J show the GDM cost and new estimates. The revised and updated costs for this option have increased to $23,205,645 from $20,270,000 in the GDM. The increase in total cost is due primarily to revised quantity estimates and new items, and to some unit cost increases. The major items accounting for most of the increase are discussed below.

The estimated cost for mobilization is increased from $1,000,000 to $3,000,000 for the drill and blast scheme ($5,000,000 for TBM) based upon the government estimate and actual bid price from the Park River tunnel project (1977). Although the diameter and total length of the proposed Town Brook tunnel are smaller than Park River, the scope of mobilization is considered similar.

General excavation is increased from 12,000 cy to 50,000 cy based upon a re-evaluation of the requirements.

The intake and outlet shafts combined costs are decreased from $3,500,000 plus $156,000 for concrete linings in Table J-3 to $1,766,000 in Table J-4. The original estimate developed by Metcalf and Eddy was based upon the assumed use of slurry walls and assumed unit prices from various sources. New estimates were made based upon the proposed designs and associated quantities.

Tunnel lining concrete is revised form 6,130 cy to 10,530 cy to adequately account for assumed overbreak and the increased finish diameter from 12.0 feet to 12.5 feet. As previously stated in this report, further hydraulic analysis may prove that this is unnecessary.

Added items shown in Tables J-4 which were not included in the GDM are tunnel instrumentation ($350,000), geologic mapping ($120,000) and an aeration system ($434,000). The aeration system requires annual maintenance estimated to cost $12,000 yearly.

59. QUINCY RESERVOIR DAM. The GDM shows $1,500,000 for rehabilitation of the dam. NED has since been directed to consider replacing the dam. No costs are shown in this DM as feature DM No. 2 concerning the dam is still in preparation.

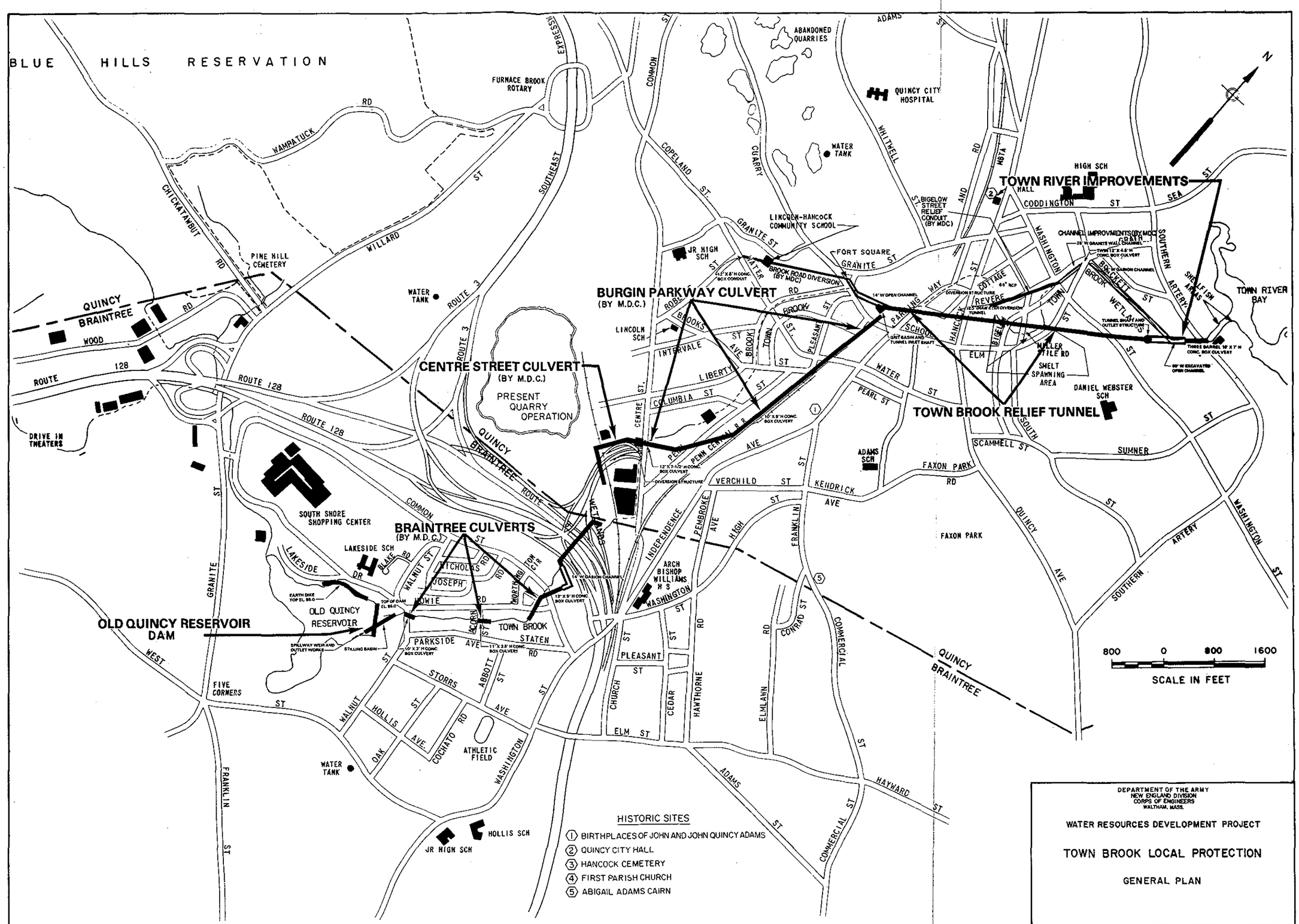
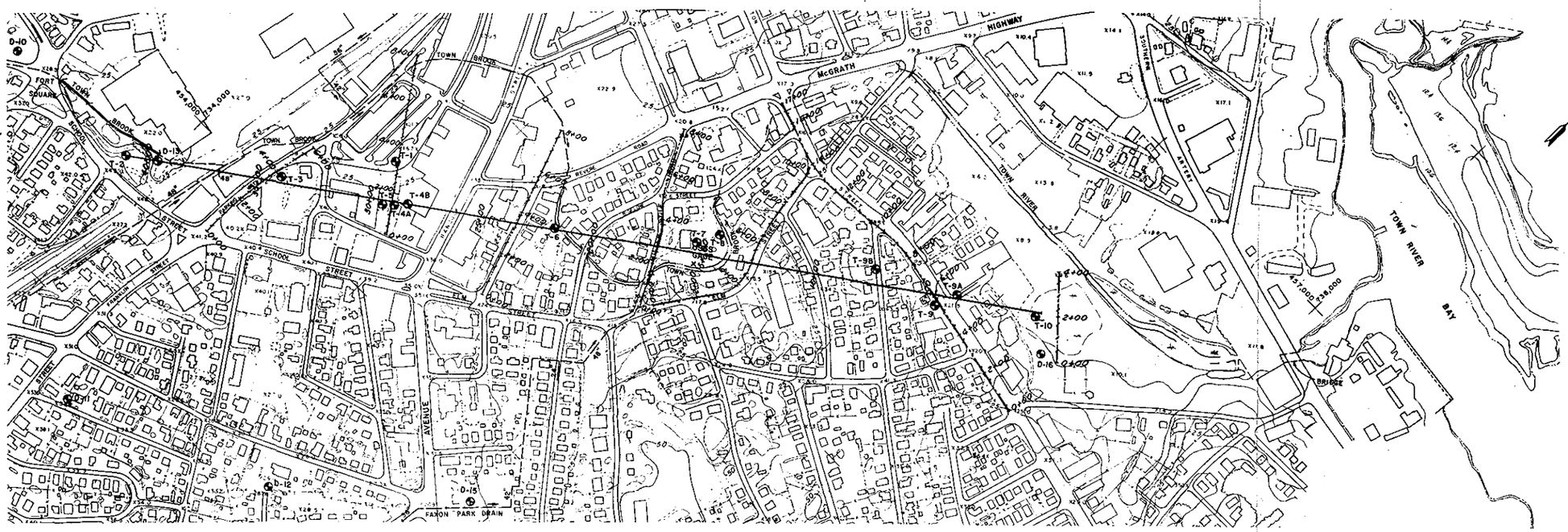

BLUE HILLS RESERVATION
FURNACE BROOK ROTARY
WAMPATUCK RD
CHICKATABUT RD
WILLARD ST
SOUTHEAST EXPRESSWAY
PINE HILL CEMETERY
QUINCY
BRAINTREE
WOOD RD
ROUTE 128
ROUTE 3
WATER TANK
DRIVE IN THEATERS
SOUTH SHORE SHOPPING CENTER
CENTRE STREET CULVERT
(BY M.D.C.)
PRESENT QUARRY OPERATION
BURGIN PARKWAY CULVERT
(BY M.D.C.)
BRAINTREE CULVERTS
(BY M.D.C.)
OLD QUINCY RESERVOIR DAM
OLD QUINCY RESERVOIR
EARTH DIKE TOP EL. 95.0
TOP OF DAM EL. 95.0
SPILLWAY WEIR AND OUTLET WORKS
STILLING BASIN
TOWN BROOK
LAKESIDE SCH
JR HIGH SCH
LINCOLN SCH
HOLLIS SCH
ATHLETIC FIELD
FIVE CORNERS
GRANITE ST
FRANKLIN ST
WASHINGTON ST
WEST ST
COMMON ST
COPELAND ST
QUARRY ST
ABANDONED QUARRIES
QUINCY CITY HOSPITAL
WHITWELL ST
ADAMS ST
LINCOLN-HANCOCK COMMUNITY SCHOOL
FORT SQUARE
GRANITE ST
BROOK ROAD DIVERSION (BY MDC)
BIGELOW STREET RELIEF CONDUIT (BY MDC)
HIGH SCH
TOWN RIVER IMPROVEMENTS
CODDINGTON ST
CHANNEL IMPROVMENTS (BY MDC)
DRAIN
SOUTHERN ARTERY
SHELLFISH AREAS
TOWN RIVER BAY
BROOKLEY ST
WETLAND
TOWN BROOK RELIEF TUNNEL
SMELT SPAWNING AREA
DANIEL WEBSTER SCH
SCAMMELL ST
SUMNER ST
FAXON PARK RD
FAXON PARK
ADAMS SCH
VERCHILD ST
KENDRICK AVE
ARCH BISHOP WILLIAMS H S
INDEPENDENCE AVE
PEMBROKE AVE
HIGH ST
FRANKLIN ST
QUINCY AVE
SOUTHERN ARTERY
WASHINGTON ST
CONRAD ST
COMMERCIAL ST
QUINCY BRAINTREE
PLEASANT ST
CHURCH ST
CEDAR ST
HAWTHORNE RD
ELMLAWN RD
ELM ST
ADAMS ST
HAYWARD ST
PEARL ST
WATER ST
LIBERTY ST
COLUMBIA ST
INTERVALE ST
PARKSIDE AVE
STATEN RD
STORRS AVE
ABBOTT ST
WALNUT ST
HOLLIS AVE
OAK ST
COCHATO RD
NICHOLS RD
JOSEPH RD
LOWIE RD
NORTHWOOD RD
TOWN CTR
LAKESIDE DR
BLAKE RD
N
800 0 800 1600
SCALE IN FEET
HISTORIC SITES
1 BIRTHPLACES OF JOHN AND JOHN QUINCY ADAMS
2 QUINCY CITY HALL
3 HANCOCK CEMETERY
4 FIRST PARISH CHURCH
5 ABIGAIL ADAMS CAIRN
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
GENERAL PLAN

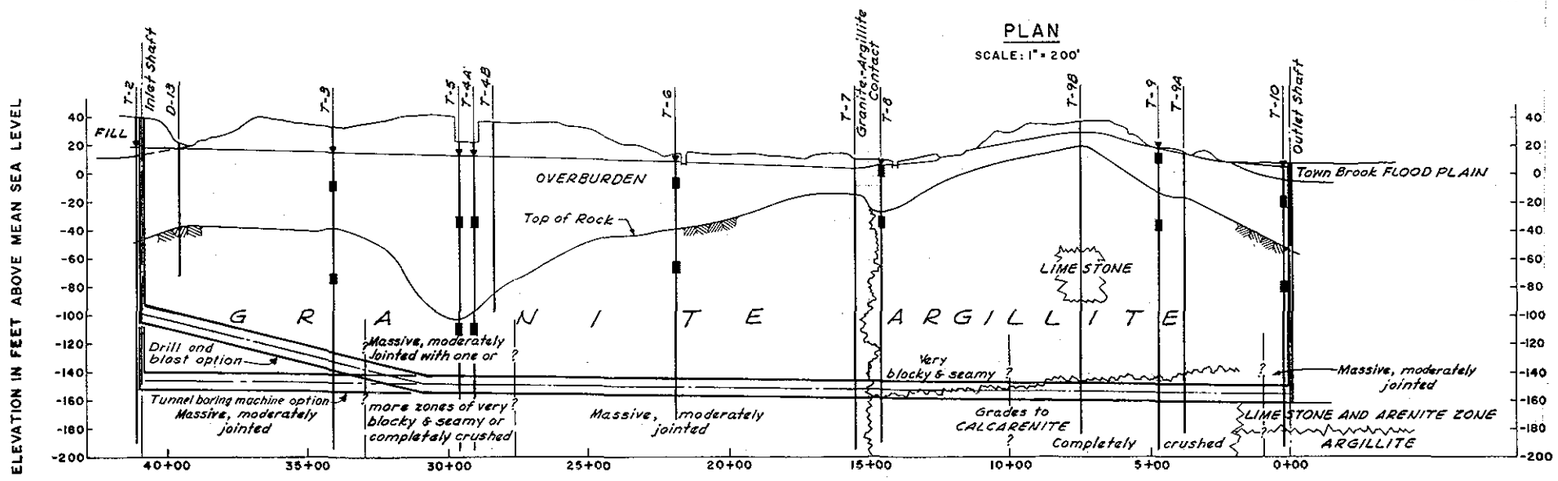
PLATE 4-1

CORPS OF ENGINEERS — U. S. ARMY



PLAN

SCALE: 1" = 200'



GEOLOGIC PROFILE ALONG ℄ OF TUNNEL

SCALE: HOR. 1" = 200'
VERT. 1" = 40'

LEGEND

- Piezometer
- Water level from bedrock piezometers
- Contact between rock types (inferred)
- T-10 Boring T-series, 1983
- D-16 Boring D-series, 1975
- Trace of Seismic Refraction Survey 1980

NOTE:
Rock quality indicated is from explorations and is for general guidance. Actual rock quality encountered during construction may be better or worse than that indicated.

GRAPHIC SCALES
SCALE 1" = 200'
200' 0 200' 400'

REVISION	DATE	DESCRIPTION	BY
		Piezometers added.	COE
		TBM option alignment added	COE

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY | DR. BY | CK. BY
SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

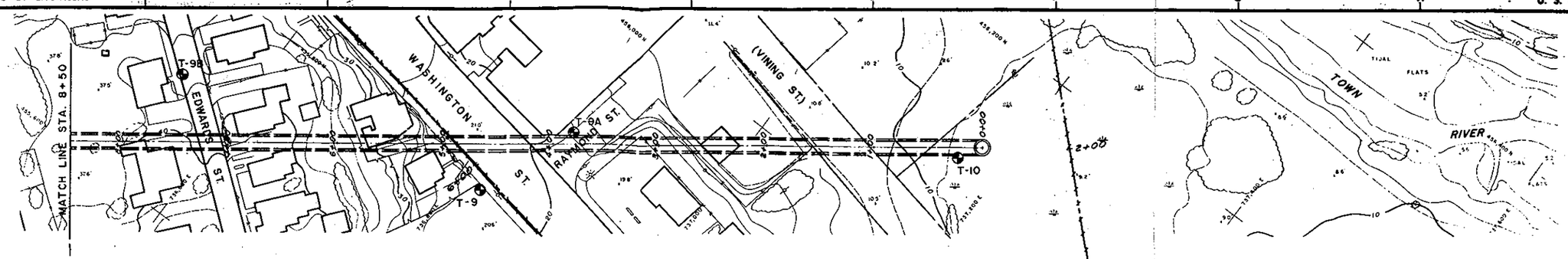
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
PLAN AND GEOLOGIC PROFILE
QUINCY AND BRAINTREE, MASSACHUSETTS

APPROVED: DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO. | DRAWING NUMBER
SHEET 5

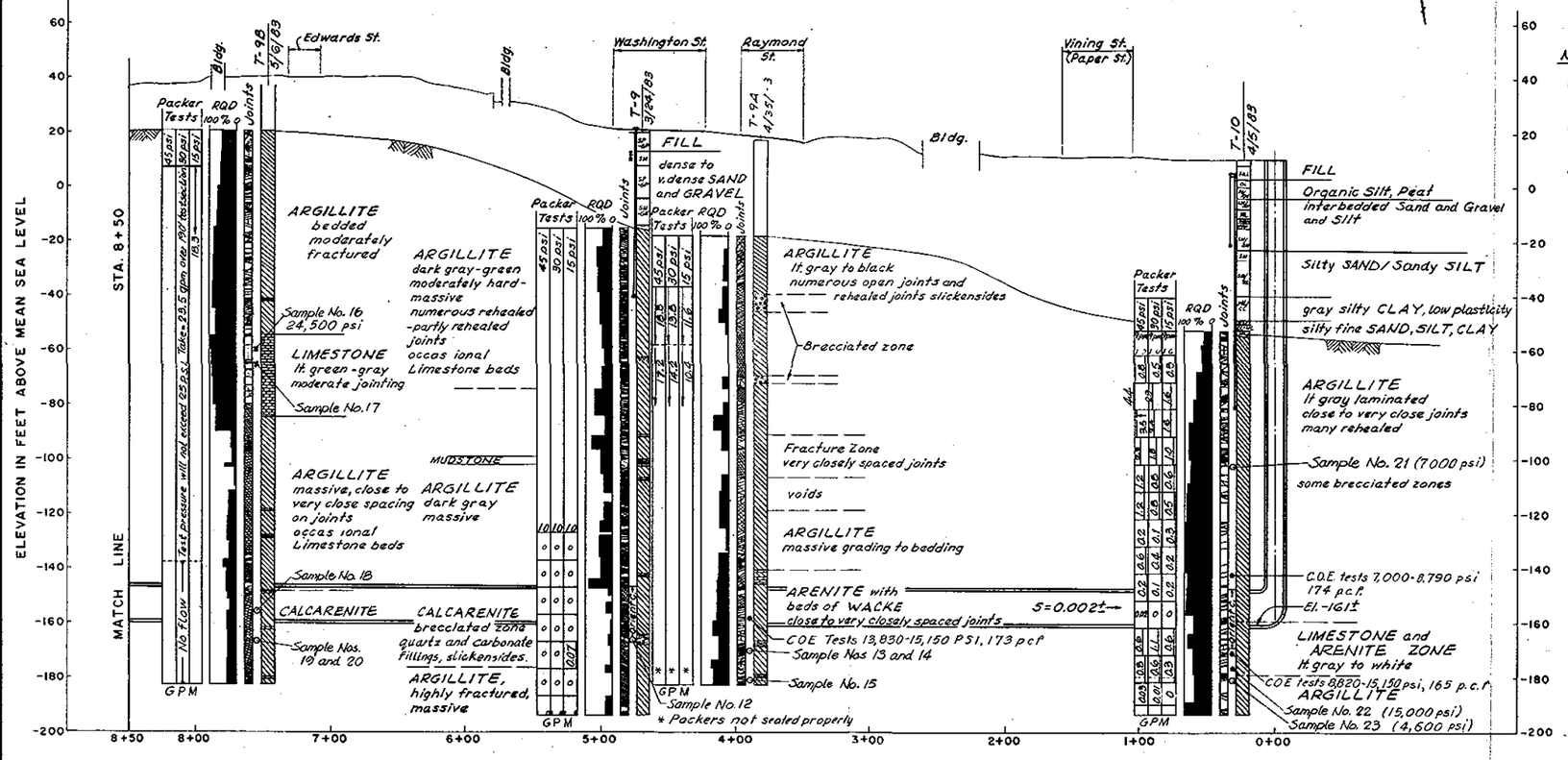
PLATE 4-2

CORPS OF ENGINEERS — U. S. ARMY



PLAN

SCALE: 1" = 40'



GEOLOGIC LOG PROFILE

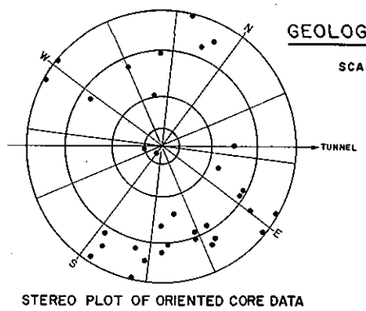
SCALE: HOR. 1" = 40' VERT. 1" = 20'

NOTES:

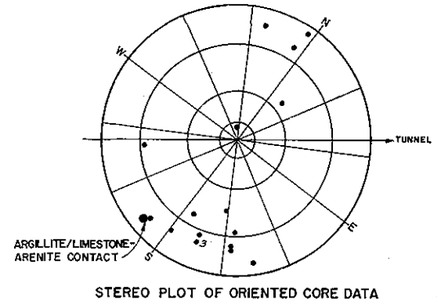
1. Overburden soils are classified according to the Unified Soil Classification System (ASTM D 2487-83)
2. RQD (Rock Quality Designation) is determined according to Deere et al, 1967.
3. Pressure tests in the rock were run at 3 pressures 45 psi, 30 psi and 15 psi. The water-loss is shown in gallons per minute.

LEGEND

- Pressure head
- Location of piezometer
- Symbols for rock type determined by petrographic examination
- ARGILLITE
- GRANITE
- Bracciated Zone
- LIME and QUARTZITE beds
- T-10 Boring locations and number
- Seismic Refraction Survey



STEREO PLOT OF ORIENTED CORE DATA IN T-9 EL. -145 TO -186



STEREO PLOT OF ORIENTED CORE DATA IN T-10 EL. -150 TO -170

GRAPHIC SCALES
SCALE IN FEET
40 0 40 80
1" = 40'

REVISION	DATE	DESCRIPTION	BY
		COE test values added.	COE
		Rock test values added.	COE
		Oriented core data plots added.	COE

METCALF & EDDY, INC.
BOSTON, MASS.

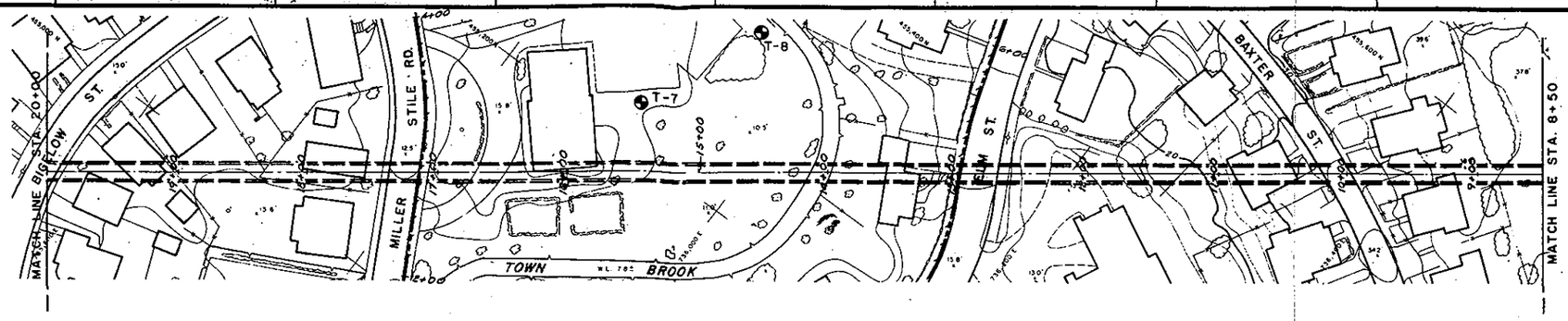
DES. BY / DR. BY / CK. BY
SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
PLAN AND GEOLOGIC LOG PROFILE
STA. 0+00 TO STA. 8+50
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE SPEC. NO.
DRAWING NUMBER
SHEET 6

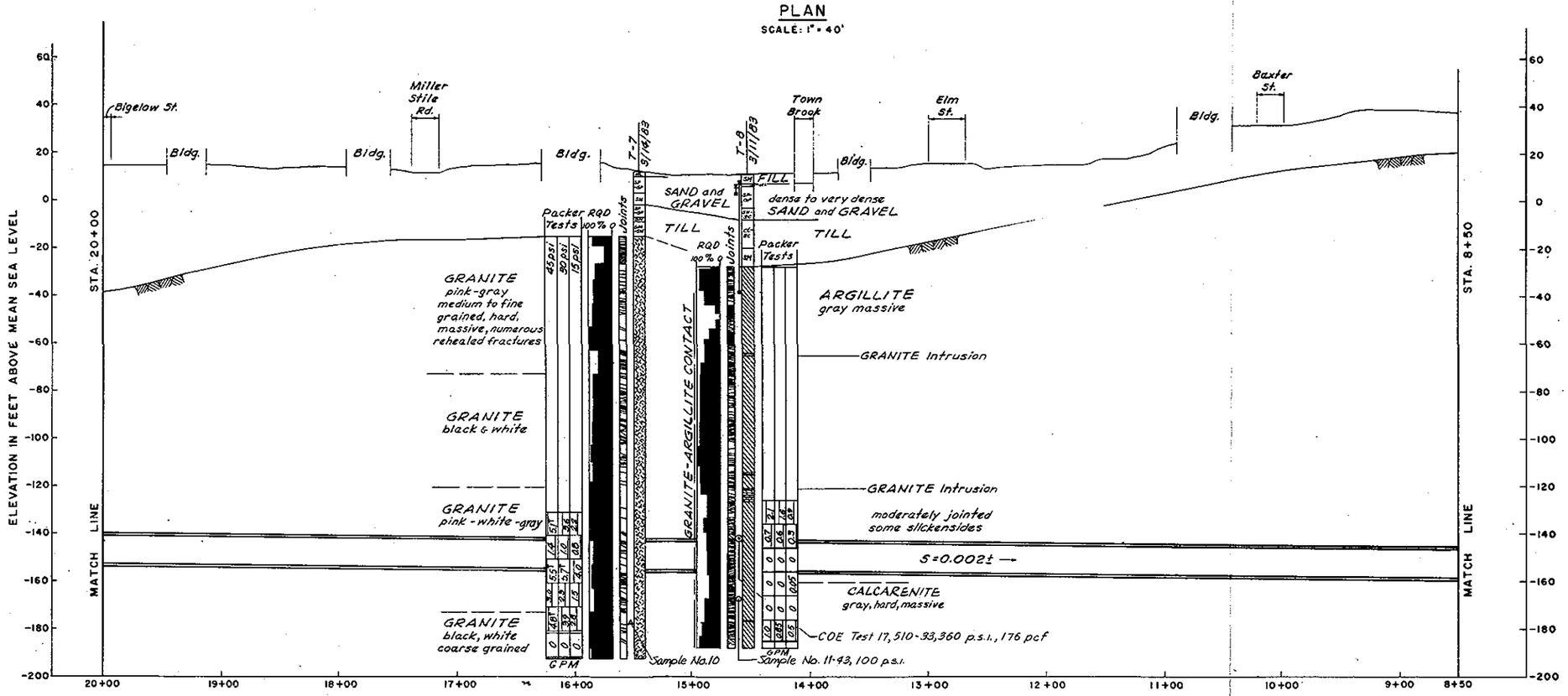
PLATE 4-3

CORPS OF ENGINEERS — U. S. ARMY



PLAN

SCALE: 1" = 40'



GEOLOGIC LOG PROFILE

SCALE: HOR. 1" = 40'
VERT. 1" = 20'

GRAPHIC SCALES
SCALE IN FEET
40 0 40 80
1" = 40'

REVISION	DATE	DESCRIPTION	BY
		Lined-unlined transition deleted.	COE
		Tunnel lining added in granite.	COE
		COE Test values added.	COE
		Rock Test values added.	COE

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY | DR. BY | CK. BY
SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
PLAN AND GEOLOGIC LOG PROFILE
STA. 8+50 TO STA. 20+00
QUINCY AND BRAINTREE — MASSACHUSETTS

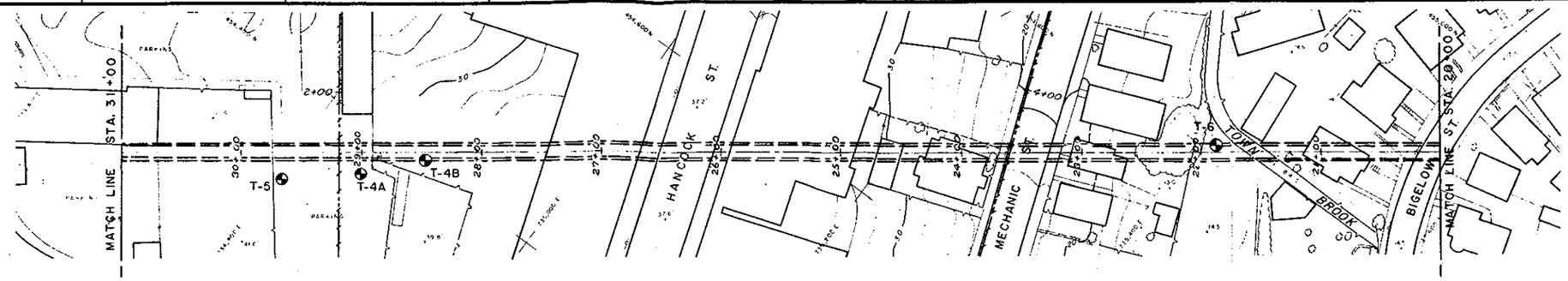
APPROVED: CHIEF, ENGINEERING DIVISION — DATE

SCALE | SPEC. NO. | DRAWING NUMBER

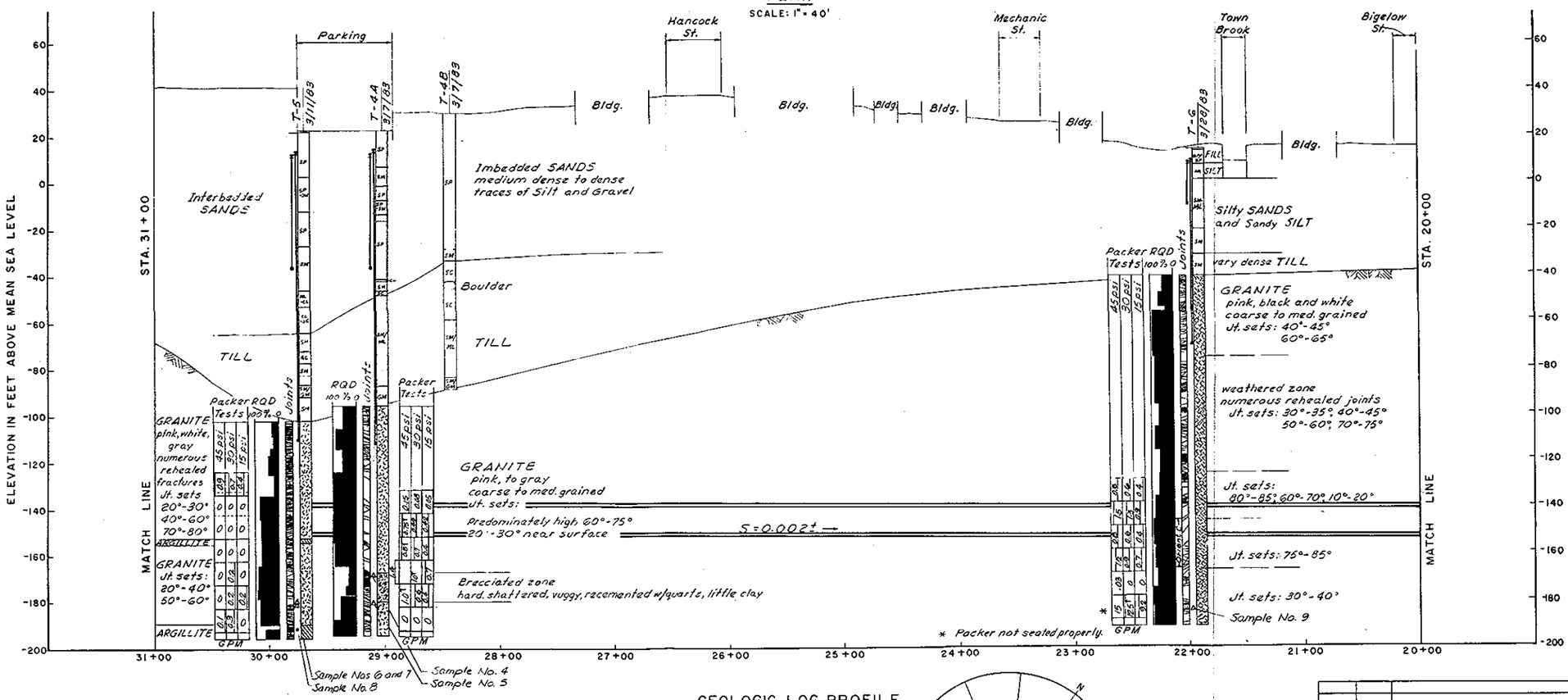
SHEET 7

PLATE 4-4

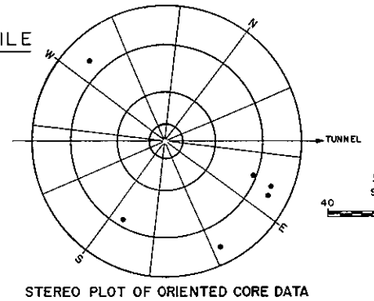
CORPS OF ENGINEERS U. S. ARMY



PLAN
SCALE: 1" = 40'



GEOLOGIC LOG PROFILE
SCALE: HOR. 1" = 40' VERT. 1" = 20'



STEREO PLOT OF ORIENTED CORE DATA
IN T-6 EL.-150 TO -166

GRAPHIC SCALES
SCALE IN FEET
40 0 40 80
1" = 40'

REVISION	DATE	DESCRIPTION	BY
		Tunnel lining added in granite.	CDE
		Oriented core data plots added.	CDE

METCALF & EDDY, INC.
BOSTON, MASS.

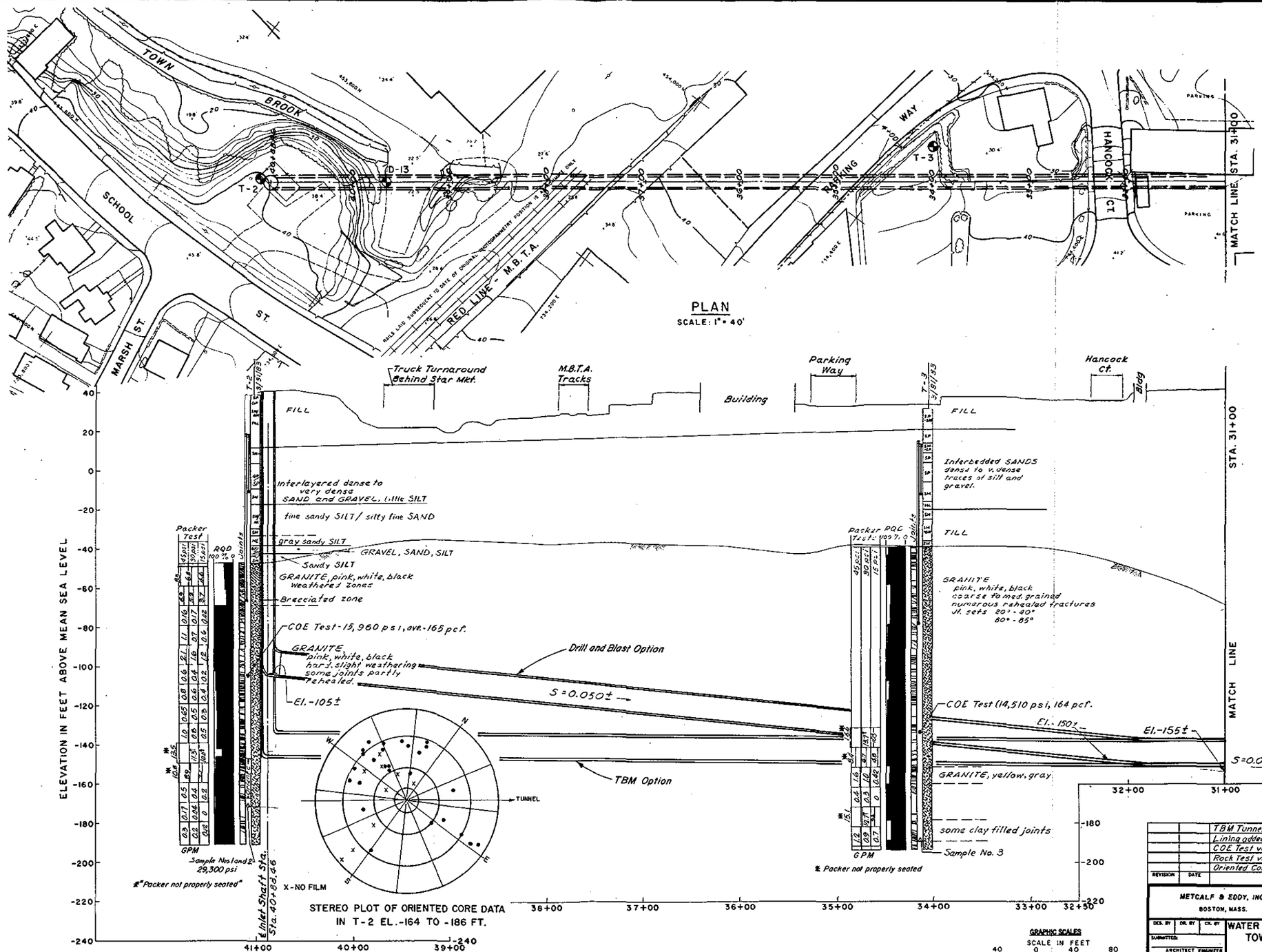
DES. BY | DR. BY | CK. BY
SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
PLAN AND GEOLOGIC LOG PROFILE
STA. 20+00 TO STA 31+00
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE AS SHOWN | SPEC NO.
DRAWING NUMBER
SHEET 8

PLATE 4-5

CORPS OF ENGINEERS U. S. ARMY



REVISION	DATE	DESCRIPTION	BY
		TBM Tunnel alignment changed.	COE
		Lining added to granite.	COE
		COE Test values added.	COE
		Rock Test values added.	COE
		Oriented Core data plots added.	COE

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY | DR. BY | CK. BY

SUBMITTED: ARCHITECT ENGINEER

APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH

REVIEWED: PROJECT MANAGER

APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

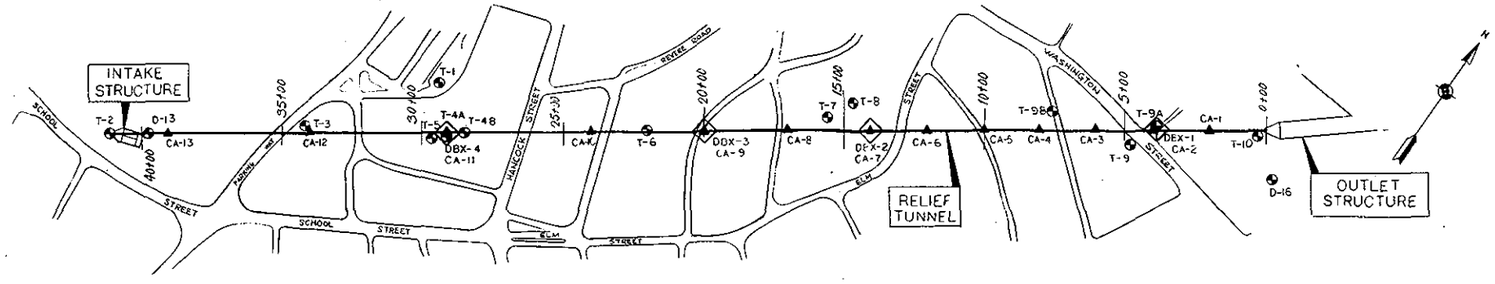
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
PLAN AND GEOLOGIC LOG PROFILE
STA. 31+00 TO STA. 40+88.46
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER

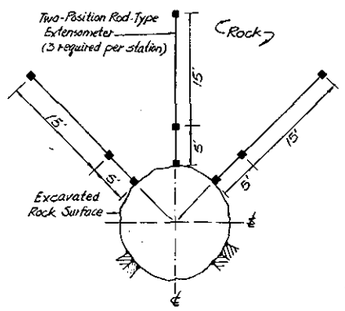
SHEET 9

PLATE 4-6

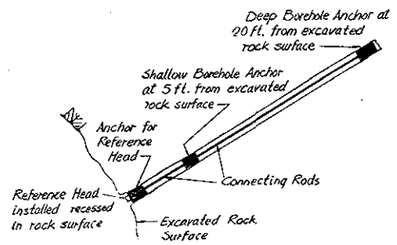


TUNNEL INSTRUMENTATION PLAN

SCALE 1" = 200'



STATION INSTALLATION

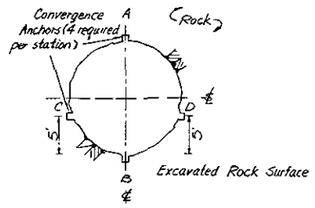


Note: Borehole length and diameter to be compatible with Extensometer Installation

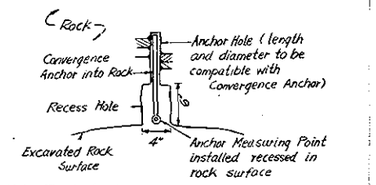
DETAIL FOR EXTENSOMETER

EXTENSOMETERS IN ROCK (160)

NOT TO SCALE



STATION INSTALLATION

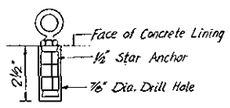


Note: The Contractor shall provide a removable metal cover to protect anchors at location B (see Station Installation)

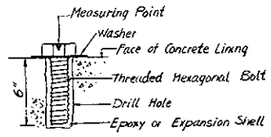
DETAIL FOR CONVERGENCE ANCHOR

CONVERGENCE ANCHORS IN ROCK (160)

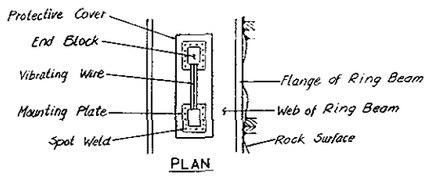
NOT TO SCALE



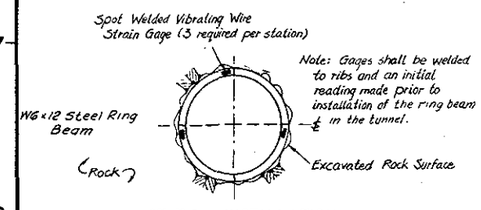
DETAIL FOR TUNNEL DIAMETER PLUG



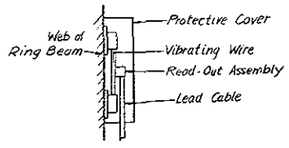
DETAIL



PLAN

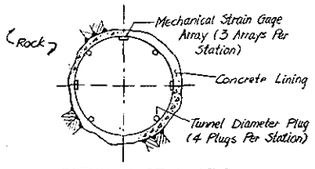


STATION INSTALLATION

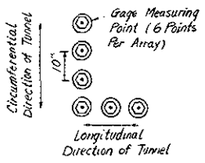


SECTION

DETAILS FOR VIBRATING WIRE STRAIN GAGES



STATION INSTALLATION



CONFIGURATION

STRAIN GAGE MEASURING POINTS

VIBRATING WIRE STRAIN GAGES FOR STEEL TUNNEL SUPPORTS (160)

NOT TO SCALE

MECHANICAL STRAIN GAGES AND TUNNEL DIAMETER PLUGS FOR CONCRETE LINING (160)

NOT TO SCALE

GRAPHIC SCALES

200' 0 200' 400'

SCALE 1" = 200'

LEGEND

- T-10 — Boring T-series, 1983
- D-16 — Boring D-series, 1975
- DBX-1 — Extensometer Station
- CA-8 — Convergence Anchor Station
- ⬡ — Payment Item Number

GENERAL NOTES

1. Stations for extensometers and convergence anchors are approximate. Final locations shall be selected during constuction by the Contracting Officer based on the following geologic conditions and distribution criteria:

Extensometers	DBX-1 in crushed argillite DBX-2 in argillite near granite contact DBX-3 in medium quality granite DBX-4 in fault zone
Convergence Anchors	at each extensometer station, plus spaced at ave. 200 ft. in argillite, plus spaced at ave. 400 ft. in granite.

2. Extensometers and convergence anchors shall be installed within 15 feet of the tunnel heading and excavation shall be stopped during installation of these instruments.

SCHEDULE OF TUNNEL INSTRUMENTATION

Type of Instrument	Instrument Station	Tunnel Station	No. of Instruments Per Station	No. of Stations	Total No. of Instruments
Extensometers	DBX-1 DBX-2 DBX-3 DBX-4	4+00 14+00 20+00 29+00	3	4	12
Convergence Anchors	CA-1 CA-2 CA-3 CA-4 CA-5 CA-6 CA-7 CA-8 CA-9 CA-10 CA-11 CA-12 CA-13	2+00 4+00 6+00 8+00 10+00 12+00 14+00 17+00 20+00 24+00 29+00 34+00 39+00	4	13	52
Vibrating Wire Strain Gages	On steel supports at average 50-foot spacing longitudinally in tunnel		3	20	60
Measuring Points for Mechanical Strain Gages	At 5 locations to be selected during construction		18	5	90
Tunnel Diameter Plugs	At 5 locations to be selected during construction		4	5	20

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY | DR. BY JSH | CK. BY

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
TUNNEL INSTRUMENTATION

QUINCY AND BRAINTREE MASSACHUSETTS

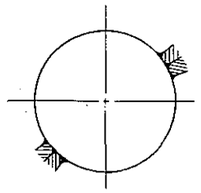
APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER

SHEET 12

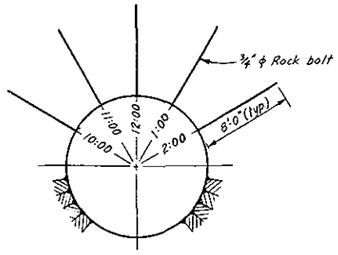
PLATE 4-7

CORPS OF ENGINEERS U. S. ARMY



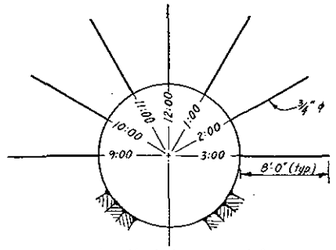
None required

In hard and intact rock
RQD = 95-100
Joint spacing 71.5 ft.



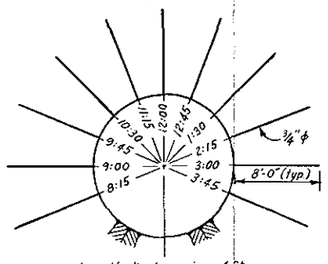
Longitudinal spacing = 7 ft

In massive moderately jointed rock
RQD = 85-95
Joint spacing 10 in. to 15 ft.



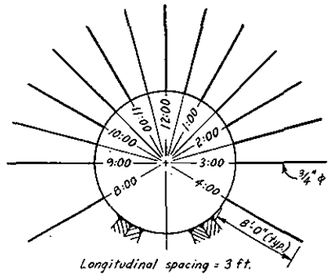
Longitudinal spacing = 6 ft.

In moderately blocky and seamy rock
RQD = 75-85
Joint spacing 6 in. to 10 in.



Longitudinal spacing = 4 ft.

In very blocky and seamy rock
RQD = 40-75
Joint spacing 4 in. to 6 in.

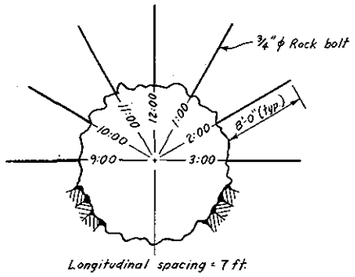


Longitudinal spacing = 3 ft.

In completely crushed rock
RQD < 40
Joint spacing < 4 in.

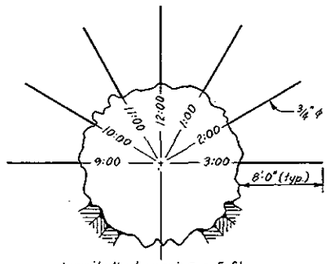
TEMPORARY ROCK SUPPORT DETAILS
EXCAVATION BY TUNNEL BORING MACHINE

N.T.S.



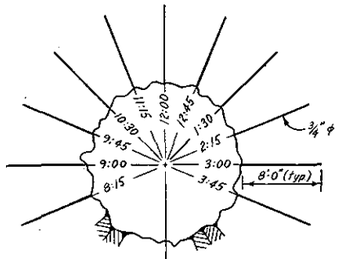
Longitudinal spacing = 7 ft.

In hard and intact rock
RQD = 95-100
Joint spacing 71.5 ft.



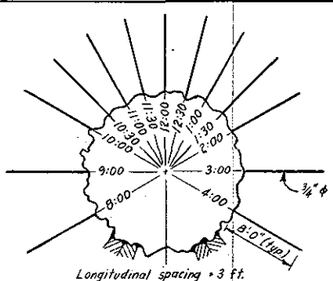
Longitudinal spacing = 5 ft.

In massive moderately jointed rock
RQD = 85-95
Joint spacing 10 in. to 15 ft.



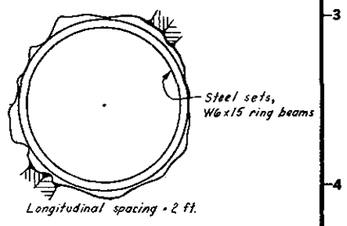
Longitudinal spacing = 4 ft.

In moderately blocky and seamy rock
RQD = 75-85
Joint spacing 6 in. to 10 in.



Longitudinal spacing = 3 ft.

In very blocky and seamy rock
RQD = 40-75
Joint spacing 4 in. to 6 in.



Longitudinal spacing = 2 ft.

In completely crushed rock
RQD < 40
Joint spacing < 4 in.

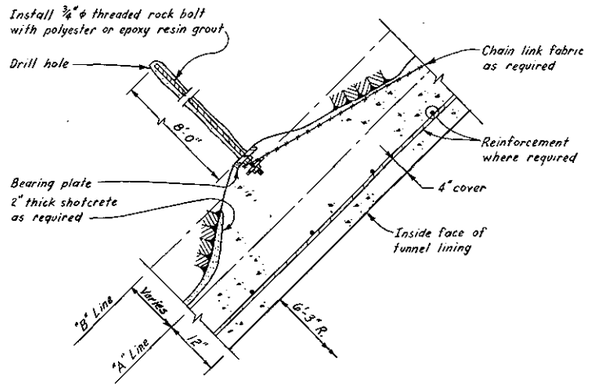
TEMPORARY ROCK SUPPORT DETAILS
EXCAVATION BY DRILL AND BLAST

N.T.S.

ROCK BOLT REQUIREMENTS					
RQD	ROCK CLASS	EXCAVATION BY TBM		EXCAVATION BY D&B	
		NUMBER	LONGITUDINAL SPACING	NUMBER	LONGITUDINAL SPACING
95-100	H & I	NONE REQUIRED	-	7	7 ft.
85-95	MMJ	5	7 ft.	7	5 ft.
75-85	MBS	7	6 ft.	11	4 ft.
40-75	VBS	11	4 ft.	13	3 ft.
<40	CC	15	3 ft.	USE STEEL SETS	2 ft.

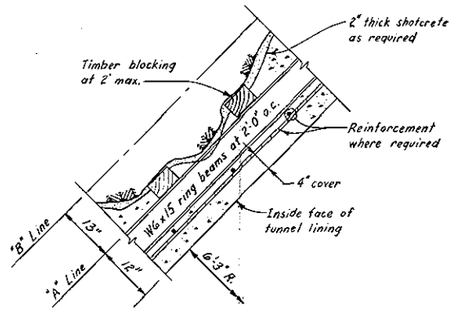
NOTE:

1. This sheet presents the temporary rock support requirements for the cast-in-place alternates.



ROCK BOLT DETAIL

SCALE: 1" = 1'-0".



STEEL SET DETAIL

SCALE: 1" = 1'-0".

GRAPHIC SCALES

1" = 1'-0" 1' 0 1' 2'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

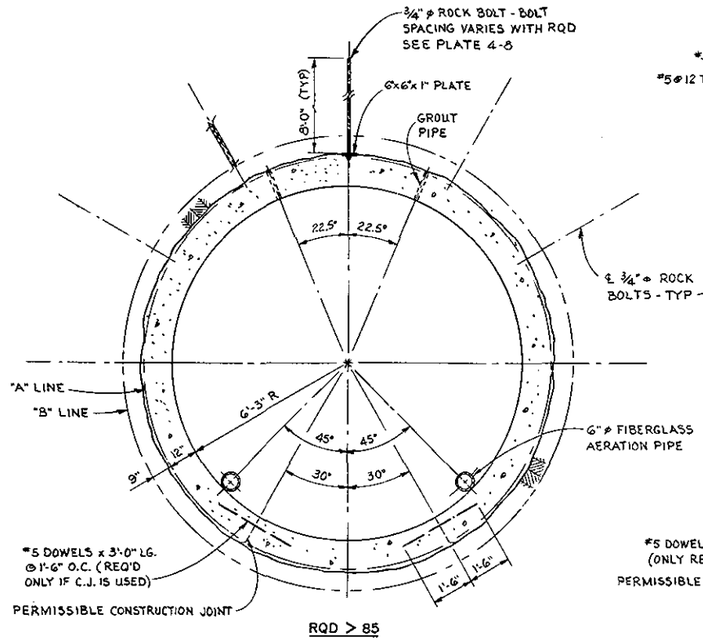
DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MANGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
QUINCY AND BRAINTREE, MASSACHUSETTS
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
TEMPORARY ROCK SUPPORT

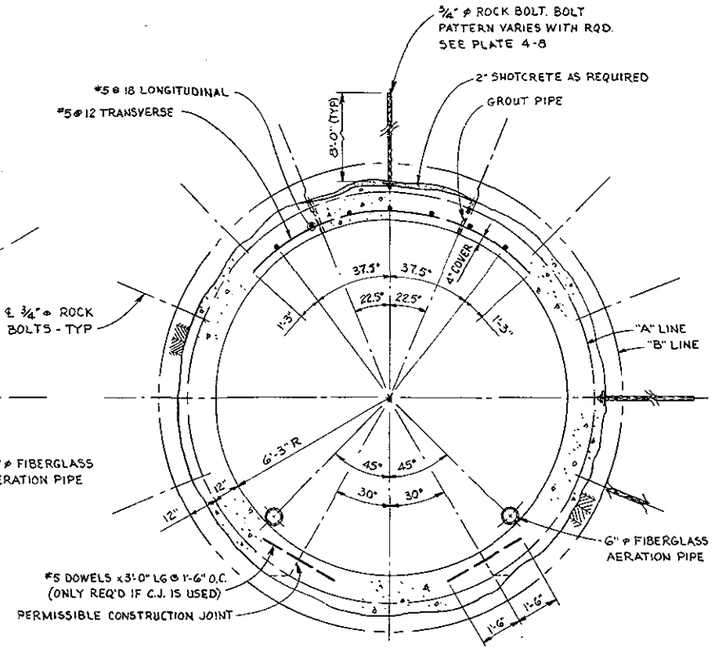
APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE AS SHOWN SPEC. NO.
DRAWING NUMBER
SHEET

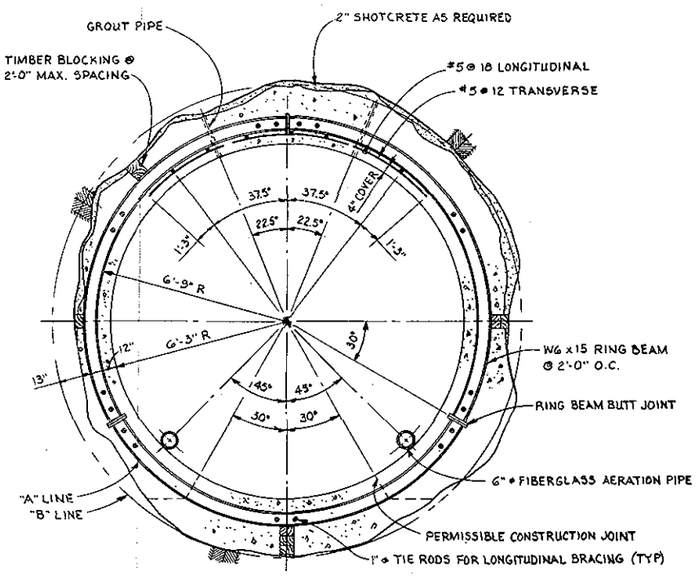
CORPS OF ENGINEERS U. S. ARMY



RQD > 85

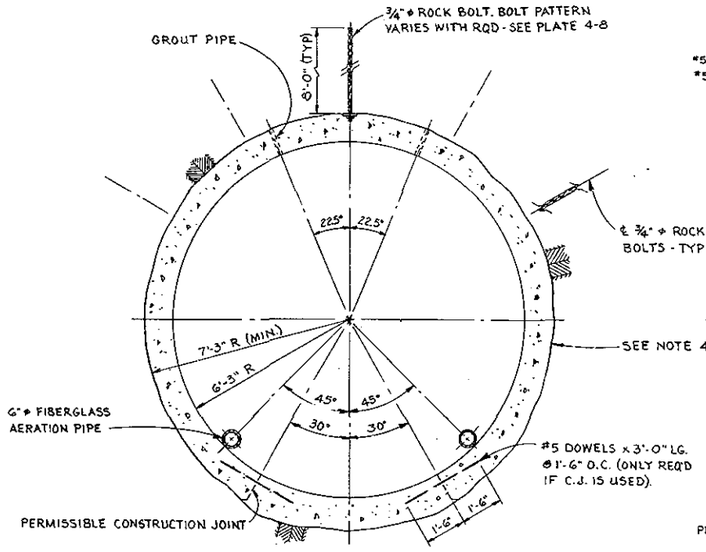


40 < RQD < 85

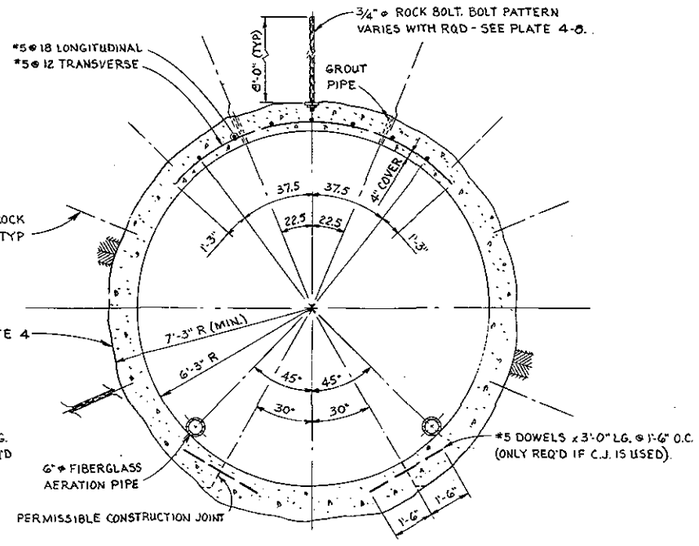


RQD < 40

TRANSVERSE TUNNEL SECTIONS
CAST-IN-PLACE LINER; DRILL AND BLAST EXCAVATION



RQD > 75



RQD < 75

TRANSVERSE TUNNEL SECTIONS
CAST-IN-PLACE LINER; EXCAVATION BY TUNNEL BORING MACHINE

NOTES:
1. THE TUNNEL LINING SHALL BE PLACED IN 30 FOOT LONG MONOLITHS.
2. LONGITUDINAL REINFORCEMENT SHALL BE CONTINUOUS THROUGH MONOLITH JOINTS.
3. ALL MONOLITH JOINTS SHALL BE WATERSTOPPED.
4. NO "A" OR "B" LINES ARE ESTABLISHED FOR EXCAVATION BY TUNNEL BORING MACHINE. EXCAVATION WILL BE PAID ON THE BASIS OF PRICE BID PER LINEAR FOOT OF TUNNEL.
5. DESIGN BASED ON A CONCRETE COMPRESSIVE STRENGTH OF 4000 POUNDS PER SQUARE INCH AT 28 DAYS AND GRADE 60 REINFORCEMENT.

GRAPHIC SCALES
1/2" = 1'-0" 1' 0 1' 3' 5'

SAFETY PAYS

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

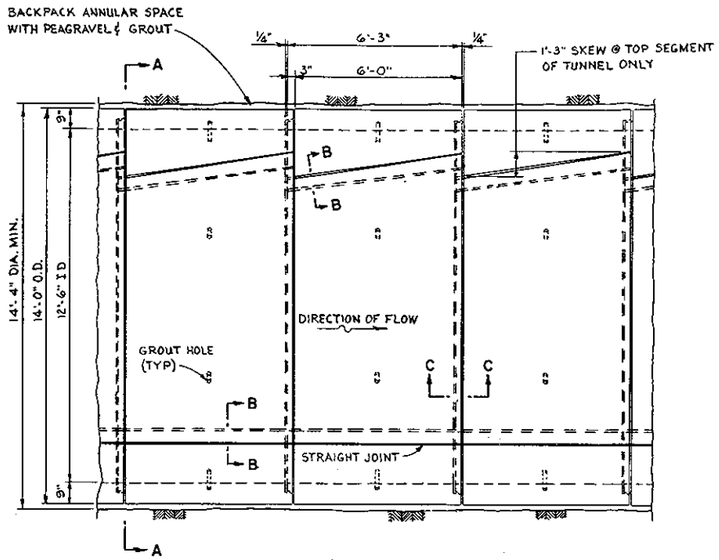
DES. BY D.D. | DR. BY R.G.P. | CK. BY

SUBMITTED:
CHIEF, GEN. ENG. SECTION
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

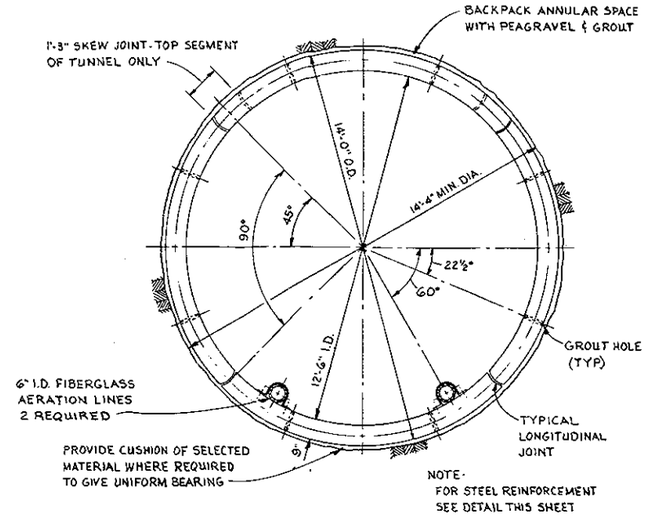
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
DETAILS-CAST IN PLACE LINER
QUINCY AND BRAINTREE MASSACHUSETTS
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE AS SHOWN SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-9

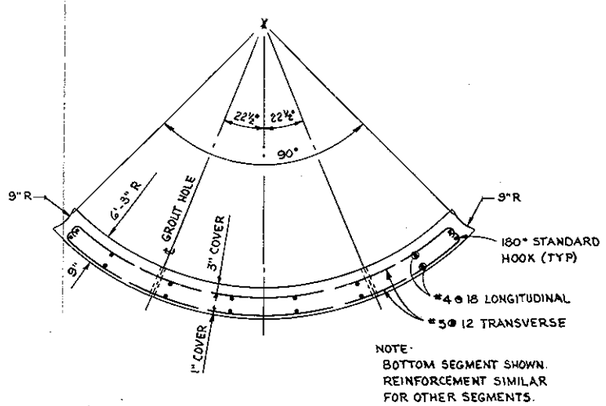
CORPS OF ENGINEERS — U. S. ARMY



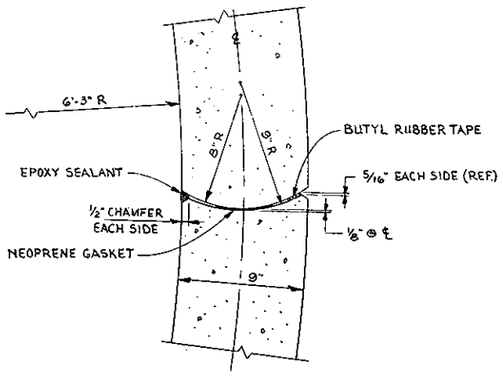
PARTIAL ELEVATION OF TUNNEL
PRECAST LINER ALTERNATE
SCALE: 1/2"=1'-0"



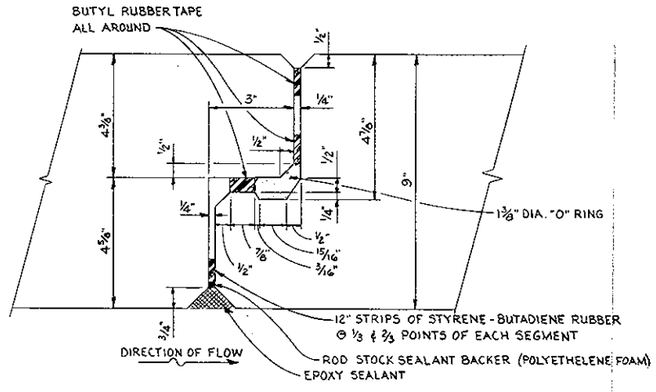
TRANSVERSE TUNNEL SECTION A-A
PRECAST LINER
SCALE: 1/2"=1'-0"



REINFORCEMENT DETAILS
PRECAST LINER
SCALE: 3/4"=1'-0"



SECTION B-B
LONGITUDINAL JOINT DETAIL
SCALE: 3"=1'-0"



SECTION C-C
TRANSVERSE JOINT DETAIL
SCALE: 6"=1'-0"

NOTE:
DESIGN OF PRECAST SEGMENTS BASED ON A CONCRETE COMPRESSIVE STRENGTH OF 5000 POUNDS PER SQUARE INCH AT 28 DAYS AND GRADE 60 REINFORCEMENT.

GRAPHIC SCALES
1/2"=1'-0" 1' 0 1' 3' 5'
3/4"=1'-0" 1' 0 1' 2' 3'
3"=1'-0" 0 6" 12"
6"=1'-0" 0 3" 6"

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY D.D. | DR. BY R.D.P. | CK. BY

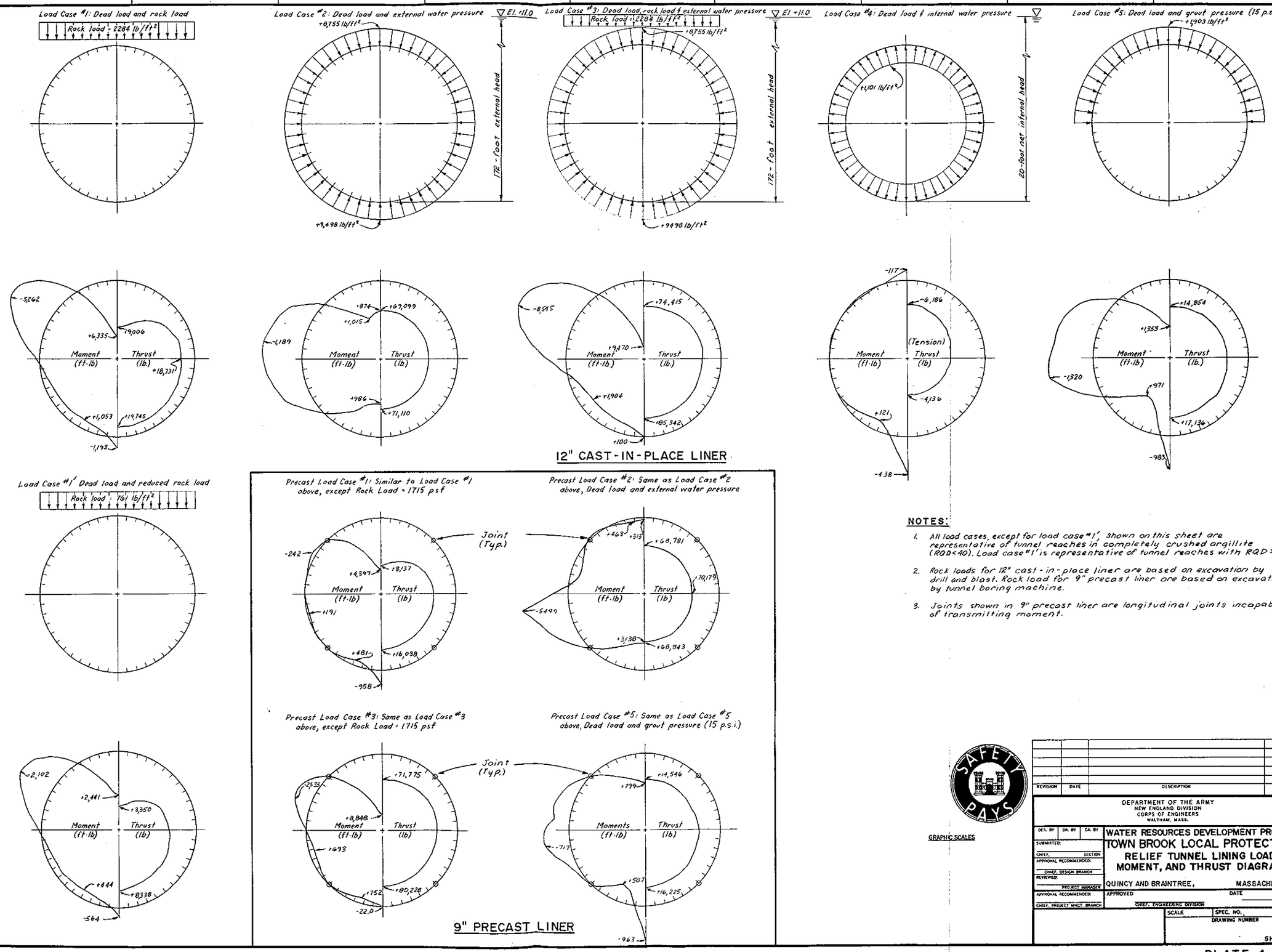
SUBMITTED: CHIEF, GEN. ENG. SECTION
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
DETAILS-PRECAST LINER
QUINCY AND BRAINTREE — MASSACUSETTS
APPROVED — DATE
CHIEF, ENGINEERING DIVISION

SCALE AS SHOWN | SPEC. NO. DACW 33
DRAWING NUMBER
SHEET

PLATE 4-10

CORPS OF ENGINEERS U. S. ARMY



NOTES:

1. All load cases, except for load case #1', shown on this sheet are representative of tunnel reaches in completely crushed argillite (RQD<40). Load case #1' is representative of tunnel reaches with RQD>85.
2. Rock loads for 12" cast-in-place liner are based on excavation by drill and blast. Rock load for 9" precast liner are based on excavation by tunnel boring machine.
3. Joints shown in 9" precast liner are longitudinal joints incapable of transmitting moment.

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY | DR. BY | CK. BY
SUBMITTED:
CHIEF, SECTION
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL LINING LOAD, MOMENT, AND THRUST DIAGRAMS

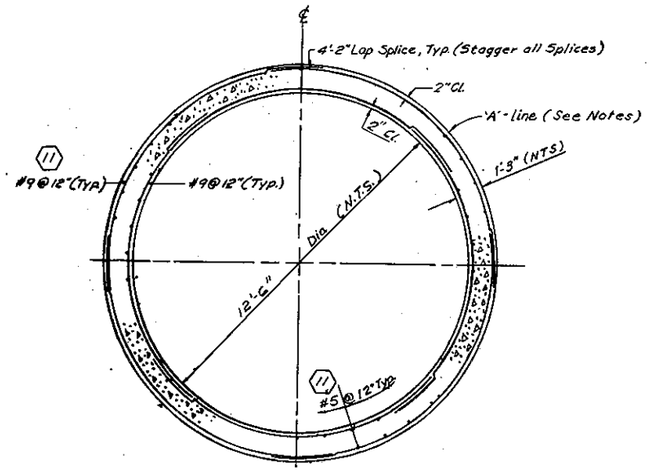
QUINCY AND BRAINTREE, MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE 4-11

CORPS OF ENGINEERS — U. S. ARMY

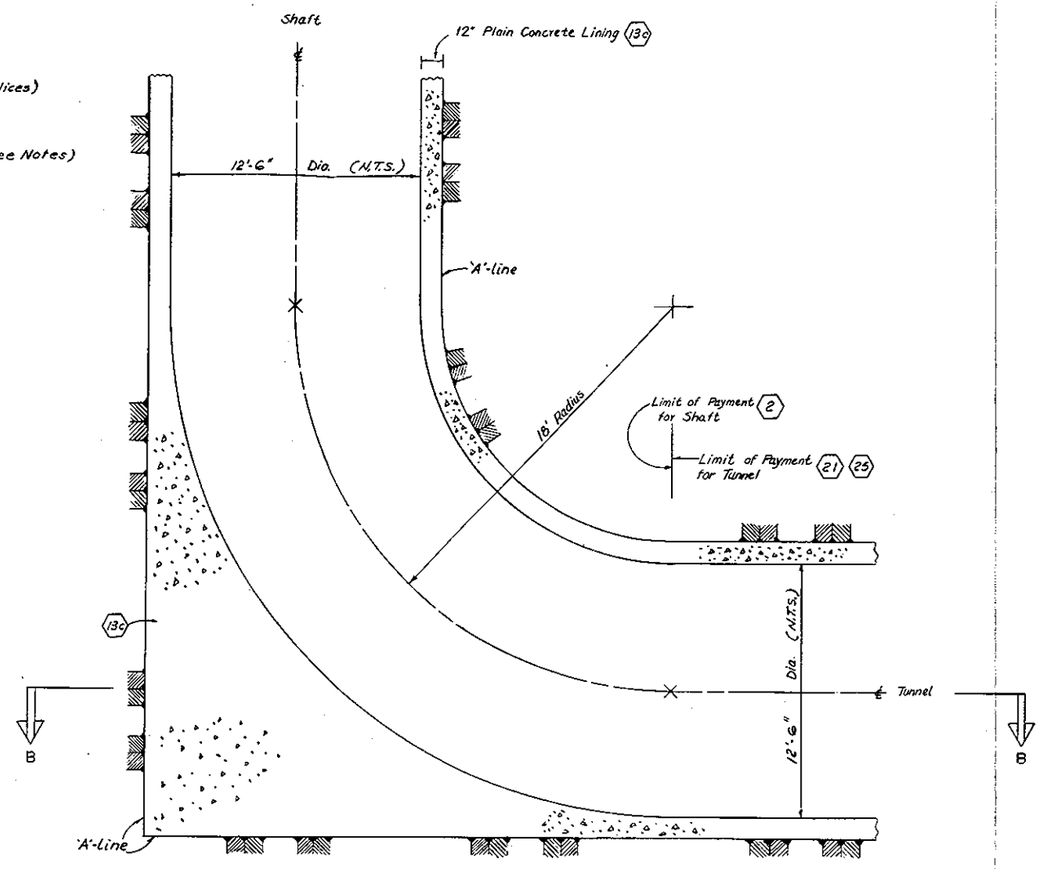


REINFORCED CONCRETE SHAFT LINING (13b) (11)

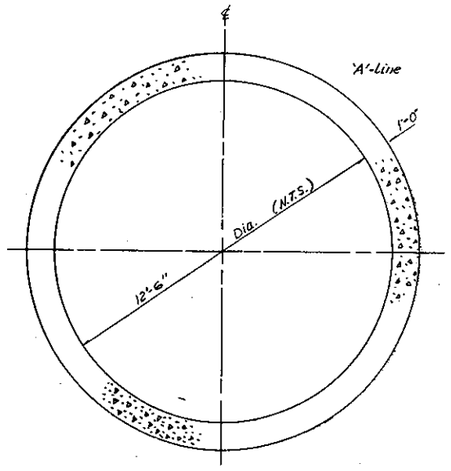
INTAKE SHAFT FROM EL.-1.0 TO EL.-70.0

OUTLET SHAFT FROM EL.-20.0 TO EL.-70.0

SCALE : 1/2" = 1'-0"



SECTIONAL ELEVATION A-A

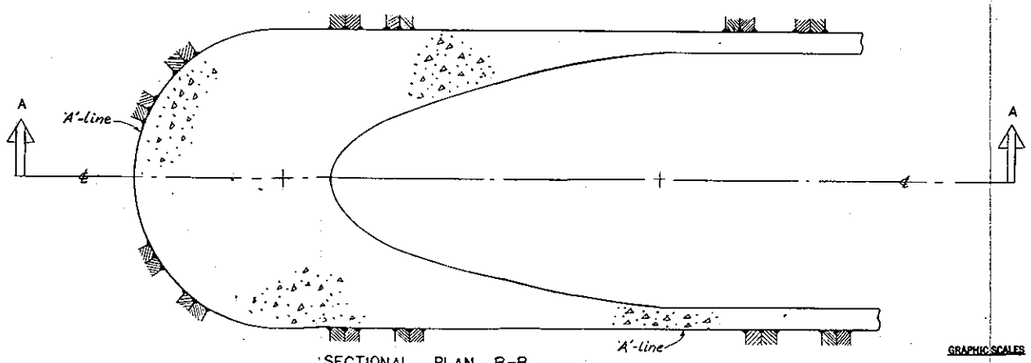


PLAIN CONCRETE SHAFT LINING (13c)

INTAKE SHAFT BELOW EL.-70.0

OUTLET SHAFT BELOW EL.-70.0

SCALE = 1/2" = 1'-0"



SECTIONAL PLAN B-B

INTAKE AND OUTLET SHAFT BOTTOM DETAILS

SCALE : 3/8" = 1'-0"

GRAPHIC SCALES

1' 0 1' 2' 3' 4' 5' 6' 7'
3/8" = 1'-0"

1' 0 1' 2' 3' 4' 5'
1/2" = 1'-0"

NOTES

1. All excavated spaces made for the Contractor's convenience (such as a tail tunnel), being outside of the permanent hydraulic structures, shall be either filled with concrete or packed tightly with excavated rock and cement grout.
2. No unexcavated or excavated material shall remain inside the 'A'-line when the concrete lining is cast.
3. No wood struts, braces, lagging or similar materials shall remain inside the 'A'-line when the concrete lining is cast.
4. The 'A'-line is a payment limit line.

REVISION	DATE	DESCRIPTION	BY
		Shaft diameter increased.	

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY — DR. BY SJH — CK. BY

SUBMITTED — ARCHITECT ENGINEER — APPROVAL RECOMMENDED — CHIEF, DESIGN BRANCH — REVIEWED — PROJECT MANAGER — APPROVAL RECOMMENDED — CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
SHAFT DETAILS

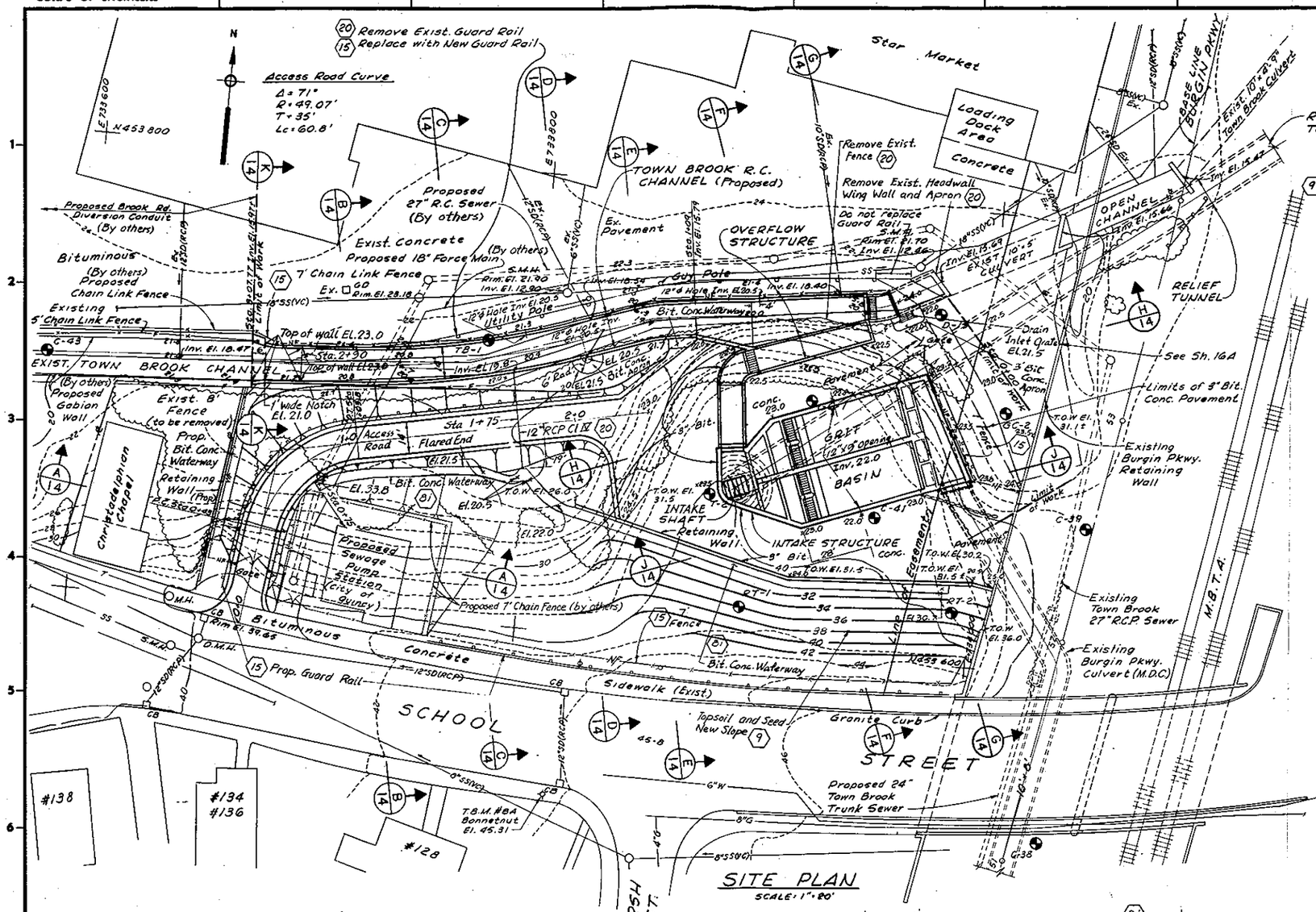
QUINCY AND BRAINTREE, MASSACHUSETTS

APPROVED — DATE — CHIEF, ENGINEERING DIVISION

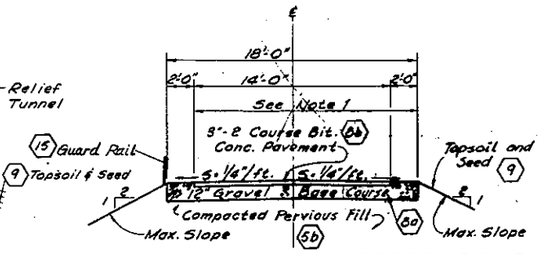
SCALE — SPEC. NO. — DRAWING NUMBER

SHEET 10

PLATE 4-12



SITE PLAN
SCALE: 1" = 20'



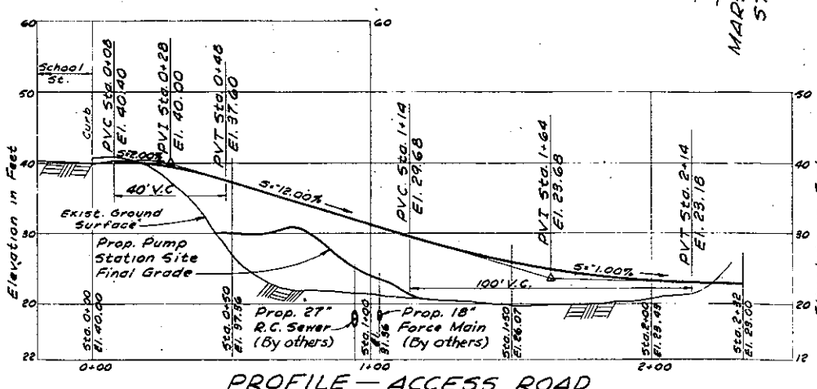
TYPICAL SECTION – ACCESS ROAD
Scale: 1" = 4'

Notes:

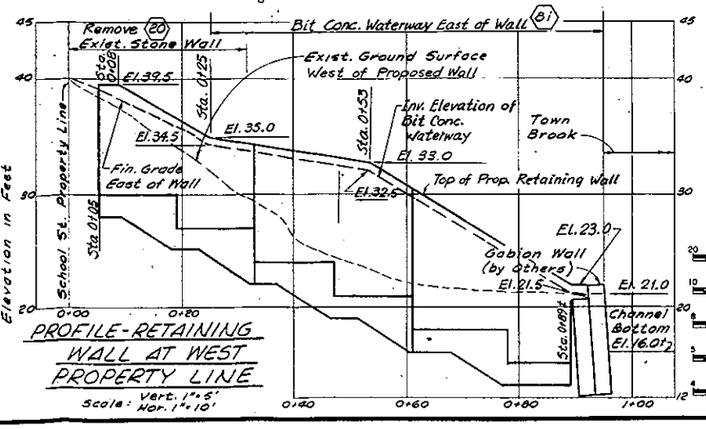
1. For Access Roadway, typical section to be used Sta. 0+00 to Sta. 0+40 and Sta. 1+20 to Sta. 2+00. Between Sta. 0+55 and Sta. 1+05 slope 14' roadway to inside of curve at 1/4"/ft. Provide transitions between Sta. 0+40 to Sta. 0+55 and Sta. 1+05 to Sta. 1+20.
2. Provide 3"-2 Course Bit. Concrete Pavement on 12" Gravel Base Course in Intake Area to limits indicated.

LEGEND

DMH	Drain Manhole
SMH	Sewer Manhole
VC	Vitrified Clay Pipe
SS	Sanitary Sewer
SD	Storm Drain
RCP	Reinforced Concrete Pipe
CB	Catch Basin
45.8	Existing Spot Elevation
x 23.0	Proposed Spot Elevation
Inv.	Invert Elevation
—36—	Proposed Contour Lines
	Existing Contour Lines
(15)	Guard Rail (New)
	Existing Fence
(15)	Proposed New Chain Link Fence
(5a)	Screened Gravel
(13a)	Concrete
(10)	Filter Fabric
	Natural Ground and Earth Cut
	Original Grade or Earth
	Rock Cut
	Boring (See Specifications for Logs)
	Payment Item



PROFILE – ACCESS ROAD
Scale: Vert. 1" = 5' Hor. 1" = 20'



PROFILE – RETAINING WALL AT WEST PROPERTY LINE
Scale: Vert. 1" = 5' Hor. 1" = 10'

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY
		Intake structure revised	COE
		Top of channel wall raised	COE

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY R.H.L. DR. BY ED ZO CK. BY R.H.L.

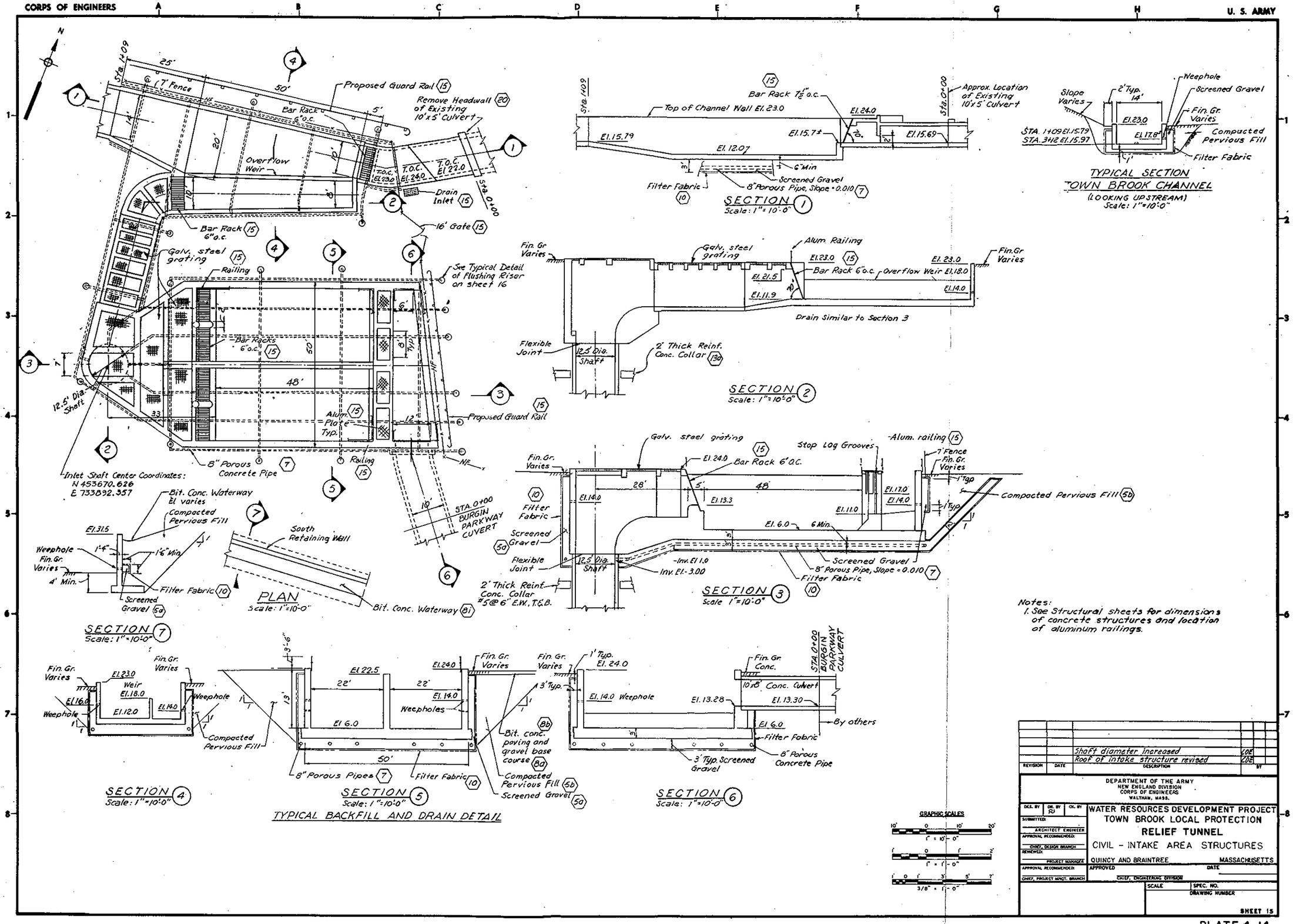
SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MGMT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
CIVIL
INTAKE AREA SITE PLAN AND DETAILS
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING BRANCH

SCALE SPEC. NO. DRAWING NUMBER
SHEET 13

CORPS OF ENGINEERS — U. S. ARMY



Notes:
1. See Structural sheets for dimensions of concrete structures and location of aluminum railings.

REVISION	DATE	DESCRIPTION	BY
		Shaft diameter increased	CDE
		Roof of intake structure revised	CDE

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

DES. BY — DR. BY PJ — CK. BY

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

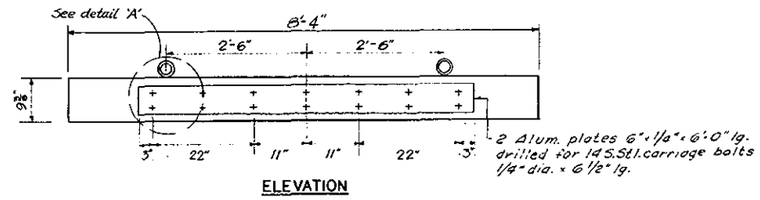
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
CIVIL - INTAKE AREA STRUCTURES
QUINCY AND BRAINTREE — MASSACHUSETTS

APPROVED — DATE
CHIEF, ENGINEERING DIVISION

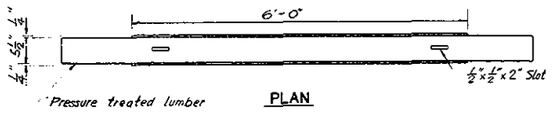
SCALE — SPEC. NO. — DRAWING NUMBER

SHEET 15

PLATE 4-14

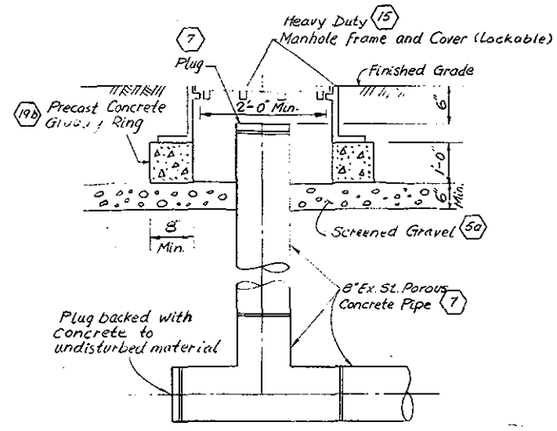


ELEVATION



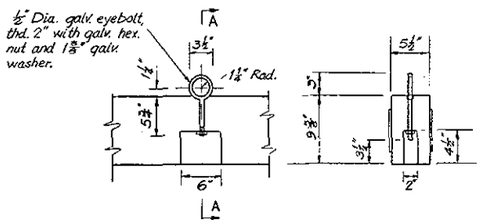
PLAN

SCALE 1"=1'-0"



DETAIL OF FLUSHING RISER

NOT TO SCALE

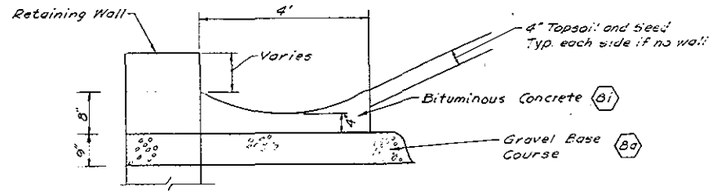


DETAIL A SECTION A-A

SCALE 1 1/2"=1'-0"

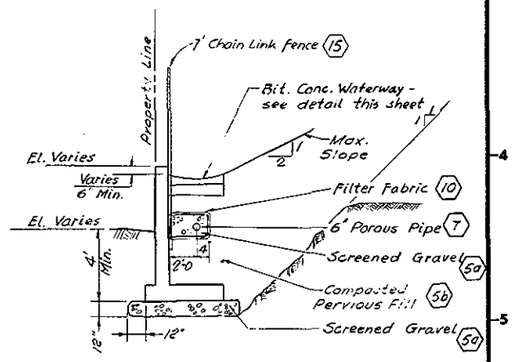
STOP LOG DETAILS (15)

(32 STOP LOGS REQUIRED)



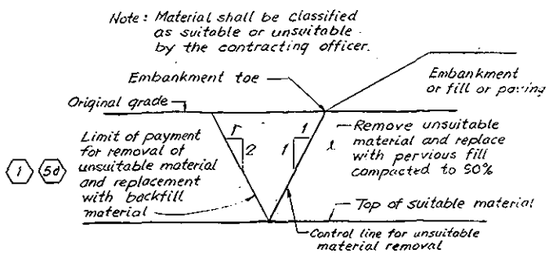
BITUMINOUS CONCRETE WATERWAY

NOT TO SCALE



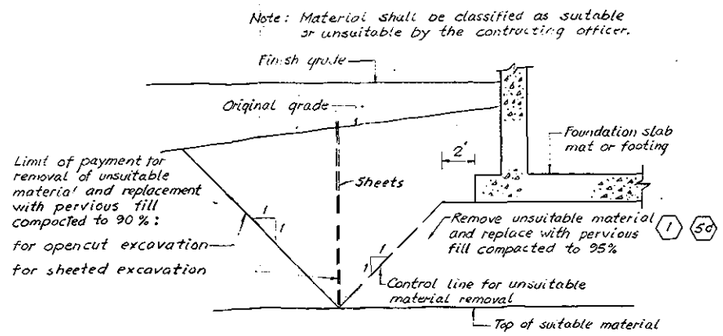
TYPICAL SECTION
RETAINING WALL AT WEST
PROPERTY LINE
(LOOKING NORTH)

SCALE : 1"=4'-0"



REMOVAL OF UNSUITABLE MATERIAL FROM
BENEATH EMBANKMENTS OR FILL
OR PAVING

NOT TO SCALE



REMOVAL OF UNSUITABLE MATERIAL FROM
BENEATH STRUCTURES

NOT TO SCALE

GRAPHIC SCALES

1' 0 1' 2'
1"=1'-0"
1' 0 1'
1 1/2"=1'-0"
4' 0 4' 8'
1"=4'-0"

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY | DR. BY SJH | CK. BY

SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MGMT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
CIVIL-MISCELLANEOUS DETAILS I

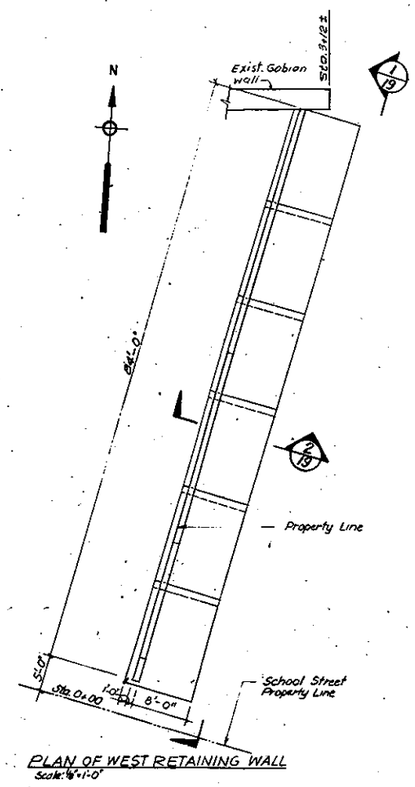
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

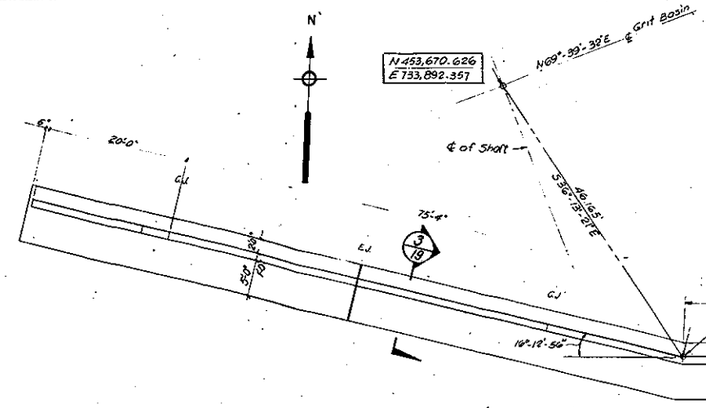
SCALE | SPEC. NO.
DRAWING NUMBER

SHEET 16

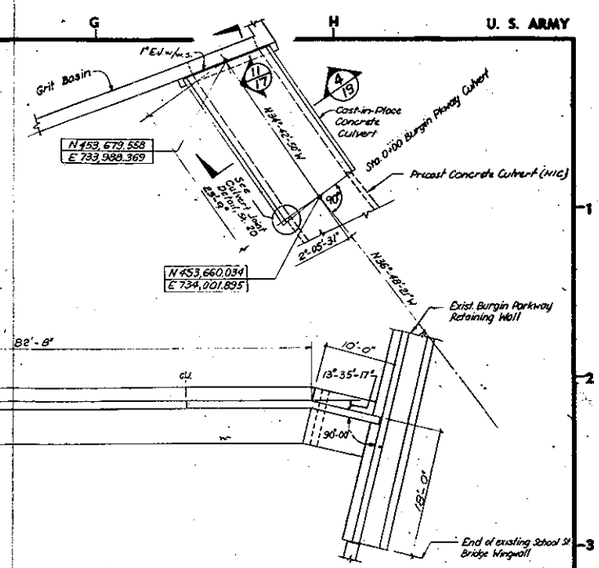
CORPS OF ENGINEERS U. S. ARMY

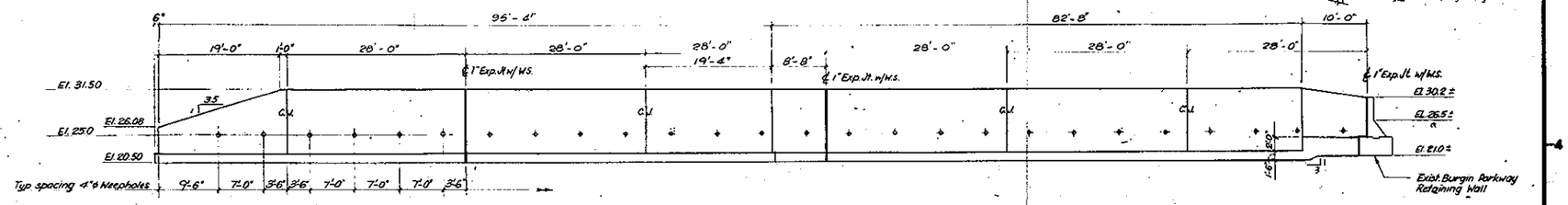


PLAN OF WEST RETAINING WALL
Scale: 1/8" = 1'-0"

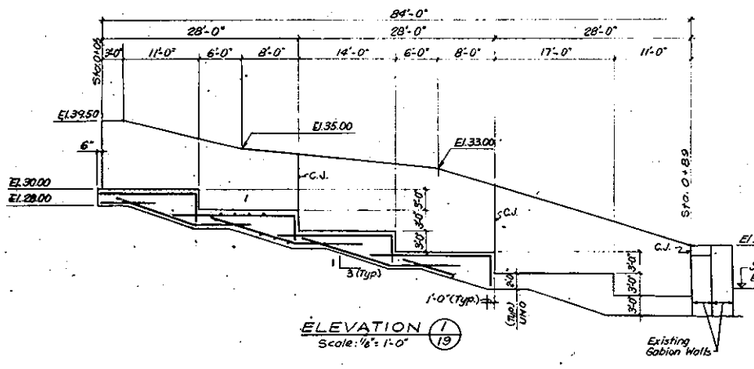


PLAN OF SOUTH RETAINING WALL
Scale: 1/8" = 1'-0"

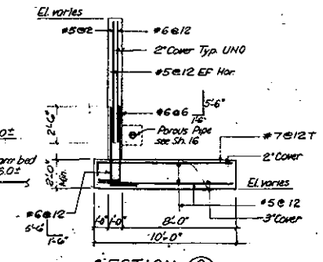




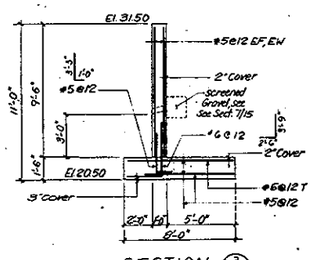
DEVELOPED ELEVATION OF SOUTH RETAINING WALL
Scale: 1/8" = 1'-0"



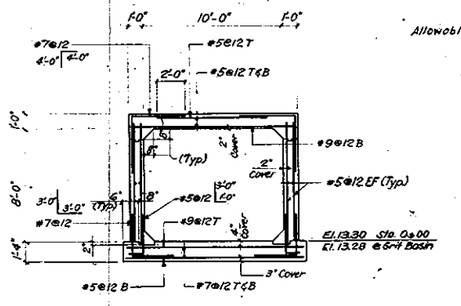
ELEVATION (1/19)
Scale: 1/8" = 1'-0"



SECTION (2/19)
Scale: 1/2" = 1'-0"



SECTION (3/19)
Scale: 1/2" = 1'-0"



SECTION (4/19)
Scale: 1/4" = 1'-0"

Allowable Soil Bearing Pressure 1.50 TSF

GRAPHIC SCALES

SCALE IN FEET
SCALE: 1/8" = 1'-0"

SCALE IN FEET
SCALE: 1/4" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY J.T.F. | DR. BY J.F. | CK. BY J.A.D.

SUBMITTED:
ARCHITECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF, DESIGN BRANCH
REVIEWED:
PROJECT MANAGER
APPROVAL RECOMMENDED:
CHIEF, PROJECT MNGT. BRANCH

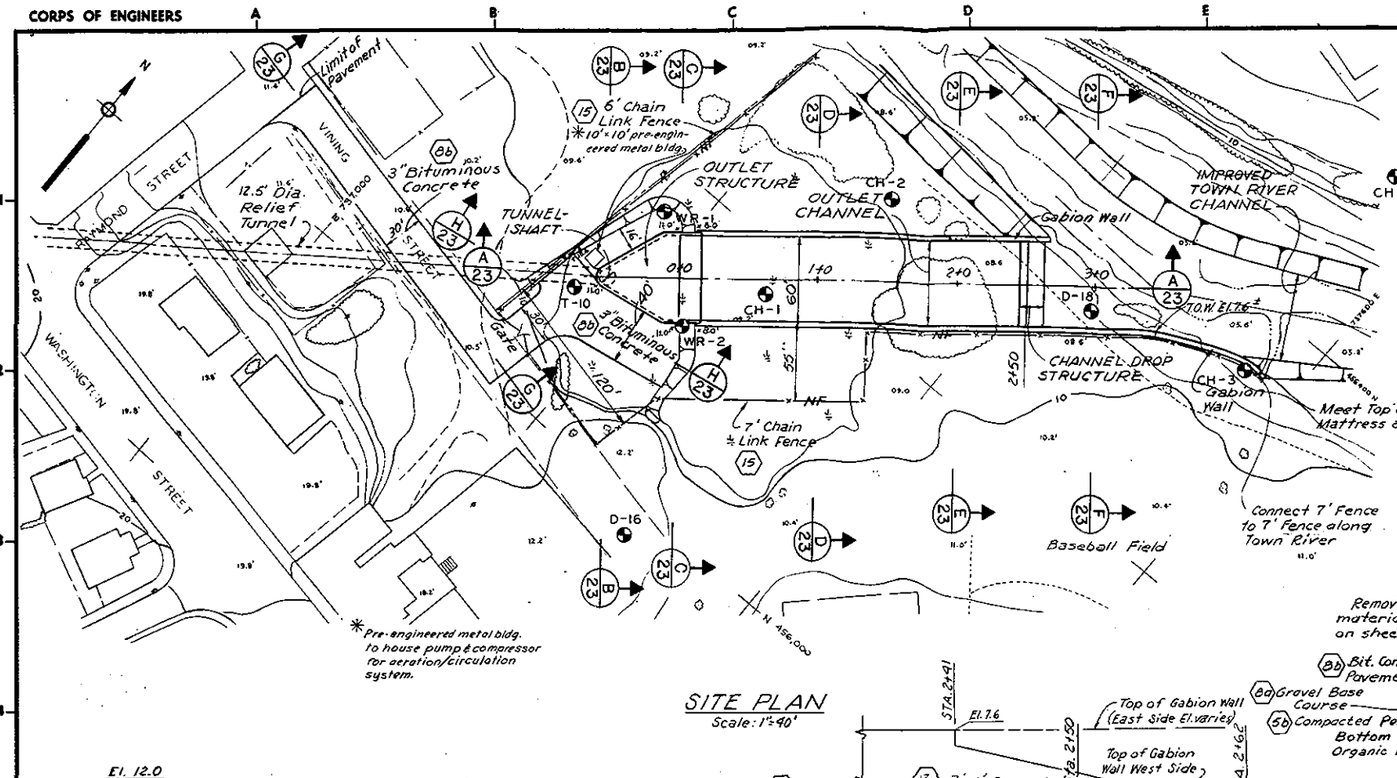
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
STRUCTURAL
RETAINING WALLS
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED: DATE
CHIEF, ENGINEERING DIVISION

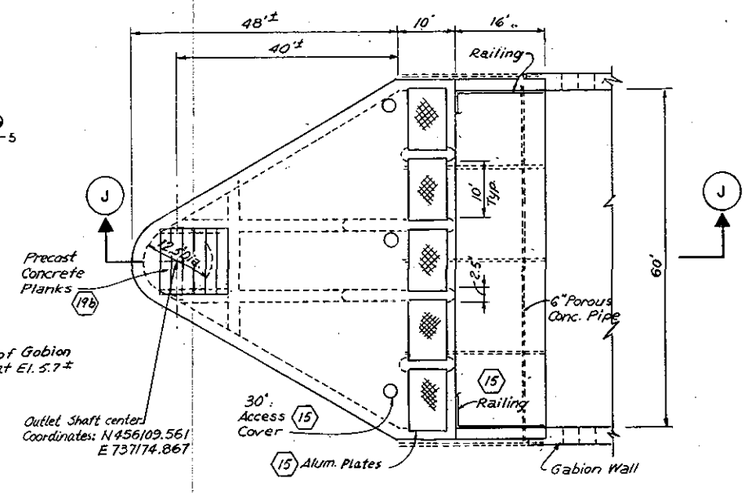
SCALE: AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET 19

PLATE 4-16

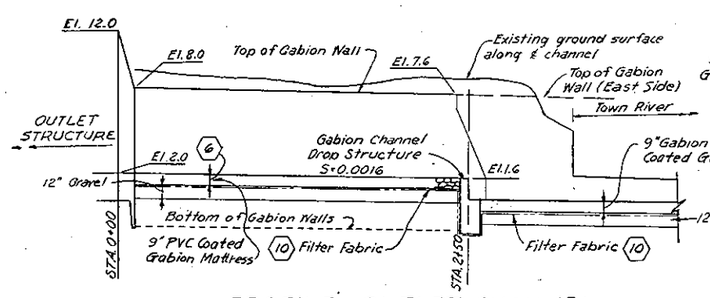
CORPS OF ENGINEERS — U. S. ARMY



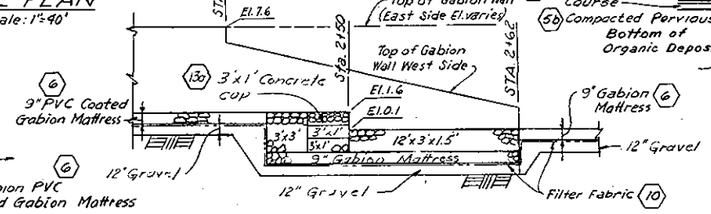
SITE PLAN
Scale: 1"=40'



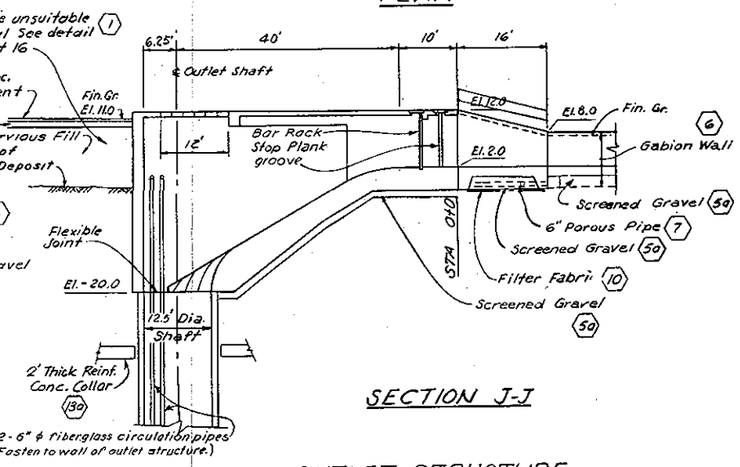
PLAN



PROFILE - OUTLET CHANNEL
Scale: Hor. 1" = 40' Ver. 1" = 4'

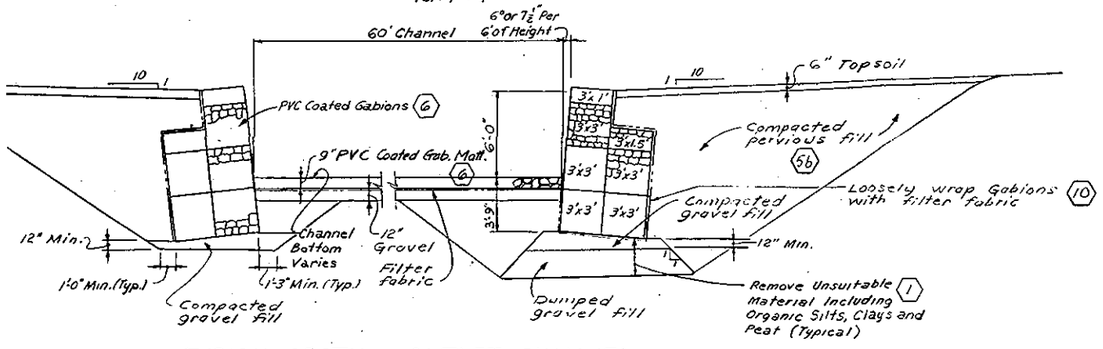


DROP STRUCTURE AT STA. 2+50
Scale: 1/4"=1'-0"



SECTION J-J

OUTLET STRUCTURE
Scale: 1"=10'



TYPICAL SECTION - OUTLET CHANNEL
Scale: 1/4"=1'-0"

Notes.
1. See Structural Sheets for dimensions of concrete structures and location of aluminum railings.
2. For boring logs see specifications.

REVISION	DATE	DESCRIPTION	BY
		Concrete caps on gabions deleted.	COE
		Pre-engineered bldg. & circulation pipes added	COE
		Shaft diameter increased	COE

GRAPHIC SCALES: 1" = 40'-0"; 1" = 4'-0"; 1/4" = 1'-0"; 1" = 10'-0"

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY R.H.L. | DR. BY R.L.J. | CK. BY R.H.L.

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
CIVIL -
OUTLET STRUCTURES AND SITE PLAN
QUINCY AND BRAINTREE MASSACHUSETTS

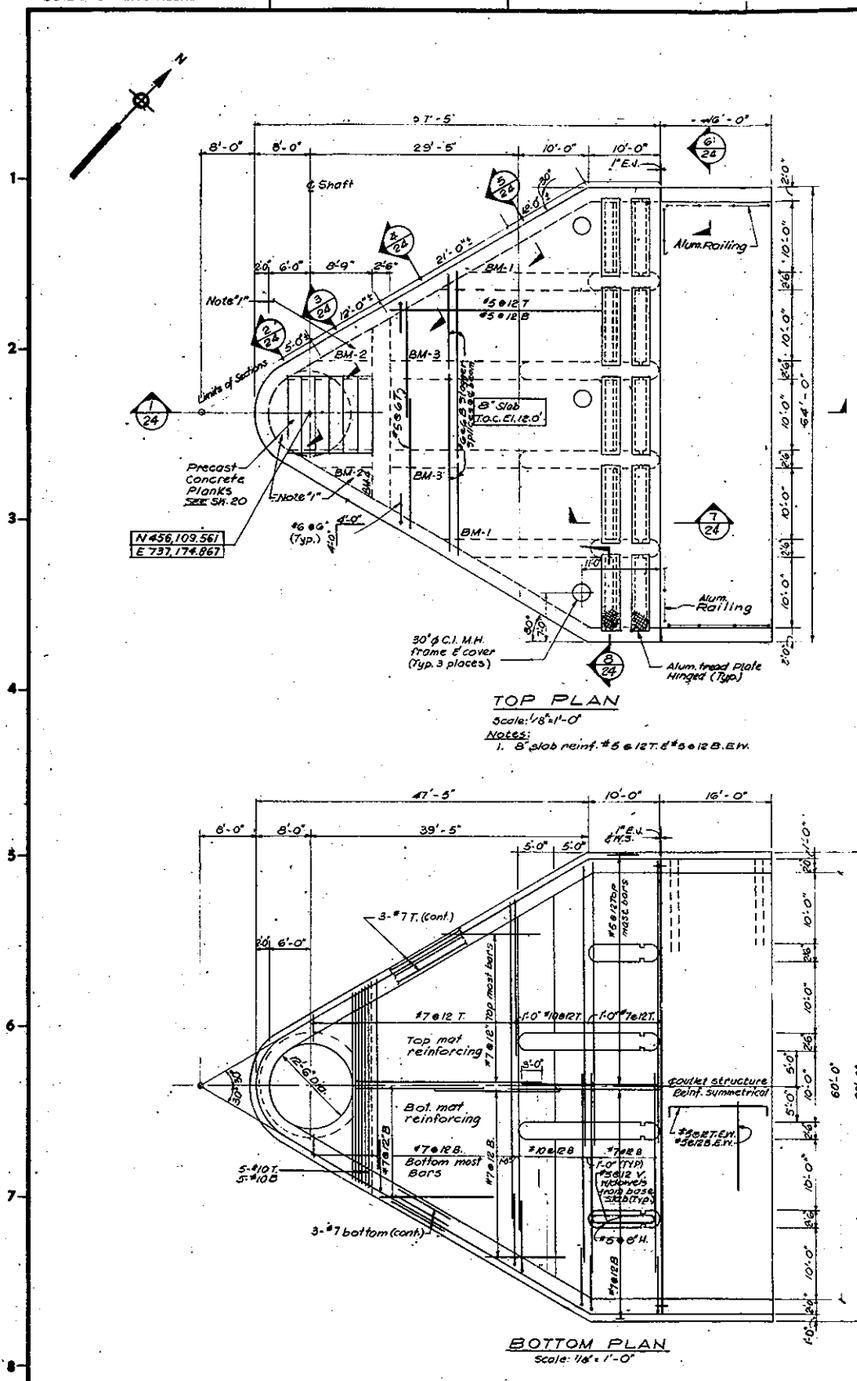
APPROVED: CHIEF, ENGINEERING DIVISION — DATE

SCALE: AS SHOWN | SPEC. NO. | DRAWING NUMBER

SHEET 22

PLATE 4-17

CORPS OF ENGINEERS — U. S. ARMY

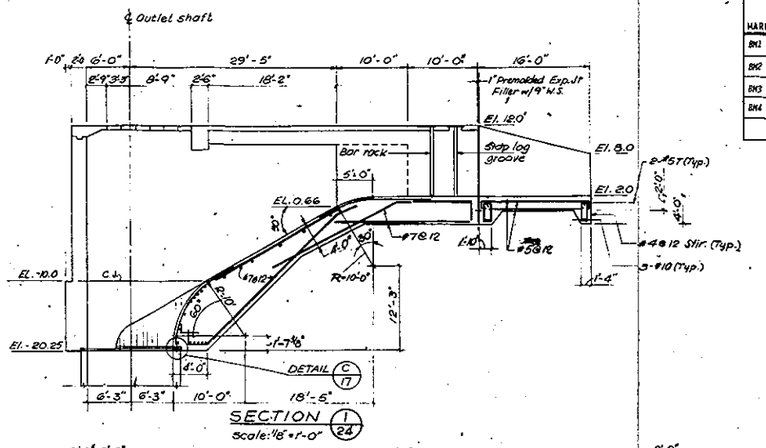


TOP PLAN
Scale: 1/8" = 1'-0"

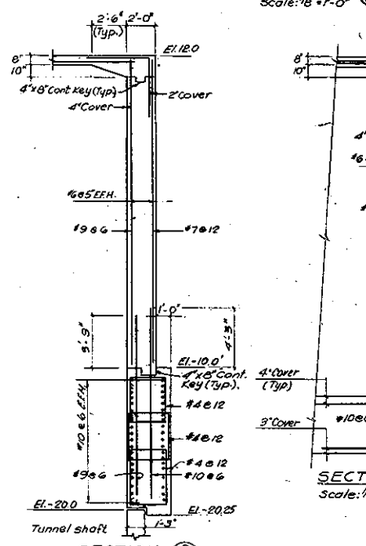
Notes:
1. 8" slab reinf. #5 @ 12 T. & #5 @ 12 B. E.W.

BOTTOM PLAN
Scale: 1/8" = 1'-0"

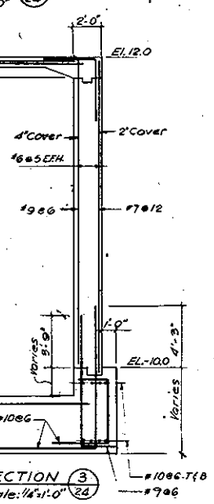
Notes:
1. Allowable soil bearing 4.5 tons/sq.ft.



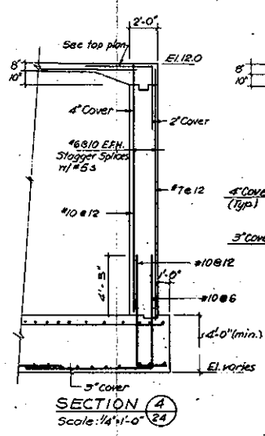
SECTION 1/24
Scale: 1/8" = 1'-0"



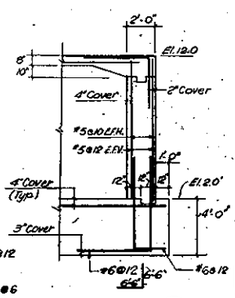
SECTION 2/24
Scale: 1/4" = 1'-0"



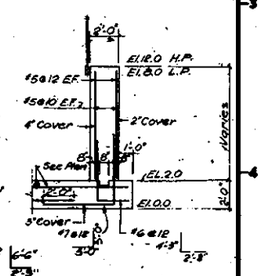
SECTION 3/24
Scale: 1/4" = 1'-0"



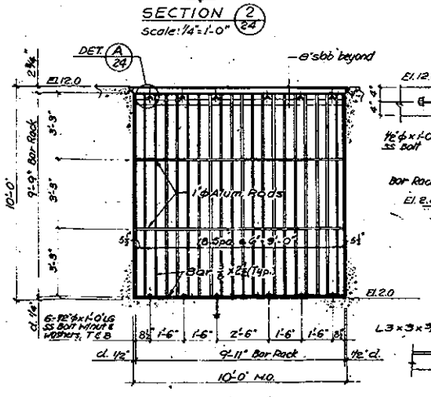
SECTION 4/24
Scale: 1/4" = 1'-0"



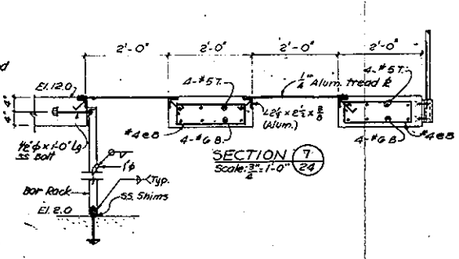
SECTION 5/24
Scale: 1/4" = 1'-0"



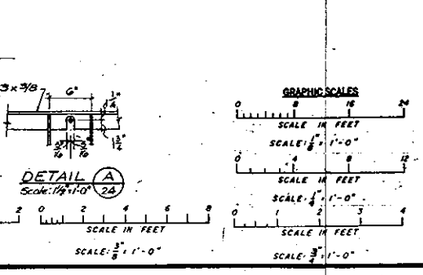
SECTION 6/24
Scale: 1/4" = 1'-0"



BAR RACK ELEVATION 8/24
Scale: 3/8" = 1'-0"



SECTION 7/24
Scale: 3/4" = 1'-0"



DETAIL A/24
Scale: 1/4" = 1'-0"

GRAPHIC SCALES

BEAM SCHEDULE (BEAM MARK OR PLAN WILL BE DESIGNATED ON LEFT END OF BEAM)

MARK	SIZE: WIDTH x DEPTH	REINFORCEMENT BOTTOM CONT	BOTTOM ADDL.	TOP ADDL. LEFT	TOP ADDL. SUPP. BARS CONT	TOP ADDL. SUPP. BARS RIGHT	STIRRUPS TYPE	STIRRUPS SIZE	STIRRUPS SPACING, EA. END	REMARKS
BM1	30x30	5-#8			3-#8			#4	1 @ 3", 4 @ 6", BAL. @ 12"	
BM2	30x30	5-#7			3-#8	2-#8		#4	1 @ 3", 4 @ 6", BAL. @ 12"	
BM3	30x30	5-#8			3-#8			#4	1 @ 3", 4 @ 6", BAL. @ 12"	
BM4	30x40	9-#10			5-#8			#4	1 @ 3", 12 @ 6", BAL. @ 12"	

REVISION	DATE	DESCRIPTION	BY
		Shaft diameter increased	

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY J.F.H. DR. BY J.F. CK. BY J.A.D.

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
RELIEF TUNNEL
STRUCTURAL
OUTLET STRUCTURE
QUINCY AND BRAINTREE, MASSACHUSETTS

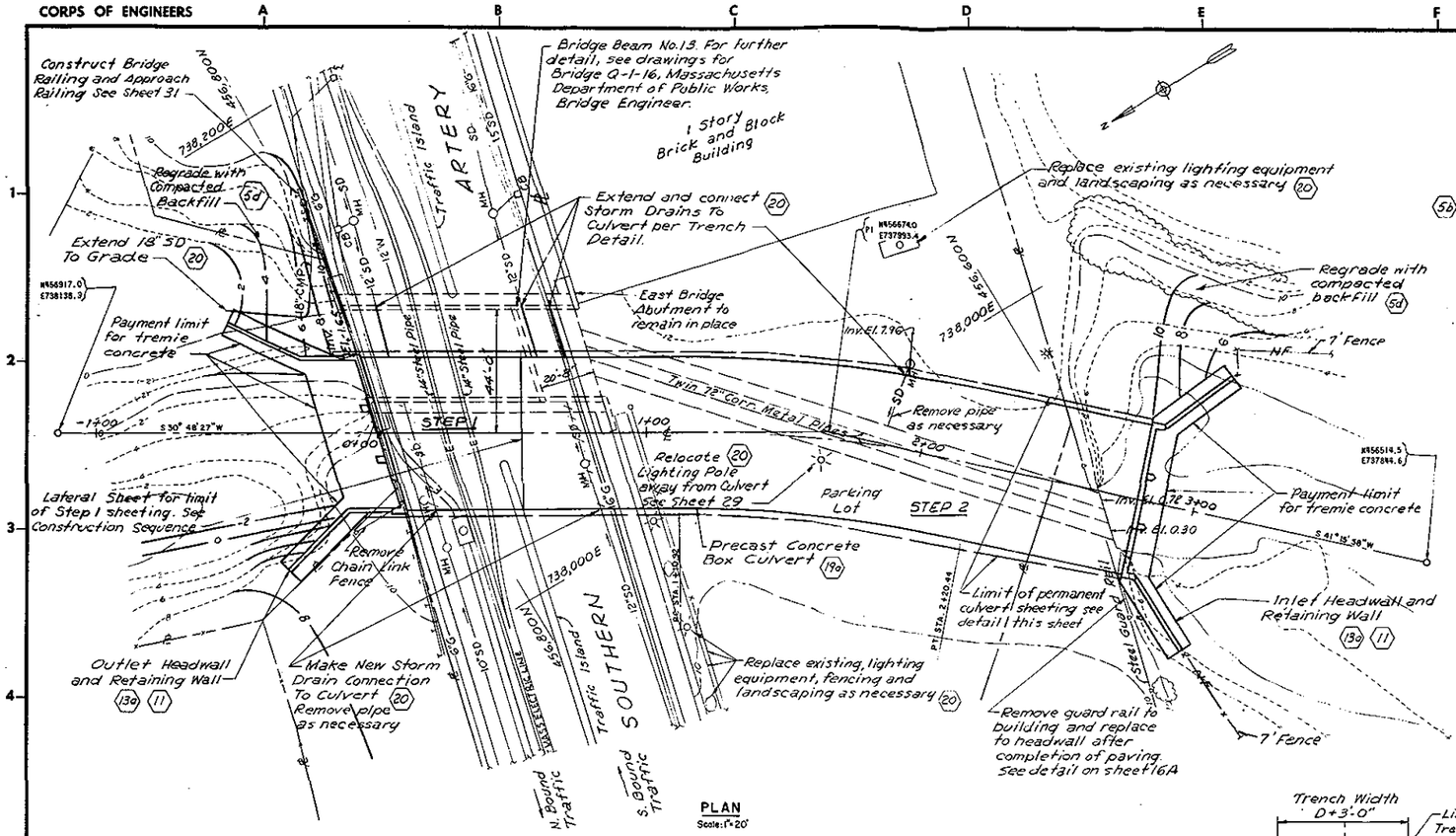
SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION
DATE

SCALE: AS SHOWN SPEC. NO.
DRAWING NUMBER

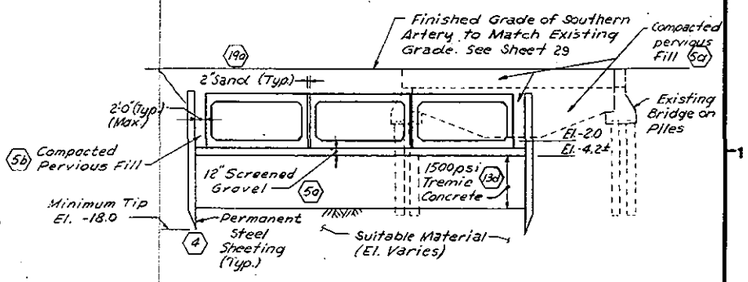
SHEET 24

PLATE 4-18

CORPS OF ENGINEERS — U. S. ARMY



PLAN
Scale: 1" = 20'



TYPICAL SECTION OF EXISTING BRIDGE AND PRECAST CONCRETE BOX CULVERT (19)

No Scale

SUGGESTED CONSTRUCTION SEQUENCE FOR THE PRECAST CONCRETE BOX CULVERT

STEP 1

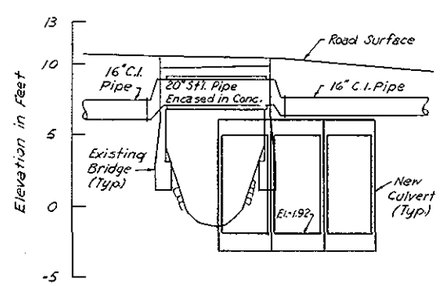
1. RE-ROUTE TRAFFIC OFF OF TWO NORTHBOUND LANES. SET UP TWO LANE SITE BYPASS BASED ON WIDTH BETWEEN EXISTING BUILDING AND BEAM NO. 13 OF BRIDGE DECK.
2. SET-UP RIVER WATER CONTROL SYSTEM.
3. DEMOLISH BRIDGE DECK AND WEST ABUTMENT AS REQUIRED FOR CULVERT CONSTRUCTION ON NORTHERN HALF OF SOUTHERN ARTERY.
4. DRIVE SHEETING FOR OUTLET HEADWALL AND RETAINING WALL SYSTEM AND CULVERT UP TO LIMITS SHOWN ON PLAN.
5. EXCAVATE WITHIN SHEETING AND COORDINATE UTILITY HANDLING.
6. PLACE TREMIE CONCRETE AND CONSTRUCT OUTLET HEADWALL AND RETAINING WALL SYSTEM AND BOX CULVERT.
7. BACKFILL, COORDINATE UTILITY HANDLING, TEMPORARY PAVE SOUTHERN ARTERY AS NECESSARY.

STEP 2

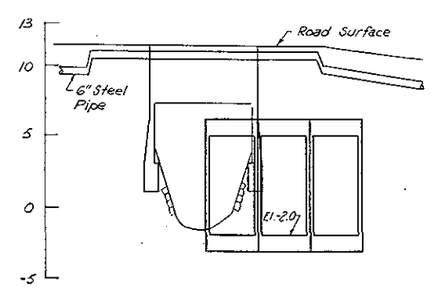
1. RE-ROUTE TRAFFIC TO NORTHBOUND LANES.
2. DEMOLISH REMAINDER OF BRIDGE DECK AND WEST ABUTMENT. DRIVE SHEETING TO INLET HEADWALL AND RETAINING WALL SYSTEM, EXCAVATE AND COORDINATE UTILITY HANDLING.
3. PULL STEP 1 LATERAL SHEET. PLACE TREMIE CONCRETE AND CONSTRUCT REMAINDER OF CULVERT AND INLET HEADWALL AND RETAINING WALL SYSTEM.
4. RETURN SOUTHERN ARTERY TO TWO-LANE TWO WAY TRAFFIC WHEN CONSTRUCTION PROCEEDS BEYOND STA. 1+00.

STEP 3

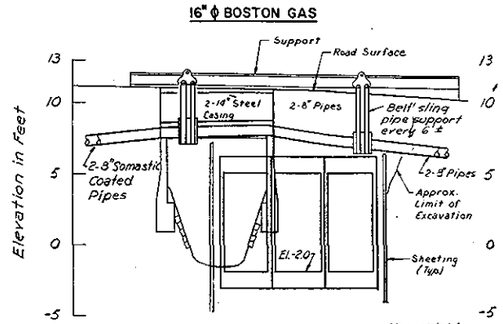
1. CUT SHEETING AT INLET AND OUTLET OF CULVERT AND DIVERT RIVER FLOW THROUGH THREE BARRELS.
2. DISMANTLE FLOW CONTROL SYSTEM, BACKFILL AS NECESSARY.
3. RESTORE SOUTHERN ARTERY, SIDEWALKS, PARKING LOT AND APPURTENANT LANDSCAPING AND EQUIPMENT AS SHOWN ON SHEET 29.



16" ϕ BOSTON GAS

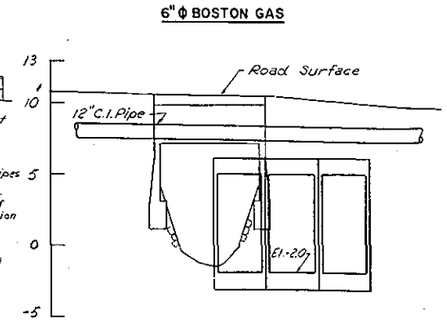


6" ϕ BOSTON GAS

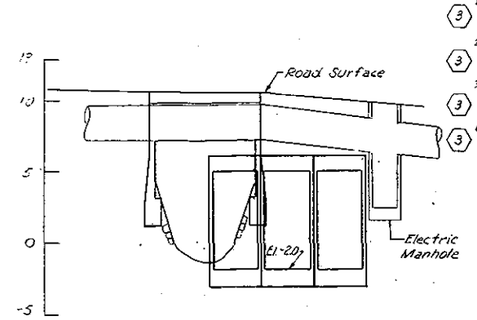


TWIN 8" ϕ MASS. ELECTRIC

Pipe Weight:
14" = 187 lbs/Ft
8" = 115 lbs/Ft
Total = 12,612 lbs/circuit



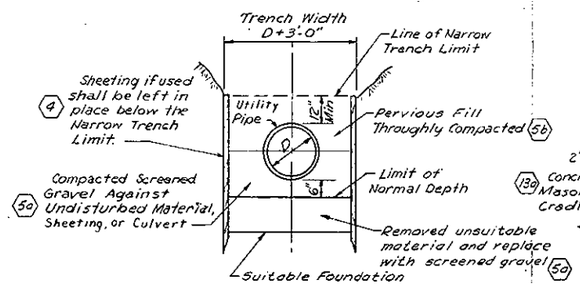
12" ϕ QUINCY WATER



2'-10" ϕ BOSTON EDISON

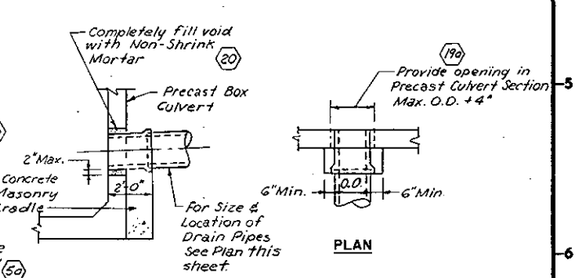
℄ PROFILES OF UTILITIES

Scale: Hor. 1" = 20' Vert. 1" = 4'



PIPE TRENCH DETAIL

No Scale



SECTION

PLAN

TYPICAL DRAIN CONNECTION (20)

No Scale

UTILITY NOTES:

(3) 1. BOTH 6-INCH AND 16-INCH GAS LINES SHALL BE REMOVED FROM SERVICE PRIOR TO CONSTRUCTION OF THE CULVERT AND REPLACED AT THE END OF CONSTRUCTION BY AUTHORIZED BOSTON GAS PERSONNEL.

(3) 2. THE QUINCY WATER LINE SHALL BE REMOVED FROM SERVICE PRIOR TO CONSTRUCTION OF THE CULVERT AND REPLACED AT THE END OF CONSTRUCTION BY THE CITY OF QUINCY PUBLIC WORKS DEPARTMENT.

(3) 3. THE TWIN MASSACHUSETTS ELECTRIC LINES SHALL BE SUPPORTED DURING CONSTRUCTION IN ACCORDANCE WITH DETAIL AS SHOWN AND AS ARRANGED WITH THE MASSACHUSETTS ELECTRIC COMPANY.

(3) 4. THE BOSTON EDISON LINE SHALL BE REMOVED FROM SERVICE PRIOR TO CONSTRUCTION OF THE CULVERT AND REPLACED AT THE END OF CONSTRUCTION BY AUTHORIZED BOSTON EDISON PERSONNEL.

GRAPHIC SCALES

20' 0 20' 40'
1" = 20'
4' 0 4' 8'
1" = 4'

REVISION	DATE	DESCRIPTION	BY

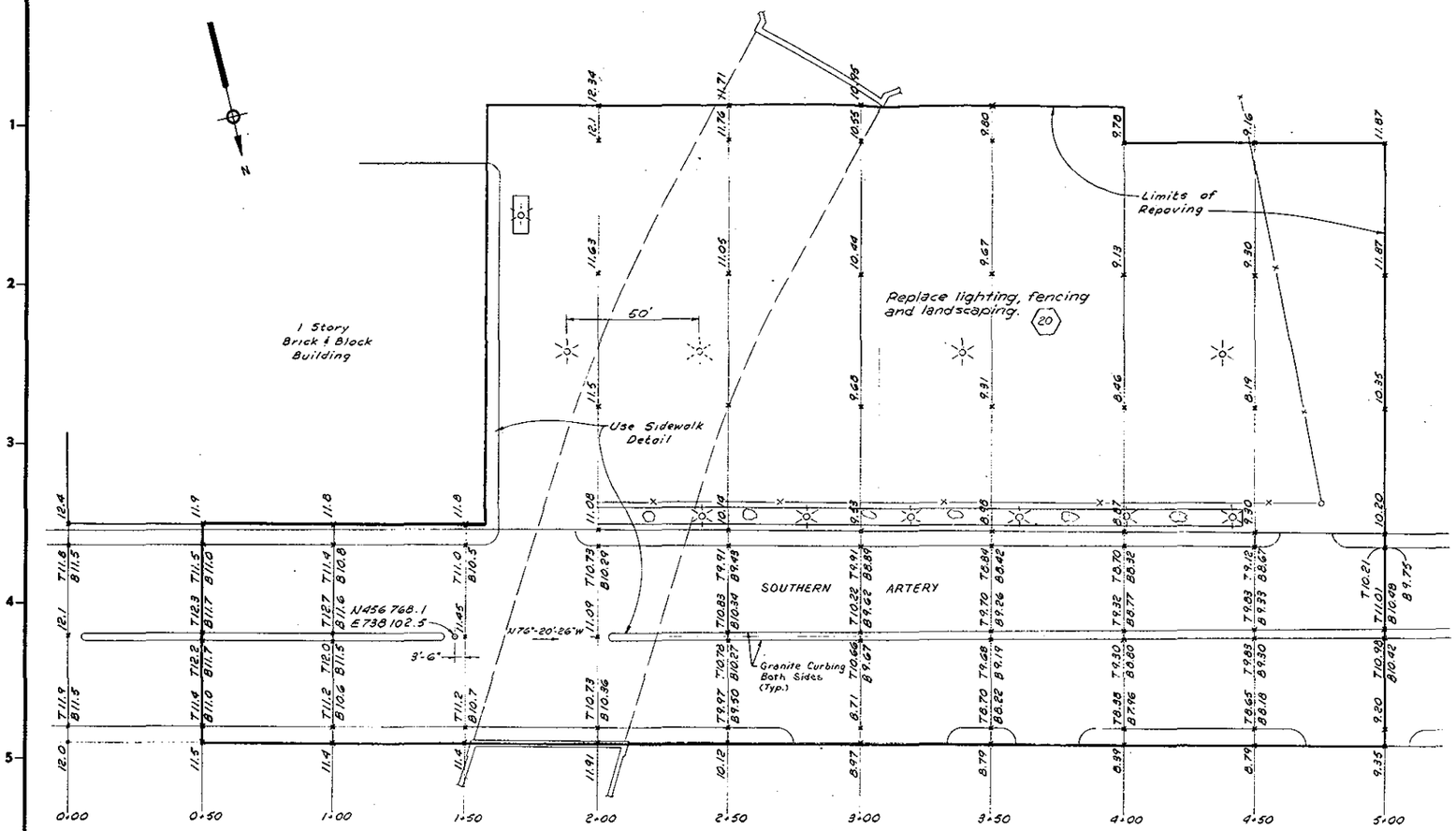
METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY W.R.K. | DR. BY R.L.J. | CK. BY W.R.K.
SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
CIVIL-CULVERT DETAILS
QUINCY AND BRAINTREE MASSACHUSETTS
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO. | DRAWING NUMBER
SHEET 28

PLATE 4-19

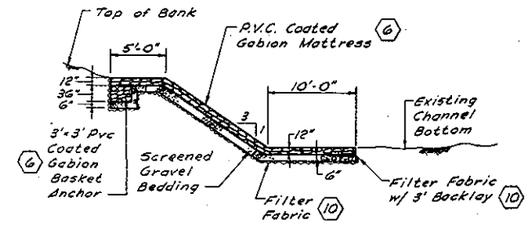
CORPS OF ENGINEERS — U. S. ARMY



PAVING PLAN

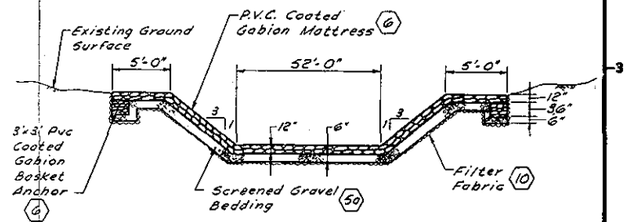
SCALE: 1" = 20'

T = TOP OF CURB
B = BOTTOM OF CURB



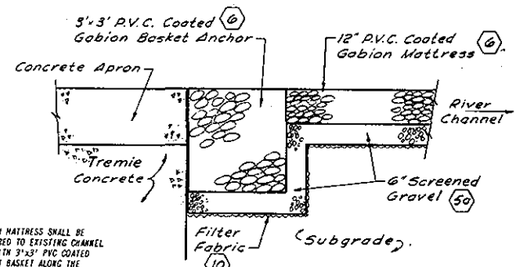
TYPICAL CHANNEL SECTION DOWNSTREAM OF CULVERT

NO SCALE



TYPICAL CHANNEL SECTION UPSTREAM OF CULVERT

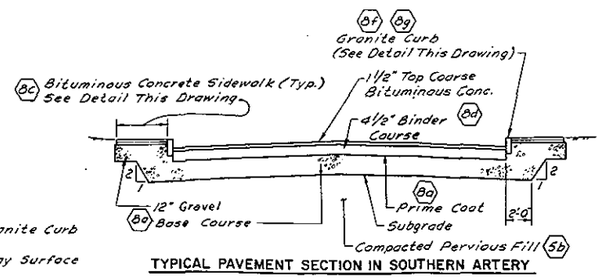
NO SCALE



NOTE: GABION MATTRESS SHALL BE ANCHORED TO EXISTING CHANNEL BED WITH 3'x3' PVC COATED GABION BASKET ALONG THE EXTREME UPSTREAM AND DOWNSTREAM CHANNEL INTERFACE SIMILAR TO ABOVE DETAIL.

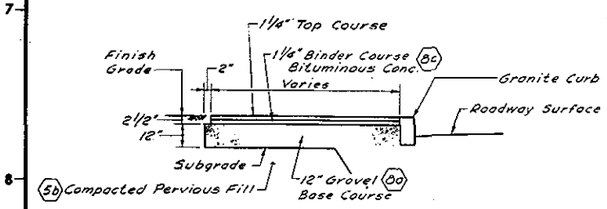
CONNECTION DETAIL AT CULVERT RETAINING WALL APRONS

NO SCALE



TYPICAL PAVEMENT SECTION IN SOUTHERN ARTERY

NO SCALE



TYPICAL FORMAT OF BITUMINOUS CONCRETE SIDEWALK

NO SCALE

PAVING NOTES:

1. ELEVATIONS, AS INDICATED ON PAVING PLANS, ARE FOR PROPOSED FINISHED GRADES.
2. PRIOR TO PLACING PAVEMENTS, SET NEW AND EXISTING MANHOLE CASTINGS, INLETS, GRATES, ETC. TO THE PROPOSED FINISHED GRADES.
3. PAVEMENT CROSS SECTION FOR PARKING LOT SHALL BE SIMILAR TO ABOVE.
4. BACKFILL TRENCH AND STRUCTURE EXCAVATIONS IN 6-INCH LIFTS AND COMPACT BY VIBRATORY ROLLER TO 100% MAXIMUM DENSITY (ASTM D 1557, METHOD D).

GRAPHIC SCALES

20' 0 20' 40'

1" = 20'

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY W.R.K. | DR. BY E.S.Z. | CK. BY W.R.K.

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

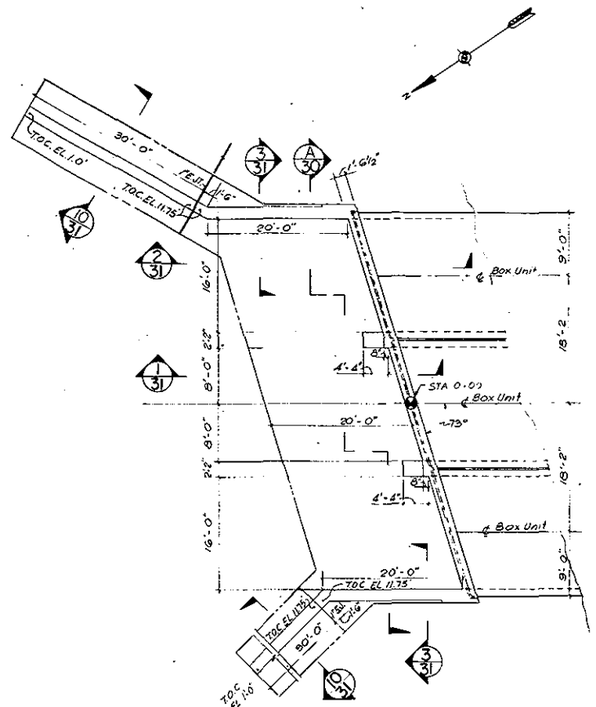
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
CIVIL-MISCELLANEOUS DETAILS
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

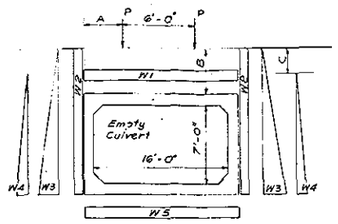
SCALE | SPEC. NO. | DRAWING NUMBER

SHEET 29

PLATE 4-20



TOP PLAN OF OUTLET
Scale: 1/8" = 1'-0"



TYPICAL LOADING DIAGRAM
For Precast Sections
No Scale

Loading Condition	Loads						Dimensions		
	P (#)	W1 (Psf)	W2 (Psf)	W3 (Pcf)	W4 (Pcf)	W5 (Psf)	A	B	C
Case 1	20,800	Rs	0	0	0	Bearing Pressure	0' To 4'-0"	0	NA
Case 2	20,800	Rs+Ob+Pd	130	65	30	Bearing Pressure	4'-0"	6'-0"	4'-9"
Case 3	0	Rs+Ob	130	65	30	Bearing Pressure	NA	6'-0"	4'-9"

Where P = HS20-44 Vehicle Load x 1.30 Impact Factor
= 16,000 x 1.3 = 20,800#

Rs = Dead Load of Roof Slab

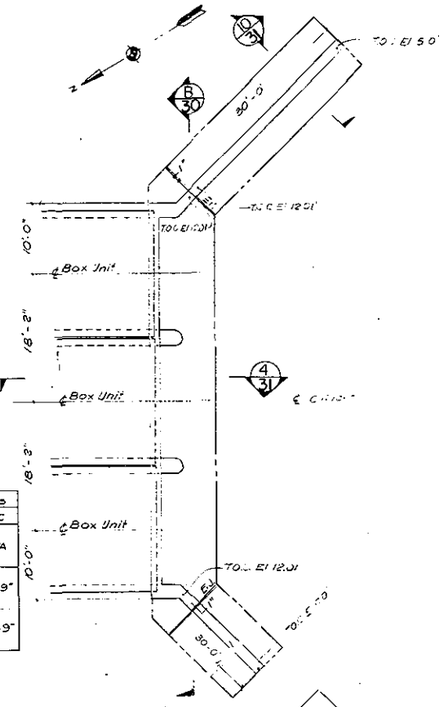
B = Height of overburden soil

Ob = Weight of overburden = B x 130 Pcf

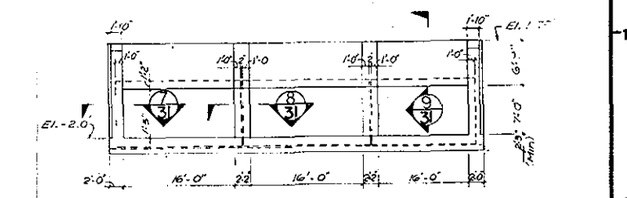
Pd = Distributed "p" load - as per AASHTO section 13.3

W2, W3 & W4 are equivalent fluid pressures

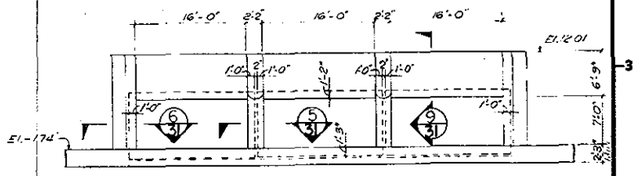
W4 = Net bearing pressure



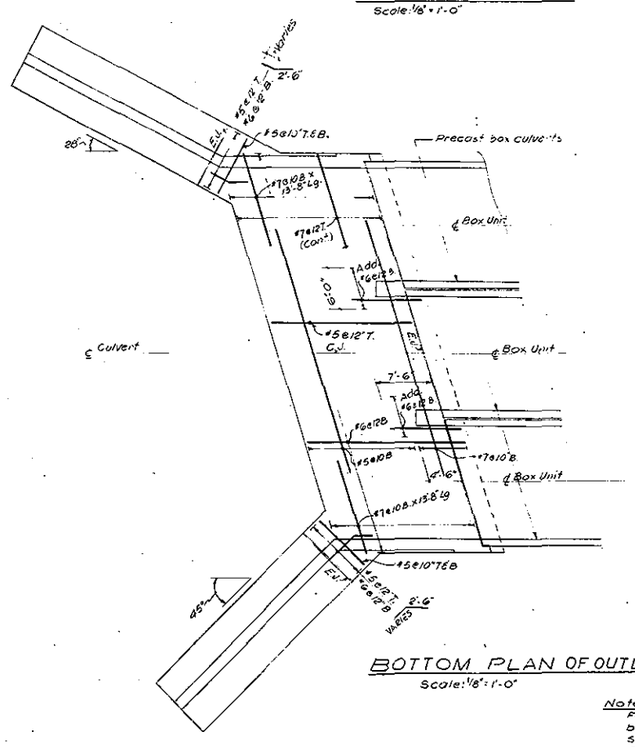
TOP PLAN OF INLET
Scale: 1/8" = 1'-0"



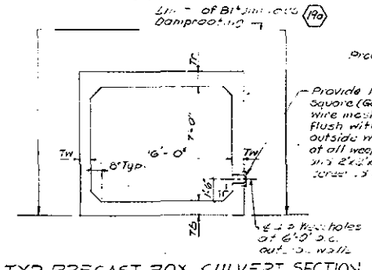
ELEVATION (A/30) OF OUTLET
Scale: 1/8" = 1'-0"



ELEVATION (B/30) OF INLET
Scale: 1/8" = 1'-0"

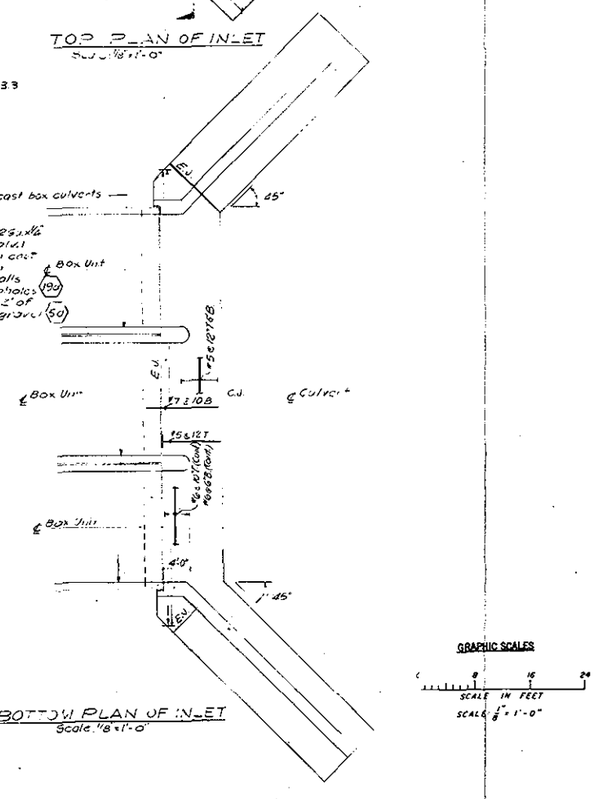


BOTTOM PLAN OF OUTLET
Scale: 1/8" = 1'-0"



TYP. PRECAST BOX CULVERT SECTION
No Scale

Culvert Minimum Thickness			Remarks
Tr	Tw	Tb	
14"	12"	15"	



BOTTOM PLAN OF INLET
Scale: 1/8" = 1'-0"

Note:
Foundation designs are based on an allowable soil bearing pressure of 1.25 tons per square foot.

GRAPHIC SCALES
SCALE IN FEET
SCALE 1/8" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY L.S. D.S. | DR. BY J.F. | CK. BY J.A.D.

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

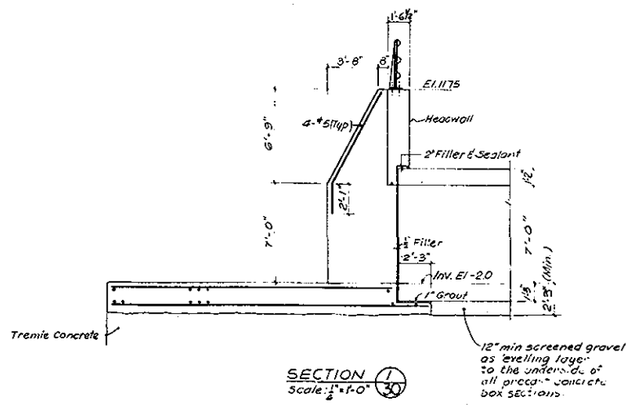
WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
STRUCTURAL-CULVERT PLANS AND ELEVATIONS
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

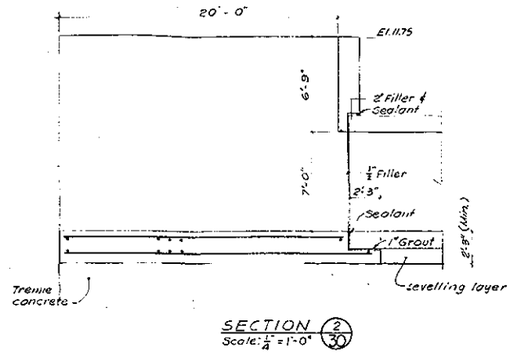
SCALE | SPEC. NO.
DRAWING NUMBER
SHEET 30

PLATE 4-21

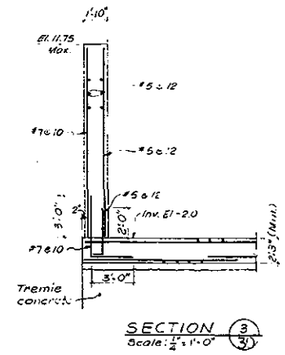
CORPS OF ENGINEERS — U. S. ARMY



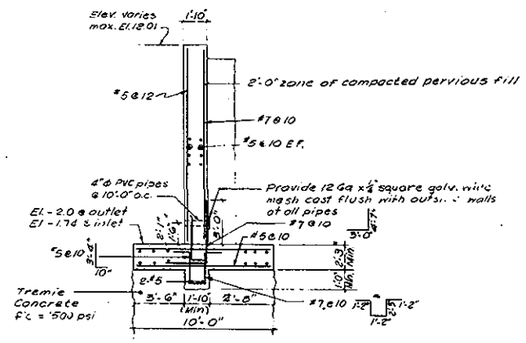
SECTION 1/30
Scale: 1/8" = 1'-0"



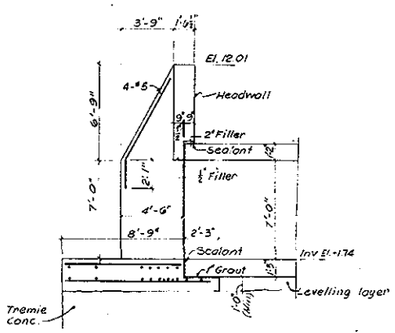
SECTION 2/30
Scale: 1/8" = 1'-0"



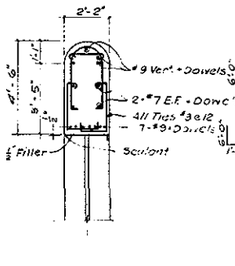
SECTION 3/30
Scale: 1/8" = 1'-0"



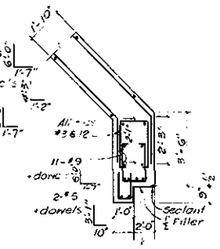
SECTION 10/30
Scale: 1/8" = 1'-0"



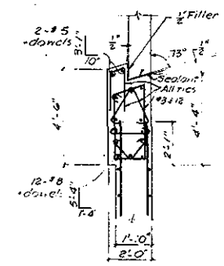
SECTION 4/30
Scale: 1/8" = 1'-0"



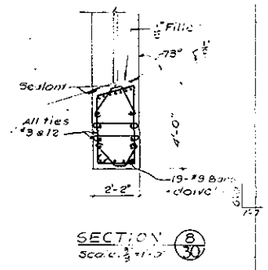
SECTION 5/30
Scale: 1/8" = 1'-0"



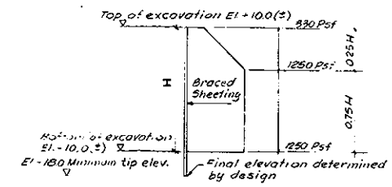
SECTION 6/30
Scale: 1/8" = 1'-0"



SECTION 7/30
Scale: 1/8" = 1'-0"



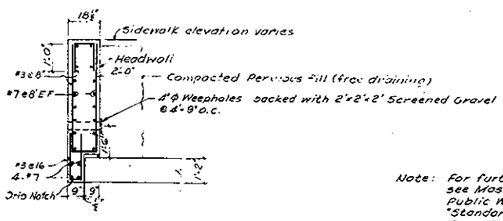
SECTION 8/30
Scale: 1/8" = 1'-0"



APPARENT PRESSURE ENVELOPE

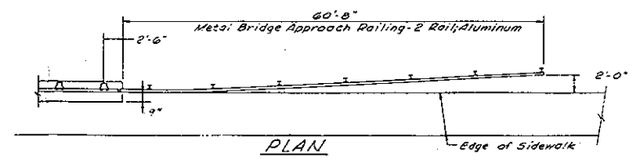
NOTES FOR BRACED SHEETING, LEFT-IN-PLACE

1. The contractor shall retain a professional engineer registered in the Commonwealth of Massachusetts to design the sheeting. The sheeting should be stable for all conditions encountered. Bracing, construction tolerance and placement of the precast box culvert sections shall be considered in the design. The sheeting shall be driven as deep as required to provide suitable support for the excavation, the adjacent structure and to prevent piping. Excavation and tremie concrete placement will be performed in the wet/submerged condition (balanced hydrostatic forces)
2. A certificate of design shall be submitted to the engineer at least 10 days prior to construction. The sheeting shall be continuous interlocking steel sheet piling.
3. The apparent pressure envelope shown on this drawing shall be the minimum envelope used for design. The apparent pressure envelope assumes balanced hydrostatic forces from the top of the braced excavation. The contractor's design shall include all additional loads such as embankment loads, sloping ground surface and unbalanced hydrostatic forces that may occur due to the construction sequence
4. The minimum tip elevation of the steel sheeting shall be El. -18.0 The final tip elevation shall be established by design.
5. All sheeting shall be left-in-place.
6. Earth or rock tie backs shall not be used. Cut-off elevation of sheeting left-in-place shall be three feet below the ground surface unless otherwise indicated or directed by the Contracting Officer.
7. After the installation of the tremie concrete and prior to the installation of the precast concrete box culvert sections, the Contractor shall not dewater to the top of the tremie cap if the river water surface is above El. 2.5.

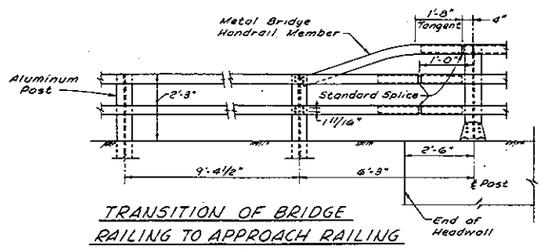


SECTION 9/30
Scale: 3/8" = 1'-0"

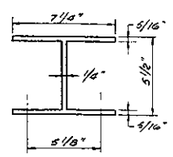
Note: For further detail of Bridge Railing, see Massachusetts Department of Public Works, Bridge Division detail, "Standard Metal Bridge Railings Type AL-3 (Aluminum)".



PLAN

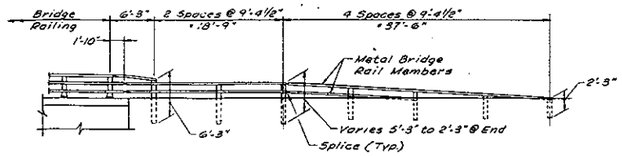


TRANSITION OF BRIDGE RAILING TO APPROACH RAILING
No Scale



SECTION ALUMINUM POST
No Scale

For detail see "Transition of Bridge Railing to Approach Railing" this sheet



ELEVATION

DETAIL OF APPROACH RAILING
No Scale

GRAPHIC SCALES
SCALE IN FEET
SCALE 1/8" = 1'-0"

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

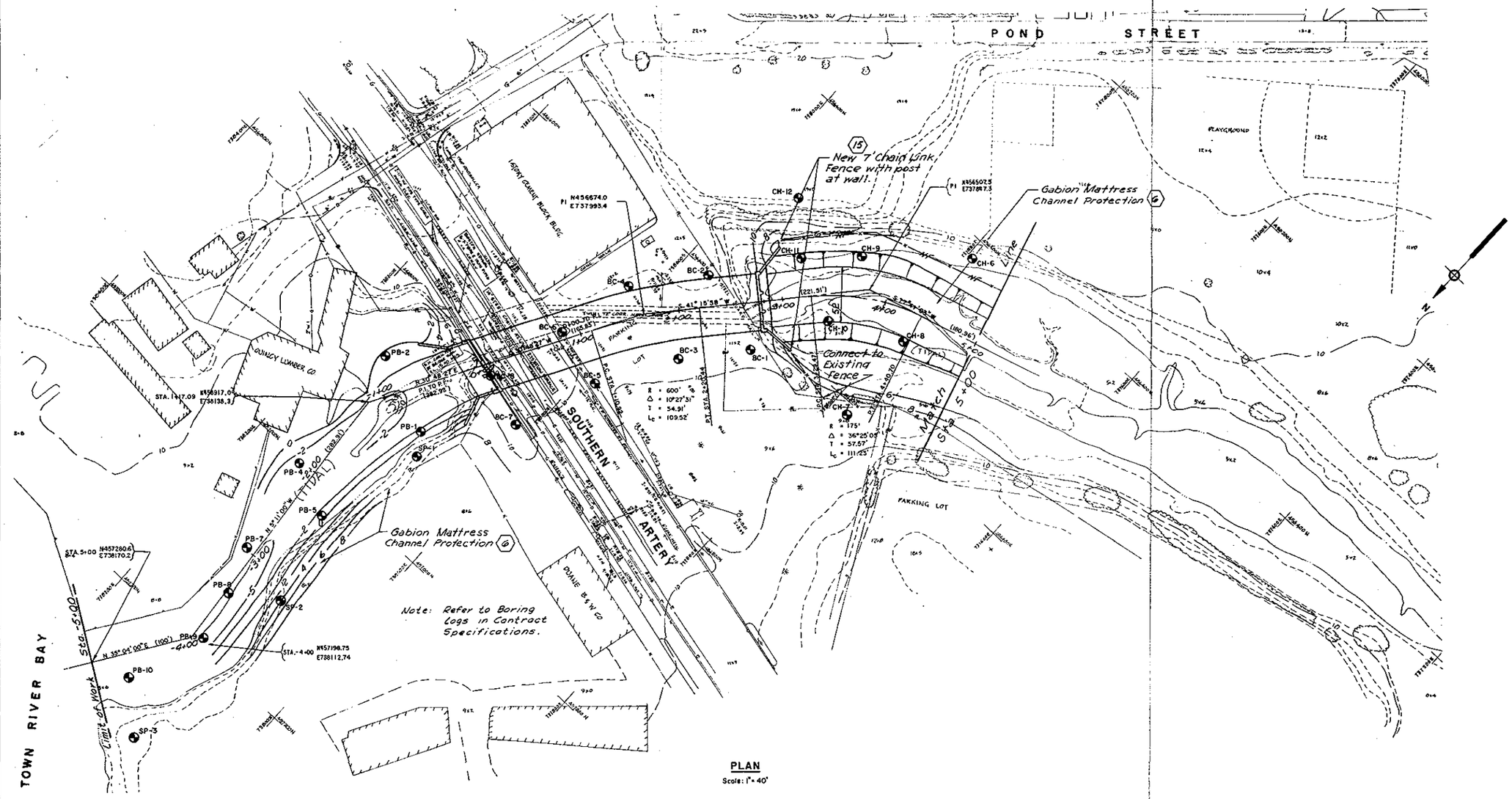
DES. BY L.S.O.S. | DR. BY J.F. | CK. BY J.A.D.

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
STRUCTURAL-CULVERT SECTIONS
QUINCY AND BRAINTREE MASSACHUSETTS
APPROVED DATE
CHIEF, ENGINEERING DIVISION
SCALE | SPEC. NO. | DRAWING NUMBER
SHEET 31

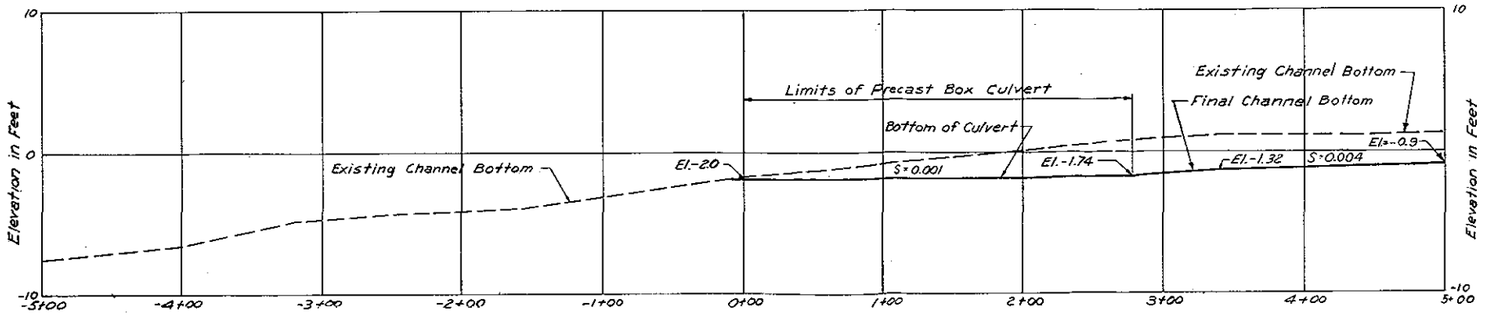
PLATE 4-22

CORPS OF ENGINEERS U. S. ARMY



PLAN

Scale: 1" = 40'



PROFILE ON ℄ OF CHANNEL AND CULVERT

Scale: Hor. 1" = 40' Vert. 1" = 4'

GRAPHIC SCALES

4' 0 4' 8'
1" = 4'-0"

40' 0 40' 80'
1" = 40'

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY W.R.K. | DR. BY R.L.J | CK. BY W.R.K.

SUBMITTED: ARCHITECT ENGINEER

APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH

REVIEWED: PROJECT MANAGER

APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
CIVIL-PLAN AND PROFILE
STA. -5+00 TO STA. 5+00
QUINCY AND BRAINTREE MASSACHUSETTS

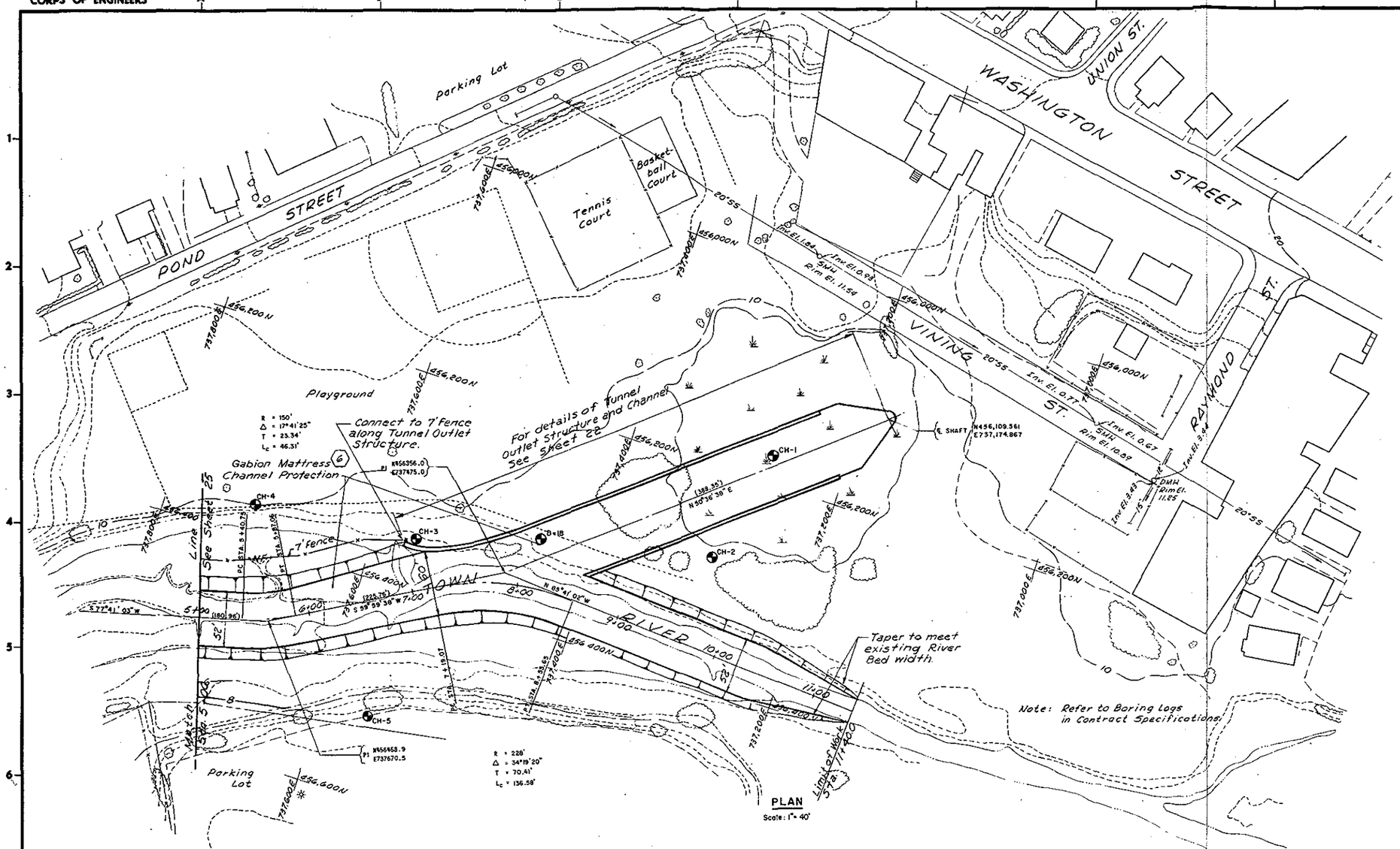
APPROVED DATE
CHIEF, ENGINEERING DIVISION

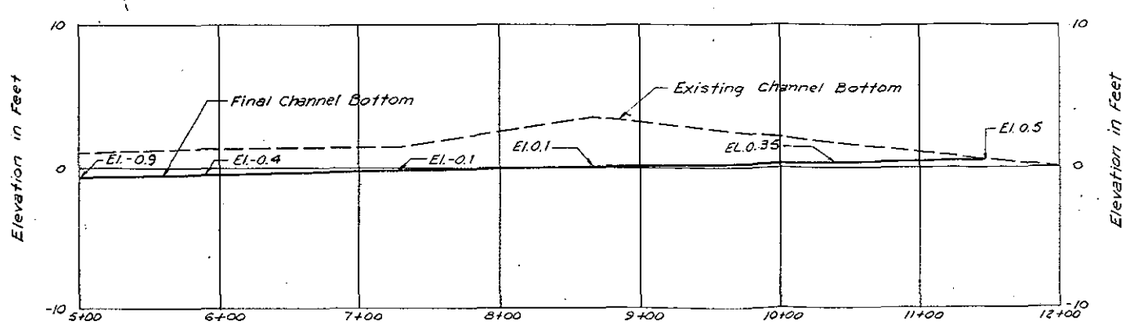
SCALE SPEC. NO
DRAWING NUMBER

SHEET 25

PLATE 4-23

CORPS OF ENGINEERS U. S. ARMY





PROFILE ON ℄ OF CHANNEL

Scale: Hor. 1" = 40' Vert. 1" = 4'

GRAPHIC SCALES

1" = 4'-0"

1" = 40'

REVISION	DATE	DESCRIPTION	BY

METCALF & EDDY, INC.
BOSTON, MASS.

DES. BY W.R.K. DR. BY R.L.J. CK. BY W.R.K.

SUBMITTED: ARCHITECT ENGINEER
APPROVAL RECOMMENDED: CHIEF, DESIGN BRANCH
REVIEWED: PROJECT MANAGER
APPROVAL RECOMMENDED: CHIEF, PROJECT MNGT. BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
TOWN BROOK LOCAL PROTECTION
TOWN RIVER IMPROVEMENTS
CIVIL-PLAN AND PROFILE
STA. 5+00 TO STA. 11+47
QUINCY AND BRAINTREE MASSACHUSETTS

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE SPEC. NO.
DRAWING NUMBER

SHEET 26

PLATE 4-24

APPENDIX A

GEOLOGIC AND ROCK MECHANICS LOGS AND DATA FOR TUNNEL

CONTENTS

A-1	Earthquake Motions for Town Brook site
A-2	Typical Drill Log
A-3	Example Detailed Graphic Geologic Log
A-4	Example Oriented Core Log

TABLES

A-1	Rock Support Defined by Q-system

SUBJECT Earthquake Motions for Town Brook, MA Site

COMPUTATION Acceleration, velocity, duration

COMPUTED BY YY CHECKED BY DATE

Source reference: "Geological Seismological Evaluation of Earthquake Hazards at Surry Mountain Damsite, New Hampshire", E. L. Krinitzsky (WES)

Town Brook project located in Krinitzsky Zone One (Fig. 6) about 70Km from Cape Ann Inner Zone. Potential earthquakes are:

Source	Distance (Km)	(Source) MMI	(Site) MMIs
Local, Zone One	0 (Nearfield)	VII	VII
Cape Ann, Inner	70 (Farfield)	IX	VII (1)

(1) Attenuated 1.9 units from Chandra Curves (Fig. 15).

MM Intensity			g	cm/sec	sec
VII Nearfield		mean	0.13	10	2
		mean+σ	0.22	19	5
		mean+2σ	0.27	27	7
VII Farfield		mean	0.1	20	6
	(selected)	mean+σ	0.15	27	9
		mean+2σ	0.2	34	12

From the site geology and seismic history, the values for the mean and one standard deviation, farfield source, are selected.

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R I

SHEET 1 OF 3
DATE
HOLE NO. T-10
LINE & STA.
OFFSET
SURF. ELEV.

TO Metcalf & Eddy, Inc.
PROJECT NAME MDC Town Brook Stormwater
REPORT SENT TO above /Tunnel
SAMPLES SENT TO "

ADDRESS Boston, Mass
LOCATION Quincy, Mass
PROJ. NO.
OUR JOB NO. 83-237

GROUND WATER OBSERVATIONS
At 1'6" after Hours
At after Hours

	CASING	SAMPLER	CORE BAR.
Type	HW NW	S/S	NQ2WL
Size O	4" 3"	1 3/8"	2"
Hammer Wt	300#	140#	BIT
Hammer Fall	24"	30"	Dia.

	Date	Time
START	4/5/83	a.m. p.m.
COMPLETE	4/14/83	a.m. p.m.
TOTAL HRS.		

BORING FOREMAN K. Allen
INSPECTOR Tracy
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	12-18	Moisture Density or Consist.	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		0'-2'	D	30	30	6	Dense		Brown fine to coarse Sand & gravel, little silt - FILL	1	24"	2"
						6						
		2'-4'	D	4	5	1		3'		2	24"	2"
						1	Soft		Root mat.& organic silt. Difficulty picking up sample-pushing gravel Fill)			
		4'-6'	D	1	1	1					24"	0
						1						
		6'-8'	D	1	1	1	"			3	24"	2"
						1		8'				
		8'-10'6"	D	3	/30"		"		Dark Brown PEAT	4	30"	4"
		10'6"-11'10"	D	2/16"			"	11'10"		5	16"	10"
		11'10"-12'	D	4/2"			"	12'	Gray clayey SILT	5A	2"	2"
		12'-14'	D	8	19	12	Dense	13'	Gray silty fine SAND	6	24"	16"
						18		14'	Gr.silty F-C SAND & Gravel			
		14'-16'	D	12	14	16	"	15'	Gr.F-C SAND&Gravel, tr. silt	7	24"	15"
						21						
		16'-18'	D	12	7	5	Stiff		Gray fine sandy SILT	8	24"	10"
						4						
		18'-19'2"	UF					19'2"		P1	14"	11"
		19'6"-21'	D	18	10	7	M/dense		Gray fine to coarse SAND & Gravel, some silt	9	18"	8"
		21'-23'	D	7	14	33	V/dense			10	24"	9"
						43						
		23'-25'	D	51	42	66	"		" little silt	11	24"	15"
						78						
		25'-27'	D	21	36	60	"		" Yellow Brown	12	24"	10"
						36						
		27'-29'	D	20	26	25				13	24"	7"
						18		29'				
		29'-31'	D	24	24	27	"		Brown silty fine SAND, some coarse sand & fine to coarse gravel	14	24"	9"
						22						
		31'-33'	D	26	30	22	Dense			15	24"	16"
						20		33'				
		33'-35'	D	21	15	16	"		Brown silty fine SAND	16	24"	10"
						16		35'				
		35'-37'	D	19	48	55	Very		Gray silty fineSAND, little c. sand & fine gravel	17	24"	11"
						105	dense					
		37'	D	Ref.								
		38'-38'3"	D	100/0"			"		" & fine to coarse grave	18	3"	3"
					65/3"	(300#)						

GROUND SURFACE TO 50 USED 4 "CASING: THEN 3" to 62'6"

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

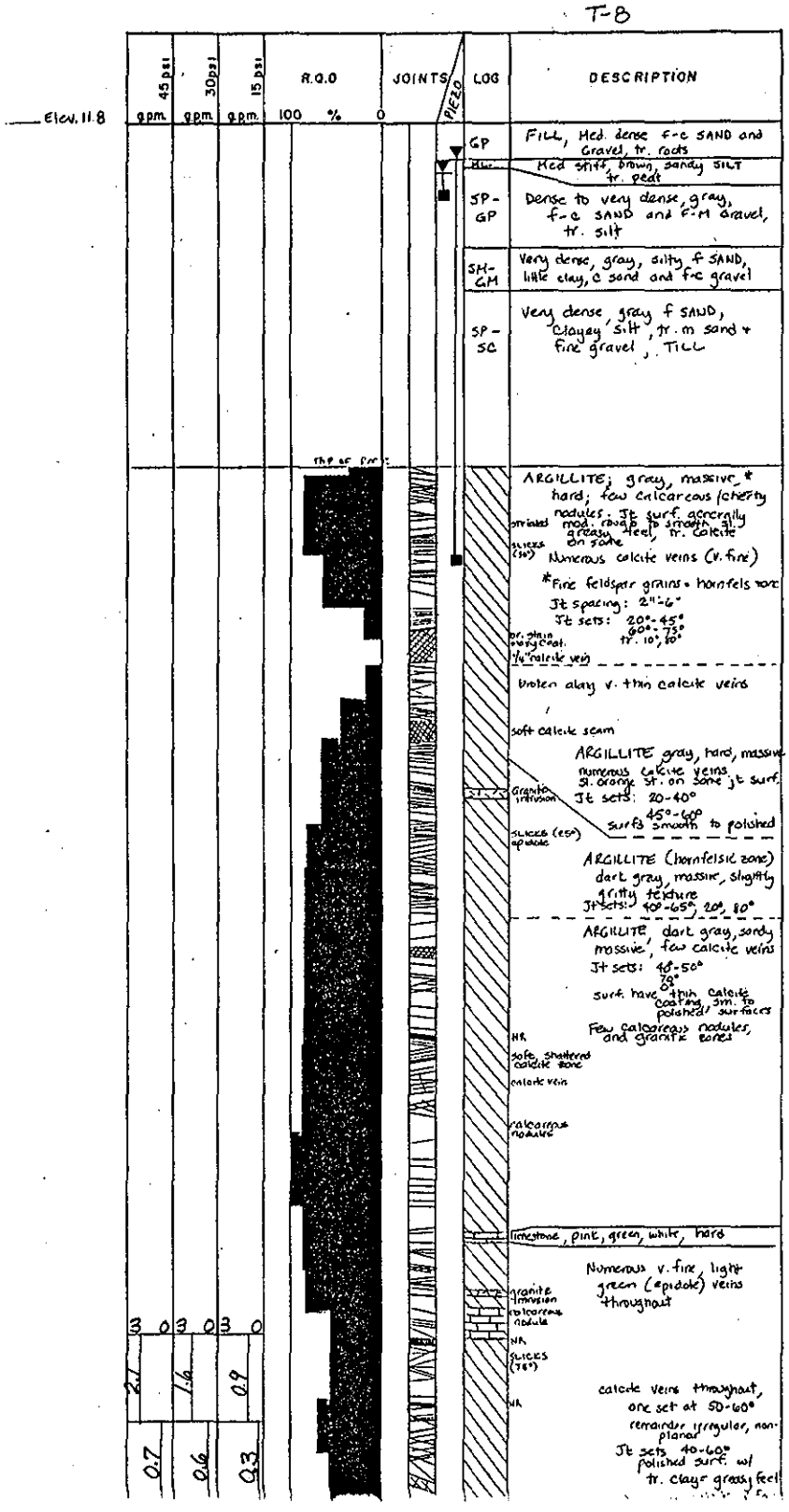
Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring 60'9"
Rock Coring 139'3"
Samples 29

HOLE NOT-10

TOWN PRESS — EAST PROV.

A-2 TYPICAL DRILL LOG



A-3 EXAMPLE DETAILED GRAPHIC GEOLOGIC LOG

McPHAIL ASSOCIATES

ROCK CORE ORIENTATION REPORT

HOLE NO. T-10

FILE NO. 1505

SHEET NO. 1 of 2

LOCATION:

ELEVATION:

DATE START: 11 April 83

DATE FINISH: 11 April 83

DRILLER: K. Allen

GEOLOGIST: J. O'Brien

FILM ANALYSIS: J. O'Brien

PROJECT: TOWN BROOK STORMWATER TUNNEL

CLIENT: Metcalf & Eddy

CONTRACTOR: Guild Drilling CO

HOLE SIZE: 1 7/8"

CORE BARREL: NQ

ORIENTATION INSTRUMENT

MANUFACTURER: Eastman

IDENTIFICATION NO.:

EXPOSURE INTERVAL: one (1) min/sec

INSTRUMENT CORRECTIONS

Declination ... 15 ..... Degrees ☐ East ☑ West

Azimuth Correction for Declination

East Declination ☐ Add to Azimuth

West Declination ☑ Subtract from Azimuth

Reference Groove Correction:

From Above, Groove is ..... 0 ..... Degrees of Orientation Lug. ☐ Right ☐ Left

☐ Right Add to Lug Azimuth

☐ Left Subtract from Lug Azimuth

REMARKS: start: 10:00
1st foot: 10:26
stop: 15:00

ROCK SUMMARY

FORMATION NAME: Weymouth

ROCK TYPE: Argillite

FOOTAGE CORED – FINISH: 177 START: 155

TOTAL FOOTAGE CORED: 12

ORIENTING DATA

FIELD DATA: EXPOSURE STOPS				FILM ANALYSIS DATA: DRIFT DIRECTION OF BOREHOLE			ORIENTATION OF THE INSTRUMENT: ORIENTING LUG				REFERENCE GROOVE		
NO.	TIME	FRAME NO.	DEPTH	ANGLE	MAGNETIC	TRUE	DIRECTION (Mag)	AZIMUTH (Mag)	DEC. CORR.	TRUE LUG AZ.	CORR.	TRUE AZ.	TRUE DIRECTION
1	1030	30	156	0	S55W	S40W	S25E	155	15	140	0	140	S40E
		31											
		32											
2	1038	38	157	0	S60W	S45W	S20W	160	15	145	0	145	S35E
		39											
		40											
3	1046	46	158	0	S62W	S47W	S24W	156	15	141	0	141	S39E
		47											
		48											
4	1054	54	159	0	S68W	S53W	S22W	158	15	143	0	143	S37E
		55											
		56											
5	1113	73	161	0	S67W	S52W	S24W	156	15	141	0	141	S39E
		74											
		75											
6	1121	81	162	1°	S61W	S46W	S23W	157	15	142	0	142	S38E
		82											
		83											
7	1133	93	163	1°	S52W	S37W	S25W	155	15	140	0	140	S40E
		94											
		95											
8	1152	112	165	0	S46W	S31W	S28W	152	15	137	0	137	S43E
		113											
		114											

A-4 EXAMPLE ORIENTED CORE LOG

TABLE A-1. ROCK SUPPORT DEFINED BY Q-SYSTEM

BOREHOLE STATION ROCK TYPE	T-2 0+88 GRANITE	T-3 34+07 GRANITE	T-4A 29+02 GRANITE	T-5 29+68 GRANITE	T-6 21+90 GRANITE	T-7 15+46 GRANITE	T-8 14+53 ARGILLITE	T-9B 7+44 CALCARENITE/ ARGILLITE	T-9 4+68 ARGILLITE/ ARENITE	T-9A 3+85 ARGILLITE/ ARENITE	T-10 0+00 ARGILLITE
RQD	98	100	100	80	85	80	55	35	15	45	90
Jr	1.5	1.5	1.5	2.0	1.5	1.5	1.0	0.5	1.0	1.0	1.5
Ja	0.75	0.75	2.0	3.0	2.0	2.0	1.0	2.0	2.0	2.0	2.0
Jn	3.0	6.0	4.0	4.0	3.0	3.0	15.0	6.0	15.0	6.0	3.0
Jw	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
SRF	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Q	65	33	18	13	21	20	3.6	1.5	0.5	3.75	22
SUPPORT REQUIRED	(1)	(1)	(1)	(1)	(1)	(1)	(1)	(2)	(3)	(1)	(1)

(1) No support required

(2) Untensioned grouted bolts 1 in. long + 2-3 cm shotcrete

(3) Untensioned grouted bolts 1 in. long + reinforced mesh or chain link mesh

APPENDIX B

PERMEABILITY DATA AND CALCULATIONS

CONTENTS

B-1 Summary of Piezometer Readings Along Tunnel

B-2 Permeability and Inflow Calculations

B-3 Example Pressure Test and Permeability Log

SUMMARY OF
PIEZOMETER READINGS
ALONG TUNNEL

Boring Number	Elevations for bottom of Piezometer (ft.)	Water Level Elevations			
		Date 4/29-5/13	6/13	7/11	7/26 Date
T-2	-10.4	+19.1	+17.6	+17.3	+16.9
	-65.9	+19.1	+17.6	+17.7	+17.5
T-3	-11.3	+13.7	+12.7	+12.7	+12.7
	-77.3	+14.9	+14.7	+14.3 vented today	+14.1
T-4A	-36.1		+11.9	+12.1	+12.1
	-111.6		+14.4	+13.9	+13.7
T-5	-36.1		+11.9	+12.3	+12.2
	-109.6		+13.4	+13.6	+13.4
T-6	-10.4	Not Available	+8.1	+7.7	+7.6
	-70.4		+8.6	+8.6	+8.5
T-8	+3.6	+6.3	+6.3	+6.0	+6.2
	-38.2	+8.2	+7.8	+7.5 vented today	+7.5
T-9	+10.2	+12.2	+12.2	Appx. +11.7	No Reading
	-39.8	>+20.2 flowing	>+20.2 flowing	>+20.2 slightly flowing	+20.1
T-10	-21.4	+6.1	+4.6	+4.3	+4.0
	-81.4	+6.1	+4.6	+4.3 vented today	+4.1

FIELD NOTE: Overburden piezometers marked with duct tape.

METCALF & EDDY

B-2. ROCK PERMEABILITY AND TUNNEL INFLOW CALCULATIONS

a. References

1. Goodman, R E., "Ground water Inflows During Tunnel Driving", Bulletin of the Association of Engineering Geologists, Vol. 1, No. 1, p. 39ff, 1965.

2. USBR, "Earth Manual", 2nd Edition, PP 576-577, Denver, 1974.

b. General Outline of Method

1. First Step - Calculate Permeability of Rock

This is done by use of the formula in Ref. 2:
$k = C*q/H$

where:

k = permeability in feet per year
q = pumping rate in test in gallons per minute
H = applied head in feet
C = coefficient from table
= 4900 for 10-ft test section in NX borehole.

The applied head is the NET head applied to the test section, as is found by taking the lesser of (a) the height from ground to the top of the test section or (b) the distance from ground to top of groundwater table; plus the pressure head; less the friction losses in the system; all expressed in feet. The net pressure has been calculated on the test sheets for this project in terms of pounds per square inch. These can be converted into feet of water by dividing by 0.433.

If q is given in cubic feet per minute, and k is desired in feet per minutes, and H is in psi, the above formula reduces to:

$k = 0.0302*q/H$

where:

k = permeability in ft/min
q = pumping rate in CFM
h = applied head in psi

2. Second Step -- Calculate Inflow into Tunnel

This is done by use of the formula in Ref. 1, in which it is assumed that the tunnel passes beneath a standing body of water, as follows:

$Q = 2*k*(h+H)/2.3*\log(r/2*h)$

where:

Q = inflow in CFM per foot of tunnel
k = permeability in FPM
h = distance from ground surface to tunnel in feet
H = depth of water body in feet
r = excavated radius tunnel.

~~From Appendix I-I,~~ the excavated diameter of both the machine-bored and conventinally excavated tunnels is 14.0 feet. The excavated radius is therefore 7.0 feet. There is no standing water over the tunnel on this project; and further, since the groundwater table lies at or below the ground surface, the distance "h" in the above equation becomes the height of the groundwater table over the tunnel, or depth of tunnel minus depth to groundwater. Therefore, the formula for inflow reduces to:

Q = 2*1*)D-d)/2.3*log(3.5/(D-d))

c. Calculations of Permeability

The permeabilities on the water pressure test data sheets have been calculated according to the above method (see B-3, example log).

There were 74 water pressure tests made in the borings. Each of these tests consisted of 5 individual runs at ascending gage pressures of 15, 30 and 45 psi, and descending pressures of 30 and 15 psi. The permeabilities resulting from these 5 runs in each test were averaged to produce a composite permeability.

In nine cases, the results were discarded because of evident leakages around the packers. In these nine cases -- which were usually in rock with very high RQD's -- the water losses and resulting permeabilities were several orders higher than could be expected, and it is vitually certain that the spurious results are more likely the product of a defect in the test procedure than actual leaky rock.

The permeabilities of the remaining 65 tests are plotted against RQD. In other geology, there is often seen to be a strong correleation between low RQD's and high permeability. In this case, there is no evident correlation between these two parameters; indeed, a number of tests with no "take" at all occurred in formations with RQD's less than 40.

The characteristic permeability was calculated simply as the arithmetic average of 65 individual tests. This average permeability is 0.000112 feet per minute, which equates to 0.00006 centimeters per second.

d. Inflow into Tunnels

The average permeability of 0.000112 feet/minute was used to calculate inflow into the tunnel in accordance with the method described above. The calculation is made for the section of tunnel near the outlet shaft, which is deepest below groundwater table and will therefore have the greatest calculable inflow, as follows:

Q = 2*k*(D-d)/(2.3* log(3.5/(D-d))
= 2*0.000112*(180-0)/(2.3*log(3.5/(180-0))
= -0.01024 cfm per foot of tunnel

This results in an anticipated total inflow for the entire tunnel of 4000*0.01024 = 41.0 cubic feet per minute.

Project Town K Tunnel Acct No [illegible] Page ___ of ___
Subject ___ Compld By ___ Date ___
Detail ___ Ck'd By ___ Date ___

$UL = \frac{13,700(Q)}{L \times NP}$ = liters/meter/minute, L = 10'

RQD Classification

K = 0.0302 g/H

SWL = 20.5'

Boring # and Test Interval	Gage psi	ft x .433 Column psi	Friction Loss psi	Net Pressure psi	Q cfm	L/m/min Lugeons		K	Avg. K	Avg RQD
T-2										
88-98	15	10.91	1.5	24.41	0.50	28.06	44-46 %	.619 x10^-3		
	30		3	37.91	0.77	27.83		.613		
	45		5	50.91	1.11	29.87	Very blocky & seamy	.658	.751 x10^-3	45
	30		5	35.91	1.06	40.44		.891		
	15		3	22.91	0.74	44.25		.975	Ft/min	
98-108	15	10.91	1.5	24.41	0.54	30.31		.668		
	30		3	37.91	0.74	26.74	38-44 %	.590		
	45		4	51.91	0.91	24.02	Very blocky & seamy to	.529	.570 x10^-3	41
	30		3	37.91	0.65	23.49		.518		
	15		1	24.91	0.45	24.75	Completely Crushed	.546	Ft/min	
108-118	15	10.91	0	25.91	0.002	0.11		.002		
	30		0	40.91	0.03	1.0	97-100 %	.022		
	45		0	55.91	0.023	0.56		.012	.010 x10^-3	98
	30		0	40.91	0.016	0.54	Hard & Intact	.012		
	15		0	25.91	0.002	0.11		.002	Ft/min	
118-128	15	10.95	0	25.95	0.067	3.54		.078		
	30		0	40.95	0.096	3.21	97-100 %	.071		
	45		.25	55.70	0.143	3.52		.078	.082 x10^-3	98
	30		0	40.95	0.080	2.68	Hard & Intact	.059		
	15		.25	25.70	0.104	5.54		.022	Ft/min	
128-138	15	10.95	.25	25.70	0.16	8.53		.188		
	30		.5	40.45	0.22	7.45	97 %	.164		
	45		.5	55.45	0.28	6.92		.152	.154 x10^-3	97
	30		0	40.95	0.09	3.01	Hard & Intact	.066		
	15		.25	25.70	0.17	9.06		.200	Ft/min	
138-148	15	10.95	.5	25.45	0.22	11.84		.261		
	30		0	40.95	0.04	1.34	92-97 %	.029		
	45		.25	55.70	0.08	1.97	Hard & Intact-to-	.043	.085 x10^-3	95
	30		0	40.95	0.06	2.01	massive, Moderately	.044		
	15		0	25.95	0.04	2.11	jointed	.047	Ft/min	
148-158	15	10.95	0	25.95	.06	3.17		.070		
	30		.25	40.70	0.09	3.03	87-92 %	.067		
	30		.25	40.70	0.08	2.69		.059	.062 x10^-3	90
	45		.25	55.70	0.11	2.71	Massive,	.060		
	30		.25	40.70	0.18	2.1	moderately jointed	.059	Ft/min	
	15		.25	25.70	0.05	2.[illegible]		.059		

APPENDIX C

ROCK LOAD CALCULATIONS

APPENDIX C

ROCK LOAD CALCULATIONS

Nomenclature as follows:

B = excavated width of tunnel
UW = unit weight of rock

EXCAVATED WIDTHS AND HEIGHTS

Assume that the thickness of the lining will be 12 inches for the cast-in-place options (including embedded steel sets, if any). Therefore:

B = 12'-6" + 12" + 12" = 14'-6"

ROCK LOAD CALCULATIONS

In Granite (UW = 165 PCF)

0.2B (UW) = 0.2 x 14.5 x 165/2000 = 0.239 TSF
0.25B (UW) = 0.25 x 14.5 x 165/2000 = 0.298 TSF
0.3B (UW) = 0.3 x 14.5 x 165/2000 = 0.358 TSF
0.4B (UW) = 0.4 x 14.5 x 165/2000 = 0.479 TSF
0.5B (UW) = 0.5 x 14.5 x 165/2000 = 0.599 TSF
0.6B (UW) = 0.6 x 14.5 x 165/2000 = 0.718 TSF
0.7B (UW) = 0.7 x 14.5 x 165/2000 = 0.838 TSF
0.8B (UW) = 0.8 x 14.5 x 165/2000 = 0.957 TSF
0.9B (UW) = 0.9 x 14.5 x 165/2000 = 1.077 TSF
1.2B (UW) = 1.2 x 14.5 x 165/2000 = 1.436 TSF
1.4B (UW) = 1.4 x 14.5 x 165/2000 = 1.675 TSF
1.8B (UW) = 1.8 x 14.5 x 165/2000 = 2.153 TSF

In Argillite (UW = 175 PCF)

0.2B (UW) = 0.2 x 14.5 x 175/2000 = 0.254 TSF
0.25B (UW) = 0.2 x 14.5 x 175/2000 = 0.317 TSF
0.3B (UW) = 0.3 x 14.5 x 175/2000 = 0.381 TSF
0.4B (UW) = 0.4 x 14.5 x 175/2000 = 0.508 TSF
0.5B (UW) = 0.5 x 14.5 x 175/2000 = 0.634 TSF
0.6B (UW) = 0.6 x 14.5 x 175/2000 = 0.761 TSF
0.7B (UW) = 0.7 x 14.5 x 175/2000 = 0.888 TSF
0.8B (UW) = 0.8 x 14.5 x 175/2000 = 1.015 TSF
0.9B (UW) = 0.9 x 14.5 x 175/2000 = 1.142 TSF
1.2B (UW) = 1.2 x 14.5 x 175/2000 = 1.523 TSF
1.4B (UW) = 1.4 x 14.5 x 175/2000 = 1.776 TSF
1.8B (UW) = 1.8 x 14.5 x 175/2000 = 2.284 TSF

APPENDIX D

DESIGN COMPUTATIONS FOR TEMPORARY SUPPORT OF TUNNELS IN ROCK

CONTENTS

Paragraph	Subject	Page
	Temporary Support by Steel Ribs	D-1
	Temporary Support by Rock Reinforcement	D-3

TABLES

D-1	Calculated Bolt Spacings in Tunnel Above Springline	D-6
D-2	Calculated Bolt Spacings in Tunnel Inverts Below Springline	D-7
D-3	Adopted Rock Bolt Patterns in Tunnel	D-7

APPENDIX D

DESIGN COMPUTATIONS FOR TEMPORARY SUPPORT OF TUNNELS IN ROCK

TEMPORARY SUPPORT BY STEEL RIBS

Method. As stated in the text, temporary support through the use of the ribs will be limited to the zones of "completely crushed rock" when excavated by drill and blast.

The method to be used will be that as presented in Appendix G, pages G-1 and G-5, EM 1110-2-2901 dated 15 Sept. 78 as changed 19 Feb. 82. The thrust in the rib will be determined as if the rock loads act hydrostatically on the ribs (as described in par. G-2,a,(9).

Given Data.

Rock class: completely crushed (RQD = 0 to 40)
Excavated diameter = 14'-6"

Assumed Data.

Rib depth = 6 inches (tentative assumption)
Rib shape = full circular
Overbreak = 12 inches (large due to poor rock)
Maximum longitudinal and blocking spacing = 2 feet
Unit weight of rock = 175 PCF in argillite
Unit weight of rock = 165 PCF in granite

Rock Load. As presented in Section 24, rock load for completely crushed argillite is 2.284 tons per square foot.

Thrust in Rib. The maximum load carried by each rib is the load per unit area times the total width of the tunnel times the rib spacing. The total width is the excavated diameter plus twice the overbreak, or:

$$\text{Diameter} = 14.5 + 1.0 + 1.0 = 16.5 \text{ feet}$$

The load per rib is:

$$W = 2.284 * 16.5 * 2.0 = 75.4 \text{ tons}$$

The thrust in the rib is half the total load, or:

$$T = W/2 = 37.7 \text{ tons}$$

Moment in Rib

Following notation similar to that on page G-4 of EM 1110-2-2901:

@ = central angle between blocking points
R = radius to neutral axis of rib, inches
h = rise of arc between blocking points, inches
M(t) = bending moment in rib, pinned segments, in-kip
M(m) = maximum bending moment, continuous, in-kip
T = thrust
S = section modulus of rib, cubic inches
A = sectional area of rib, square inches
f(r) = stress in rib, psi
f(y) = yield strength of steel, psi
f(all) = allowable fiber stress
= 0.73 f(y) for temporary construction, psi

Calculations:

@ = 2*PI*(max blocking spacing)/perimeter
= 2*PI*(max blocking spacing)/PI*(exc. dia.)
= 2*(max blocking spacing)/(exc. dia.)
= 2*2.0/14.5 = 0.2759 radians
= 15.8 degrees, @/2 = 7.9 degrees
R = (14.5/2)*12 - one-half rib depth
= 87 -6/2 = 84 inches
h = R(1-cos [@/2])
= 84(1-cos [7.9])
= 0.797 inches
T = 37.7 tons = 75,400 pounds (from above)
M(t) = h*T = 0.797 * 75,400 = 60,100 inch-pounds
M(m) = 0.67 * M(t)
= 0.67 * 60,100 = 40,300 inch-pounds
f(y) = 36,000 psi (for A36 steel)
f(all) = 0.73*f(y) = 26,300 psi

Assume W6x20 as trial:

S = 13.4 cubic inches
A = 5.88 sq. in. - 0.29 sq. in. (for 1-inch tie rods)
= 5.59 sq. in.
f(r) = T/A + M(m)/S
= 75,400/5.59 + 40,300/13.4
= 13,490 + 3,010 = 16,500 psi <<< 26,300
"SECTION IS TOO LARGE"

Assume W6x15 as trial:

$S = 9.72$ cubic in.
$A = 4.43 - 0.29 = 4.14$ sq. in.
$f(r) = 75{,}400/4.14 + 40{,}300/9.72$
$= 18{,}210 + 4{,}150 = 22{,}360$ psi $<26{,}300$
"SECTION IS O.K."

Therefore, use W6x15 for ribs.

TEMPORARY SUPPORT BY ROCK REINFORCEMENT

Method. Rock reinforcement will be used in all areas of the tunnel except in rock that is completely crushed and excavated by D&B. Tensioned rock bolts, with continuous rolled-in threadlike deformations, will be installed with polyester or epoxy resin grout.

The method used for design of rock bolts will be as presented in Par. 3-5 of EM 1110-2-2901.

Design of the size and spacing is based on the criteria contained in Tables 3-7 and 3-8 of EM 1110-2-2901.

Diameter. In general, diameter of bolts will be selected toward the small end of the acceptable range, so as to result in smaller spacings, thereby reducing likelihood of fallouts between bolts.

Initially try #6 (3/4-inch) bolts, 60,000 psi yield stress. Since the area of this bar is 0.44 square inches, the yield strength is 26,400 pounds.

Spacing. Since rock loads in Table 3 are generally greater than the 0.2B as specified in Par. I of Table 3-8 of EM 1110-2-2901, use loads in Table 3 for calculation of spacing. Use 100 percent of yield strength, as implied by Par. Ib of Table 3-8 of EM 1110-2-2901.

Rock loads in granite and argillite, for excavation by tunnel boring machine (TBM) and by drill and blast (D&B), for all classes of rock to be encountered are discussed in Section 24.

Table 3-8 of EM 1110-2-2901 suggests (under IIa) that loads for design of rock reinforcement below the spring line should be taken as one-half the rock loads above the spring line. Spacing is given by the formula:

$$S = SQRT(f(y)/W)$$

where S = spacing in feet,
f(y) = yield strength of bolt in pounds, and
W = rock load in pounds per square foot.

Minimum spacing will occur in tunnels driven by TBM in completely crushed argillite. Load = 1.776 TSF (from Appendix C). Therefore, spacing = SQRT(26,400/1.776*2,000) = 2.7 feet.

This appears to be reasonable minimum average spacing.

Spacing in other rock classes, excavation methods, and rock types are similarly calculated, and yield the results in Table D-1 for spacing of rock bolts above the spring line of the tunnel.

Similar calculations taking the rock loads as one-half the above figures yield spacings of rock bolts below the spring line of the tunnel as shown in Table D-2.

Spacings of greater than one-half quadrant, about 5.5 feet in this tunnel, are neither reasonable or practical. Further installation of rock bolts in the bottom two half-quandrants of tunnels is generally unnecessary. Therefore, to achieve more practical and effective rock bolt patterns, the following adjustments were made to the spacings in Tables D-1 and D-2:

1. Calculate the theoretical number of rock bolts indicated;

2. Round to a number convenient for pattern installation; then

3. Distribute these bolts in the crown and ribs according to judgment.

The total number of rock bolts and their longitudinal spacings that result from this procedure are given in Table D-3. Lengths of bolts are established empirically by criteria such as presented in Table 3-7 of EM 1110-2-2901. Based on this guidance, a bolt length of 8 feet is selected. The foregoing bolt numbers, spacings and lengths are depicted on Plate 4-8.

The bolting patterns for the drill and blast scheme create a compression zone that circumferentially ranges in width from about 1.5 feet (RQD 85-100) to about 4.5 feet (RQD 40-75). In the longitudinal direction, the width varies from 1 foot (RQD 95-100, spacing 7 feet) to 5 feet (RQD 40-75, spacing 5 feet). As the narrowest compression zones occur in the most massive rock condition, in the competent granite, additional bolting is not considered necessary.

The bolting patterns in the TBM option result in compression zones that are at least 3 feet wide circumferentially and 1 foot wide longitudinally (for 7-foot spacing). Supplemental bolting to increase the longitudinal width of the compression zone is not considered necessary as the pattern will be used mainly in the massive granite.

TABLE D-1 - CALCULATED BOLT SPACINGS IN TUNNEL ABOVE SPRINGLINE

Rock Class	Excavation by TBM						Excavation by D&B					
	Granite			Argillite			Granite			Argillite		
	Rock ft	Load TSF	S ft	Rock ft	Load TSF	S ft	Rock ft	Load TSF	S ft	Rock ft	Load TSF	S ft
H&I	0	0	--	0	0	--	0.25B	0.298	6.7	0.25B	0.317	6.5
MMJ	0.2B	0.239	7.4	0.2B	0.254	7.2	0.4B	0.479	5.3	0.4B	0.508	5.1
MBS	0.3B	0.358	6.1	0.3B	0.381	5.9	0.8B	0.957	3.7	0.8B	1.015	3.6
VBS	0.8B	0.957	3.7	0.8B	1.015	3.6	1.2B	1.436	3.0	1.2B	1.523	2.9
CC	1.4B	1.675	2.8	1.4B	1.776	2.7	--- STEEL SETS TO BE USED ---					

TABLE D-2 - CALCULATED BOLT SPACINGS IN TUNNEL INVERTS BELOW SPRINGLINE

Rock Class	Excavation by TBM		Excavation by D&B	
	Granite	Argillite	Granite	Argillite
H&I	---None Required---		9.4 ft.	9.1 ft.
MMJ	10.5 ft.	10.2 ft.	7.4 ft.	7.2 ft.
MBS	8.6 ft.	8.3 ft.	5.3 ft.	5.1 ft.
VBS	5.3 ft.	5.1 ft.	4.3 ft.	4.1 ft.
CC	4.0 ft.	3.9 ft.	--Steel sets to be used--	

TABLE D-3 - ADOPTED ROCK BOLT PATTERNS IN TUNNEL

Rock Class	Excavation by TBM		Excavation by D&B	
	Number	Longitudinal Spacing	Number	Longitudinal Spacing
H&I	None Required		7	7 ft.
MMJ	5	7 ft.	7	5 ft.
MBS	7	6 ft.	11	4 ft.
VBS	11	4 ft.	13	3 ft.
CC	15	3 ft.	--Steel sets to be used--	

APPENDIX E

DESIGN COMPUTATIONS FOR PERMANENT CONCRETE LINING OF TUNNEL

Subject	Page
Cast-in-Place Concrete Liner, D&B Excavation	
Computer Model	E-1
Load Cases	E-3
Computer Input	E-9
Computer Output	E-18
Stress Summary - Unreinforced Liner	E-38
Stress Computation - Reinforced Liner	E-44
Precast Liner - TBM Excavation	
Computer Model	E-47
Load Cases	E-49
Computer Input	E-53
Computer Output	E-61
Stress Computation - Design of Reinforcement	E-75

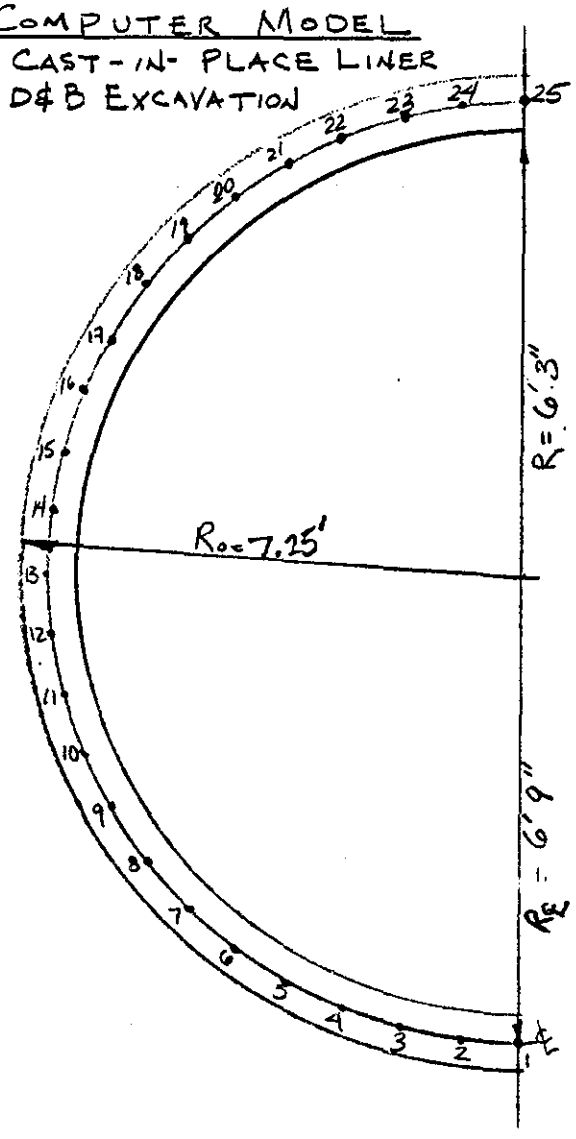
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Town Brook Tunnel Cast-In-Place D=12.5'
COMPUTATION Geometry = X-Y Coordinates.
COMPUTED BY E Nestorides CHECKED BY DATE 2-28-86

COMPUTER MODEL
CAST-IN-PLACE LINER
D&B EXCAVATION

$\phi = 12.5'$ $D = 12''$ 1-foot length



NODE	X-COORD.	Y-COORD.
1	0.000	-6.750
2	-0.881	-6.692
3	-1.747	-6.519
4	-2.583	-6.236
5	-3.375	-5.846
6	-4.109	-5.355
7	-4.773	-4.773
8	-5.355	-4.109
9	-5.846	-3.375
10	-6.236	-2.583
11	-6.519	-1.747
12	-6.692	-0.881
-13-	-6.750	0.000
14	-6.692	0.881
15	-6.519	1.747
16	-6.236	2.583
17	-5.846	3.375
18	-5.355	4.109
19	-4.773	4.773
20	-4.109	5.355
21	-3.375	5.846
22	-2.583	6.236
23	-1.747	6.519
24	-0.881	6.692
25	0.000	6.750

- Continuity release for every member ⇒ code 0

- Support modulus for nodes 1,25 ⇒ 0.00 ft²
  otherwise, SUM = 0.0127 ft² (See p. E-2)

Based on Modulus of Deformation (E_d) for Argillite E_d = 500,000 psi = 72,000,000 psf

WES Computer Program: "TUNNEL" No. 713-F3-R0019

NED FORM 223
27 Sept 49

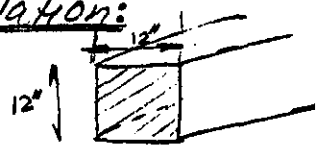
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE E-2

SUBJECT Townbrook Tunnel Cast-in-Place D=12.5'
COMPUTATION Parameter Computations
COMPUTED BY E Nestorides CHECKED BY DATE 2-28-86

Parameter Calculation:

① $I = \frac{bd^3}{12}$



1-ft section

$A_{cs} = 144\ in^2 \Rightarrow \boxed{1 ft^2}$

$I = \frac{12(12)^3}{12} \Rightarrow 1728\ in^4$

$\Rightarrow \boxed{0.0833 ft^4}$

② Radius to ℄ $\Rightarrow$ 6' 3" + 6" $\Rightarrow$ 6' 9" or 6.75'

$S = r\sin\theta \Rightarrow 6.75 \sin(7.5) \Rightarrow \underline{.881\ ft}$

$A_{surface} \Rightarrow$ $\boxed{A_s = .881\ ft^2}$

③ Young's Modulus

$E_c = w^{1.5}(33)\sqrt{f_c'} = (150)^{1.5}(33)\sqrt{4000}$

$E_c = 3{,}834{,}253.513\ psi \cdot \frac{144\ in^2}{ft^2}$

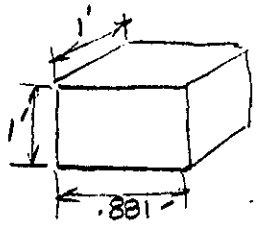
$\boxed{E_c = 552{,}132{,}506\ psf}$

④ $R = \frac{E_d}{R_2(1+V_r)} \Rightarrow \frac{72{,}000{,}000}{(7.25)(1.25)} = 7{,}944{,}827.6\ lb/ft^3$

$SMM = \frac{R}{E_c} \cdot A_s \Rightarrow \frac{7944828}{552132506} \times (.881) \Rightarrow 0.0127 \times 1ft$ length

$= \boxed{0.0127\ ft^2}$

⑤



$V = .881\ A^3 \quad \gamma = 150\ lb/ft^3$

$W_c \Rightarrow \boxed{132.15\ lb}$

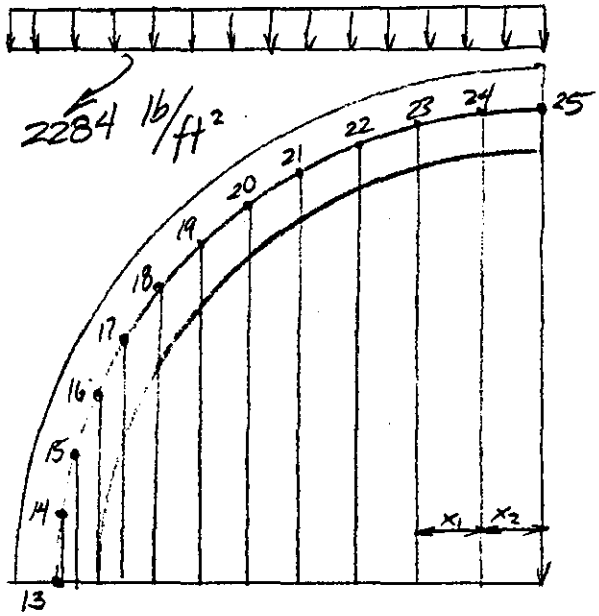
at nodes 1, 25 $\Rightarrow W_c/2 \Rightarrow \boxed{66.08\ lb}$

DEAD LOAD

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: TOWNBROOK TUNNEL Cast-in-Place Liner
COMPUTATION: Dead + Rock Load LOAD CASE #1
COMPUTED BY: ENestorides CHECKED BY: DATE: 4-7-86



DEAD LOAD:
nodes 1,2 ⇒ 66.08 lb.
others ⇒ 132.15 lb.

ROCK LOAD:

for example node 24: 2284 psf $\left(\frac{x_1+x_2}{2}\right)$

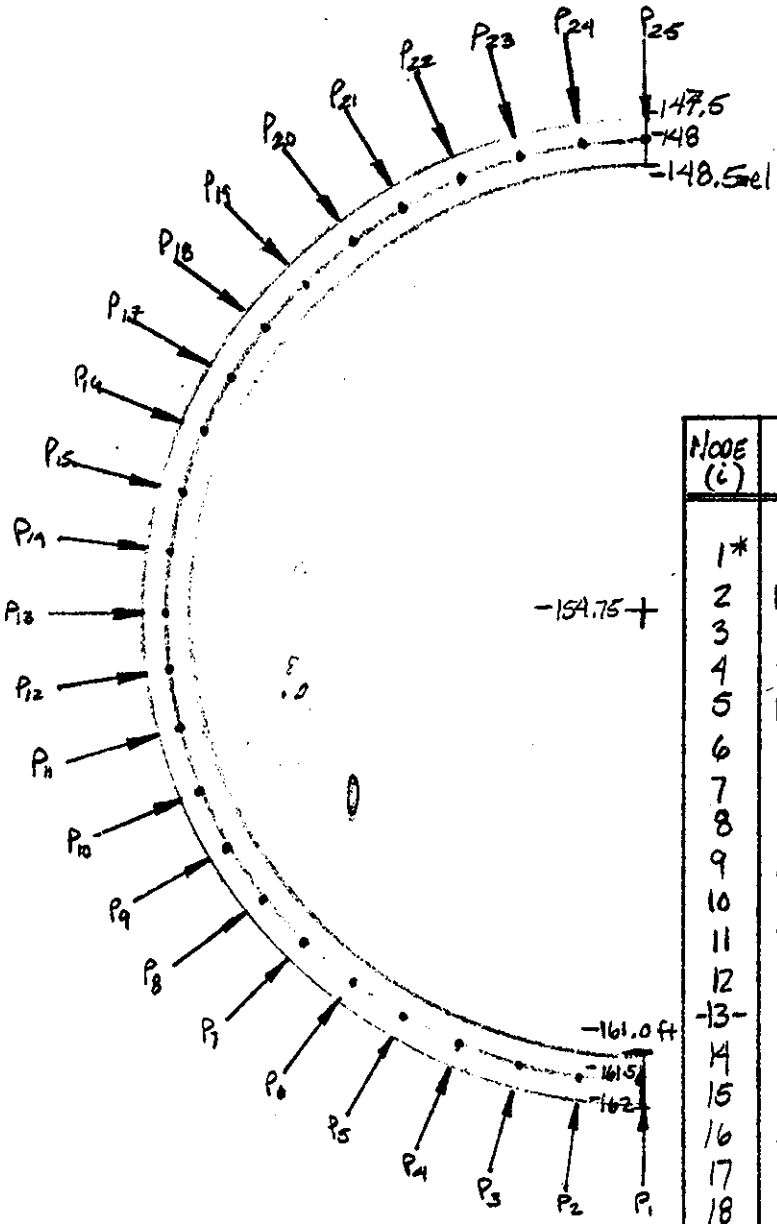
LOAD Case 1:

NODE	Fx	M	F_{YR} ROCK LOAD x_1+x_2	2284 $\left(\frac{x_1+x_2}{2}\right)$	F_{YD} DEAD LOAD	F_Y TOTAL
1	0	0		0	-66.08	-66.08
2					-132.15	-132.15
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						-132.15
13			.030	-68.52		-200.67
14			.115	-262.66		-394.81
15			.225	-513.90		-646.05
16			.335	-765.14		-897.29
17			.440	-1004.96		-1005.96 1136.11
18			.540	-1233.36		-1365.51
19			.625	-1427.50		-1559.65
20			.695	-1587.38		-1719.53
21			.765	-1747.26		-1879.41
22			.815	-1861.46		-1993.61
23			.850	-1941.40		-2073.55
24			.875	-1998.50	-132.15	-2130.65
25			.440	-1004.96	-66.08	-1071.04

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Town Brook Tunnel Cast-in-Place Dia = 12.5'
COMPUTATION: Dead + External Water Pressure LC #2
COMPUTED BY: E Nestorides CHECKED BY: DATE: 2-28-86



① DEAD LOAD: nodes 1,25 : 66.08 lb
all others : 132.15 lb

② EXTERNAL WATER PRESSURE: [head ▽ +11]

$A_{contact} = .881\ ft^2$ $\gamma_w = 62.5\ pcf$

$W_w = 55.0625\ lb/ft$

* nodes 1, 25 ⇒ $P_i/2$

NODE (i)	h_i (ft)	P_i (lb)	F_x (lb)	F_y (lb)	F_{TOTAL} (lb)
1*	172.50	4749.14	0.00	4749.14	4683.06
2	172.44	9494.98	1239.34	9413.75	9281.60
3	172.27	9485.62	2455.06	9162.45	9030.26
4	171.99	9470.20	3624.09	8749.32	8617.17
5	171.60	9448.73	4724.37	8182.84	8050.69
6	171.11	9421.74	5735.59	7474.77	7342.62
7	170.52	9389.26	6639.21	6639.21	6507.06
8	169.86	9352.92	7420.17	5693.70	5561.55
9	169.13	9312.72	8065.05	4656.36	4524.31
10	168.33	9268.67	8563.13	3546.97	3414.82
11	167.50	9222.97	8908.70	2387.08	2254.93
12	166.63	9175.06	9096.57	1197.59	1065.44
13	165.75	9126.61	9126.61	0.00	-132.15
14	164.87	9078.15	9000.49	-1184.94	-1317.09
15	164.00	9030.25	8722.55	-2337.20	-2469.35
16	163.17	8984.55	8300.64	-3438.24	-3570.39
17	162.37	8940.50	7742.70	-4470.25	-4602.40
18	161.64	8900.30	7061.08	-5418.16	-5550.31
19	160.98	8863.96	6267.77	-6267.77	-6399.92
20	160.39	8831.47	5376.26	-7006.48	-7138.63
21	159.90	8804.49	4402.25	-7624.9	-7757.06
22	159.51	8783.02	3361.12	-8114.45	-8246.60
23	159.23	8767.60	2269.22	-8468.85	-8601.00
24	159.06	8758.24	1143.18	-8683.31	-8815.46
25*	159.00	4377.47	0.00	-4377.47	-4443.55

LOAD CASE 2

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Town Brook Tunnel Cast-in-Place Dia = 12.5'
COMPUTATION Dead, Rock, and Ex. Water Load. LC #3
COMPUTED BY E Nestorides CHECKED BY DATE 4-7-86

LOAD CASE #3: Refer to LC #1 and LC #2 write-ups
Dead Load + Rock Load + Ex. Water Pressure.

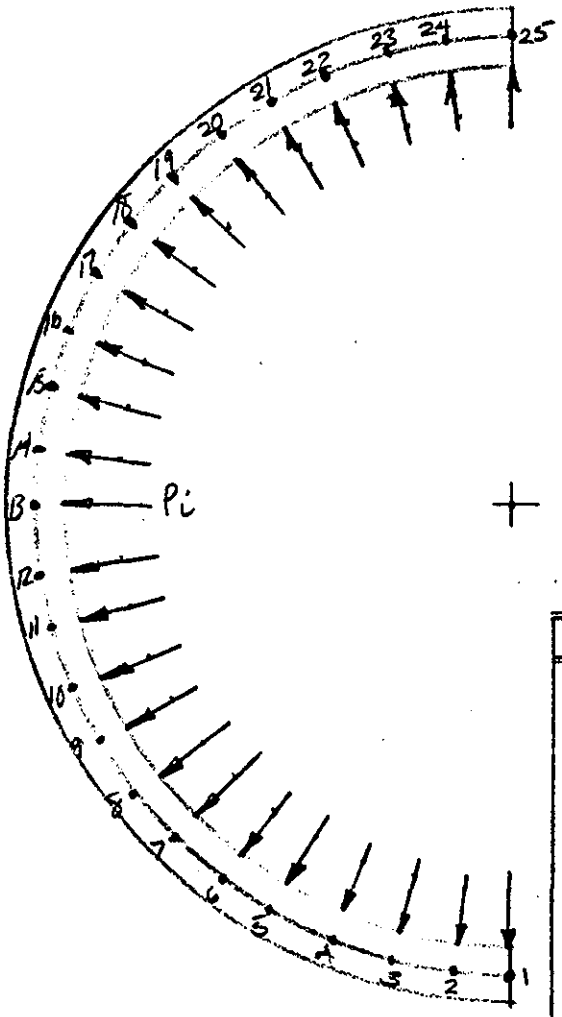
NODE (i)	Fx (lbs)	Fy (lbs)
1	0.00	4683.06
2	1239.34	9281.60
3	2455.06	9030.26
4	3624.09	8617.17
5	4724.37	8050.69
6	5735.59	7342.62
7	6639.21	6507.06
8	7420.17	5561.55
9	8065.05	4524.21
10	8563.13	3414.82
11	8908.70	2254.93
12	9096.57	1065.44
13	9126.61	-200.67
14	9000.49	-1579.75
15	8722.55	-2983.25
16	8300.64	-4335.53
17	7742.70	-5476.21
18	7061.80	-6783.67
19	6267.77	-7827.42
20	5376.26	-8726.01
21	4402.25	-9511.16
22	3361.12	-10108.06
23	2269.22	-10542.40
24	1143.18	-10813.96
25	0.00	-5448.51

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE E-6

SUBJECT Townbrook Tunnel → Cast-in-Place Dia = 12.5'
COMPUTATION Dead Load and Internal water Pressure L.C. # 4
COMPUTED BY E Nestorides CHECKED BY DATE 3-3-86



INTERNAL HEAD = 20ft

$A_s = .881\ ft^2$ $\gamma_w = 62.5\ lb/ft^3$

$P_i = 1101.25$ lb

except: $P_1, P_{25} = P_i/2 \Rightarrow 550.625$ lb.

DEAD LOAD:

nodes 1,25 ⇒ 66.08 lb

other nodes ⇒ 132.15 lb

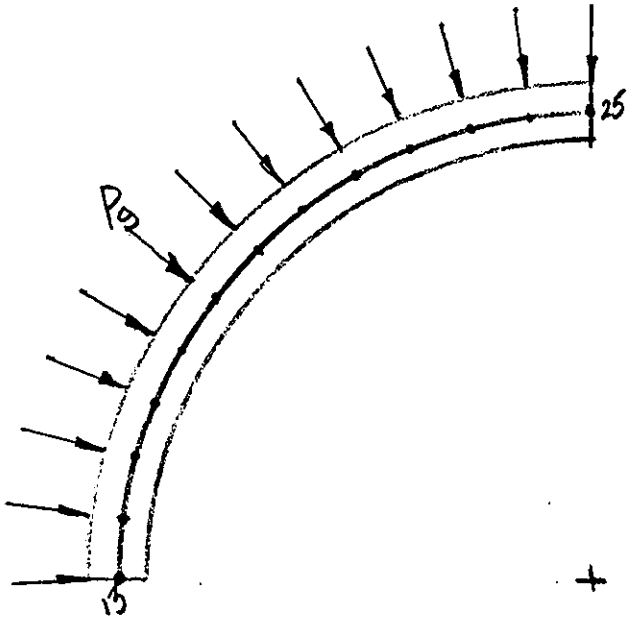
NODE	Px (lb)	Py (lb)	Py TOTAL (lb)
1	0.00	-550.63	-616.71
2	-143.74	-1091.83	-1223.98
3	-285.02	-1063.73	-1195.88
4	-421.43	-1017.42	-1149.57
5	-550.63	-953.71	-1085.86
6	-670.40	-873.68	-1005.83
7	-778.70	-778.70	-910.85
8	-873.68	-670.40	-802.55
9	-953.71	-550.63	-682.78
10	-1017.42	-421.43	-553.58
11	-1063.73	-285.02	-417.17
12	-1091.83	-143.74	-275.89
-13-	-1101.25	0.00	-132.15
14	-1091.83	143.74	11.59
15	-1063.73	285.02	152.87
16	-1017.42	421.43	289.29
17	-953.71	550.63	418.48
18	-873.68	670.40	539.25
19	-778.70	778.70	646.55
20	-670.40	873.68	741.53
21	-550.63	953.71	821.56
22	-421.43	1017.42	885.27
23	-285.02	1063.73	931.58
24	-143.74	1091.83	959.68
25	0.00	550.63	484.55

LOAD Case 4

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5'
COMPUTATION Dead Load + Grout pressure L.C. #5
COMPUTED BY E. Nestorides CHECKED BY DATE 3-3-86



LOAD Case 5

① Dead Load:

nodes 1, 25 ⇒ 66.08 lb
other nodes ⇒ 132.15 lb

② Grout Pressure: 15 psi $A_G = .881\ ft^2$

$$P_g = 15\ lb/in^2 \times 144\ in^2/ft^2 \times .881\ ft^2$$

$$P_g = 1902.96\ lb$$

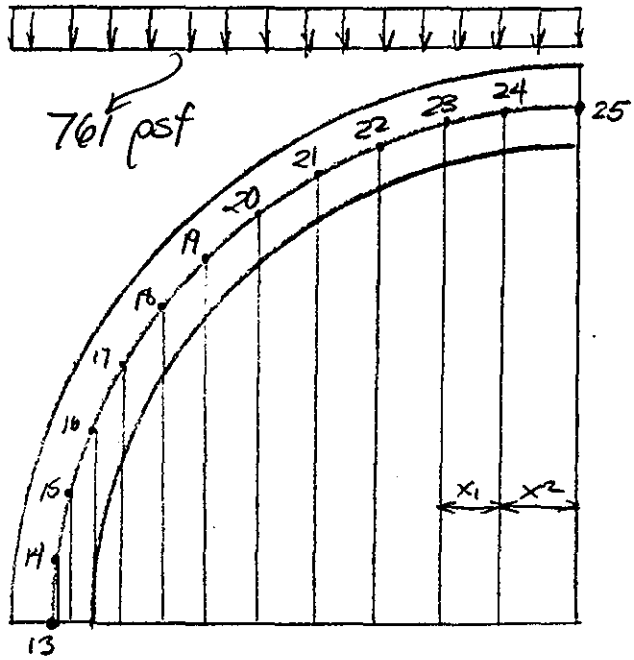
except node 13, 25 ⇒ $P_g/2 = 951.48$

Node (i)	P_{gx}	P_{gy}	P_{TOTAL}
1	0.00	0.00	-66.08
2			-132.15
3			
4			
5			
6			
7			
8			
9			
10			
11			
12	0.00		
-13-	951.48	0.00	-132.15
14	1886.68	-248.39	-380.54
15	1838.12	-492.52	-624.67
16	1758.11	-728.23	-860.38
17	1648.01	-951.48	-1083.63
18	1509.72	-1158.45	-1290.60
19	1345.60	-1345.60	-1477.75
20	1158.45	-1509.72	-1641.87
21	951.48	-1648.01	-1780.16
22	728.23	-1758.11	-1890.26
23	492.52	-1838.12	-1970.27
24	248.39	-1886.68	-2018.93
25	0.00	-951.48	-1083.63

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5 ft
COMPUTATION Reduced Rock Load + Dead Load (761 psf) LC #1'
COMPUTED BY E Nestorides CHECKED BY DATE 4-7-86



DEAD LOAD:

nodes 1, 25 : 66.08 lb
others : 132.15 lb.

ROCK LOAD:

example node 24: $761\left(\frac{x_1 + x_2}{2}\right)$

LOAD CASE #1'

NODE (i)	F_{YR} ROCK LOAD $\frac{x_1+x_2}{2}$	$761\left(\frac{x_1+x_2}{2}\right)$	F_{YD} DEAD LOAD	F_{YT} TOTAL
1		0	-66.08	-66.08
2			-132.15	-132.15
⋮				
12		0		-132.15
13	.030	-22.83		-154.98
14	.115	-87.52		-219.67
15	.225	-171.23		-303.38
16	.335	-254.94		-387.09
17	.440	-334.84		-466.99
18	.54	-410.94		-543.09
19	.625	-475.63		-607.78
20	.695	-528.90		-661.05
21	.765	-582.17		-714.32
22	.815	-620.22		-752.37
23	.850	-646.85		-779.00
24	.875	-665.88	-132.15	-798.03
25	.440	-334.84	-66.08	-400.92

```
1000 "TOWNBROOK TUNNEL:CAST-IN-PLACE(12.5 FT)"
1100 25,552132506.5
1110 0.000,-6.750,0.0000
1120 -0.881,-6.692,0.0127
1130 -1.747,-6.519,0.0127
1140 -2.583,-6.236,0.0127
1150 -3.375,-5.846,0.0127
1160 -4.109,-5.355,0.0127
1170 -4.773,-4.773,0.0127
1180 -5.355,-4.109,0.0127
1190 -5.846,-3.375,0.0127
1200 -6.236,-2.583,0.0127
1210 -6.519,-1.747,0.0127
1220 -6.692,-0.881,0.0127
1230 -6.750,0.000,0.0127
1240 -6.692,0.881,0.0127
1250 -6.519,1.747,0.0127
1260 -6.236,2.583,0.0127
1270 -5.846,3.375,0.0127
1280 -5.355,4.109,0.0127
1290 -4.773,4.773,0.0127
1300 -4.109,5.355,0.0127
1310 -3.375,5.846,0.0127
1320 -2.583,6.236,0.0127
1330 -1.747,6.519,0.0127
1340 -0.881,6.692,0.0127
1350 0.000,6.750,0.0000
1400 0.0833,1.0,0
1410 0.0833,1.0,0
1420 0.0833,1.0,0
1430 0.0833,1.0,0
1440 0.0833,1.0,0
1450 0.0833,1.0,0
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1460 0.0833,1.0,0
1470 0.0833,1.0,0
1480 0.0833,1.0,0
1490 0.0833,1.0,0
1500 0.0833,1.0,0
1510 0.0833,1.0,0
1520 0.0833,1.0,0
1530 0.0833,1.0,0
1540 0.0833,1.0,0
1550 0.0833,1.0,0
1560 0.0833,1.0,0
1570 0.0833,1.0,0
1580 0.0833,1.0,0
1590 0.0833,1.0,0
1600 0.0833,1.0,0
1610 0.0833,1.0,0
1620 0.0833,1.0,0
1630 0.0833,1.0,0
1700 2,2
1710 3,1
1800 "L.C.#1:DEAD LOAD & ROCK LOAD (2284 PSF)"
1805 25
1810 1,0.00,0,-66.08
1820 2,0.00,0,-132.15
1830 3,0.00,0,-132.15
1840 4,0.00,0,-132.15
1850 5,0.00,0,-132.15
1860 6,0.00,0,-132.15
1870 7,0.00,0,-132.15
1880 8,0.00,0,-132.15
1890 9,0.00,0,-132.15
1900 10,0.00,0,-132.15
1910 11,0.00,0,-132.15
1920 12,0.00,0,-132.15
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E-10

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1930 13,0.00,0,-200.67
1940 14,0.00,0,-394.81
1950 15,0.00,0,-646.05
1960 16,0.00,0,-897.29
1970 17,0.00,0,-1136.11
1980 18,0.00,0,-1365.51
1990 19,0.00,0,-1559.65
2000 20,0.00,0,-1719.53
2010 21,0.00,0,-1879.41
2020 22,0.00,0,-1993.61
2030 23,0.00,0,-2073.55
2040 24,0.00,0,-2130.65
2050 25,0.00,0,-1071.04
2100 "L.C.#2:DEAD LOAD & EXTERNAL WATER PRESSURE"
2105 25
2110 1,0.00,0,4683.00
2120 2,1239.34,0,9281.60
2130 3,2455.06,0,9030.26
2140 4,3624.09,0,8617.17
2150 5,4724.37,0,8050.69
2160 6,5735.59,0,7342.62
2170 7,6639.21,0,6507.06
2180 8,7420.17,0,5561.55
2190 9,8065.05,0,4524.21
2200 10,8563.13,0,3414.82
2210 11,8908.70,0,2254.93
2220 12,9096.57,0,1065.44
2230 13,9126.61,0,-132.15
2240 14,9000.49,0,-1317.09
2250 15,8722.55,0,-2469.35
2260 16,8300.64,0,-3570.39
2270 17,7742.70,0,-4602.40
2280 18,7061.08,0,-5550.31
2290 19,6267.77,0,-6399.92
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2300 20,5376.26,0,-7138.63
2310 21,4402.25,0,-7757.06
2320 22,3361.12,0,-8246.60
2330 23,2269.22,0,-8601.00
2340 24,1143.18,0,-8815.46
2350 25,0.00,0,-4443.55
2400 "L.C.#3:DEAD,ROCK, & EX. WATER LOAD"
2405 25
2410 1,0.00,0,4683.06
2420 2,1239.34,0,9281.60
2430 3,2455.06,0,9030.26
2440 4,3624.09,0,8617.17
2450 5,4724.37,0,8050.69
2460 6,5735.59,0,7342.62
2470 7,6639.21,0,6507.06
2480 8,7420.17,0,5561.55
2490 9,8065.05,0,4524.21
2500 10,8563.13,0,3414.82
2510 11,8908.70,0,2254.93
2520 12,9096.57,0,1065.44
2530 13,9126.61,0,-200.67
2540 14,9000.49,0,-1579.75
2550 15,8722.55,0,-2983.25
2560 16,8300.64,0,-4335.53
2570 17,7742.70,0,-5476.21
2580 18,7061.80,0,-6783.67
2590 19,6267.77,0,-7827.42
2600 20,5376.26,0,-8726.01
2610 21,4402.25,0,-9511.16
2620 22,3361.12,0,-10108.06
2630 23,2269.22,0,-10542.40
2640 24,1143.18,0,-10813.96
2650 25,0.00,0,-5448.51
2700 "L.C.#4: DEAD LOAD & INTERNAL WATER PRESSURE"
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2705 25
2710 1,0.00,0,-616.71
2720 2,-143.74,0,-1223.98
2730 3,-285.02,0,-1195.88
2740 4,-421.43,0,-1149.57
2750 5,-550.63,0,-1085.86
2760 6,-670.40,0,-1005.83
2770 7,-778.70,0,-910.85
2780 8,-873.68,0,-802.55
2790 9,-953.71,0,-682.78
2800 10,-1017.42,0,-553.58
2810 11,-1063.73,0,-417.17
2820 12,-1091.83,0,-275.89
2830 13,-1101.25,0,-132.15
2840 14,-1091.83,0,11.59
2850 15,-1063.73,0,152.87
2860 16,-1017.42,0,289.28
2870 17,-953.71,0,418.48
2880 18,-873.68,0,538.25
2890 19,-778.70,0,646.55
2900 20,-670.40,0,741.53
2910 21,-550.63,0,821.56
2920 22,-421.43,0,885.27
2930 23,-285.02,0,931.58
2940 24,-143.74,0,959.68
2950 25,0.00,0,484.55
3000 "L.C.#5:DEAD LOAD & GROUT PRESSURE"
3005 25
3010 1,0.00,0,-66.08
3020 2,0.00,0,-132.15
3030 3,0.00,0,-132.15
3040 4,0.00,0,-132.15
3050 5,0.00,0,-132.15
3060 6,0.00,0,-132.15
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3070 7,0.00,0,-132.15
3080 8,0.00,0,-132.15
3090 9,0.00,0,-132.15
3100 10,0.00,0,-132.15
3110 11,0.00,0,-132.15
3120 12,0.00,0,-132.15
3130 13,951.48,0,-132.15
3140 14,1886.68,0,-380.54
3150 15,1838.12,0,-624.67
3160 16,1758.11,0,-860.38
3170 17,1648.01,0,-1083.63
3180 18,1509.72,0,-1290.60
3190 19,1345.60,0,-1477.75
3200 20,1158.45,0,-1641.87
3210 21,951.48,0,-1780.16
3220 22,728.23,0,-1890.26
3230 23,492.52,0,-1970.27
3240 24,248.39,0,-2018.83
3250 25,0.00,0,-1083.63
/
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E-14

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1000 "TOWNBROOK TUNNEL:CAST-IN-PLACE (12.5 FT)"
1100 25,552132506,1
1110 0.000,-6.750,0.0000
1120 -0.881,-6.692,0.0127
1130 -1.747,-6.519,0.0127
1140 -2.583,-6.236,0.0127
1150 -3.375,-5.846,0.0127
1160 -4.109,-5.355,0.0127
1170 -4.773,-4.773,0.0127
1180 -5.355,-4.109,0.0127
1190 -5.846,-3.375,0.0127
1200 -6.236,-2.583,0.0127
1210 -6.519,-1.747,0.0127
1220 -6.692,-0.881,0.0127
1230 -6.750,0.000,0.0127
1240 -6.692,0.881,0.0127
1250 -6.519,1.747,0.0127
1260 -6.236,2.583,0.0127
1270 -5.846,3.375,0.0127
1280 -5.355,4.109,0.0127
1290 -4.773,4.773,0.0127
1300 -4.109,5.355,0.0127
1310 -3.375,5.846,0.0127
1320 -2.583,6.236,0.0127
1330 -1.747,6.519,0.0127
1340 -0.881,6.692,0.0127
1350 0.000,6.750,0.0000
1400 0.0833,1.0,0
1410 0.0833,1.0,0
1420 0.0833,1.0,0
1430 0.0833,1.0,0
1440 0.0833,1.0,0
1450 0.0833,1.0,0
```

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1460 0.0833,1.0,0
1470 0.0833,1.0,0
1480 0.0833,1.0,0
1490 0.0833,1.0,0
1500 0.0833,1.0,0
1510 0.0833,1.0,0
1520 0.0833,1.0,0
1530 0.0833,1.0,0
1540 0.0833,1.0,0
1550 0.0833,1.0,0
1560 0.0833,1.0,0
1570 0.0833,1.0,0
1580 0.0833,1.0,0
1590 0.0833,1.0,0
1600 0.0833,1.0,0
1610 0.0833,1.0,0
1620 0.0833,1.0,0
1630 0.0833,1.0,0
1700 2,2
1710 3,1
1800 "L.C.#1':DEAD LOAD & ROCK LOAD(761 LB/FT)"
1805 25
1810 1,0,0,-66.08
1820 2,0,0,-132.15
1830 3,0,0,-132.15
1840 4,0,0,-132.15
1850 5,0,0,-132.15
1860 6,0,0,-132.15
1870 7,0,0,-132.15
1880 8,0,0,-132.15
1890 9,0,0,-132.15
1900 10,0,0,-132.15
1910 11,0,0,-132.15
1920 12,0,0,-132.15
```

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1930 13,0,0,-154.98
1940 14,0,0,-219.67
1950 15,0,0,-303.38
1960 16,0,0,-387.09
1970 17,0,0,-466.99
1980 18,0,0,-543.09
1990 19,0,0,-607.78
2000 20,0,0,-661.05
2010 21,0,0,-714.32
2020 22,0,0,-752.37
2030 23,0,0,-779.00
2040 24,0,0,-798.03
2050 25,0,0,-400.92
/
```

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"TOWNBROOK TUNNEL:CAST-IN-PLACE(12.5 FT)"

"L.C.#1:DEAD LOAD & ROCK LOAD (2284 PSF)"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00048	-.00048
2	.00003	-.00001	-.00048	-.00047
3	.00006	-.00000	-.00048	-.00044
4	.00009	.00001	-.00049	-.00042
5	.00011	.00003	-.00052	-.00039
6	.00012	.00005	-.00057	-.00038
7	.00011	.00007	-.00063	-.00037
8	.00007	.00009	-.00070	-.00037
9	.00002	.00011	-.00077	-.00037
10	-.00006	.00012	-.00084	-.00038
11	-.00016	.00014	-.00091	-.00039
12	-.00028	.00015	-.00096	-.00040
13	-.00040	.00014	-.00100	-.00040
14	-.00052	.00012	-.00102	-.00038
15	-.00062	.00008	-.00103	-.00033
16	-.00067	.00002	-.00105	-.00022
17	-.00066	-.00007	-.00108	-.00003
18	-.00059	-.00017	-.00116	.00024
19	-.00046	-.00026	-.00131	.00060

20	-.00031	-.00031	-.00152	.00102
21	-.00017	-.00032	-.00176	.00144
22	-.00006	-.00029	-.00202	.00184
23	-.00000	-.00022	-.00224	.00216
24	.00001	-.00012	-.00239	.00237
25	0.00000	0.00000	-.00244	.00244

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	1366.10174	12.82120	19744.70907
2	784.24324	705.39502	19585.85662
3	322.75503	990.25902	19479.89390
4	71.05194	1052.98480	19403.46067
5	-23.85559	1031.91831	19334.15287
6	-65.47895	974.10296	19259.32873
7	-43.35294	935.82400	19172.84472
8	-55.13581	887.13446	19074.21917
9	-79.36114	817.07318	18968.38622
10	-199.42457	641.06081	18864.28440
11	-463.95304	231.33879	18779.34666
12	-789.16270	-465.41859	18730.50189
13	-1154.28164	-1484.54208	18657.34275
14	-1468.66933	-2781.54009	18437.73412
15	-1611.64539	-4203.98035	18011.66717
16	-1198.09177	-5261.67488	17365.18927
17	225.82422	-5062.25317	16443.39377
18	1457.25390	-3775.55491	15248.22618
19	2270.13985	-1771.11012	13897.69945
20	2624.83975	546.84334	12529.62900
21	2565.09582	2811.35084	11244.87391
22	2125.37217	4687.20702	10173.57733

23	1385.39903	5910.66811	9408.62444
24	480.45874	6334.86856	9005.94312

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-66.08000
2	3163.43054
3	2983.09788
4	2799.31058
5	2649.61002
6	2550.45208
7	2501.70652
8	2498.15368
9	2533.24479
10	2599.10063
11	2694.71640
12	2787.77081
13	2820.41529
14	2749.73027
15	2496.31735
16	1905.68580
17	808.99386
18	831.41466
19	1102.83909
20	1364.05490
21	1627.65251
22	1842.40321
23	2002.89002
24	2112.26895
25	1071.04000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-19791.79945	1193.31976	66.08000
LAST	-8954.95955	6334.86856	1071.04000

"L.C.#2:DEAD LOAD & EXTERNAL WATER PRESSURE"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	.00247	.00247
2	.00011	.00002	.00245	.00244
3	.00022	.00004	.00240	.00238
4	.00031	.00005	.00233	.00227
5	.00039	.00007	.00223	.00213
6	.00045	.00007	.00212	.00196
7	.00049	.00008	.00199	.00176
8	.00052	.00008	.00186	.00154
9	.00052	.00007	.00173	.00132
10	.00052	.00006	.00160	.00109
11	.00051	.00005	.00148	.00088
12	.00050	.00003	.00136	.00067
13	.00049	.00001	.00125	.00049
14	.00048	-.00001	.00114	.00033
15	.00048	-.00004	.00102	.00020
16	.00049	-.00006	.00090	.00011
17	.00049	-.00007	.00078	.00004

18	.00049	-.00008	.00064	-.00001
19	.00047	-.00009	.00051	-.00003
20	.00043	-.00008	.00038	-.00004
21	.00038	-.00007	.00026	-.00004
22	.00031	-.00005	.00016	-.00004
23	.00021	-.00004	.00009	-.00003
24	.00011	-.00002	.00005	-.00003
25	0.00000	0.00000	.00003	-.00003

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	-11.65775	976.00347	71110.06228
2	-36.70461	943.58923	71101.18714
3	-213.86392	754.83266	71083.04726
4	-247.18396	536.61472	71055.98608
5	-210.89127	350.38006	71019.77873
6	-297.16985	87.99069	70973.46707
7	-275.16708	-154.97109	70917.48661
8	-331.56172	-447.76788	70850.75138
9	-237.25578	-657.22107	70773.91443
10	-185.01782	-820.51803	70685.86261
11	-251.62830	-1042.73374	70586.28184
12	-145.61552	-1171.29873	70476.01644
13	-20.50555	-1189.40322	70354.51683
14	115.93733	-1087.01769	70222.00533
15	111.08112	-988.97735	70079.32476
16	251.38992	-767.04632	69926.72990
17	458.52157	-362.13335	69764.85854
18	485.49320	66.53818	69599.45540
19	451.57223	465.25881	69444.14027
20	270.84097	704.43408	69315.30260

21	220.28970	898.90934	69216.87325
22	132.17328	1015.56564	69149.38875
23	-64.31147	958.77148	69110.80142
24	-95.92029	874.08277	69099.09925

BLOCKING POINT THRUSTS

JOINT	THRUST
1	4683.00000
2	9363.96901
3	9357.97799
4	9348.06744
5	9334.28454
6	9316.90874
7	9295.81666
8	9272.45968
9	9246.64752
10	9218.18093
11	9188.76093
12	9157.74208
13	9126.61000
14	9095.47488
15	9064.45243
16	9035.03044
17	9006.57054
18	9032.99446
19	9164.63784
20	9200.93520
21	9187.22603
22	9155.50522
23	9123.87656
24	9107.74953

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25	4443.55000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-70955.83503	-986.29618	-4683.00000
LAST	-68956.27564	874.08277	4443.55000

"L.C.#3:DEAD,ROCK, & EX. WATER LOAD"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00032	-.00032
2	.00014	.00001	-.00033	-.00031
3	.00027	.00003	-.00037	-.00029
4	.00038	.00006	-.00046	-.00027
5	.00047	.00009	-.00058	-.00026
6	.00053	.00013	-.00073	-.00026
7	.00055	.00016	-.00092	-.00026
8	.00052	.00020	-.00113	-.00028
9	.00043	.00024	-.00135	-.00030
10	.00029	.00027	-.00157	-.00033
11	.00010	.00029	-.00178	-.00036
12	-.00012	.00029	-.00196	-.00038
13	-.00035	.00026	-.00211	-.00035
14	-.00056	.00018	-.00223	-.00026
15	-.00069	.00006	-.00234	-.00006

16	-.00072	-.00009	-.00248	.00028
17	-.00064	-.00025	-.00266	.00077
18	-.00048	-.00039	-.00293	.00141
19	-.00027	-.00049	-.00329	.00214
20	-.00006	-.00054	-.00372	.00292
21	.00010	-.00053	-.00419	.00368
22	.00019	-.00046	-.00464	.00436
23	.00019	-.00034	-.00501	.00489
24	.00011	-.00018	-.00526	.00523
25	0.00000	0.00000	-.00535	.00535

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	925.21727	916.89153	85341.78417
2	608.38188	1454.16024	85228.81368
3	213.98770	1643.02606	85141.20852
4	93.55840	1725.62090	85063.68585
5	128.97176	1839.51378	84982.51028
6	38.40002	1873.41949	84892.18821
7	35.70425	1904.94495	84793.66506
8	-168.71113	1755.95889	84696.07731
9	-360.19449	1437.97345	84616.58104
10	-803.53427	728.77309	84577.20529
11	-1596.64392	-681.24069	84604.72711
12	-2282.46901	-2696.44885	84723.58965
13	-2758.85075	-5132.25785	84854.37691
14	-2581.09932	-7411.65500	84818.63458
15	-1284.28198	-8545.16390	84443.32631
16	408.58714	-8184.45666	83663.41882
17	1936.54786	-6474.32262	82635.05701
18	2859.45822	-3949.53274	81238.68194

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19	3463.70430	-891.21717	79717.32613
20	3535.05831	2230.53556	78211.05716
21	3321.02821	5162.39223	76812.99742
22	2686.38298	7533.39725	75661.47035
23	1632.21570	8974.82485	74844.31790
24	561.23411	9470.34245	74415.19096

BLOCKING POINT THRUSTS

JOINT	THRUST
1	4683.06000
2	11517.47885
3	11394.78553
4	11274.23391
5	11179.93045
6	11133.73296
7	11145.71239
8	11222.30939
9	11357.95503
10	11530.19053
11	11715.94798
12	11804.09288
13	11615.28097
14	10965.01251
15	9635.56429
16	9327.32881
17	9443.44679
18	9732.28396
19	9966.80443
20	10195.52885
21	10438.06701
22	10625.42015

23	10770.48831
24	10870.50564
25	5448.51000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-85218.36085	-100.01060	-4683.06000
LAST	-74217.74458	9470.34245	5448.51000

"L.C.#4: DEAD LOAD & INTERNAL WATER PRESSURE"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00012	-.00012
2	-.00001	-.00001	-.00012	-.00012
3	-.00001	-.00001	-.00011	-.00011
4	-.00002	-.00001	-.00011	-.00010
5	-.00002	-.00000	-.00010	-.00010
6	-.00003	-.00000	-.00010	-.00009
7	-.00003	.00000	-.00009	-.00009
8	-.00004	.00000	-.00009	-.00008
9	-.00004	.00000	-.00008	-.00008
10	-.00005	.00000	-.00008	-.00007
11	-.00005	.00000	-.00007	-.00007
12	-.00006	.00000	-.00006	-.00006
13	-.00006	.00000	-.00005	-.00006

14	-.00006	.00000	-.00004	-.00006
15	-.00006	.00000	-.00004	-.00005
16	-.00006	.00000	-.00003	-.00005
17	-.00006	.00000	-.00002	-.00004
18	-.00005	.00000	-.00001	-.00004
19	-.00005	.00000	-.00000	-.00003
20	-.00004	.00000	.00001	-.00003
21	-.00004	.00000	.00001	-.00003
22	-.00003	.00000	.00002	-.00003
23	-.00002	.00000	.00002	-.00003
24	-.00001	.00000	.00003	-.00003
25	0.00000	0.00000	.00003	-.00003

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	345.77329	-132.54720	-4135.71360
2	189.80440	35.07114	-4188.93057
3	83.97613	109.18858	-4240.93047
4	13.38199	121.00241	-4297.12714
5	-25.69948	98.30762	-4362.48664
6	-33.00171	69.16839	-4439.43845
7	-33.14699	39.90089	-4528.73146
8	-21.86098	20.59582	-4630.05758
9	-18.60299	4.17280	-4742.12076
10	-12.93756	-7.24590	-4863.22815
11	-.28222	-7.49513	-4990.28190
12	-.52425	-7.95800	-5120.90892
13	-2.14659	-9.85324	-5253.16916
14	-4.13449	-13.50445	-5384.66265
15	5.64634	-8.52098	-5512.67751
16	5.29793	-3.84388	-5634.99839

17	.87490	-3.07127	-5750.06489
18	7.37127	3.43727	-5855.79791
19	4.98446	7.83835	-5950.25735
20	7.54932	14.50504	-6031.56367
21	-6.71339	8.57834	-6097.87237
22	-24.04797	-12.64643	-6147.23228
23	-39.70771	-47.71274	-6177.38050
24	-78.32457	-116.86606	-6186.35695

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-616.71000
2	-391.04517
3	-438.72894
4	-489.08386
5	-532.14252
6	-564.91554
7	-588.78167
8	-606.66462
9	-621.38463
10	-635.12111
11	-648.89683
12	-663.75681
13	-679.87586
14	-696.60109
15	-713.42625
16	-730.19569
17	-746.48092
18	-760.96551
19	-772.90477
20	-781.48376

21	-785.75540
22	-785.16263
23	-780.20432
24	-773.38984
25	-484.55000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	4104.08415	437.83291	616.71000
LAST	6167.86328	-116.86606	-484.55000

"L.C.#5:DEAD LOAD & GROUT PRESSURE"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00042	-.00042
2	.00003	-.00001	-.00042	-.00041
3	.00006	-.00000	-.00042	-.00039
4	.00008	.00001	-.00043	-.00037
5	.00009	.00003	-.00046	-.00035
6	.00010	.00005	-.00051	-.00034
7	.00008	.00007	-.00056	-.00034
8	.00005	.00008	-.00062	-.00034
9	.00001	.00008	-.00068	-.00033
10	-.00004	.00008	-.00074	-.00032
11	-.00009	.00006	-.00078	-.00029

12	-.00013	.00004	-.00082	-.00024
13	-.00016	.00002	-.00085	-.00016
14	-.00016	-.00001	-.00087	-.00005
15	-.00015	-.00003	-.00090	.00009
16	-.00013	-.00005	-.00094	.00024
17	-.00009	-.00007	-.00098	.00041
18	-.00005	-.00008	-.00104	.00059
19	-.00001	-.00009	-.00110	.00077
20	.00002	-.00009	-.00118	.00095
21	.00004	-.00008	-.00125	.00110
22	.00005	-.00007	-.00132	.00124
23	.00004	-.00005	-.00138	.00134
24	.00002	-.00003	-.00141	.00140
25	0.00000	0.00000	-.00143	.00143

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	1194.34887	71.28217	17135.84226
2	693.47083	683.69386	16994.26385
3	285.81601	935.95544	16896.57228
4	40.56291	971.76502	16824.56458
5	-96.27005	886.75050	16762.05552
6	-218.84503	693.51889	16701.94137
7	-312.91895	417.22368	16643.29077
8	-439.66085	28.96630	16587.27608
9	-527.68053	-436.87848	16536.49523
10	-558.22897	-929.57209	16485.40397
11	-441.89242	-1319.81214	16422.24324
12	53.15081	-1272.88491	16316.52106
13	139.05545	-1150.11185	16171.43123
14	4.84706	-1145.83136	16031.72796

15	129.53885	-1031.50021	15895.10437
16	249.92023	-810.86664	15746.44164
17	348.01763	-503.53791	15592.16801
18	382.01856	-166.23045	15439.80061
19	403.92805	190.42224	15294.43070
20	376.06127	522.51586	15162.73965
21	351.16137	832.52671	15048.50979
22	292.30173	1090.51259	14957.00951
23	188.87680	1257.31177	14891.61374
24	108.09630	1352.75076	14853.65201

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-66.08000
2	2741.11205
3	2595.21982
4	2454.57803
5	2351.71432
6	2297.77076
7	2283.60361
8	2286.75302
9	2272.48229
10	2194.51304
11	2008.83205
12	1655.97262
13	2048.48186
14	2248.69092
15	1937.16538
16	1953.44678
17	1969.02918
18	1983.42291

19	1996.40993
20	2007.79195
21	2017.40629
22	2025.08990
23	2030.60788
24	2033.97025
25	1083.63000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-17177.28513	983.21697	66.08000
LAST	-14814.49869	1352.75076	1083.63000

"TOWNBROOK TUNNEL:CAST-IN-PLACE (12.5 FT)"

"L.C.#1':DEAD LOAD & ROCK LOAD(761 LB/FT)"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00022	-.00022
2	.00001	-.00001	-.00022	-.00021
3	.00003	-.00000	-.00021	-.00020
4	.00004	.00000	-.00022	-.00019
5	.00005	.00001	-.00023	-.00017
6	.00005	.00002	-.00025	-.00017
7	.00005	.00003	-.00027	-.00016
8	.00003	.00004	-.00030	-.00016
9	.00001	.00004	-.00033	-.00016
10	-.00002	.00005	-.00036	-.00016
11	-.00006	.00006	-.00039	-.00016
12	-.00011	.00006	-.00041	-.00016
13	-.00016	.00006	-.00042	-.00016
14	-.00021	.00005	-.00043	-.00015
15	-.00025	.00003	-.00044	-.00012
16	-.00027	.00001	-.00044	-.00008
17	-.00026	-.00003	-.00045	-.00000
18	-.00023	-.00007	-.00049	.00011
19	-.00018	-.00010	-.00054	.00026

20	-.00012	-.00012	-.00062	.00042
21	-.00007	-.00013	-.00072	.00059
22	-.00002	-.00011	-.00082	.00075
23	-.00000	-.00008	-.00090	.00087
24	.00000	-.00005	-.00096	.00096
25	0.00000	0.00000	-.00098	.00098

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	615.15255	-21.20450	8338.04987
2	352.15219	289.78497	8257.04164
3	144.51842	417.33711	8190.60378
4	30.17540	443.97644	8128.54773
5	-14.14942	431.48131	8061.28479
6	-33.04005	402.30824	7983.72180
7	-23.38956	381.65617	7893.78917
8	-26.48493	358.26776	7791.98986
9	-34.19511	328.07977	7681.29286
10	-80.96232	256.62233	7566.45730
11	-186.73970	91.71045	7455.88606
12	-318.52658	-189.51894	7357.83679
13	-467.93487	-602.66197	7254.29742
14	-595.32533	-1128.40031	7106.03462
15	-645.26224	-1697.90958	6892.63749
16	-457.95645	-2102.20084	6606.59470
17	153.73411	-1966.44073	6221.38901
18	601.63838	-1435.21755	5740.44625
19	891.69574	-647.88505	5211.47402
20	1010.85870	244.78789	4684.18636
21	977.76548	1107.97485	4194.81994
22	805.34007	1818.76902	3789.67096

23	523.11337	2280.73618	3501.67473
24	181.21071	2440.72840	3350.49130

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-66.08000
2	1352.22792
3	1269.26624
4	1183.54233
5	1112.01573
6	1061.91786
7	1033.02333
8	1022.80221
9	1028.22312
10	1045.90085
11	1075.58412
12	1104.31679
13	1108.98695
14	1070.15315
15	953.76705
16	697.74128
17	235.69714
18	330.66985
19	429.76536
20	524.39241
21	618.63284
22	695.30595
23	752.45416
24	791.14542
25	400.92000

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-8360.44787	564.32707	66.08000
LAST	-3331.36195	2440.72840	400.92000

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel Cast-in Place Dia = 12.5 PAGE E-38
COMPUTATION Stress Summary - Load Case 1 = Dead + Rock Load
COMPUTED BY ENestorides CHECKED BY DATE 4-9-86

LC #1

* Max allowable stresses:
for compression $.45 f_c' = 1800$ psi
tension $1.6\sqrt{f_c'} = 101$ psi
Prin. diagonal tension $2\sqrt{f_c'} = 126$ psi

NODE (i)	SHEAR (lb)	MOMENT (ft·lb)	THRUST (lb)	f_b (psi) inside face	f_b (psi) outside face	f_t (psi) prin. Ten. Stress
1	1366	-1193	19745	52	87	-1
2	1366	13	19745	144	138	-1
3	784	705	19586	112	165	0
4	323	990	19480	99	177	0
5	71	1053	19403	96	179	0
6	-24	1032	19334	96	177	0
7	-65	974	19259	98	174	0
8	-43	936	19173	99	172	0
9	-55	887	19074	100	169	0
10	-79	817	18968	103	166	0
11	-199	641	18864	110	158	0
12	-464	231	18779	127	140	0
13	-789	-465	18731	157	111	0
14	-1154	-1485	18657	201	68	0
15	-1469	-2782	18438	256	12	-1
16	-1612	-4204	18012	316	-50	-1
17	-1198	-5262	17365	357	-99	-1
18	226	-5062	16443	342	-98	0
19	1457	-3776	15248	277	-51	-1
20	2270	-1771	13898	179	23	-3
21	2625	547	12530	68	110	-4
22	2565	2811	11245	-41	195	-4
23	2125	4687	10174	-131	266	-3
24	1385	5911	9409	-190	312	-1
25	480	6335	9006	-212	327	0

SIGN CONVENTION: + Thrust = Compression; + Shear is directed toward center of tunnel;
+ Moment causes tension on inside fibers of liner; + f_b = compression
- f_t = tension.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE E-39

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5'
COMPUTATION Stress Summary: Load Case 2 = Dead Load and Ex. Water Pressure
COMPUTED BY ENestoridis CHECKED BY _____ DATE 4-9-86

LC # 2

NODE (i)	SHEAR (lb)	MOMENT (ft·lb)	THRUST (lb)	f_b (psi) inside face	f_b (psi) outside face	f_t (psi) prin. ten. stress
1	-12	986	71110	476	535	0
2	-12	976	71110	476	534	0
3	-37	944	71101	478	533	0
4	-214	756	71083	486	525	0
5	-247	537	71056	495	516	0
6	-211	350	71020	503	508	0
7	-297	88	70973	514	497	0
8	-275	-155	70917	524	486	0
9	-332	-448	70851	537	473	0
10	-237	-657	70774	545	464	0
11	-185	-821	70686	552	457	0
12	-252	-1043	70586	561	447	0
13	-146	-1171	70476	566	441	0
14	-21	-1189	70355	566	439	0
15	116	-1087	70222	560	442	0
16	111	-989	70079	555	445	0
17	251	-767	69927	544	454	0
18	459	-362	69765	525	469	0
19	485	67	69599	505	486	0
20	452	465	69444	487	502	0
21	271	704	69315	475	511	0
22	220	899	69217	466	518	0
23	132	1016	69149	460	523	0
24	-64	959	69111	463	520	0
25	-96	874	69099	466	516	0

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5' PAGE E-40
COMPUTATION Stress Summary: Load Case 3 = Dead, Rock, Ex. Water Pressure
COMPUTED BY E. Nestorides CHECKED BY _____ DATE 4-9-86

LC #3

NODE (i)	SHEAR (lb)	MOMENT (ft·lb)	THRUST (lb)	f_b (psi) inside face	f_b (psi) outside face	f_t (psi) prin. ten. stress
1	925	100	85342	619	597	0
2	925	917	85342	583	631	0
3	608	1454	85229	558	652	0
4	214	1643	85141	550	660	0
5	94	1726	85064	545	663	0
6	129	1840	84983	540	667	0
7	38	1873	84892	538	668	0
8	36	1905	84794	536	668	0
9	-169	1756	84696	541	661	0
10	-360	1438	84617	555	648	0
11	-804	729	84577	585	618	0
12	-1597	-681	84605	647	559	0
13	-2282	-2696	84724	737	476	0
14	-2759	-5132	84854	844	375	-1
15	-2581	-7412	84819	944	280	-1
16	-1284	-8545	84443	991	230	0
17	409	-8184	83663	969	240	0
18	1937	-6474	82635	887	304	0
19	2859	-3950	81239	766	400	-1
20	3464	-891	79717	621	516	-1
21	3535	2231	78211	473	636	-1
22	3321	5162	76813	335	749	-1
23	2686	7533	75661	222	839	-1
24	1632	8975	74844	153	894	0
25	561	9470	74415	128	911	0

NED FORM 223
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NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5' PAGE E-41
COMPUTATION Stress Summary ⇒ Load Case 4 = Dead Load + Int. Water Pressure
COMPUTED BY E Nestorides CHECKED BY _____ DATE 4-9-86

LC #4

NODE (i)	SHEAR (lb)	MOMENT (ft·lb)	THRUST (lb)	f_b (psi) inside face	f_b (psi) outside face	f_t (psi) prin. ten. stress
1	346	-438	-4136	-11	-47	-29
2	346	-133	-4136	-24	-34	-29
3	190	35	-4189	-32	-28	-29
4	84	109	-4241	-36	-25	-29
5	13	121	-4297	-37	-25	-30
6	-26	98	-4362	-36	-26	-30
7	-33	69	-4439	-35	-28	-31
8	-33	40	-4529	-35	-30	-31
9	-22	21	-4630	-35	-31	-32
10	-19	4	-4742	-35	-33	-33
11	-13	-7	-4863	-35	-34	-34
12	0	-7	-4990	-36	-35	-35
13	-1	-8	-5121	-37	-36	-36
14	-2	-10	-5253	-38	-37	-36
15	-4	-14	-5385	-39	-38	-37
16	6	-9	-5513	-40	-39	-38
17	5	-4	-5635	-41	-39	-39
18	1	-3	-5750	-42	-40	-40
19	7	3	-5856	-43	-41	-41
20	5	8	-5950	-44	-41	-41
21	8	15	-6032	-45	-41	-42
22	-7	9	-6098	-45	-42	-42
23	-24	-13	-6147	-44	-43	-43
24	-40	-48	-6177	-43	-45	-43
25	-78	-117	-6186	-40	-48	-43

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5'

COMPUTATION Stress Summary: Load Case #5 = Dead Load and Grout Pressure

COMPUTED BY E Nestorides CHECKED BY _____ DATE 4-9-86

PAGE E-42

LC #5

NODE (i)	SHEAR (lb)	MOMENT (ft.lb)	THRUST (lb)	f_b (psi) inside face	f_b (psi) outside face	f_b (psi) prin. ten. stress
1	1194	-983	17136	168	78	-1
2	1194	71	17136	122	122	-1
3	693	684	16994	94	147	0
4	286	936	16897	82	156	0
5	41	972	16825	80	157	0
6	-96	887	16762	84	153	0
7	-219	694	16702	92	145	0
8	-313	417	16643	103	133	0
9	-440	29	16587	120	116	0
10	-528	-437	16536	140	97	0
11	-558	-930	16485	161	76	0
12	-442	-1320	16422	178	59	0
13	53	-1273	16317	175	60	0
14	139	-1150	16171	168	64	0
15	5	-1146	16032	167	64	0
16	130	-1032	15895	161	67	0
17	250	-811	15746	150	76	0
18	348	-504	15592	136	87	0
19	382	-166	15440	120	100	0
20	404	190	15294	103	114	0
21	376	523	15163	88	127	0
22	351	833	15049	73	139	0
23	392	1091	14957	61	149	0
24	189	1257	14892	54	156	0
25	108	1353	14854	49	160	0

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel TBM Liner
COMPUTATION Dead + Rock + Ex. Water Pressure LOADING LC #5
COMPUTED BY ENestorides CHECKED BY DATE 4-78 3

LOAD CASE #3

Refer to LC #1 and LC #2 write-ups.
Dead Load + Rock Load + Ex. Water Press.

NODE	Fx	Fy
1	0	4610.99
2	1215.99	9139.04
3	2408.79	8892.43
4	3555.79	8487.12
5	4635.59	7931.77
6	5628.14	7237.42
7	6515.57	6418.26
8	7282.40	5490.68
9	7915.78	4472.87
10	8405.62	3384.41
11	8745.35	2246.00
12	8930.83	1078.46
13	8961.40	-146.19
14	8838.11	-1454.66
15	8566.23	-2775.92
16	8152.39	-4041.81
17	7605.36	-5230.01
18	6936.28	-6321.80
19	6157.38	-7303.41
20	5282.24	-8157.75
21	4325.54	-8873.91
22	3302.76	-9441.18
23	2233.75	-9851.18
24	1123.34	-10,100.50
25	0	-5091.88

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

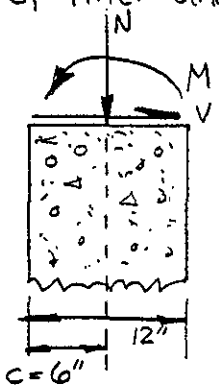
SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5'
COMPUTATION Stress Computation
COMPUTED BY ENestorides CHECKED BY DATE 4-10-86

I. Unreinforced Strength Limitations: $f_c = \left(\frac{N}{bD} \pm \frac{6M}{bD^2}\right) F^*$

$D = 12''$ (liner thickness) $b = 12''$ (length of liner analysed)

$c = 6''$ $R = 6.75'$ or $81''$

$I = 1728 \text{ in}^4$ $f_c' = 4000 \text{ psi}$



ⓐ Max. compressive stress ⇒ $.45 f_c' = \underline{1800 \text{ psi}}$

ⓑ Max tensile stress ⇒ $1.6\sqrt{f_c'} = \underline{101 \text{ psi}}$

ⓒ Prin. diagonal tensile* stress ⇒ $2\sqrt{f_c'} = \underline{126.5 \text{ psi}}$

*This restriction controls (shear and diagonal tension requirements are met when this is met).

ⓓ Curved Beam Correction factor (F^*) ⇒ inside face of liner.

$$F^* = 1 + .5\left(\frac{I}{bc^2}\right)\left[\frac{1}{R-c} + \frac{1}{R}\right]$$
$$F^* = 1 + .5\,\frac{1728}{12(36)}\left[\frac{1}{81-6} + \frac{1}{81}\right]$$
$$\underline{\underline{F^* = 1.0513}}$$

II. Steel Area Requirements:

From EM ⇒ $f_s = 20{,}000 \text{ psi}$ $f_c' = 4{,}000 \text{ psi}$

ⓐ Min. transverse steel (in each face): $.002\,bD \Rightarrow .002(12)(12) = .288 \text{ in}^2$

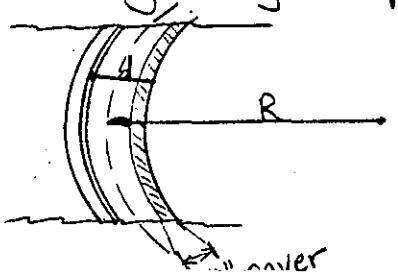
try #5 bars at 12": $\underline{A_s = .31 \text{ in}^2}$ OK

ⓑ Min. Longitudinal steel: $.001\,bD \Rightarrow .001(144) = .144 \text{ in}^2$

try #5 bars at 18": $\underline{A_s = .207 \text{ in}^2}$ OK

III. Calculations of Transverse Stresses:

— neglecting compression steel — $\phi_{\#5} = .625$



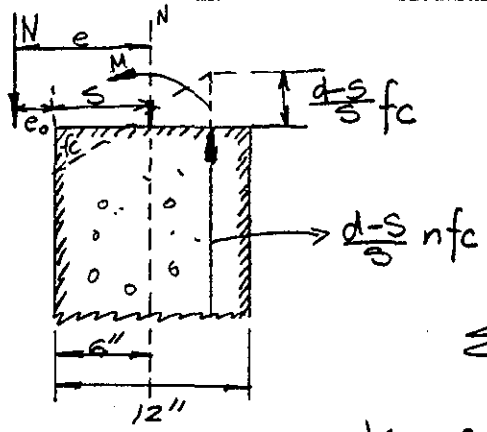
$$d^+ = 12'' - 4''(\text{cover}) - \frac{\phi}{2} = 7.688''$$

$$n = \frac{E_s}{E_c} = \frac{30{,}000 \text{ ksi}}{3834 \text{ ksi}} = 7.8 \Rightarrow \underline{n \approx 8}$$

SUBJECT Townbrook Tunnel Cast-in-Place Dia = 12.5'

COMPUTATION Stress Computation - REINFORCED LINER

COMPUTED BY ENestorides CHECKED BY DATE 4-10-86



$$e = M/N = \frac{6335(12)}{9006} = 8.44$$

$$e_o = e - D/2 = 8.44'' - 6'' = 2.44''$$

Crown Node L.C. #1	M = 6335	ft.lb
	N = 9006	lb
	V = 480	lb

$$\Sigma M_{\text{about } N} = 0$$

$$\Rightarrow \tfrac{1}{2} b f_c s \left(\tfrac{s}{3} + e_o\right) - \frac{d-s}{s} n f_c A_s (d + e_o) = 0$$

$$\tfrac{1}{2}(12)\left(\frac{s^2}{3} + 2.44(s)\right) - \left(\frac{7.688-s}{s}\right)(8)(.31)\underbrace{(7.688+2.44)}_{25.147} = 0$$

$$2s^3 + 14.64s^2 - 193.33 + 25.147(s) = 0$$

$$s^3 + 7.32s^2 + 12.57s - 96.67 = 0$$

trial and error $\Rightarrow s \cong \underline{\underline{2.56}}$

① $$f_c = \frac{N F^*}{\frac{sb}{2} - \frac{d-s}{s} n A_s} = \frac{9006}{\frac{2.56(12)}{2} - \frac{7.688-2.56}{2.56}(8)(.31)}$$

$f_c = 866.61$ psi < 1800 psi OK ✓

② $$f_s = \left(\frac{d-s}{s}\right) n f_c F^* = \left(\frac{7.688-2.56}{2.56}\right)(8)(866.61)(1.0513)$$

$f_s = 14{,}599.8$ psi $\leq 20{,}000$ psi OK ✓

It's "OK" to use #5 bars at 12" for L.C. #1 (transverse steel)

IV. Principal Tensile Stress:

$$f_t = \frac{N}{bD} - \sqrt{\left(\frac{N}{2bD}\right)^2 + \left(\frac{V}{bD}\right)^2}$$

① Crown node
N = 9006# M = 6335 '#
V = 480 #

$f_t \cong 0$ psi < 126.5 psi ✓

② Max shear occurs at node 21
N = 12530
V = 2625 #

$f_t \cong -4$ psi < 126.5 psi ✓

SUBJECT Town Brook Tunnel Cast-in-Place Dia = 12.5
COMPUTATION Reinforced liner Design
COMPUTED BY ENestorides CHECKED BY DATE 4-10-86

1. Determine cut-off for transverse steel. (Reinforcement required at the crown only.)

From Moment Table on p. -38 — Inflection point occurs between nodes 21 and 20.

Therefore, terminate reinforcement at node 20, which is located 37.5° from the crown, plus a development length ℓd.

$$\ell d = 0.04\, A_b f_y / \sqrt{f_c'} \quad \text{but not less than } .0004\, d_b f_y$$

for #5 bar, grade 60:

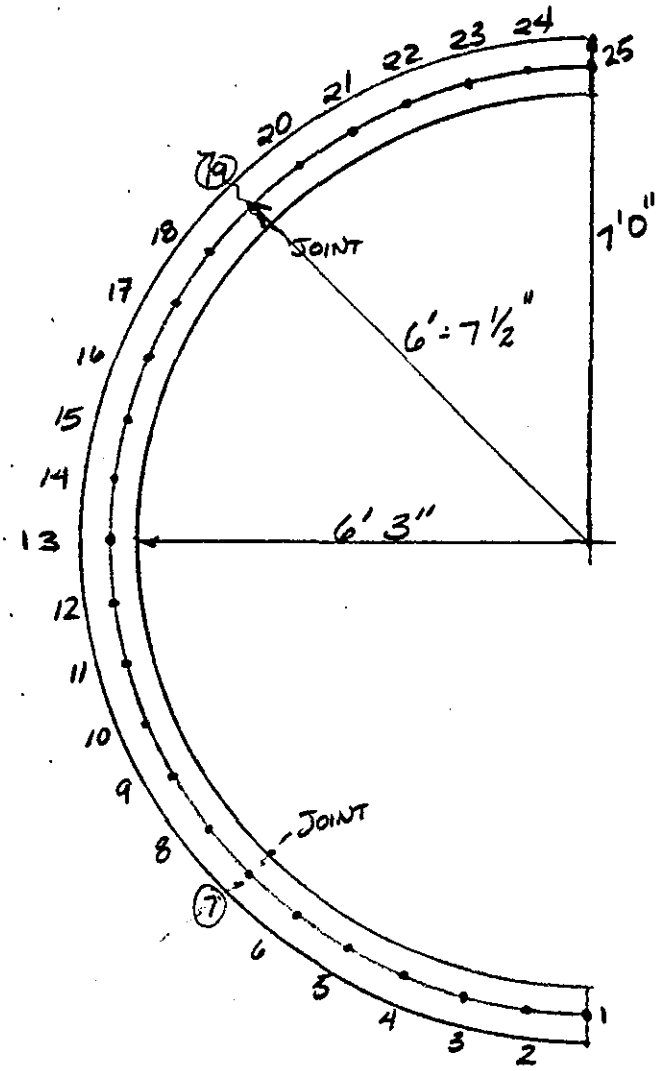
$$\ell d = .04(.31)(60{,}000)/\sqrt{4000} = 11.76''$$

but, $\ell d \geq .0004(.625)(60{,}000) = 15''$

$\therefore$, use $\ell d = 15''$

SUBJECT: Town Brook Tunnel TBM - Precast liner
COMPUTATION: X-Y Coordinate Description (D=12.5')
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-24-86

COMPUTER MODEL PRECAST LINER WITH TBM EXCAVATION



NODE	X-coord.	Y-Coord
1	0	-6.625
2	-.865	-6.568
3	-1.715	-6.399
4	-2.535	-6.121
5	-3.313	-5.737
6	-4.033	-5.256
7	-4.685	-4.685
8	-5.256	-4.033
9	-5.737	-3.313
10	-6.121	-2.535
11	-6.399	-1.715
12	-6.568	-.865
13	-6.625	0
14	-6.568	.865
15	-6.399	1.715
16	-6.121	2.535
17	-5.737	3.313
18	-5.256	4.033
19	-4.685	4.685
20	-4.033	5.256
21	-3.313	5.737
22	-2.535	6.121
23	-1.715	6.399
24	-.865	6.568
25	0	6.625

SSM for nodes 1, 25 = 0.00 ft²

for rest: 0.0115 ft²

— Continuity Release for each joint

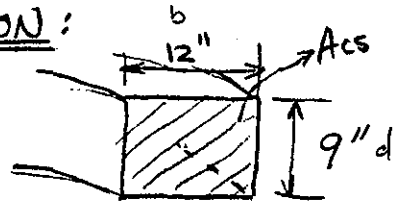
Shear and thrust are transmitted } No Moment] ⇒ Code 3

Otherwise, code 0

SUBJECT Townbrook TUNNEL
COMPUTATION ReComputation of Tunnel liner D=12.5'
COMPUTED BY ENestondes CHECKED BY DATE 2-24-86

HALF-CIRCLE

PARAMETER COMPUTATION:



1-foot section

$A_{cs} \Rightarrow 108\ in^2 \Rightarrow .75\ ft^2$

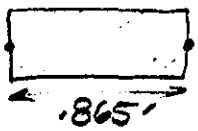
① $I = \frac{bd^3}{12}$

$$I = \frac{12(9)^3}{12} \Rightarrow \underline{729\ in^4} \ \underline{or} \ \boxed{.035\ ft^4}$$

② radius of tunnel at node axis: $6'3'' + 4.5''$

$\Rightarrow 6'7.5'' \Rightarrow 6.625\ ft.$

$s = r\sin\theta \Rightarrow (6.625\ ft)(\sin 7.5^\circ) \Rightarrow .865\ ft.$

$A_{surface} \Rightarrow$ 

$\Rightarrow .865\ ft^2$

③ Young's Modulus.

$$E_c = w^{1.5}(33)\sqrt{f_c'} \Rightarrow (150)^{1.5}(33)(\sqrt{5000})$$

$$= 4286825.748\ lb/in^2 \cdot \frac{144\ in^2}{ft^2}$$

$$\boxed{E_c = 617302907.8\ lb/ft^2}$$

④ $R \Rightarrow 8,228,571\ lb/ft^3$

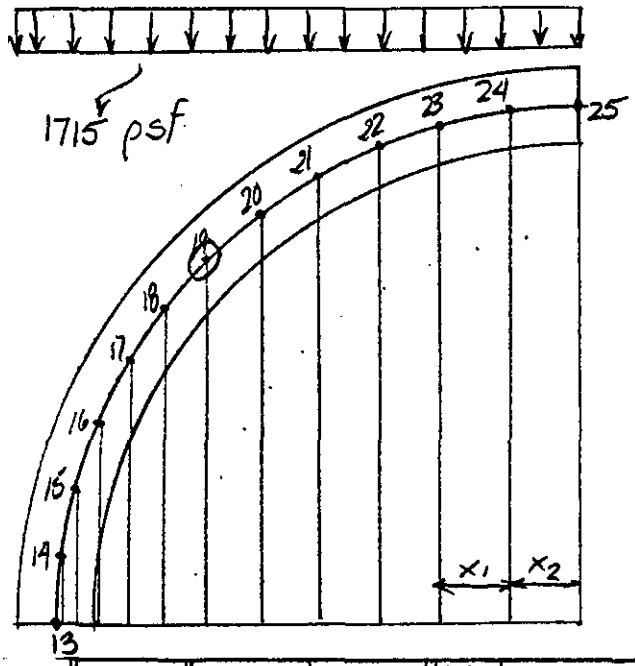
$$SMM = \frac{R}{E_c} \cdot A_s \Rightarrow \frac{8228,571}{617302907.8} \times (.865)$$

$$\boxed{SMM \Rightarrow 0.0115} \times \underline{1ft\ length} \Rightarrow \underline{.0115\ ft^2}$$

NED FORM 223
27 Sept 49

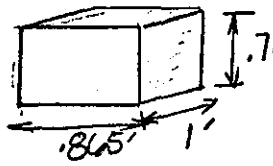
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel TBM - Precast Liner
COMPUTATION Rock Load + Dead Load Load Case #1
COMPUTED BY E Nestorides CHECKED BY DATE 4-7-86



LOAD CASE 1

DEAD LOAD:



$\Rightarrow .649\ ft^3$

$\gamma_c = 150\ lb/ft^3$

$W_c = 97.3\ lb$

at nodes 1, 25 $\Rightarrow W_c/2 \Rightarrow 48.66.$

ROCK LOAD:

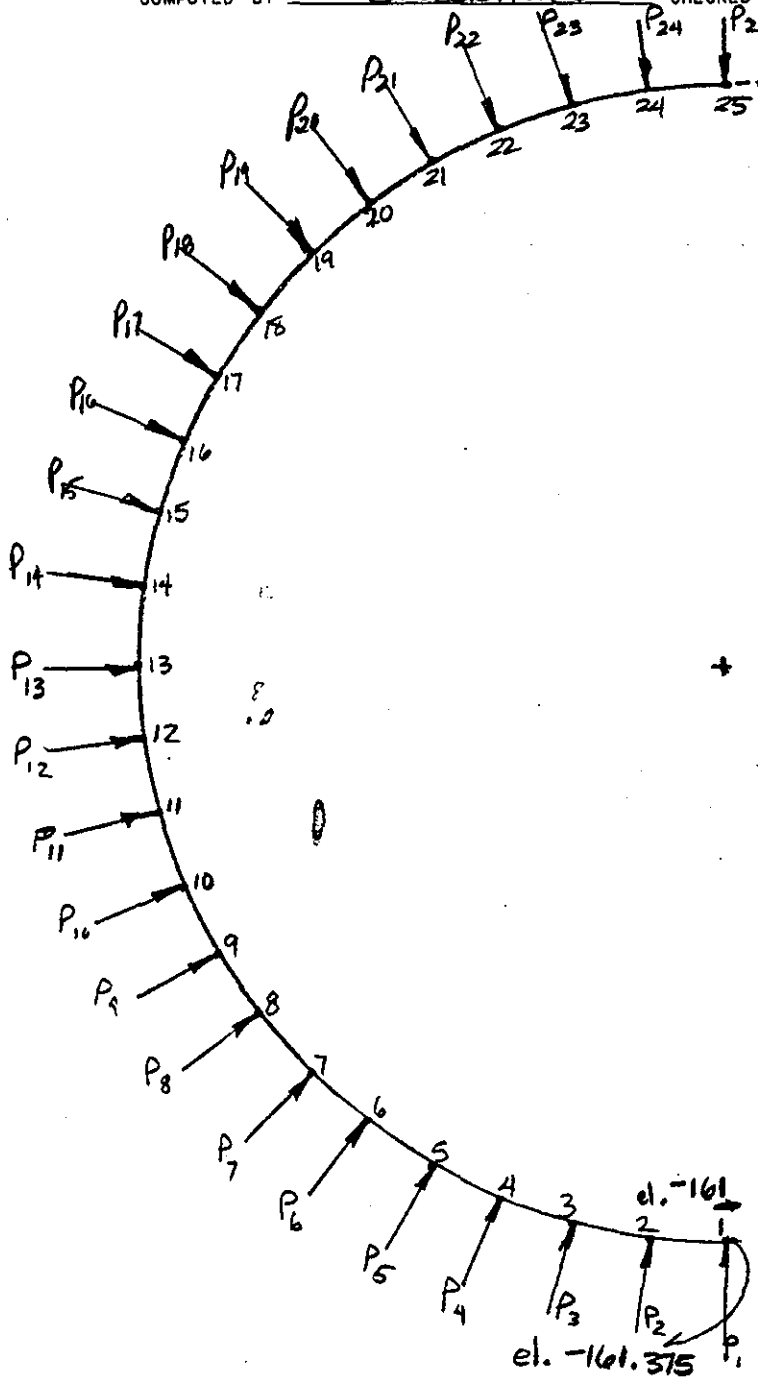
example for node 24: $1715\left(\frac{x_1+x_2}{2}\right)$

NODE	Fx	M	Fyr ROCK LOAD $\frac{x_1+x_2}{2}$	$1715\frac{x_1+x_2}{2}$	Fyd DEAD LOAD	TOTAL Fy
1	0	0		0	-48.66	-48.66
2					-97.31	-97.31
3						
4						
5						
6						
7						
8						
9						
10						
11						
12				0		-97.31
13			.0285	-48.88		-146.19
14			.113	-193.80		-291.10
15			.2235	-383.30		-480.61
16			.331	-567.67		-664.98
17			.4325	-741.74		-839.05
18			.526	-902.09		-999.40
19			.6115	-1048.72		-1146.03
20			.686	-1176.49		-1273.80
21			.749	-1284.54		-1381.85
22			.799	-1370.30		-1467.60
23			.835	-1432.03		-1529.34
24			.8575	-1470.61	-97.31	-1567.92
25			.4325	-741.74	-48.66	-790.40

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Town Brook Tunnel
COMPUTATION: Dead Load + External Water Pressure [LC #2]
COMPUTED BY: ENestorides CHECKED BY: _____ DATE: 2-24-86



① DEAD LOAD (see LC #1)

a) nodes 1, 25 ⇒ 48.66
b) all others ⇒ 97.31

② EXTERNAL WATER PRESSURE

$A_{contact\,sur} = .865\ ft^2$

$\gamma_w = 62.5\ pcf \Rightarrow w_w \Rightarrow 54.0625$ lb/ft

Water Level ▽ +11 feet

Node (i)	h_i (ft)	P_i (lb)	F_x (lb)	F_y (lb)	F_{Ty} (lb)
1*	172.38	4659.65	0	4659.65	4610.99
2	172.32	9316.05	1215.99	9236.35	9139.04
3	172.15	9306.86	2408.79	8989.74	8892.43
4	171.87	9291.72	3555.79	8584.43	8487.12
5	171.49	9271.18	4635.59	8029.08	7931.77
6	171.01	9245.23	5628.14	7334.73	7237.42
7	170.44	9214.41	6515.57	6515.57	6418.26
8	169.79	9179.27	7282.40	5587.99	5490.68
9	169.07	9140.35	7915.78	4570.18	4472.87
10	168.29	9098.18	8405.62	3481.72	3384.41
11	167.47	9053.85	8745.35	2343.31	2246.00
12	166.62	9007.89	8930.83	1175.77	1078.46
13	165.76	8961.40	8961.4	0	-97.31
14	164.89	8914.37	8838.11	-1163.56	-1260.87
15	164.04	8868.41	8566.23	-2295.31	-2392.62
16	163.22	8824.08	8152.39	-3376.83	-3474.14
17	162.44	8781.91	7605.36	-4390.96	-4488.27
18	161.72	8742.99	6936.28	-5322.40	-5419.71
19	161.07	8707.85	6157.38	-6157.38	-6254.69
20	160.50	8677.03	5282.24	-6883.95	-6981.26
21	160.02	8651.08	4325.54	-7492.06	-7589.37
22	159.64	8630.54	3302.76	-7973.58	-8070.89
23	159.36	8615.40	2233.75	-8321.84	-8419.15
24	159.19	8606.21	1123.34	-8532.58	-8629.89
25*	159.13	4301.48	0	-4301.48	-43[illegible]4

* nodes 1 & 25 ⇒ ½ P_i

LOAD CASE 2

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Townbrook Tunnel TBM Liner
COMPUTATION Dead + Rock + Ex. Water Pressure LOADING LC #5
COMPUTED BY ENestorides CHECKED BY DATE 4-78 3

LOAD CASE #3

Refer to LC #1 and LC #2 write-ups.
Dead Load + Rock Load + Ex. Water Press.

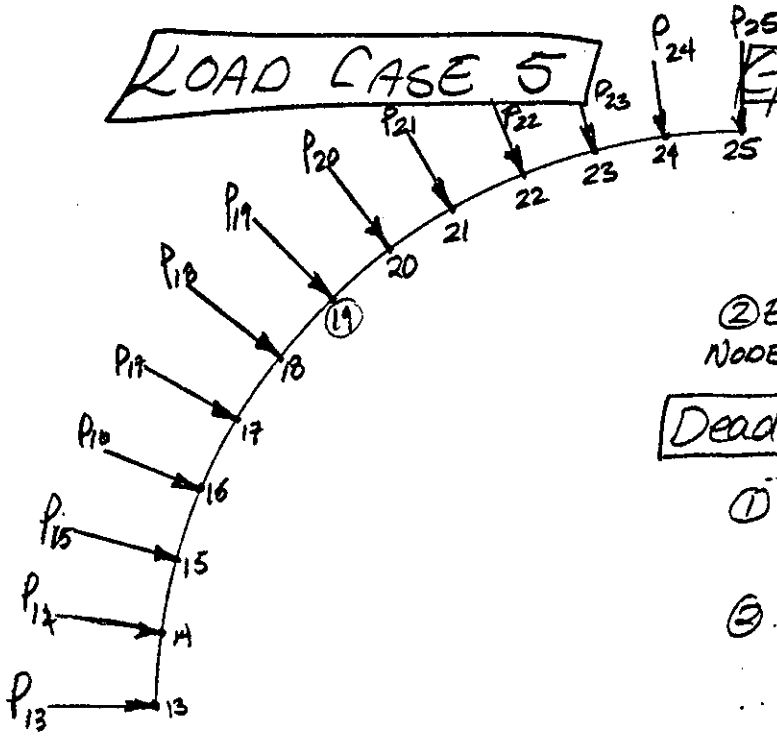
NODE	Fx	Fy
1	0	4610.99
2	1215.99	9139.04
3	2408.79	8892.43
4	3555.79	8487.12
5	4635.59	7931.77
6	5628.14	7237.42
7	6515.57	6418.26
8	7282.40	5490.68
9	7915.78	4472.87
10	8405.62	3384.41
11	8745.35	2246.00
12	8930.83	1078.46
13	8961.40	-146.19
14	8838.11	-1454.66
15	8566.23	-2775.92
16	8152.39	-4041.81
17	7605.36	-5230.01
18	6936.28	-6321.80
19	6157.38	-7303.41
20	5282.24	-8157.75
21	4325.54	-8873.91
22	3302.76	-9441.18
23	2233.75	-9851.18
24	1123.34	-10,100.50
25	0	-5091.88

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Town Brook Tunnel
COMPUTATION Dead Load and Grout Pressure LC #5
COMPUTED BY E Nestorides CHECKED BY DATE 2-24-86

LOAD CASE 5



Grout Pressure 15 psi

① all nodes

$F_G = 15\,lb/in^2 \left(\frac{144\,in^2}{ft^2}\right) \times .865\,ft^2$

$= \underline{1868.40\ lb}$

② EXCEPT
NODES 13, 25 ⇒ $F_G/2 = \underline{934.20\ lb}$

Dead Load

① all nodes
$w_1 = \underline{93.71\ lb.}$

② nodes 1, 25 = $\underline{48.66\ lb}$

NODE (i)	P_{ix}	P_{iy}	P_{YT} w/ Grout Pressure
1	0	0	-48.66
			-97.31
12	0		
13	934.20	0	-97.31
14	1852.42	-243.88	-341.19
15	1804.74	-483.58	-580.89
16	1726.18	-715.01	-812.32
17	1618.08	-934.20	-1031.51
18	1482.30	-1137.41	-1234.72
19	1321.16	-1321.16	-1418.47
20	1137.41	-1482.30	-1579.61
21	934.20	-1618.08	-1715.39
22	715.01	-1726.18	-1823.49
23	483.58	-1804.74	-1902.05
24	243.88	-1852.42	-1949.73
25	0	-934.20	-982.86

```
1100 "TOWNBROOK TUNNEL: PRECAST LINER(12.5 FT)"
1200 25,617302907.8,2
1300 0.00,-6.625,0.00
1400 -0.865,-6.568,0.0115
1500 -1.715,-6.399,0.0115
1600 -2.535,-6.121,0.0115
1700 -3.313,-5.737,0.0115
1800 -4.033,-5.256,0.0115
1900 -4.685,-4.685,0.0115
2000 -5.256,-4.033,0.0115
2010 -5.737,-3.313,0.0115
2020 -6.121,-2.535,0.0115
2030 -6.399,-1.715,0.0115
2040 -6.568,-0.865,0.0115
2050 -6.625,0.00,0.0115
2060 -6.568,0.865,0.0115
2070 -6.399,1.715,0.0115
2080 -6.121,2.535,0.0115
2090 -5.737,3.313,0.0115
2100 -5.256,4.033,0.0115
2110 -4.685,4.685,0.0115
2120 -4.033,5.256,0.0115
2130 -3.313,5.737,0.0115
2140 -2.535,6.121,0.0115
2150 -1.715,6.399,0.0115
2160 -0.865,6.568,0.0115
2170 0.00,6.625,0.00
2200 0.035,0.75,0
2210 0.035,0.75,0
2220 0.035,0.75,0
2230 0.035,0.75,0
2240 0.035,0.75,0
2250 0.035,0.75,3
```

```
2260 0.035,0.75,0
2270 0.035,0.75,0
2280 0.035,0.75,0
2290 0.035,0.75,0
2300 0.035,0.75,0
2310 0.035,0.75,0
2320 0.035,0.75,0
2330 0.035,0.75,0
2340 0.035,0.75,0
2350 0.035,0.75,0
2360 0.035,0.75,0
2370 0.035,0.75,3
2380 0.035,0.75,0
2390 0.035,0.75,0
2400 0.035,0.75,0
2410 0.035,0.75,0
2420 0.035,0.75,0
2430 0.035,0.75,0
2500 2,2
2510 3,1
2600 "ROCK (1715 PSF) AND DEAD LOAD"
2605 25
2610 1,0,0,-48.66
2620 2,0,0,-97.31
2630 3,0,0,-97.31
2640 4,0,0,-97.31
2650 5,0,0,-97.31
2660 6,0,0,-97.31
2670 7,0,0,-97.31
2680 8,0,0,-97.31
2690 9,0,0,-97.31
2700 10,0,0,-97.31
2710 11,0,0,-97.31
2720 12,0,0,-97.31
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2730 13,0,0,-146.310
2740 14,0,0,-291.100
2750 15,0,0,-480.610
2760 16,0,0,-664.980
2770 17,0,0,-839.050
2780 18,0,0,-999.400
2790 19,0,0,-1146.030
2800 20,0,0,-1273.800
2810 21,0,0,-1381.850
2820 22,0,0,-1467.600
2830 23,0,0,-1529.340
2840 24,0,0,-1567.920
2850 25,0,0,-790.400
2900 "DEAD LOAD AND EX. WATER PRESSURE"
2905 25
2910 1,0.00,0,4610.99
2920 2,1215.99,0,9139.04
2930 3,2408.79,0,8892.43
2940 4,3555.79,0,8487.12
2950 5,4635.59,0,7931.77
2960 6,5628.14,0,7237.42
2970 7,6515.57,0,6418.26
2980 8,7282.40,0,5490.68
2990 9,7915.78,0,4472.87
3000 10,8405.62,0,3384.41
3010 11,8745.35,0,2246.00
3020 12,1175.77,0,1078.46
3030 13,8961.40,0,-97.31
3040 14,8838.11,0,-1260.87
3050 15,8566.23,0,-2392.62
3060 16,8152.39,0,-3474.14
3070 17,7605.36,0,-4488.27
3080 18,6936.28,0,-5419.71
3090 19,6157.38,0,-6254.69
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3100 20,5282.24,0,-6981.26
3110 21,4325.54,0,-7589.37
3120 22,3302.76,0,-8070.89
3130 23,2233.75,0,-8419.15
3140 24,1123.34,0,-8629.89
3150 25,0.00,0,-4350.14
/
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1100 "TOWNBROOK TUNNEL: PRECAST LINER(12.5 FT)"
1200 25,617302907.8,2
1300 0.00,-6.625,0.00
1400 -0.865,-6.568,0.0115
1500 -1.715,-6.399,0.0115
1600 -2.535,-6.121,0.0115
1700 -3.313,-5.737,0.0115
1800 -4.033,-5.256,0.0115
1900 -4.685,-4.685,0.0115
2000 -5.256,-4.033,0.0115
2010 -5.737,-3.313,0.0115
2020 -6.121,-2.535,0.0115
2030 -6.399,-1.715,0.0115
2040 -6.568,-0.865,0.0115
2050 -6.625,0.00,0.0115
2060 -6.568,0.865,0.0115
2070 -6.399,1.715,0.0115
2080 -6.121,2.535,0.0115
2090 -5.737,3.313,0.0115
2100 -5.256,4.033,0.0115
2110 -4.685,4.685,0.0115
2120 -4.033,5.256,0.0115
2130 -3.313,5.737,0.0115
2140 -2.535,6.121,0.0115
2150 -1.715,6.399,0.0115
2160 -0.865,6.568,0.0115
2170 0.00,6.625,0.00
2200 0.035,0.75,0
2210 0.035,0.75,0
2220 0.035,0.75,0
2230 0.035,0.75,0
2240 0.035,0.75,0
2250 0.035,0.75,3
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2260 0.035,0.75,0
2270 0.035,0.75,0
2280 0.035,0.75,0
2290 0.035,0.75,0
2300 0.035,0.75,0
2310 0.035,0.75,0
2320 0.035,0.75,0
2330 0.035,0.75,0
2340 0.035,0.75,0
2350 0.035,0.75,0
2360 0.035,0.75,0
2370 0.035,3.75,3
2380 0.035,0.75,0
2390 0.035,0.75,0
2400 0.035,0.75,0
2410 0.035,0.75,0
2420 0.035,0.75,0
2430 0.035,0.75,0
2500 2,2
2510 3,1
2600 "DEAD,ROCK(1715 PSF),AND EX.WATER LOAD"
2605 25
2610 1,0.00,0,4610.99
2620 2,1215.99,0,9139.04
2630 3,2408.79,0,8892.43
2640 4,3555.79,0,8487.12
2650 5,4635.59,0,7931.77
2660 6,5628.14,0,7237.42
2670 7,6515.57,0,6418.26
2680 8,7282.40,0,5490.68
2690 9,7915.78,0,4472.87
2700 10,8405.62,0,3384.41
2710 11,8745.35,0,2246.00
2720 12,8930.83,0,1078.46
```

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2730 13,8961.40,0,-146.19
2740 14,8838.11,0,-1454.66
2750 15,8566.23,0,-2775.92
2760 16,8152.39,0,-4041.81
2770 17,7605.36,0,-5230.01
2780 18,6936.28,0,-6321.80
2790 19,6157.38,0,-7303.41
2800 20,5282.24,0,-8157.75
2810 21,4325.54,0,-8873.91
2820 22,3302.76,0,-9441.18
2830 23,2233.75,0,-9851.18
2840 24,1123.34,0,-10100.50
2850 25,0.00,0,-5091.88
2900 "DEAD LOAD AND GROUT PRESSURE"
2905 25
2910 1,0,0,-48.66
2920 2,0,0,-97.31
2930 3,0,0,-97.31
2940 4,0,0,-97.31
2950 5,0,0,-97.31
2960 6,0,0,-97.31
2970 7,0,0,-97.31
2980 8,0,0,-97.31
2990 9,0,0,-97.31
3000 10,0,0,-97.31
3010 11,0,0,-97.31
3020 12,0,0,-97.31
3030 13,934.20,0,-97.31
3040 14,1852.42,0,-341.19
3050 15,1804.74,0,-580.89
3060 16,1726.18,0,-812.32
3070 17,1618.08,0,-1031.51
3080 18,1482.30,0,-1234.72
3090 19,1321.16,0,-1418.47
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3100 20,1137.41,0,-1579.61
3110 21,934.20,0,-1715.39
3120 22,715.01,0,-1823.49
3130 23,483.58,0,-1902.05
3140 24,243.88,0,-1949.73
3150 25,0.00,0,-982.86
/
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"TOWNBROOK TUNNEL: PRECAST LINER(12.5 FT)"

"ROCK (1715 PSF) AND DEAD LOAD"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00041	-.00041
2	.00003	-.00002	-.00040	-.00039
3	.00006	-.00001	-.00039	-.00036
4	.00009	.00001	-.00040	-.00034
5	.00011	.00002	-.00043	-.00031
6	.00012	.00003	-.00047	-.00030
7	.00012	.00009	-.00051	-.00027
8	.00008	.00009	-.00058	-.00029
9	.00003	.00009	-.00065	-.00030
10	-.00003	.00010	-.00071	-.00030
11	-.00010	.00010	-.00077	-.00030
12	-.00019	.00011	-.00081	-.00029
13	-.00028	.00012	-.00085	-.00028
14	-.00039	.00012	-.00087	-.00027
15	-.00049	.00012	-.00088	-.00025
16	-.00060	.00011	-.00087	-.00022
17	-.00069	.00010	-.00085	-.00017
18	-.00078	.00009	-.00083	-.00012
19	-.00085	-.00059	-.00079	-.00004

20	-.00054	-.00058	-.00119	.00062
21	-.00028	-.00053	-.00160	.00124
22	-.00011	-.00045	-.00199	.00180
23	-.00002	-.00033	-.00232	.00223
24	.00001	-.00017	-.00253	.00251
25	0.00000	0.00000	-.00261	.00261

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	1105.61953	.29026	16038.22295
2	485.74843	421.25819	15921.50134
3	69.27820	481.24223	15859.91164
4	-142.40499	357.69082	15827.57378
5	-225.58727	162.35765	15802.69963
6	-187.33152	0.00000	15770.36794
7	27.08955	23.47814	15712.02154
8	85.96212	97.91165	15627.19606
9	70.80905	159.34600	15532.64424
10	37.10299	191.47137	15435.29164
11	-24.21790	170.48322	15340.31109
12	-84.49169	97.23940	15250.74962
13	-102.48912	8.39404	15116.44359
14	-112.65336	-89.23561	14841.26063
15	-110.28251	-184.72295	14390.53102
16	-66.69980	-242.59209	13786.60069
17	29.39566	-217.13876	13060.82450
18	250.53907	0.00000	12247.78804
19	688.17611	596.43275	11373.47542
20	1100.93732	1549.72051	10480.04955
21	1215.90844	2604.64985	9636.14169
22	1080.23249	3539.96153	8921.98680

23	732.95237	4175.16573	8407.09845
24	255.90537	4397.00395	8137.20437

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-48.65999
2	2701.85958
3	2489.42948
4	2295.21766
5	2144.95425
6	2017.36427
7	1866.58646
8	1981.57067
9	2046.39407
10	2070.31166
11	2069.40093
12	2054.23459
13	2014.70313
14	1962.94949
15	1905.12272
16	1809.59981
17	1654.31267
18	1426.99994
19	1124.34967
20	1010.89498
21	1196.58296
22	1355.93487
23	1477.31031
24	1554.47720
25	790.39988

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END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-16076.18566	958.14478	48.65998
LAST	-8103.21086	4397.00395	790.39975

"DEAD LOAD AND EX. WATER PRESSURE"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	.00453	.00453
2	.00013	.00012	.00447	.00444
3	.00022	.00024	.00428	.00419
4	.00026	.00034	.00400	.00380
5	.00023	.00042	.00364	.00327
6	.00012	.00048	.00325	.00265
7	-.00006	.00037	.00284	.00196
8	-.00022	.00035	.00254	.00137
9	-.00038	.00029	.00227	.00081
10	-.00051	.00019	.00206	.00032
11	-.00057	.00005	.00191	-.00006
12	-.00051	-.00015	.00179	-.00027
13	-.00030	-.00029	.00167	-.00030
14	-.00003	-.00032	.00152	-.00023
15	.00020	-.00029	.00134	-.00015
16	.00038	-.00024	.00114	-.00009
17	.00050	-.00021	.00094	-.00004

18	.00056	-.00018	.00074	-.00000
19	.00060	-.00006	.00054	.00004
20	.00053	-.00006	.00042	-.00001
21	.00046	-.00006	.00030	-.00003
22	.00036	-.00005	.00020	-.00005
23	.00025	-.00003	.00013	-.00006
24	.00013	-.00002	.00008	-.00006
25	0.00000	0.00000	.00007	-.00007

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	-84.72968	3064.69859	68839.74373
2	-329.12199	2779.46905	68855.27487
3	-554.39768	2299.44773	68885.64059
4	-692.75293	1698.41121	68930.32692
5	-889.45198	928.24593	68986.12391
6	-1071.02884	0.00000	69051.12969
7	-1085.18054	-940.51102	69124.97738
8	-1199.92642	-1979.51219	69200.22003
9	-1264.65537	-3076.73466	69275.59568
10	-1208.55082	-4123.14986	69348.69289
11	-1588.14079	-5499.49262	69439.52473
12	4316.38548	-1757.72165	70178.86399
13	2438.15687	355.85803	69636.24594
14	913.43122	1147.47201	69321.79469
15	-23.46413	1127.15576	69167.27536
16	-412.09583	769.61898	69105.74350
17	-528.41262	312.07316	69083.85639
18	-360.07629	0.00000	69061.06230
19	-10.99447	-9.52876	69016.67922
20	175.95412	142.82770	68949.92479

21	184.80818	303.16337	68876.47776
22	165.95952	446.86325	68816.19158
23	18.06501	462.51907	68784.78047
24	-172.81584	312.70917	68781.01167

BLOCKING POINT THRUSTS

JOINT	THRUST
1	4610.98928
2	9219.57450
3	9212.86315
4	9201.81508
5	9186.91859
6	9168.00053
7	9145.59890
8	9120.06180
9	9091.68503
10	9060.94576
11	9435.18503
12	3209.11735
13	11067.47563
14	10579.51612
15	9971.78787
16	9481.60512
17	9124.51592
18	8825.83861
19	8776.65887
20	8795.66090
21	8975.92625
22	9074.46324
23	9126.51372
24	9157.75295

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25	4350.13932

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-68687.78000	-3138.14872	-4610.98855
LAST	-68645.96339	312.70917	4350.13863

"TOWNBROOK TUNNEL: PRECAST LINER(12.5 FT)"

"DEAD,ROCK(1715 PSF),AND EX.WATER LOAD"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00024	-.00024
2	.00015	.00001	-.00026	-.00024
3	.00029	.00004	-.00031	-.00022
4	.00042	.00007	-.00040	-.00021
5	.00053	.00009	-.00052	-.00019
6	.00060	.00010	-.00068	-.00017
7	.00066	.00022	-.00084	-.00013
8	.00061	.00022	-.00108	-.00017
9	.00053	.00024	-.00132	-.00020
10	.00041	.00026	-.00154	-.00022
11	.00023	.00028	-.00176	-.00023
12	.00001	.00031	-.00196	-.00025
13	-.00025	.00032	-.00212	-.00025
14	-.00053	.00031	-.00225	-.00024
15	-.00081	.00026	-.00235	-.00017
16	-.00103	.00017	-.00243	-.00003
17	-.00120	.00007	-.00251	.00022
18	-.00130	-.00001	-.00262	.00056
19	-.00130	-.00130	-.00266	.00096

20	-.00067	-.00126	-.00359	.00244
21	-.00021	-.00114	-.00454	.00383
22	.00007	-.00094	-.00541	.00503
23	.00017	-.00067	-.00612	.00596
24	.00012	-.00035	-.00658	.00654
25	0.00000	0.00000	-.00674	.00674

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	665.56769	554.54336	80225.83552
2	227.38755	751.60599	80155.98535
3	-89.75297	673.89402	80119.52797
4	-212.80708	489.26134	80102.05446
5	-309.52775	221.24516	80088.52454
6	-255.27712	0.00000	80062.50902
7	230.91404	200.13002	79995.16329
8	289.93801	451.18369	79887.30064
9	193.96208	619.46634	79770.14966
10	85.49041	693.48760	79661.71662
11	-189.08605	529.61849	79576.47636
12	-555.33078	48.21557	79527.24458
13	-852.34673	-690.66336	79473.49778
14	-1125.58806	-1666.14047	79314.65554
15	-1001.20747	-2533.02889	78986.35318
16	93.94393	-2451.52258	78430.28063
17	1067.72432	-1526.99354	77627.39488
18	1761.87587	0.00000	76644.61278
19	2303.61962	1996.51538	75563.21168
20	2384.60069	4061.31109	74483.98436
21	2182.86040	5955.17366	73491.92328
22	1808.01718	7520.63257	72666.24307

E - 72

23	1158.19870	8524.37129	72081.78236
24	373.22100	8847.90762	71775.30531

BLOCKING POINT THRUSTS

JOINT	THRUST
1	4610.98928
2	10891.79798
3	10774.99193
4	10660.10026
5	10539.18091
6	10372.37424
7	10092.41049
8	10350.13011
9	10503.67502
10	10593.62054
11	10664.82630
12	10736.70115
13	10751.20445
14	10623.32861
15	10205.58180
16	9260.02004
17	9201.58095
18	9351.05929
19	9518.21589
20	9687.92293
21	9847.66709
22	9986.47374
23	10093.72657
24	10160.68501
25	5091.87920

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-80098.56579	22.42130	-4610.98855
LAST	-71598.28888	8847.90762	5091.87840

"DEAD LOAD AND GROUT PRESSURE"

JOINT DISPLACEMENTS (GLOBAL COORDINATES)

JOINT	X	ROTATION	Y	RADIAL
1	0.00000	0.00000	-.00041	-.00041
2	.00003	-.00002	-.00041	-.00040
3	.00006	-.00001	-.00040	-.00037
4	.00009	.00001	-.00041	-.00034
5	.00011	.00003	-.00043	-.00032
6	.00012	.00004	-.00047	-.00030
7	.00012	.00010	-.00052	-.00028
8	.00008	.00010	-.00060	-.00030
9	.00002	.00009	-.00067	-.00031
10	-.00003	.00009	-.00073	-.00031
11	-.00009	.00007	-.00078	-.00029
12	-.00013	.00004	-.00082	-.00024
13	-.00015	.00002	-.00085	-.00015
14	-.00016	-.00000	-.00088	-.00005
15	-.00016	-.00002	-.00091	.00008
16	-.00014	-.00004	-.00094	.00023
17	-.00011	-.00006	-.00099	.00039

18	-.00008	-.00007	-.00104	.00057
19	-.00004	-.00012	-.00109	.00074
20	.00001	-.00012	-.00118	.00094
21	.00004	-.00011	-.00128	.00113
22	.00005	-.00009	-.00137	.00128
23	.00005	-.00006	-.00144	.00140
24	.00003	-.00003	-.00148	.00147
25	0.00000	0.00000	-.00150	.00150

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	1117.90267	5.84217	16224.62501
2	497.78354	437.24018	16106.31461
3	80.83195	507.22795	16043.18163
4	-134.55720	390.48533	16009.56688
5	-230.48415	190.91200	15984.49759
6	-220.27811	0.00000	15954.63647
7	-53.13252	-46.04922	15903.78530
8	-78.74963	-114.23753	15834.93663
9	-191.51866	-280.40025	15768.29061
10	-275.77769	-519.18042	15708.84427
11	-228.75283	-717.42626	15647.69686
12	146.68741	-590.26647	15556.39133
13	152.43022	-458.12836	15439.15964
14	-56.23330	-506.86226	15336.43553
15	45.79467	-467.21127	15242.47496
16	138.53422	-347.01816	15140.18592
17	188.64099	-183.67631	15034.60751
18	211.92942	0.00000	14930.36395
19	241.61102	209.40094	14831.36639
20	221.33250	401.04997	14742.54908

21	195.76709	562.22259	14566.94292
22	152.67425	694.41451	14607.35985
23	93.83608	775.73640	14566.46480
24	26.49420	798.70359	14545.61858

BLOCKING POINT THRUSTS

JOINT	THRUST
1	-48.65999
2	2726.30489
3	2513.92852
4	2322.93790
5	2181.38699
6	2069.18695
7	1938.60898
8	2091.89052
9	2172.70635
10	2154.24374
11	1999.19432
12	1659.40768
13	2032.33089
14	2214.33941
15	1893.57102
16	1905.63588
17	1917.06972
18	1927.60539
19	1937.21095
20	1945.62461
21	1952.66183
22	1958.31083
23	1962.40407
24	1964.88070

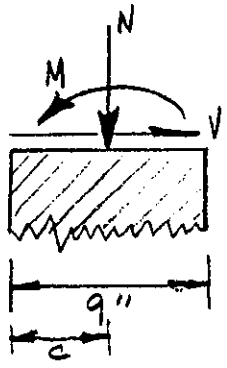
25	982.85985

END JOINT REACTIONS

JOINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-16262.99199	963.24083	48.65998
LAST	-14512.94903	798.70359	982.85969

SUBJECT Townbrook Tunnel
COMPUTATION Stress computation - TBM Precast Lining
COMPUTED BY ENestorides CHECKED BY DATE 2-26-86

A. Problem Definition:



N = thrust
M = moment (in lb)
V = shear

D = 9" (thickness of liner)
b = 12" (1 foot analysed)
c = 4.5"

R = 79.5"
I = 729 in⁴

$f_c' = 5000$ psi

① max. compressive stress = $.45 f_c' = 2250$ psi

② max. tensile stress = $1.6\sqrt{f_c'} = 113$ psi

③ Prin. Diag. ten. stress $(f_t) \Rightarrow 2\sqrt{f_c'} \Rightarrow 141.42$ psi } This restriction controls. Shear + Diagnal tension req. are met.

- Curved Beam Correction factor (inside face of liner)

$$F^* = 1 + .5\frac{I}{bc^2}\left[\frac{1}{R-c} + \frac{1}{R}\right]$$

so for our problem: $F^* = 1 + .5\left(\frac{729}{12(4.5)^2}\right)\left[\frac{1}{79.5-4.5} + \frac{1}{79.5}\right]$

$$F^* = 1 + .5(3)(.0259)$$

$$F^* = 1.039$$

B. Formulas Used: when N falls inside kern:

Compressive stress in
a) outside fiber ⇒ $f_c = \left(\frac{N}{bD} + \frac{6M}{bD^2}\right)$
b) inside fiber ⇒ $f_c = \left(\frac{N}{bD} - \frac{6N}{bD^2}\right)F^*$

Principal Tensile Stress

$$f_t = \frac{N}{2bD} - \sqrt{\left(\frac{N}{2bD}\right)^2 + \left(\frac{V}{bD}\right)^2}$$

SUBJECT Townbrook Tunnel — TBM, Precast Liner
COMPUTATION Stress Computation (cont.)
COMPUTED BY ENestorides. CHECKED BY DATE 2-26-86

C. Steel Area Requirements:

use: $f_c' = 5000$ psi $f_s = 20{,}000$ psi

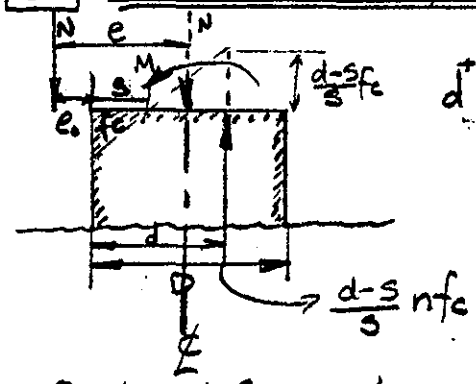
① Minimum Transverse Steel: .002 bD in each face

$\Rightarrow .002(12)(9) \Rightarrow \underline{.216 \text{ in}^2}$ } try #5 at 12" $\underline{.31 \text{ in}^2/\text{ft}}$

② Min. Longitudinal Steel: .001bD

$\Rightarrow .001(12)(9) \Rightarrow \underline{.108 \text{ in}^2}$ } try #4 at 18" $= \underline{.13 \text{ in}^2/\text{ft}}$

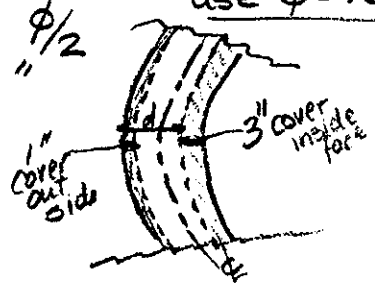
D. Calculations of Transverse Stresses:



$d' = 9'' - 3''$ (cover) $- \phi/2$

$d = \underline{5.69''}$

use $\phi = .625$



① Load Case 1: Rock + Dead Load

$e = M/N$

$M_{MAX} = 4397$ ft. lb
$N = 8137$ lb } node 25

$e = \frac{(4397)12}{8137} = 6.48$

$n = \frac{E_s}{E_c} = \frac{30{,}000 \text{ ksi}}{4287 \text{ ksi}} = 7.0$

$e_o = e - D/2 \Rightarrow 6.48 - 4.5 \Rightarrow 1.98$

ΣM about $N \Rightarrow 0$ — neglecting compression steel —

$$\tfrac{1}{2} b f_c s\left(s/3 + e_o\right) - \frac{d-s}{s} n f_c A_s (d + e_o) = 0$$

$$2s^2 + 11.94s - \left(\frac{5.69 - s}{s}\right)(7.0)(.31)(7.67) = 0$$

$$s^3 + 5.95s^2 - \tfrac{1}{2}(5.69 - s)(16.654) = 0$$

$$s^3 + 5.95s^2 + 8.33s - 47.38 = 0$$

trial & error $s \Rightarrow \underline{1.975}$

SUBJECT: Townbrook Tunnel - TBM Precast Liner
COMPUTATION: Stress Computation (cont.)
COMPUTED BY: ENestorides CHECKED BY: DATE: 2-26-86

D. Calculations of transverse stress (cont.)

① LC #1

$$f_c = \frac{N F^*}{\frac{Sb}{2} - \left(\frac{d-s}{s}\right) n A_s} = \frac{8137}{\frac{(1.975)(12)}{2} - \left(\frac{5.69-1.975}{1.975}\right)(7)(.31)}$$

$f_c = \underline{1047.5}$ < 2250 psi OK ✓

$$f_s = \left(\frac{d-s}{s}\right) n f_c F^* = \left(\frac{5.69-1.975}{1.975}\right)(7)(1047.5)(1.04)$$

$f_s = \underline{14{,}343.89}$ < 20,000 psi ✓ OK

— its ok to use #5 at 12" for LC #1 — (transverse steel)

Max. shear occurs at node 22 $V = 1216$, $N = 9636$

$$f_t = \frac{9636}{2(108)} - \sqrt{\left(\frac{9636}{2(108)}\right)^2 + \left(\frac{1216}{108}\right)^2}$$

$= 44.61 - 46.01$

$= \underline{-1.40 \text{ psi}}$ < 141.42 ✓ OK.

② Load Case #2: Dead + Ex. Water:

$M_{max} = -5499$ ft. lb
$N = 69440$ lb
$V = -1588$ lb

$d = 4.69''$ $e = M/N = -.95''$

N falls in kern. $e < D/2$

$$f_c \text{ outside} = \left(\frac{69440}{108}\right) + \frac{6(-5499)\cancel{12}}{\cancel{12}(9)^2} = 236 \text{ psi}$$

$$f_c \text{ inside} = \left(\frac{69440}{108} - \frac{6(-5499)\cancel{12}}{\cancel{12}(9)^2}\right)1.04 = 1092 \text{ psi}$$

< 2250 ✓ OK

SUBJECT Townbrook Tunnel — TBM Precast Liner
COMPUTATION Stress Computation (cont.)
COMPUTED BY E Nestoridis CHECKED BY DATE 2-26-86

Let us also check node 1 $M = +3138$ ft. lb $V = 85$ lb
$N = 68843$ lb

N falls in kern $e < D/2$

$$f_c \text{ outside} = \left(\frac{68843}{108}\right) + \frac{6(3138)\cancel{12}}{\cancel{12}(81)} \Rightarrow 870 \text{ psi}$$

$$f_c \text{ inside} = \left(\frac{68843}{108} - \frac{6(3138)\cancel{12}}{\cancel{12}(81)}\right)1.04 = 421 \text{ psi}$$

< 2250 psi ✓OK

Max shear occurs Node 12 / $V = 4316.3$ $N = 70179$

$$f_t = \frac{70179}{216} - \sqrt{\left(\frac{70179}{216}\right)^2 + \left(\frac{4316.3}{1.08}\right)^2}$$

$f_t = -2.45$ psi < 141.42

③ Load Case #3: Dead + Rock + Ex. Water

ⓐ node 25 ⇒ $V = 373$ lb, $N = 71775$ lb, $M_{max} = 8848$ ft.lb

ⓑ node 16 ⇒ $V = -1001$ lb, $M = -2533$ ft lb, $N = 78986$ lb

ⓒ max $V = 2385$, $N = 74484$ } node 21

$e = M/N < D/2$ "N" falls inside kern.

ⓐ $f_c \text{ outside} = \left(\frac{71775}{108} + \frac{6(8848)\cancel{12}}{\cancel{12}(81)}\right) \Rightarrow 1320$ psi

$f_c \text{ inside} = \left(\frac{71775}{108} - \frac{6(8848)}{81}\right)1.04 \Rightarrow 10$ psi

< 2250 psi OK ✓

ⓑ $f_c \text{ out.} \Rightarrow \left(\frac{78986}{108} + \frac{6(-2533)}{81}\right) \Rightarrow 544$ psi

$f_c \text{ inside} \Rightarrow \left(\frac{78986}{108} - \frac{6(-2533)}{81}\right)1.04 \Rightarrow 919$ psi

< 2250 psi ✓OK

ⓒ $$f_t = \frac{74484}{216} - \sqrt{\left(\frac{74484}{216}\right)^2 + \left(\frac{2385}{108}\right)^2} \Rightarrow -.71$$

< 141.42 psi ✓OK

④ Load Case #4: Dead + Grout pressure

node 1 $M_{max} \Rightarrow -963$ ft lb $N = 16{,}225$ lb $V_{max} = 1118$

node 25 $M = +799$ ft lb $N = 14546$ lb $V = 26$

e (in both cases) $< D/2$

SUBJECT Townbrook Tunnel – TBM Precast Liner
COMPUTATION Stress Computation (cont.)
COMPUTED BY E Nestorides CHECKED BY DATE 2-26-86

N falls inside kern:

node 1

$$f_{c\,outside} = \left(\frac{16225}{108} + \frac{6(-963)(12)}{12(81)}\right) = 79 \text{ psi}$$

$$f_{c\,inside} = \left(\frac{16225}{108} - \frac{6(-963)}{81}\right)1.04 = 230 \text{ psi}$$

$$f_t = \frac{16225}{216} - \sqrt{\left(\frac{16225}{216}\right)^2 + \left(\frac{1118}{108}\right)^2} = -.71 \text{ psi}$$

node 25

$$f_{c\,outside} = \left(\frac{14546}{108} + \frac{6(799)}{81}\right) = 194 \text{ psi}$$

$$f_{c\,inside} = \left(\frac{14546}{108} - \frac{6(799)}{81}\right)1.04 = 79 \text{ psi}$$

$$f_t = \frac{14546}{216} - \sqrt{\left(\frac{14546}{216}\right)^2 + \left(\frac{26}{108}\right)^2} = -.0004 \sim 0 \text{ psi}$$

[E] Reinforcement Developmental Length:

$$l_d = .04\, A_b f_y / \sqrt{f_c'} \quad \text{but not} < .0004\, d_b f_y$$

– for #5 bars, grade 60:

$$l_d = .04(.31)(60{,}000)/\sqrt{5000} = 10.5''$$

$$.0004(.625)(60{,}000) \Rightarrow \underline{\underline{15''}} \text{ controls} \qquad \underline{\underline{l_d = 15''}}$$

– for #4 bars, grade

$$l_d = .04(.2)(60{,}000)/\sqrt{5000} = 6.8''$$

$$.0004(.5)(60000) \Rightarrow 12'' \text{ controls} \qquad \underline{\underline{l_d = 12''}}$$

APPENDIX F

DESIGN COMPUTATIONS FOR INTAKE AND OUTLET SHAFTS AND APPURTENANT STRUCTURES

Subject	Page
Physical and Mechanical Properties	1
Overflow Structure	2
Grit Basin (Intake Structure)	23
Box Culverts	54
Retaining Walls	70
Intake Shaft	76
Outlet Structure	89
Town River Culvert	

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

DC-1

PHYSICAL AND MECHANICAL PROPERTIES

BASED ON WORKING STRESS METHOD:-

CAST-IN-PLACE CONCRETE

f'_c =	4,000 PSI	@ 28 DAYS	(TYPE II CEMENT) (5" ± A.C)
f_c =	1,400 PSI	($0.35 f'_c$)	
v =	70 PSI	($1.1\sqrt{f'_c}$)	
f_t =	126 PSI	($2\sqrt{f'_c}$)	
n =	8		
R =	221		
k =	0.359		
j =	0.880		
p =	0.0126		
a =	1.44		

REINFORCING STEEL

f_s = 20,000 PSI (INTERMEDIATE GRADE STEEL)

PRECAST CONCRETE

f'_c =	5000 PSI	
f_c =	1750 PSI	($0.35 f'_c$)
v =	78 PSI	($1.1\sqrt{f'_c}$)
n =	7	
R =	292	
a =	1.44	

DESIGN LOADS

UNIT WEIGHT OF SOIL (DRY) 125 PCF

LATERAL PRESSURE OF SOIL (DRY)	62.5 PCF	AT REST
(SATURATED)	93.7 PCF	AT REST
(MIN., RETAINING WALLS)	33.8 PCF	ACTIVE

STRUCTURES BURIED UNDER DRIVES H-20 (AASHTO)

STRUCTURES WITH FRAMED SLABS H-10 (AASHTO)

OVERFLOW

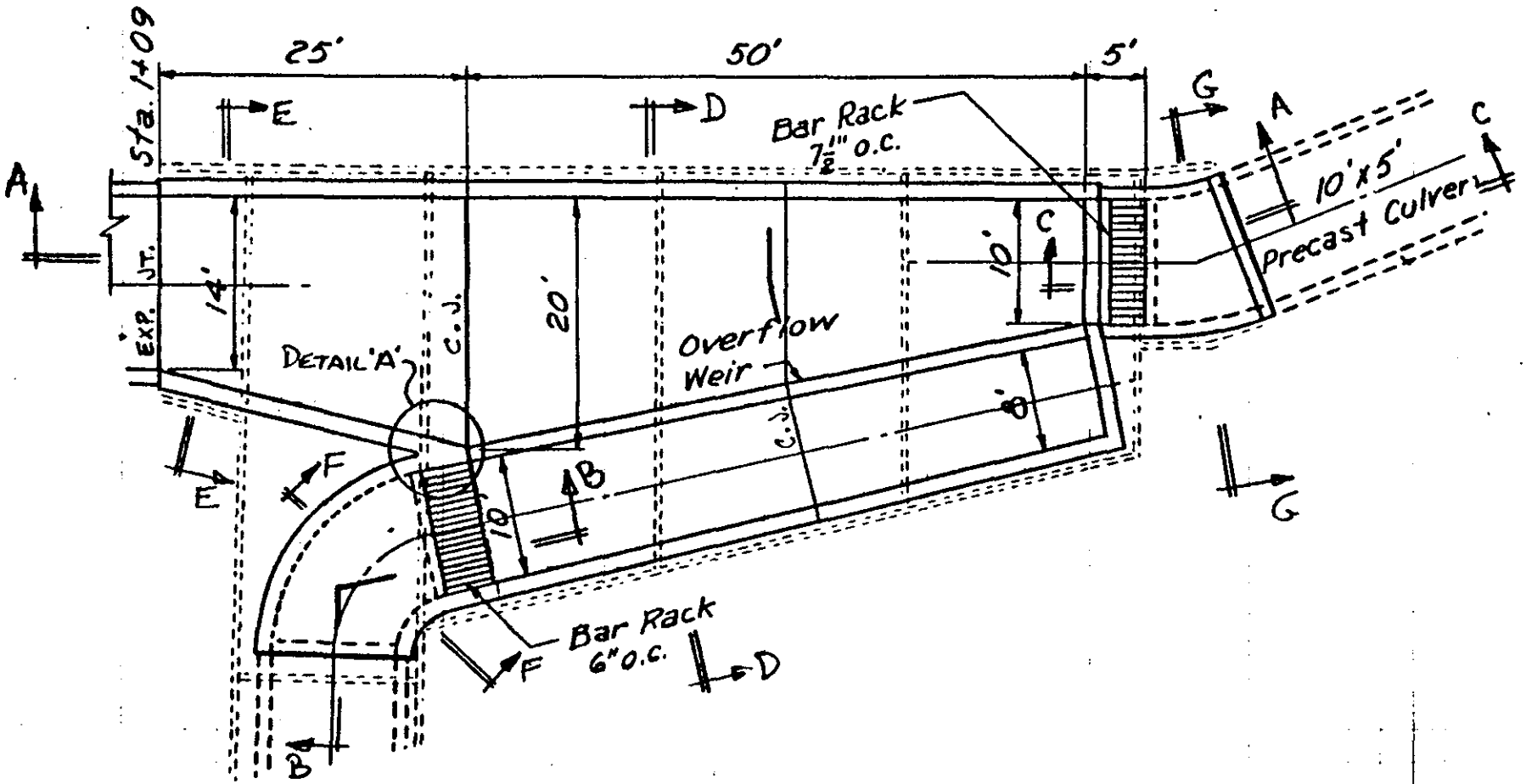
STRUCTURE

METCALF & EDDY, ENGINEERS

Project MDC- TOWN BROOK Acct. No. 6041 Page 2 of 12

Subject OVERFLOW STRUCTURE Comptd. By Date

Detail GENERAL ARRANGEMENT Ck'd. By Date

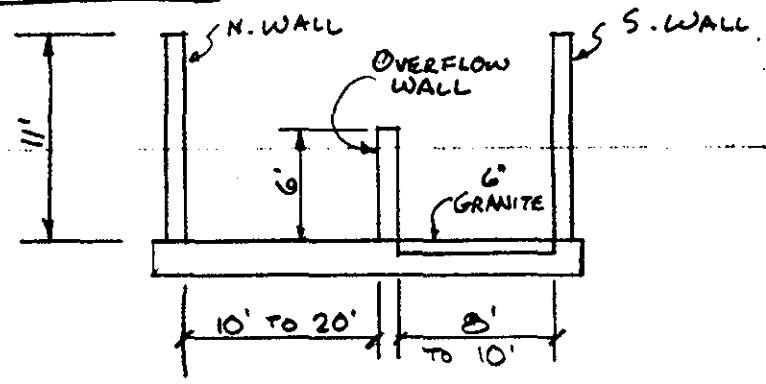


OF-0

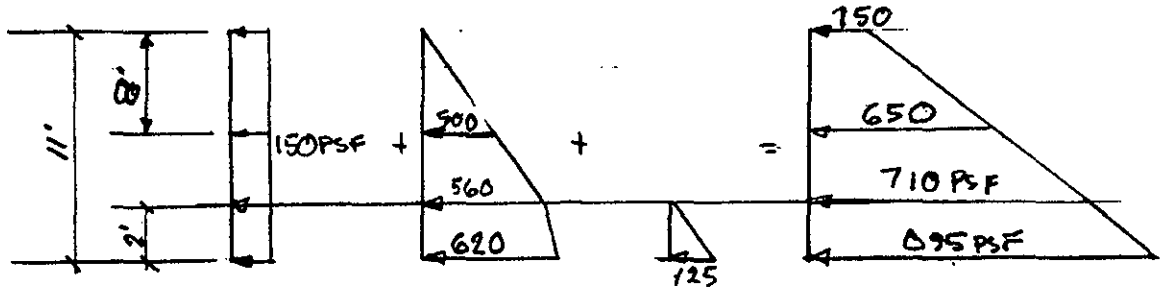
Project TOWN BROOK Acct. No. 6041 Page 3 of 121
Subject RELIEF TUNNEL Comptd. By RITTMEYER Date 6/18/84
Detail INTAKE STRUCTURE Ck'd. By [signature] Date 8/15/84

OF-1

OVERFLOW WEIR



- EARTH PRES. @ N. & S. WALL



$R_{BASE} = 5.48^{k}/_{1}$ $M_{BASE} = 22.92^{'k}/_{1}$

@ S. WALL; $M = 22.92 + 5.48 \times .5 = 25.66^{'k}/_{1}$

TRY 18" WALL $d = 18 - 2 - .5 = 15.5"$ $v = 29.5$ PSI

RQ'D $A_s = 1.150"/_{1} \rightarrow$ #7@6

CHECK MOMENT @ 8' BELOW TOP

$M = .15(8)(4) + .37\left(\frac{8^2}{6}\right) = \cancel{8.75}\ 10.1^{'k}/_{1}$ (0.150)

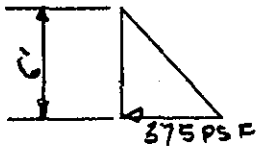
$A_s = \cancel{.39\square"/_{1}\ USE}$ #7@6

J.D.Y. 8/15/84

$A_s = \frac{10.1^{'k}}{15.5 \times 1.44} = 0.45 in^2$

USE #6@12

- OVERFLOW WALL



$M = 2.25^{'k}/_{1}$

USE 12" WALL w/ #5@12 E.F.

- TRANSVERSE REINFORCEMENT

USE .0018

N. & S. WALL: $.0018 \times 9" \times 12" = .20\square"/_{1}$ E.F. USE #5@12

CENTER WALL: $.0018 \times 6 \times 12 = .13\square"/_{1}$ E.F. "

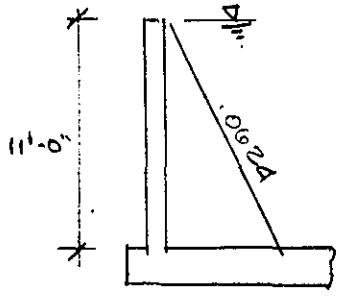
NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

OF-2

HOLD

OVERFLOW WEIR

N&S WALLS INTERIOR LOADING



$$-M = \frac{.0624 \times 11^3}{6} = 13.8'k$$

18" WALL

$d = 18" - 4" - 0.5" = 13.5"$

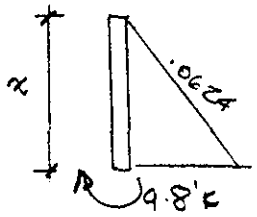
$$A_{s\,REQ'D} = \frac{13.8'k}{1.44 \times 13.5} = 0.71$$

USE #6@6

$A_s = 0.88\ in^2$

CAPACITY OF #6@12"

$A_s = 0.44\ in^2$ — $M = 0.44\ in^2 \times 1.44 \times 13.5" = 8.6'k$



$$\frac{.0624 \times \bar{x}^3}{6} = 8.6'k$$

$$x = \sqrt[3]{\frac{8.6'k \times 6}{.0624\ kcf}} = 9.4'$$

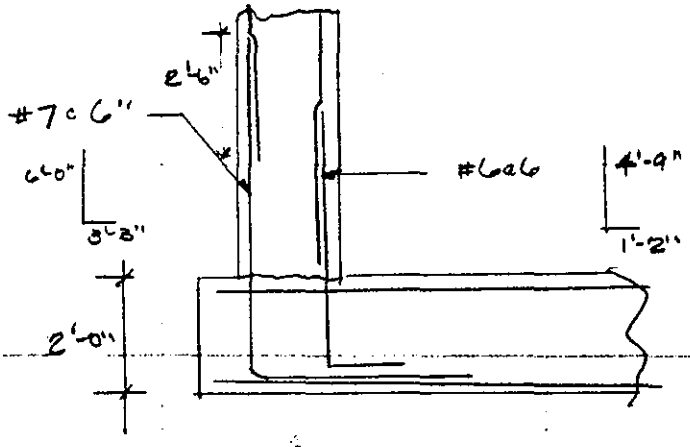
#7@12" BAR CUT-OFF : $11' - 9.4' = [1.2' + d]$

$= 1.6' + 13.5"/12 = 2.7'$

SAY 2'-9"

SPLICE W/#6@12 CLASS 'C' SPLICE

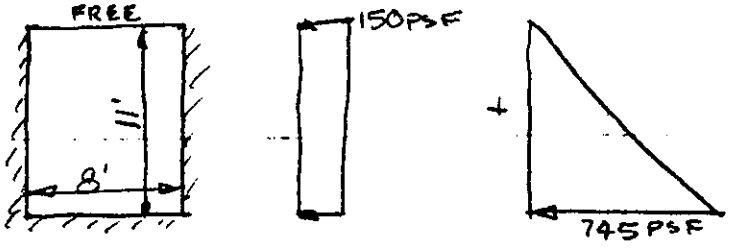
$= 1.7\ell_d = 1.7(18") = 30.6"$ SAY 2'-6"



NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC - TOWN BROOK Acct. No. 6041 Page 5 of 121
Subject RELIEF TUNNEL Comptd. By RITTMEYER Date 6-18-84
Detail INTAKE STRUCTURE Ck'd. By [signature] Date 8/15/84

- END WALL 8' WIDTH OF- 3



$\frac{a}{b} = \frac{4}{11} = .36$ SAY $\frac{3}{8}$

USING BUREAU OF RECLAMATION TABLES FOR RECT. PLATES:

FOR 150 PSF : $pb^2 = 18.15$
FOR 745 PSF : $pb^2 = 90.145$

$M_{BASE} = .0200 \times 90.145 + .0288(18.15) = 2.33$ 'K/'

$M_{CORNER} = .0208 \times 90.145 + .0379(18.15) = 2.56$ 'K/'

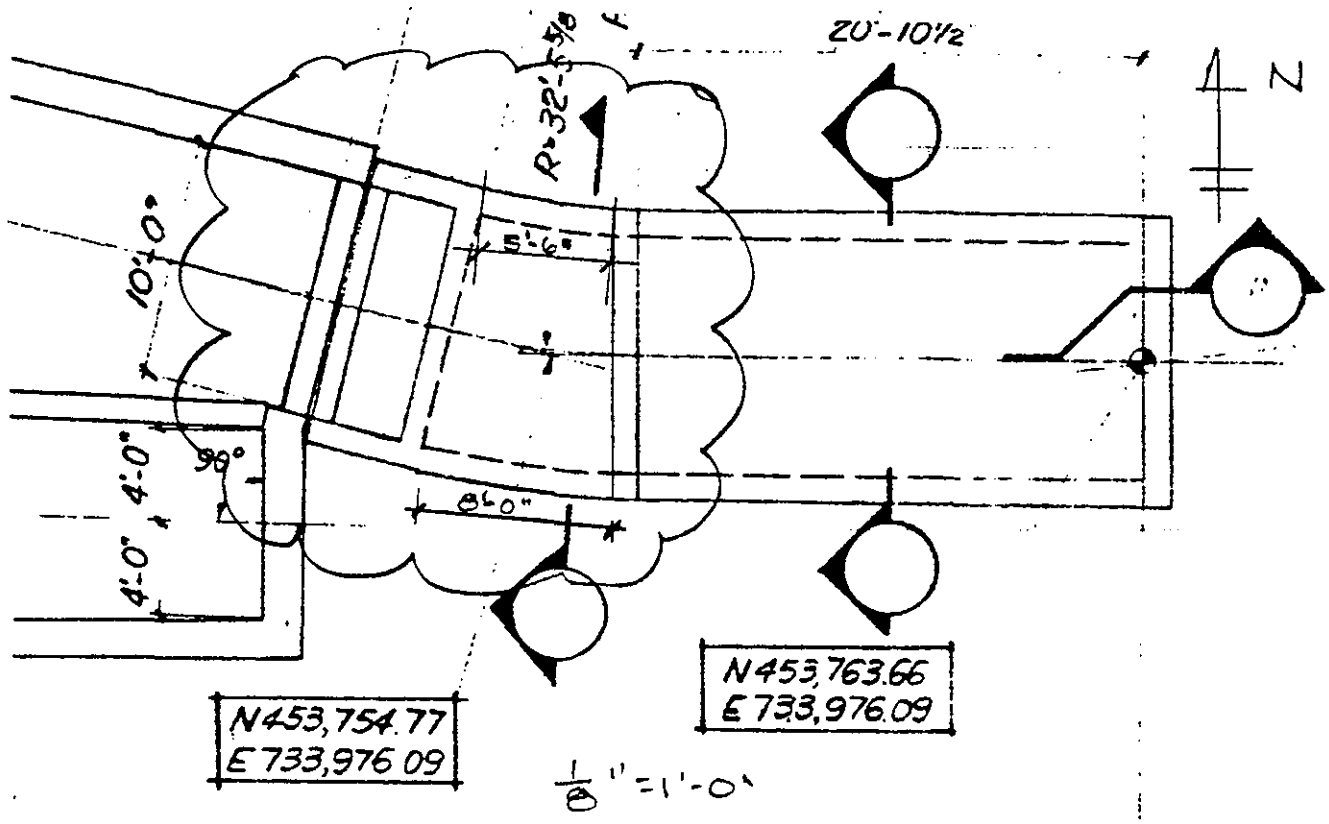
FOR #5@12 IN 18" WALL w/ d = 14"

$M_w = 1.44 \times .31 \times 14 = 6.25$ 'K/'

USE TYPICAL WALL STEEL
SEE COMPS. ON SHEET OF-1

[signature] 8/15/84

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS



SLAB DESIGN

LOADING H-10 TRUCK LOAD

AXLE LOAD = 16K

WHEEL LOAD = 8K × 1.3 (IMPACT) = 10.4K

D.L = 12" SLAB = 150 PSF

WHEEL LOAD DISTRIBUTION

ASSUME TWO WAY SLAB ACTION : 6'-6" × 11'-0" (c/c)

$$\% \text{ E-W} = \frac{11^3}{6.5^3 + 11^3} = 0.83$$

$$\% \text{ N-S} = 1.0 - 0.83 = 0.17$$

ASSUME 4'-0" WHEEL LOAD SPREAD = E (CONSERVATIVE).

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC TOWN BROOK Acct. No. 6041 Page 12 of 121
Subject RELIEF TUNNEL Comptd. By J. FINITZ Date 8/6/84
Detail INTAKE STRUCTURE Ck'd. By J. Howard Date 9-11-84

OF - 10

ANALYSE SLAB AS SIMPLE SPAN

UNIFORM LOAD DISTRIBUTION ACI 318-63 METHOD 3, CASE I

$m = \frac{6.5'}{11.0'} = 0.59$ % N-S = 0.11
% E-W = 0.89

$+M_{E-W} = \left[\frac{10.4^k \times 0.83}{4.0'}\right] \times \frac{8.0'}{4} + (0.150\,KSF \times 0.89) \times \frac{\overline{8}^2}{8} = 5.4'^k$

$V_{max} = \frac{10.4^k}{4.0'} + (0.150 \times .89) \times \frac{8'}{2} = 3.1^k$

12" SLAB
$d = 12.0" - 2" - 0.5" = 9.5"$
$V_c = 12" \times 9.5" \times 0.07\,KSI = 8.0^k$

$A_{S\,REQD} = \frac{5.4'^k}{1.44 \times 9.5"} = 0.39\,IN^2$

USE #6 @ 12 BEW
#5 @ 12 TEW

BEAM DESIGN (WEST BEAM).

LOADING
SLAB : $0.150\,KSF \times \frac{6.5}{3} \times \left(\frac{3 - (6.5/11)^2}{2}\right) =$ 0.43 kli
BEAM : $(1' \times 2.0' + 0.58' \times 2.0'/2)\,0.150\,KCF =$ 0.39 kli
RACK: (ASSUMED) 0.4 kli
1.22 kli

HIO - LOAD = 10.4^k

ASSUME SIMPLE SUPPORT SPAN = 11'-0" CENTER to CENTER

$+M = \frac{10.4^k \times \overline{11}'}{4} + \frac{1.22^{kli} \times \overline{11}^2}{8} = 47'^k$

17^k

24" DEEP BEAM $d = 24 - 2' - 1" = 21" = 1.75'$

$A_{S\,REQD} = \frac{47'^k}{1.44 \times 21} = 1.6\,IN^2$

3 @ - #7 B.C.
2 - #5 T.C.
#4 @ 12

$V_c = 21" \times 12" \times .07 = 17^k$
$V_{cd} = 10.4^k \times \frac{8.25'}{10'} + \left(1.22^{kli} \times \frac{10'}{2} - 1.22 \times 1.75\right) = 12.5^k < V_c$ OK

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

OF - 11

EAST BEAM

LOADING:

SLAB : 0.43 kli

BEAM : 6.31' x 1.0' x .150 KCF = 0.95 kli

1.38 kli

$$+M = \frac{10.4^k \times 11'}{4} + \frac{1.38 \text{ kli} \times 11^2}{8} = 49.5'^k$$

DEPTH = 6.31' = 75.7" SAY 75"

d = 75 - 4 = 71"

$$A_s = \frac{49.5'^k}{1.44 \times 71} = 0.48 \text{ IN}^2 \times 4/3 = 0.64$$

LESS $A_{s\,MIN}$

2-#5 BL.
2-#5 T.G.

MIN A_s = .0033 x 12' x 71" = 2.8 IN²

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

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Subject RELIEF TUNNEL Comptd. By J. FINITZ Date 8/8/84
Detail INTAKE STRUCTURE Ck'd. By J. Howard Date 9-11-84

OF - 14

BAR RACK AREA @ CONNECTING CULVERT TO 12' ϕ SHAFT

ROOF SLAB ASSUME 10'-0" SIMPLE SPAN

L.L. H10 LOADING W/IMPACT = 10.4^K

SPREAD = 4 + .06(10') = 4.6'

18" SLAB (DL) = 1.5 x .150 KCF = 0.23 KSF

$$+M = \left(\frac{10.4^k}{4.6'}\right) \times \frac{\overline{10}'}{4} + \frac{0.23\,kl/LF \times \overline{10}^2}{8} = 8.5'k$$

18" SLAB $d = 18.0 - 2 - 0.5 = 15.5"$

$$As_{REQD} = \frac{8.5'k}{1.44 \times 15.5} = 0.38\,IN^2$$

USE # 6 @ 12" B
5 @ 12 T

BEAM @ BAR RACK SPAN = 10' SIMPLE SPAN

$$w_{DL} = \underbrace{0.39\,kl}_{\text{BEAM}} + \underbrace{0.4\,kl}_{\text{RACK}} + 0.23\,KSF \times 10'/3 = 1.56\,kl$$

$$+M = \frac{1.56\,kl \times \overline{10}^2}{8} + \frac{10.4^k \times 10'}{4} = 45.5'k$$

$$V_{MAX} = 10.4^k + \frac{1.56\,kl \times 10'}{2} = 18.2^k \cong V_c \quad OK$$

$d = 24" - 3" = 21"$
$V_c = 12 \times 21 \times .07 = 17.6k$

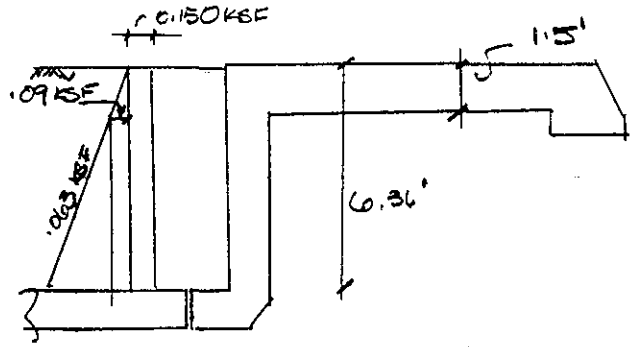
$$As_{REQD} = \frac{45.5'k}{1.44 \times 21"} = 1.51\,IN^2$$

USE 3 - #7 B.C
2 - 2 - #5 TC
#4 ⬠ @ 12

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

BAR RACK AREA @ CONNECTING CULVERT TO 12' ϕ SHAFT

REF SECTION 8 OF SHEET 18

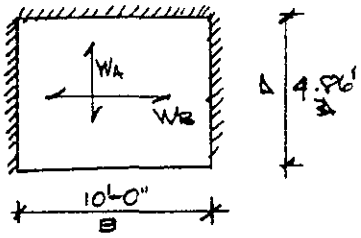


12" WALL

$d_{OF} = 12 - 2 - 0.5 = 9.5"$

$d_{IF} = 12 - 3 - 0.5 = 8.5"$

LOAD DISTRIBUTION REF ACI 318-63 METHOD 3



$m = \frac{A}{B} = \frac{4.86}{10.0'} = 0.44$ SAY 0.5

$W_A = 0.89$

$W_B = 0.11$

NEG MOMENT @ TOP (VERTICLE)

$$-M = \frac{(.09\,KSF + .150\,KSF)0.89 \times \overline{4.86}^2}{2} + \frac{(.063 \times 0.89) \times \overline{4.86}^3}{3} = 4.7'k$$

$$A_{s\,REQD} = \frac{4.7}{1.44 \times 9.5} = 0.34$$

SAY #5@20c

NEG MOM @ SIDES HORZ

PRESSURE @ BOTTOM $= [.063 \times 6.36' + .150 \times 6.36] \times 0.11 = 0.15\,KSF$ (WB)

$$-M = \frac{wL^2}{12} = \frac{.150 \times \overline{10}^2}{12} = 1.3'k$$

POS MOM. (HORZ)

ASSUME WALLS RELAX SO $+M = wL^2/8$

$$+M = \frac{.150 \times \overline{10}^2}{8} = 1.8'k$$

#5@12" EW EF OS

NONREPRODUCIBLE GRID FORM 145
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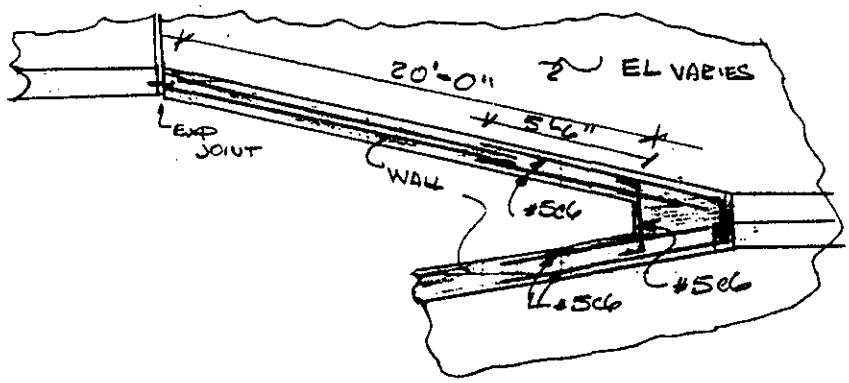
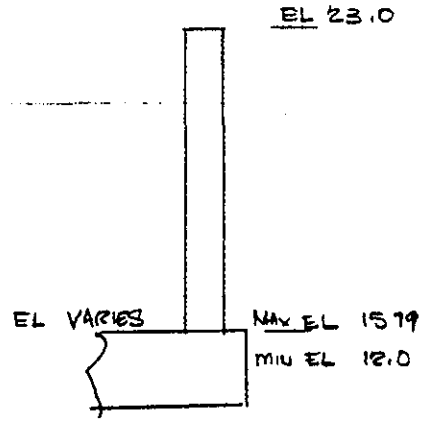
Subject RELIEF TUNNEL Comptd. By J. D. FINITZ Date 8/14/84
Detail STRUCTURAL Ck'd. By J. Howard Date 9-11-84

OF - 16

SOUTH WALL @ FORK



PLAN

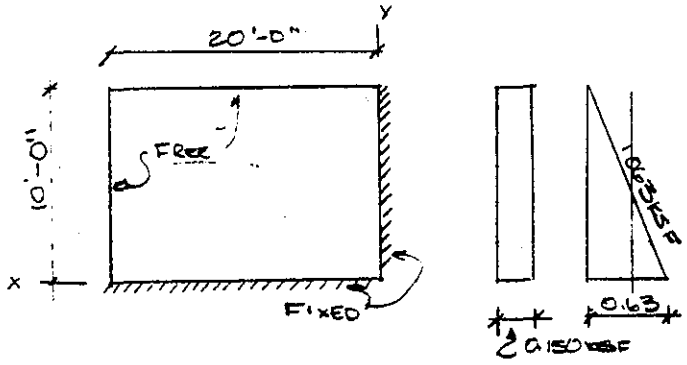


USE AVG EL FOR ANALYSIS

$\frac{15.79+12}{2} = 13.9$ SAY 13

$23 - 13 = 10'$

EXTERIOR LOADING



VERTICAL MOMENTS
REF TYPICAL WALL
STEEL, SECTION 3

HORIZONTAL MOMENTS

TAKE 2/3 OF TRIAN. LOAD AS A UNIFORM LOAD.

TOTAL UNIFORM LOAD = 0.150 KSF + 0.63 KSF × 2/3 = 0.57 KSF

LOAD DISTRIBUTION - REF. ACI 318-63, METHOD 4, CASE 4

$m = \frac{10}{20} = 0.5 \rightarrow W_B$ - (HORIZONTAL) = 0.06

$-M = \frac{0.57\,KSF \times (.06) \times \overline{20}^2}{2} = 6.8'K$ 12" WALL ; d = 9.5"

$A_S = \frac{6.8'k}{1.44 \times 9.5"} = 0.50\,IN^2$ USE #5c6" OF

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

OF-17

SOUTH WALL @ FORK CON'T

DETERMINE CUT-OFF OF #5@6

CAP #5@12" = 0.31 x 9.5" x 1.44 = 4.2 'K

$L = \sqrt{\frac{4.2'k \times 2}{(0.57 \times .06)}} = 15.7'$ 20' - 15.7' = 4.3' + 1.0' = 5.3'
SAY 5'-6"

INTERIOR

UNIFORM (TL) = (.063 KCF x 10') x $\frac{2}{3}$ = 0.42 KSF

HORIZONTAL LOAD DIST.

$-M = \frac{wL^2}{2} = \frac{(0.42 KSF \times .06) \times \overline{20}^2}{2} = 5.04 'K$

12" WALL

d = 12.0 - 3.0 - 0.5" = 8.5"

$A_s = \frac{5.04'k}{1.44 \times 8.5"} = 0.42 IN^2$ USE #5@6" T.F

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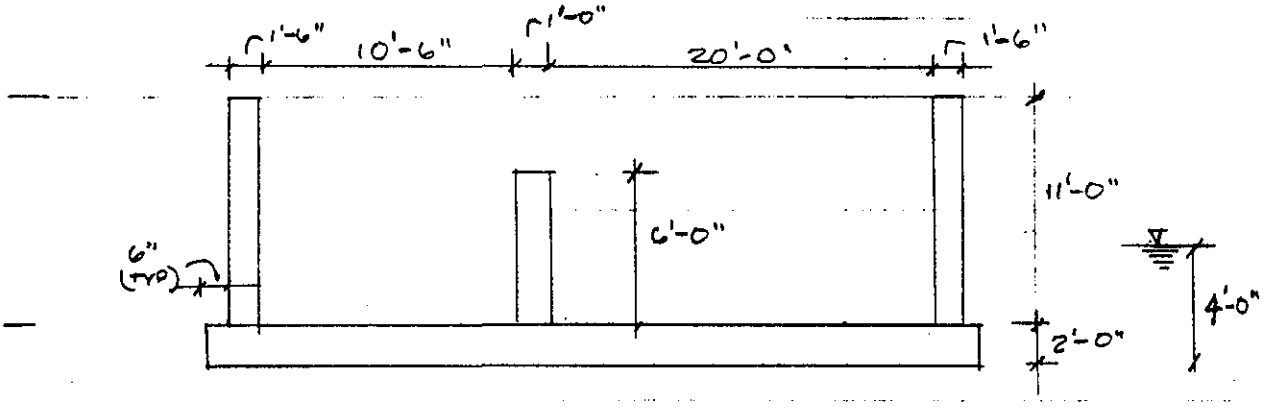
CHECK FLOTATION

1'-0" WIDE STRIP

SECTION 1-1



DEAD LOAD

WALLS: $1.5' \times 11.0' \times 1.0' \times 0.150\,KCF \times 2 =$ 5.0K

C/WALL: $1.0' \times 6.0' \times 1.0 \times .150 =$ 0.9K

MAT: $2.0' \times 1.0' (\underset{\text{LIP}}{1.0'} + \underset{\text{O.WALL}}{3.0'} + \underset{\text{C/WALL}}{1.0'} + 30.5') \times .150 =$ 10.7K

16.6K/LF ↓

UPLIFT

WALLS: $(3.0' + 10.5' + 1.0' + 20.0') \times 2.0' \times .063 =$ 4.3K/LF ↑

MAT: $(35.5')(2.0') \times .063 =$ 4.5K/LF ↑

8.8 K/LF ↑

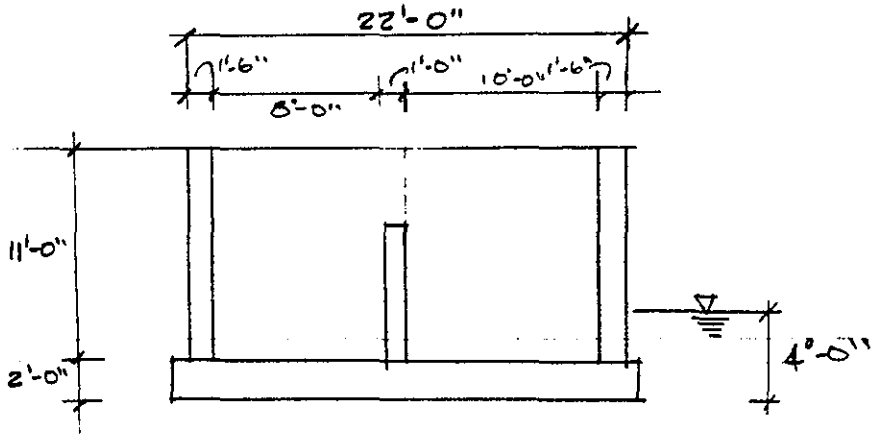
$$F.S. = \frac{16.6^K}{8.8^K} = 1.9 > 1.15 \text{ OK}$$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

CHECK FLOTATION

SECTION E-2 1'-0" WIDE STRIP



D.L.

O. WALLS :	1.5' x 11.0' x 0.150 KCF =	2.5 K/LF
I. WALLS :	1.0' x 6.0' x 0.150 KCF =	0.9 K/LF.
MAT :	23.0' x 2.0' x .150 KCF =	6.9 K/LF.
		10.3 K/LF

UPLIFT

WALLS :	22.0' x 2.0' x .063 KCF =	2.8 K/LF.
SLAB :	23.0' x 2.0' x .063 =	2.9 K/LF.
		5.7 K/LF.

$$F.S. = \frac{10.3}{5.7} = 1.8 > 1.15 \quad OK$$

STRUCTURE WILL NOT FLOAT

GRIT BASIN
(INTAKE STRUCTURE)

NONREPRODUCIBLE GRID FORM 145
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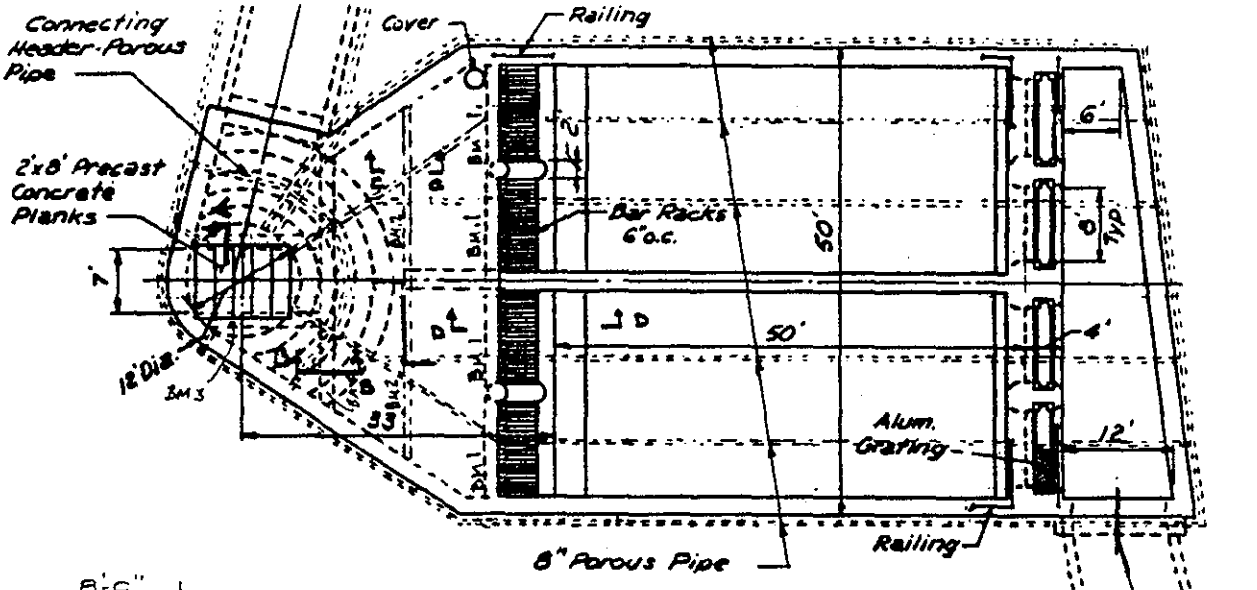
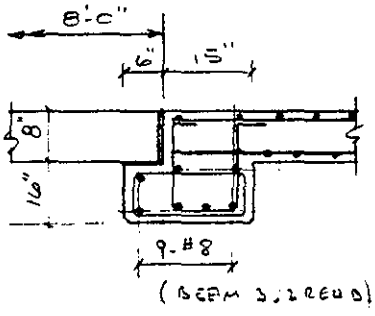
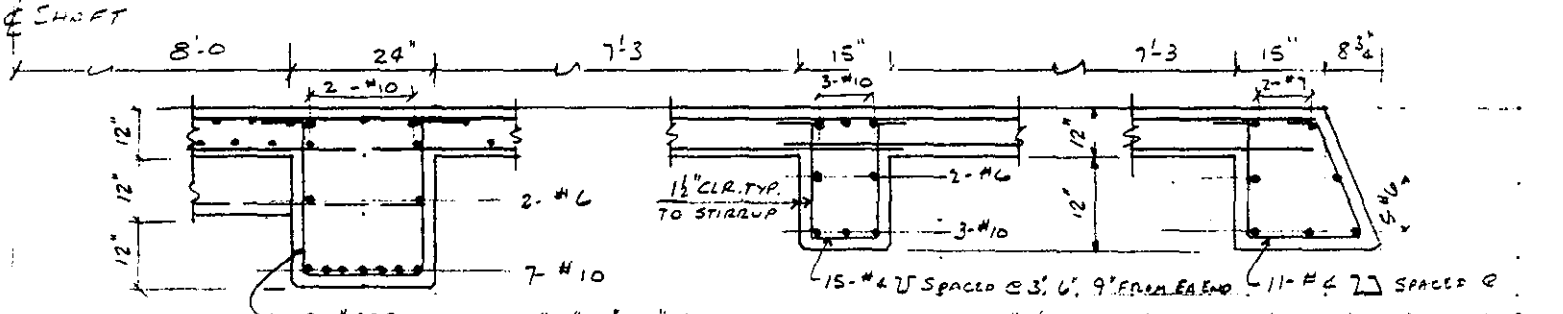


GENERAL PLAN
SCALE: 1" = 20'



SECTION A-A



SECTION B-B SECTION C-C SECTION D-D

SCALE: 3/8" = 1'-0"

Project MDC - TOWN RIVER (QUINCY) Acct. No. 6041 Page 27 of 121
Subject INTAKE GRIT BASIN Comptd. By J.T. HOWARD Date 6-15-84
Detail TOP DECK - SLAB Ck'd. By G. Volpin Date 8/7/84

GB - 5

DL (12" SLAB) = 0.15 $^{k}L_{F}$

FEM = $wL^2/12$ = 0.15 (16.0)^2/12 = 3.2 ^{1K}
0.15 (9.0)^2/12 = 1.0 ^{1K}
0.15 (8.5)^2/12 = 0.9 ^{1K}

16/16 = 1.00 ÷ 2.78 = 0.36
16/9 = 1.78 ÷ 2.78 = 0.64
2.78 1.00
1.78 ÷ 3.66 = 0.49
16/8.5 = 1.88 ÷ 3.66 = 0.51
3.66 1.00

w = 0.15 $^{k}L_{F}$

	.36	.64	.49	.51	
	16.0'		9.0'		8.5'
	−3.20	+3.20	−1.00	+1.00	−0.90 +0.9
	+3.20	−.79	−1.41	−0.05	−0.05 −0.9
	−0.40	+1.60	−0.03	−0.71	−0.45 −0.03
	+0.40	−0.57	−1.00	+0.57	+0.59 +0.03
	−0.29	+0.20	+0.28	−0.50	+0.01 +.29
	+0.29	−0.17	−0.30	+0.24	+0.25 −.29
	−0−	+3.47	−3.46	+0.55	−0.55 −0−
$V_S = wL/2$	1.20 ↑	1.20 ↑	0.68 ↑	0.68 ↑	0.64 ↑ 0.64 ↑
$V_M = M/L$	0.22 ↓	.22 ↑	0.32 ↑	0.32 ↓	0.06 ↑ 0.06 ↓
	0.98 ↑	1.42 ↑	1.00 ↑	0.36 ↑	0.70 ↑ 0.58 ↑

LOAD CASE A. LIVE LOAD = 150 PSF

w = 0.15 $^{k}L_{F}$

	.36	.64	.49	.51	
	−3.20	+3.20	−1.00	+1.00	−0.00 +0.00
	+3.20	−0.79	−1.41	−0.49	−0.51 −0.00
	−0.40	+1.60	−0.25	−0.71	−0.00 −0.26
	+0.40	−0.49	−0.86	+0.35	+0.36 +0.26
	−0.25	+0.20	+0.18	−0.43	+0.13 +0.18
	+0.25	−0.14	−0.24	+0.15	+0.15 −0.18
	−0−	+3.58	−3.58	−0.13	+0.13 −0−
$V_S = wL/2$	1.20 ↑	1.20 ↑	0.68 ↑	0.68 ↑	0.0 ↑ 0.0 ↑
$V_M = M/L$	.22 ↓	.22 ↑	0.41 ↑	0.41 ↓	0.015 ↓ 0.015 ↑
	0.98 ↑	1.42 ↑	1.09 ↑	0.27 ↑	0.015 ↓ 0.015 ↑

V = 0 @ x = 0.98/0.15 = 6.53'
+M = 0.98 × 6.53 − 0.15(6.53)^2/2
= 3.20 ^{1K}

	−3.20	+3.20	−0.00	+0.00	−0.90 +0.90
	+3.20	−1.15	−2.05	+0.44	+0.46 −0.90
	−0.55	+1.60	+0.22	−1.03	−0.45 +0.23
	+0.58	−0.66	−1.16	+0.73	+0.75 −0.23
	−0.33	+0.29	+0.37	−0.58	−0.12 +0.38
	+0.33	−0.24	−0.42	+0.34	+0.36 −0.38
	−0−	+3.04	−3.04	−0.10	+0.10 −0−
$V_S = wL/2$	1.20 ↑	1.20 ↑	0.0 ↑	0.0 ↑	↑ 0.64 0.64 ↑
$V_M = M/L$	0.19 ↓	0.19 ↑	0.35 ↑	0.35 ↓	↓ 0.01 0.01 ↑
	1.01 ↑	1.39 ↑	0.35 ↑	0.35 ↓	↑ 0.63 0.65

V = 0 @ x = 1.01/0.15 = 6.73'
+M = 1.01 × 6.73 − 0.15(6.73)^2
= 3.40 ^{1K}

NONREPRODUCIBLE GRID FORM 145
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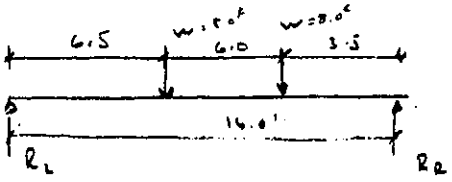
GB-6

LOAD CASE B. LIVE LOAD = ONE H-10 TRUCK NORMAL TO REINF.

$$M = \frac{S+2}{32} P \qquad \text{(AASHTO, REINF} \perp \text{TRAFFIC)}$$

SUBSTITUTING L FOR S; & W FOR P

$$= \frac{L+2}{32} W = \frac{16+2}{32} 8.0 = 4.5^{1k} \qquad \times 0.8 = 3.6^{1k}$$



$$R_L = (9.5 + 3.5)\frac{8.0}{16.0} = 6.5^k$$

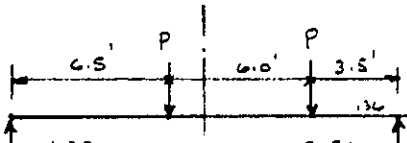
$$R_R = (12.5 + 6.5)\frac{8.0}{16.0} = \frac{9.5}{16.0^k}$$

$$M = \frac{R_L \times 6.5}{E} = 4.5 \qquad E = \frac{6.5 \times 6.5}{4.5} = 9.39' \qquad \text{USE } E = 7.0'$$

$$P = \frac{W(1+I)}{E} = \frac{8.0 \times 1.3}{7.0} = 1.50^k$$

$$FEM_L = \frac{P}{L^2}\sum(ab^2) = \frac{1.50}{(16.0)^2}\left(6.5 \times 9.5^2 + 12.5 \times 3.5^2\right) = 4.33^{1k}$$

$$FEM_R = \frac{1.50}{(16.0)^2}\left(6.5^2 \times 9.5 + 12.5^2 \times 3.5\right) = 5.56^k$$



		.36	.64	.49	.51
-4.33	+5.56	-0.00	+0.00	-0.00	+0.00
+4.33	-2.00	-3.52	0.00	0.00	0.00
-1.00	+2.17	0.00	-1.78	0.00	0.00
+1.00	-0.78	-1.39	+0.87	+0.91	0.00
-0.39	+0.50	+0.44	-0.70	0.00	+0.46
+0.39	-0.34	-0.60	+0.34	+0.36	-0.46
-0-	+5.11	-5.11	-1.27	+1.27	-0-
X					
V_S	1.22 ↑	1.78 ↑			
V_M	0.32 ↓	0.32 ↑			
	0.90 ↑	2.10 ↑			

COMBINE DL + LL FOR MAX. +M

$$R_L = 1.98 + 0.90 = 1.88^k \qquad \left(-0.15 \times 6.5 - 1.50 = -0.60^k\right); V = 0 \text{ @ } x = 6.5'$$

$$+M = 1.88 \times 6.5 - 0.15\frac{(6.5)^2}{2} = 9.05^{1k}$$

$$V_{MAX} = 1.42 - 0.15 \times \overset{1.30}{\frac{9.5}{12}} + \overset{2.03}{\frac{1.5}{16.0}}(15.2 + 9.2)1.15 = 3.93^k$$

$$v = \frac{3,930}{12 \times 9.5} = 35.0 \text{ psi}$$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC - TOWN BROOK (QUINCY) Acct. No. 6041 Page 29 of 121
Subject INTAKE GRIT BASIN Comptd. By J.T. HOWARD Date 6-15-84
Detail TOP DECK - SLAB Ck'd. By A. Volpini Date 8/7/84

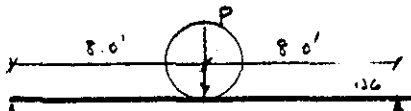
GB-7

LOAD CASE C. LIVE LOAD = ONE H-10 TRUCK PARALLEL TO REINF.

$E = 4.0 + 0.06S$ (AASHTO)
$= 4.0 + 0.06(16.0) = 4.96$ SAY 5.0'

$P = \frac{W(1+I)}{E} = \frac{8.0 \times 1.3}{5} = 2.08^K$

$FEM_L = FEM_R = \frac{PL}{8} = \frac{2.08 \times 16}{8} = 4.16^{'K}$



	.36	.64	.49	.51	
-4.16	+4.16	0.00	0.00	0.00	0.00
+4.16	-1.50	-2.66	0.00	0.00	0.00
-0.75	+2.08	0.00	-1.33	0.00	0.00
+0.75	-0.75	-1.33	+0.65	+0.68	0.00
-0.38	+0.38	+0.33	-0.67	0.00	+0.34
+0.38	-0.26	-0.45	+0.33	+0.34	-0.34
-0-	+4.11	-4.11	-1.02	+1.02	-0-

x

V_S = 1.04↑ 1.04↑
V_M = 0.26↓ 0.26↑
0.78↑ 1.30↑

COMBINE DL+LL FOR MAX +M

$R_L = 0.98 + 0.78 = 1.76^K$ $V = 0$ @ $x = 8.0'$

$+M = 1.76 \times 8.0 - 0.15(8.0)^2/2 = 9.28^{'K}$ $\left(\frac{PL}{4} + \frac{wL^2}{8} - \frac{4.11+3.47}{2} = 9.33^{'K}\right)$ ✓

$V = 1.30 + 2.08 \times \frac{15.2}{16.0} = 3.28^K$

$v = \frac{3280}{12 \times 9.5} = 29.0$ psi

MAX -M = $3.47 + 5.11 = 8.58^{'K}$

$-A_s = -M/ad = 8.58/1.44 \times 9.5 = 0.63 \text{ in.}^2$

MAX +M = $9.28^{'K}$ (CASE C)

$+A_s = +M/ad = 9.28/1.44 \times 9.5 = 0.68 \text{ in.}^2$

USE #6 @ 8" T&B
$A_s = 0.66 \text{ in.}^2$
#5 @ 12" DIST

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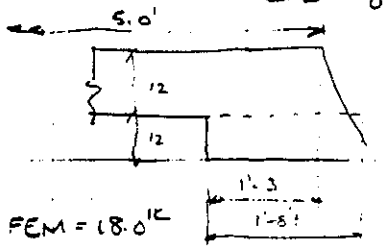

DL (SLAB) $1.0 \times 5.0 \times 0.15 = 0.75^{k}/_{LF}$
(BEAM) $1.0 \times 1.67 \times 0.15 = 0.25$
$1.00^{k}/_{LF}$

$FEM = \frac{wL^2}{12} = \frac{1.0(12.0)^2}{12} = 12.0^{k}$ $M_S = 1.5\,FEM = 18.0^{k}$

		.5	.5	.5	.5	.5	.5	
DL	-12.0	+12.0	-12.0	+12.0	-12.0	+12.0	-12.0	+12.0
	+12.0	0.0	0.0	0.0	0.0	0.0	0.0	-12.0
	0.0	+6.0	0.0	0.0	0.0	0.0	-6.0	0.0
	0.0	-3.0	-3.0	0.0	0.0	+3.0	+3.0	0.0
	-1.5	0.0	0.0	-1.5	+1.5	0.0	0.0	+1.5
	+1.5	0.0	0.0	0.0	0.0	0.0	0.0	-1.5
	-0-	+15.0	-15.0	+10.5	-10.5	+15.0	-15.0	-0-
$V_S = \frac{wL}{2}$	↑ 6.0	6.0 ↑	↑ 6.0	6.0 ↑	↑ 6.0	6.0 ↑	↑ 6.0	6.0 ↑
$V_M = \frac{M}{L}$	↓ 1.25	1.25 ↑	↑ 0.38	0.38 ↓	↓ 0.38	0.38 ↑	↑ 1.25	1.25 ↓

LOAD CASE A : LIVELOAD = 100 PSF $FEM = wL^2/12 = (0.10 \times 5.0)(12.0)^2/12 = 6.0^{k}$

+LL	-6.0	+6.0	0.0	0.0	-6.0	+6.0	0.0	0.0
	+6.0	-3.0	-3.0	+3.0	+3.0	-3.0	-3.0	0.0
	-1.5	+3.0	+1.5	-1.5	-1.5	+1.5	0.0	-1.5
	+1.5	-2.2	-2.3	+1.5	+1.5	-0.8	-0.7	+1.5
	-1.1	+0.8	+0.8	-1.2	-0.4	+0.8	+0.8	-0.4
	+1.1	-0.8	-0.8	+0.8	+0.8	-0.8	-0.8	+0.4
	-0-	+3.8	-3.8	+2.6	-2.6	+3.7	-3.7	-0-
$V_S =$	↑ 3.0	3.0 ↑	↑ -0-	-0- ↑	↑ 3.0	3.0 ↑	↑ -0-	-0- ↑
$V_M =$	↓ 0.32	0.32 ↑	↑	↓	↓	↑	↑	↓

LL = .50 k/LF

-LL	-6.0	+6.0	-6.0	+6.0	0.0	0.0	0.0	0.0
	+6.0	0.0	0.0	-3.0	-3.0	0.0	0.0	0.0
	0.0	+3.0	-1.5	0.0	0.0	-1.5	0.0	0.0
	0.0	-0.8	-0.7	0.0	0.0	+0.8	+0.7	0.0
	-0.4	0.0	0.0	-0.4	+0.4	0.0	0.0	+0.4
	+0.4	0.0	0.0	0.0	0.0	0.0	0.0	-0.4
	-0-	+8.2	-8.2	+2.6	-2.6	-0.7	+0.7	-0-
$V_S = \frac{wL}{2}$	↑ 3.0	3.0 ↑						
$V_M = \frac{M}{L}$	↓ 0.65	0.68 ↑						

$R_{MAX} = R_{DL} + R_{LL} = 6.0 + 1.25 + 3.0 + 0.68 = 10.93^{k}$ $v = \frac{+0930}{15 \times 21.5} = 34$ psi

$-M = -M_{DL} + (-M_{LL}) = 15.0 + 8.2 = 23.2^{'k}$ $-A_S = 23.2/1.44 \times 21.5 = 0.75\,in^2$

$+M$: $R = 6.0 - 1.25 + 3.0 - 0.32 = 7.43^{k}$ $x = 7.43 \div (1.0 + 0.5) = 4.95'$

$+M = 7.43 \times 4.95 - (1.0 + 0.5)(4.95)^2/2 = 18.5^{'k}$ $-A_S = 18.5/1.44 \times 21.5 = 0.60\,in^2$

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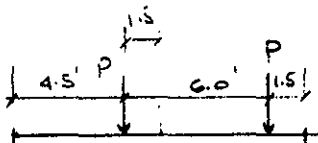
LOAD CASE B: ONE H10 TRUCK ON END SPAN

$W_R = 8.0^k$ $\quad P = WI = 8.0 \times 1.3 = 10.4^k$

$$FEM_L = \frac{10.4}{(12.0)^2}\left[4.5(7.5)^2 + 10.5(1.5)^2\right] = 20.0^{'k}$$

$$FEM_R = \frac{10.4}{(12.0)^2}\left[(4.5)^2 7.5 + (10.5)^2 1.5\right] = 22.9$$



-20.0	+22.9	0.0	0.0	0.0	0.0	0.0	0.0
+20.0	-11.4	-11.5	0.0	0.0	0.0	0.0	0.0
-5.7	+10.0	0.0	-5.8	0.0	0.0	0.0	0.0
+5.7	-5.0	-5.0	+2.9	+2.9	0.0	0.0	0.0
-2.5	+2.8	+1.5	-2.5	0.0	+1.5	0.0	0.0
+2.5	-2.2	-2.1	+1.2	+1.3	-0.8	70.7	0.0
-0-	+17.1	-17.1					

V_S	↑ 7.80	13.00 ↑
V_M	↓ 1.42	1.42 ↑
V	6.38	14.42 ↑

COMBINE DL & LL

$$-M = -M_{DL} + (-M_{LL}) = 15.0 + 17.1 = 32.1^{'k}$$

$$-A_s = -M/ad = 32.1/1.44 \times 21.5 = 1.04 \text{ IN.}^2$$

USE - 2-#7 T. CONT SPLICE @ ℄ SPAN ($A_s = 1.20 \text{ IN}^2$)

$$R_L = R_{DL} + R_{LL} = 4.75^k + 6.38 = 11.13^k$$

$+M$ = MAX WHERE $V = 0$ UNDER OUTBOARD TRUCK WHEEL

$$+M = 11.13 \times 4.5 - 1.0(4.5)^2/2 = 40.0^{'k}$$

$$+A_s = 40.0/1.44 \times 21.5 = 1.29 \text{ IN.}^2$$

USE: 3-#6 B. CONT. SPLICE @ ℄ SUPPORT ($A_s = 1.32 \text{ IN.}^2$)

$$R_{MAX} = 7.25 + 14.42 = 21.67^k$$

$$v = \frac{21{,}670}{15 \times 21.5} = 67 \text{ PSI} < 70 \text{ PSI} \quad \text{OK}$$

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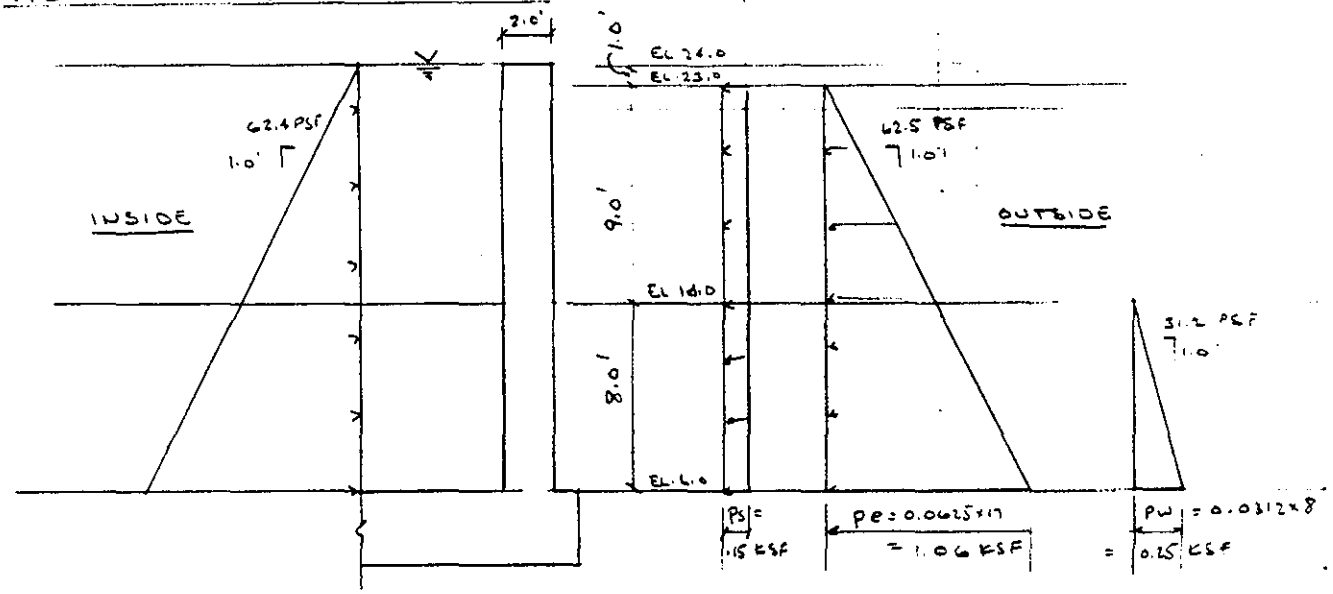
ASSUME PURE CANTILEVER:



EXTERIOR LOADING:-

BASE:

$V = 0.15 \times 17 + 1.06 \times 17/2 + 0.25 \times 8/2 = 2.55 + 9.0 + 1.0 = 12.55^{K}_{F}$

$v = \frac{V}{bd} = \frac{12,500}{12 \times 21.5} = 48$ PSI

$M = 2.55 \times 17/2 + 9.0 \times 17/3 + 1.0 \times 8/3 = 21.7 + 51.0 + 2.7 = 75.4^{'K}$

$M_R = 0.221 (21.5)^2 = 102.0^{'K} > 75.4^{'K}$

$A_S = M/a\,d = 75.4/1.44 \times 21.5 = 2.44\ IN^2$ USE: #10 @ 6" VOF, $A_S = 2.54\ IN^2$

4.5' ABOVE BASE:-

$M = 0.15(12.5)^2/2 + 0.0625(12.5)^3/6 + 0.0312(3.5)^3/6$

$= 11.7 + 20.4 + 0.2 = 32.3^{'K}$

$A_S = M/a\,d = 32.3/1.44 \times 21.5 = 1.04\ IN^2$ USE: #10 @ 12" VOF ($A_S = 1.27\ IN^2$)

INTERIOR LOADING:-

BASE.

$V = 0.0624(18)^2/2 = 10.1^{K}$

$v = \frac{V}{bd} = \frac{10,100}{12 \times 19.5} = 43$ PSI

$M = 10.1 \times 18.0/3 = 60.6^{'K}$

$A_S = \frac{M}{ad} = \frac{60.6}{1.44 \times 19.5} = 2.16\ IN^2$ USE: #10 @ 6" VOF ($A_S = 2.54\ IN^2$)

4.5' ABOVE BASE

$M = 0.0624(13.5)^3/6 = 25.6^{'K}$

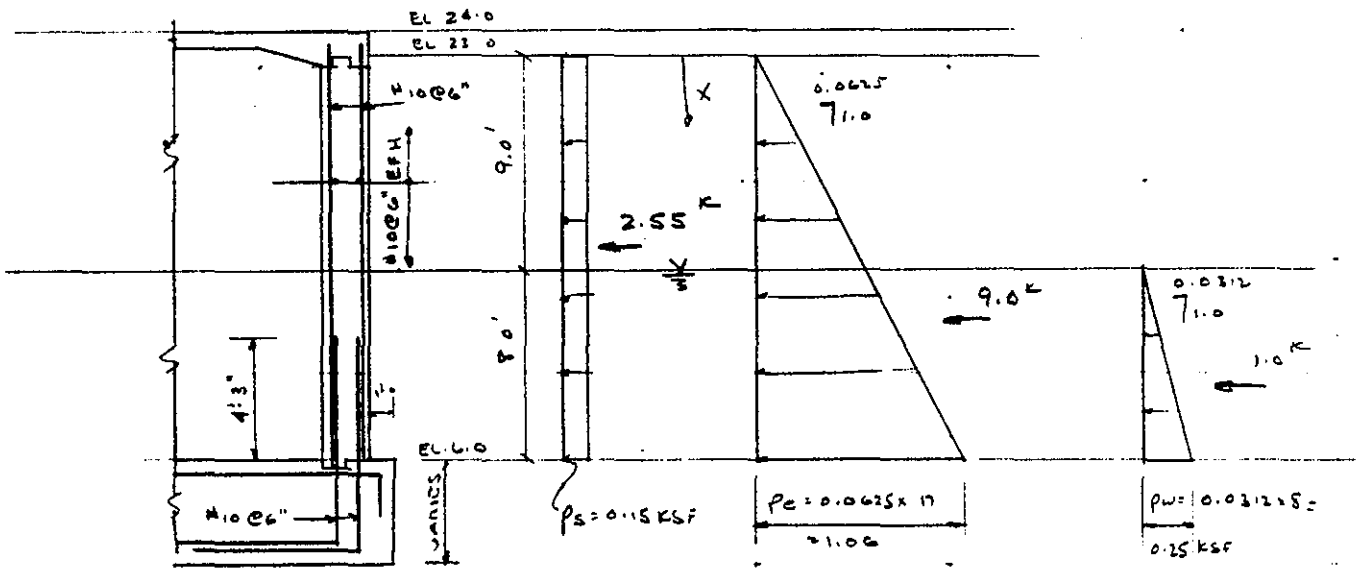
$A_S = \frac{M}{ad} = \frac{25.6}{1.44 \times 19.5} = 0.91\ IN^2$ USE: #10 @ 12" VOF ($A_S = 1.27\ IN^2$)

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC - TOWN BROOK (QUINCY) Acct. No. 6041 Page 40 of 121
Subject INTAKE GRIT BASIN Comptd. By J.T. HOWARD Date 7-27-84
Detail SPILLWAY WALL H=17' Ck'd. By A. Volpini Date 8/7/84

GB - 18

WALL SPANS VERTICALLY :-
ASSUME FIXED BASE ; HINGED TOP



EXTERNAL LOADING :-

$$-M_{BASE} = \frac{0.15(17.0)^2}{8} + \frac{1.06(17.0)^2}{15} + \frac{0.25(17.0)^2}{120} Z = 5.42 + 20.42 + 1.73 = 27.57^{'K}$$

$$Z = K^2(20 - 15K + 3K^2) = \left(\frac{8}{17}\right)^2\left[20 - \frac{15 \times 8}{17} + 3\left(\frac{8}{17}\right)^2\right] = 2.87$$

$$R_T = \left[\frac{0.15(17.0)^2}{2} + \frac{0.0625(17.0)^3}{6} + \frac{0.0312(8.0)^3}{6} - 27.6\right]\frac{1}{17} = 2.82$$

$$R_B = \left[2.55 \times \frac{17}{2} + 9.0 \times \frac{2}{3} \times 17 + 1.0\left(9.0 + \frac{2}{3}.8\right) + 27.6\right]\frac{1}{17} = \frac{9.74}{12.56^K}$$

$V = 0$ @ $x = 7.4'$

$$+M = 2.82 \times 7.40 - \frac{0.15(7.4)^2}{2} - \frac{0.0625(7.40)^3}{6} = 20.87 - 4.11 - 4.22 = 12.54^{'K}$$

$$V_{BOT} = 9.74 - 0.15 \times \frac{21.5}{12} - \frac{0.0625}{2}\left(\frac{21.5}{12}\right)^2 - \frac{0.0312}{2}\left(\frac{21.5}{12}\right)^2 = 9.32^K$$

$$v_{BOT} = \frac{V}{bd} = \frac{9320}{12 \times 21.5} = 36.0 \text{ PSI}$$

$$v_{TOP} = \frac{7820}{12 \times 21.5} = 11.0 \text{ PSI}$$

$$-A_S = -M/ad = 27.6/1.44 \times 21.5 = 0.89 \text{ IN}^2$$

$$+A_S = +M/ad = 12.6/1.44 \times 19.5 = 0.45 \text{ IN}^2$$

CONTINUE CANTILEVER WALL STEEL AROUND SUMP $\left(\begin{matrix}-A_S = 2.54 \\ +A_S = 1.27\end{matrix}\right)$
BECAUSE THIS WALL WILL REACT HORIZONTALLY
SPANNING WALL AROUND SHAFT.

NONREPRODUCIBLE GRID FORM 145
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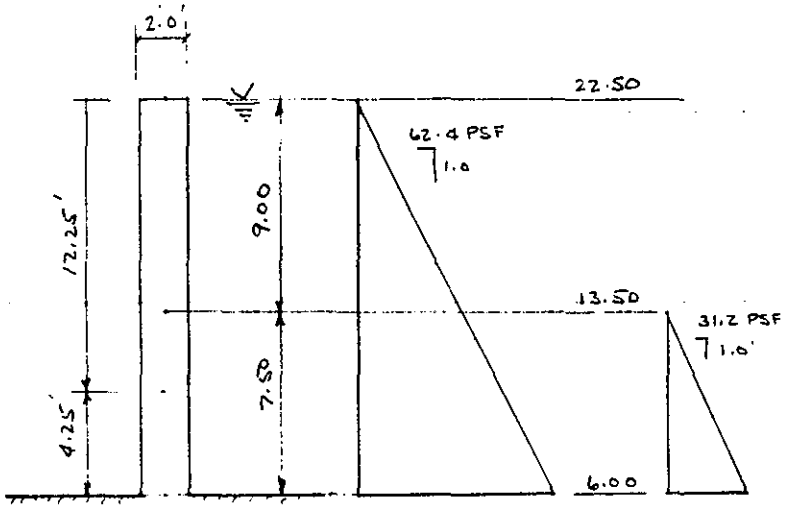
NEGLIGIBLE END SUPPORT : USE CANTILEVER
ASSUME ONE CELL EMPTY ; LOADED CELL FULL OF WATER & GRIT



SHEAR:-

$$R_{BASE} = R_W + R_G = 0.0624(16.5)^2/2 + 0.0312(7.5)^2/2 = 8.50 + 0.88 = 9.38^k$$

$$v = \frac{R}{bd} = \frac{9380}{12 \times 19.4} = 40.3 \text{ PSI}$$

MOMENT:-

$$M_{BASE} = 0.0624(16.5)^3/6 + 0.0312(7.5)^3/6 = 46.7 + 2.2 = 48.9^{'k}$$

$$A_s = \frac{M}{ad} = \frac{48.9}{1.44 \times 19.4} = 1.76 \text{ in}^2$$

USE: # 9 @ 6" EF ($A_s = 2.06 \text{ in}^2$)

$$M_{+4.25} = 0.0624(12.25)^3/6 + 0.0312(3.25)^3/6 = 19.1 + 0.2 = 19.3^{'k}$$

$$A_s = \frac{M}{a \cdot d} = \frac{19.3}{1.44 \times 19.4} = 0.70 \text{ in}^2$$

USE: # 8 @ 12" EF ($A_s = 0.79 \text{ in}^2$)

HORIZONTAL $A_{s_{MIN}} = 0.0025 \times 12 \times 24 / 2 = 0.36 \text{ in}^2$ (MIN)

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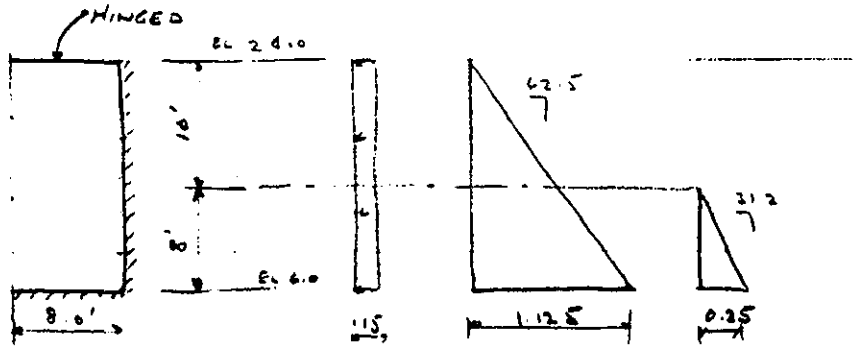
CORNER BARS:



$a = 8.0$ $b = 18.0$ $a/b = 8.0/18.0 = 0.44$

REF. BUREAU OF RECLAMATION MONOGRAPH #27
@ $Y/b = 0.4$

$M = 0.0441\,(18.0)^2\,0.15 = 2.14^{1K}$ FIG. 10
$0.0238\,(18.0)^2\,1.125 = 8.68$ FIG. 13
$0.0089\,(18.0)^2\,0.25 = 0.72$
11.54

$A_s = \frac{M}{ad} = \frac{11.54}{1.44 \times 18} = 0.44 \text{ IN.}^2$

USE: # 8 @12" DFH

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METCALF & EDDY, ENGINEERS

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CENTER WALL AS BEAM SPANNING GRIT BASIN:-

UPLIFT PRESSURE:

$$u = \left[0.0624\,(15.0-3.0) - 0.15 \times 3.0'\right] \times 24 \times 6/8 \times 2 = 9.0\ ^{k}/_{F}$$

DEAD LOAD OF WALL:

$$w = 2.0\,(22.5-6.0)\,0.15 = 5.0\ ^{k}/_{F}$$

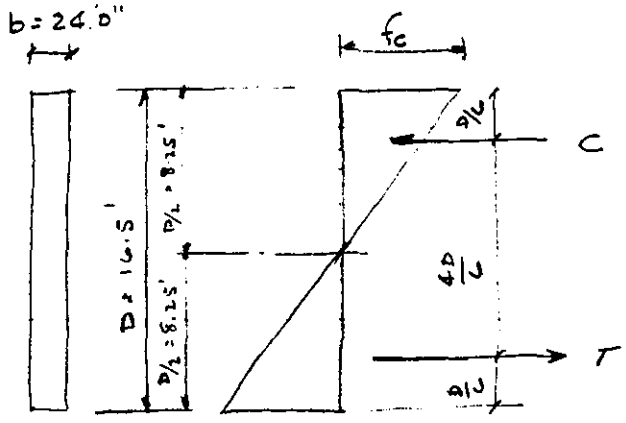
$$q = u - w = 9.0 - 5.0 = 4.0\ ^{k}/_{F} \uparrow$$

$$L = 50.0'$$

$$M = \frac{qL^2}{8} = \frac{4.0\,(50.0)^2}{8} = 1250.0\ ^{'k}$$

$$M_R = 2.0\,(155.5)^2\,0.221 = 16{,}893.\ ^{'k}$$

USE HOMOGENEOUS BEAM THEORUM



EXTERNAL MOMENT = INTERNAL MOMENT

$$M = C \cdot \frac{2}{3} D = 1250\ ^{'k}$$

$$C = \frac{1250 \times 3}{16.5 \times 2} = 114.\ ^{k} = T$$

$$C = f_c \cdot \frac{D}{2} \cdot \frac{b}{2} \qquad f_c = \frac{4C}{Db} = \frac{4 \times 114}{16.5 \times 2.0} = 13.8\ ^{k}/_{SF}\ \left(\times \frac{1000}{144} = 96\ psi\right)$$

$$A_s = \frac{13.8\ ^{k}/_{SF}}{20.0\ ^{k}/_{S.I.}} = 0.69\ S.I./_{SF} \times 2.0' = 1.38\ in^2/_{FT.}$$

USE: #8@12" EF = 1.58 in²/FT

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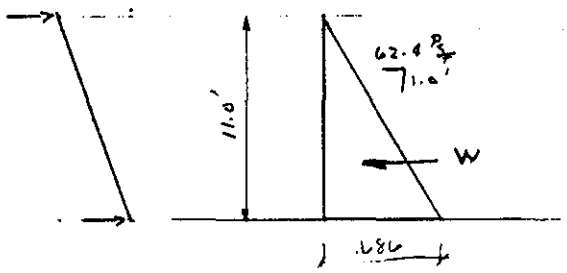
LOAD CASE A: HYDRAULIC LOADING

ASSUME:
1. BARS @ 6" O/C
2. RACK PLUGGED
3. FULL HEAD OF WATER
4. 6061-T6 ALUM



$W = 0.0624(11.0)^2/2 = 3.78\ ^{k}/_{FT} \times 0.50 = 1.89\ ^{k}/_{BAR}$

$M = 0.1283WL = 0.1283 \times 1.89 \times 11.0 = 2.66^{IK} \times 12 = 31.97^{IN\text{-}K}$

SAY 32.0 IN-K

$F = 16.0\ ^{K}/_{IN^2}$ (ASCE FOR WELD AREA)

$S = \frac{M}{F} = \frac{32.0\ IN\text{-}K}{16.0\ ^{K}/_{IN^2}} = 2.0\ IN^3 = \frac{td^2}{6}$

N/16THS	t	$d = (6 \times 2/t)^{1/2}$	d ACTUAL	$L = 676\ t^2/d$ *	M	10.5/M
4	0.250	6.93	7.0	6.03 IN.	3	3.500' = 42.0"
5	0.312	6.20	6.5	10.12	4	2.625 = 31.5"
6	0.375	5.66	6.0	15.84	5	2.100 = 25.2"
7	0.437	5.18	5.5	24.52	6	1.750 = 21.0"
8	0.500	4.90	5.0	33.80	7	1.500' = 18.0"
9	0.562	4.62	5.0		8	1.3125 = 15.75
10	0.625	4.38	4.5	58.68		
11	0.687	4.18	4.5			
12	0.750	4.00	4.0	95.06		

M = NUMBER OF PANELS: MATCH W/L

* ASCE SLENDERNESS RATIO = $\frac{d}{t}\sqrt{\frac{L}{d}} = 26$

LOAD CASE B: DYNAMIC IMPACT (LOG)

$F = ma$ $m = \pi(0.5)^2\ 8.0 \times 60 = 377$ $a = 2.0\ FT/SEC^2$

$= 377 \times 2 = 754^{LB}$

$M = \frac{FL}{4} = \frac{.754 \times 11}{4} = 2.07^{IK}$

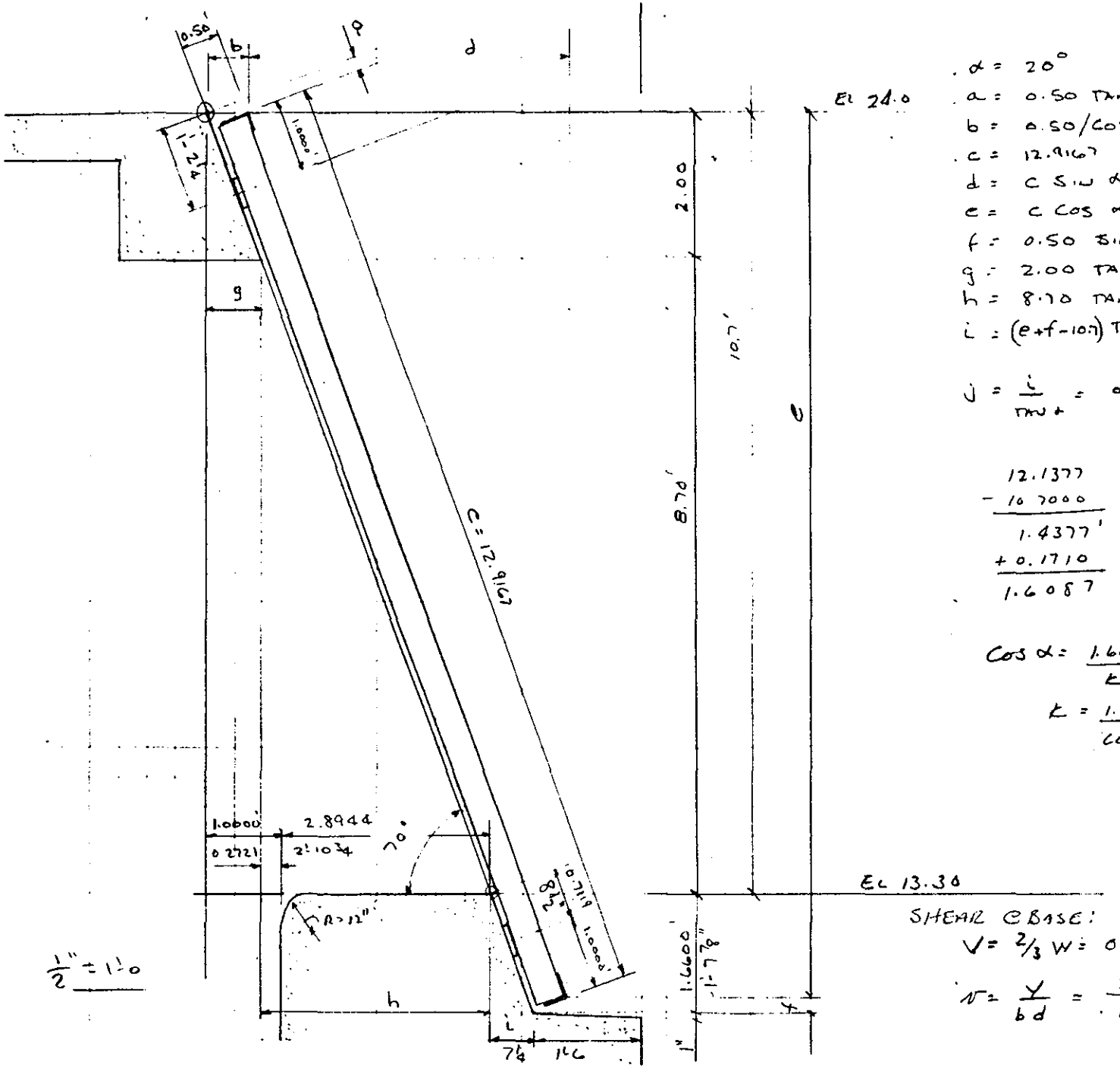
$f = \frac{M}{S} = \frac{2.07 \times 12 \times 6}{0.50(5.0)^2} = 11.9\ ^{K}/_{IN^2} < 16.0\ ^{K}/_{IN^2}$ OK

USE: BAR $\frac{1}{2}$" x 5" @ 6" O/C CROSS BARS @ 32" ± O/C

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

METCALF & EDDY, ENGINEERS

Project: MDC - TOWN BROOK (QUINCY) | Acct. No. 6041 | Page 47 of 121
Subject: INTAKE GRIT BASIN | Comptd. By J. T. Howard | Date 6-26-84
Detail: TRASH RACK GEOMETRY | Ck'd. By [illegible] | Date 8/7/84



$\alpha = 20^\circ$

$a = 0.50 \tan\alpha = 0.1820$

$b = 0.50/\cos\alpha = 0.5321$

$c = 12.9167 = 12.9167$

$d = c \sin\alpha = 4.4188$

$e = c \cos\alpha = 12.1377$

$f = 0.50 \sin\alpha = 0.1710$

$g = 2.00 \tan\alpha = 0.7279$

$h = 8.70 \tan\alpha = 3.1665$

$i = (e+f-10.7) \tan\alpha = 0.5855$

$i = 0.6042$

REV. 7¼"

$j = \frac{i}{\tan\alpha} = 0.6042/\tan 20^\circ = 1.6600' = 1'-7\frac{7}{8}"$

$$\begin{array}{r} 12.1377 \\ -\ 10.7000 \\ \hline 1.4377' \\ +\ 0.1710 \\ \hline 1.6087 \end{array}$$

$\cos\alpha = \frac{1.6087}{L}$

$L = \frac{1.6087}{\cos\alpha} = 1.7119$

$-\ 1.6000$

0.1119

SHEAR @ BASE:

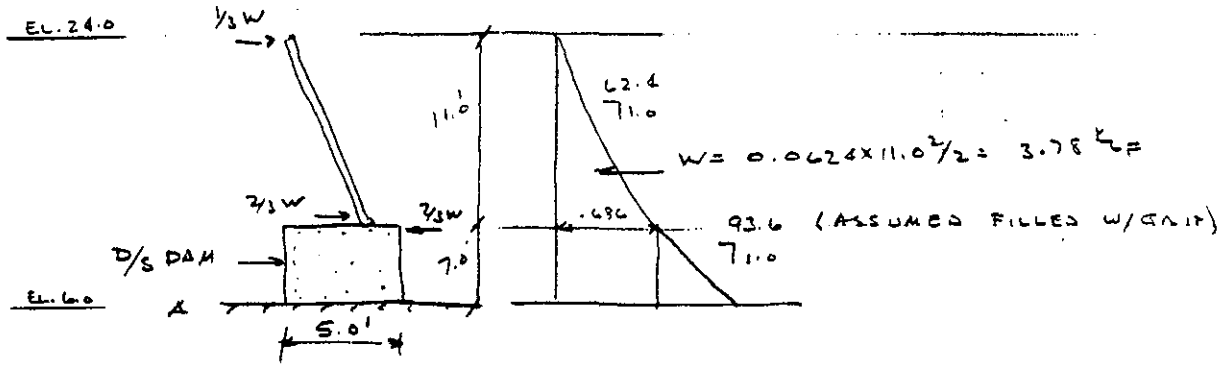
$V = \frac{2}{3} W = 0.67 \times 3.78\ \xi_F = 2.54\ \xi_F$

$v = \frac{V}{bd} = \frac{2540}{12 \times 36'} = 6\ \text{psi}$

GB-25

GB-26

DAM



$$M_A = \frac{2}{3} W \times 7.0 + 0.686 (7.0)^2/2 + 0.094 (7.0)^3/6 = 17.62 + 16.81 + 5.37 = 39.80^{'K}$$

$$A_s = \frac{39.82}{1.44 \times 55} = 0.50 \text{ in}^2$$

$$A_{s_{MIN_V}} = 60 \times 12 \times 0.0015/2 = 0.54 \text{ in}^2$$

USE #7 @ 12" EF ($A_s = 0.60$)

$$A_{s_{MIN_H}} = 60 \times 78 \times 0.0025 = 11.7 \text{ in}^2$$

USE 20-#7 ($A_s = 12.0 \text{ in}^2$)

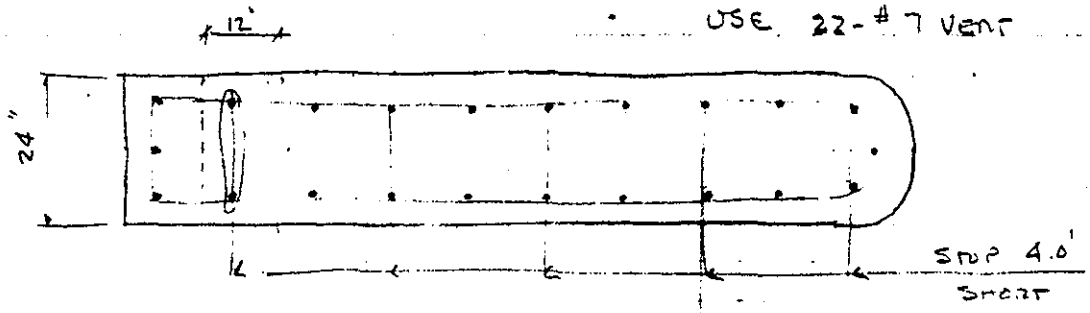
PIER

VERY LITTLE LOAD ON PIER COMPARED WITH ITS DEPTH ETC.

CALCULATE MIN STEEL

$$A_{s_V} = 24 \times 12 \times 0.0015 = 0.44 \text{ in}^2$$

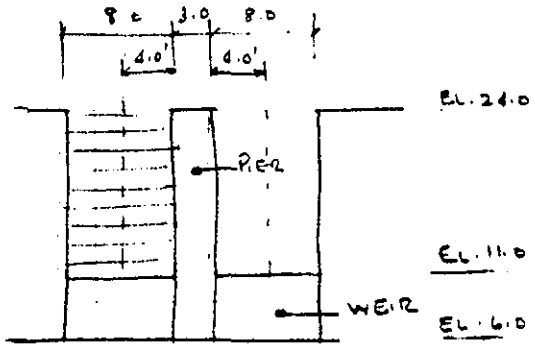
USE 22-#7 VERT

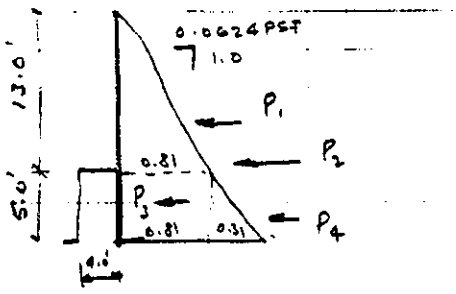


NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

GB-27

STOP LOGS "IN"





$$P_1 = \frac{0.0624}{2}(13.0)^2 \times 8.0 = 42.18^{K}$$

$$P_2 = \frac{0.0624}{2}(18.0)^2 \times 3.0 = 30.33^{K}$$

$$P_3 = 0.0624 \times 13.0 \times 5.0 = 4.06^{K}/_{LF}$$

$$P_4 = \frac{0.0624}{2}(5.0)^2 = 0.78^{K}/_{LF}$$

AT BASE OF PIER

$$V = P_1 + P_2 = 42.18 + 30.33 = 72.51^{K}$$

$$v = \frac{72510}{36 \times 66} = 31.0 \text{ psi}$$

$$M = P_1 h_1 + P_2 h_2 = 42.18\left(5.0 + \frac{13}{3}\right) + 30.33 \times \frac{18}{3} = 393.5 + 182.0 = 575.5^{'K}$$

$$A_s = \frac{M}{ad} = \frac{575.5}{1.44 \times 66} = 6.06 \text{ in}^2$$

USE 5-#10 (A_s = 6.35 in^2)

AT BASE OF WEIR

$$V = P_3 + P_4 = 4.06 + 0.78 = 4.84^{K}/_{FT}$$

$$v = \frac{4840}{12 \times 43} = 10 \text{ psi}$$

$$M = P_3 h_3 + P_4 h_4 = 4.06 \times 5/2 + 0.78 \times 5/3 = 10.2 + 1.3 = 11.5^{'K}$$

$$A_s = \frac{M}{ad} = \frac{11.5}{1.44 \times 43} = 0.19 \text{ in}^2$$

$$A_{s_{MIN_V}} = 0.0015 \times 12 \times 48/2 = 0.44 \text{ in}^2$$

USE #6 @ 12" EF V (A_s = 0.44 in^2)

$$A_{s_{MIN_H}} = 0.0025 \times 48 \times 60 = 7.2 \text{ in}^2$$

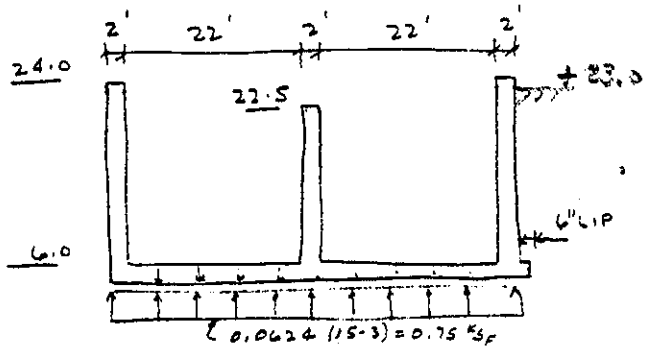
USE 12-#7 HORIZ. (A_s = 7.2 in^2)

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC-TOWN BROOK (QUINCY) Acct. No. 6041 Page 53 of 121
Subject INTAKE GRIT BASIN Comptd. By J.T HOWARD Date 6-20-84
Detail BOTTOM SLAB Ck'd. By A. Volpini Date 8/21/84

GB-31

CHECK 1.0' STRIP



DEAD WEIGHT OF STRUCTURE: -

CONCRETE

4.0 × 18.0 = 72.0 × .15/2 = 5.4
2.0 × 16.5 = 33.0 × .15 = 5.0
3.0 × 50.0 = 150.0
255.0 × 0.15 = 38.25 K

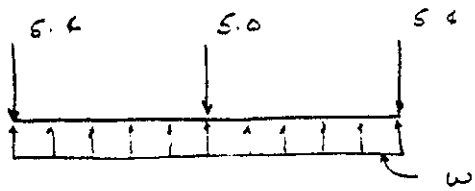
LIP

3.0 × 1.0 = 3.0 × 0.15 = 0.45
17.0 × 1.0 = 17.0 × 0.06 = 1.02
39.72

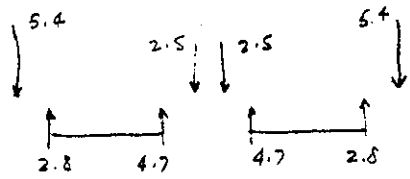
ASSUME • 3.0' THICK SLAB
• UNDERDRAINS PLUGGED

	W/O LIP	W/ LIP	W/ LIP
WITH G.W @ EL 14 (WEEPHOLES)			
F.S = 38.25 / ((14-3) × 50 × 0.0624) = 1.11 (34.32)		39.72 / 34.32 = 1.16	1.2
WITH G.W @ EL 15 (WH +1.0)			
FS = 38.25 / ((15-3) × 50 × 0.0624) = 1.02 (37.44)		39.72 / 37.44 = 1.06	1.1

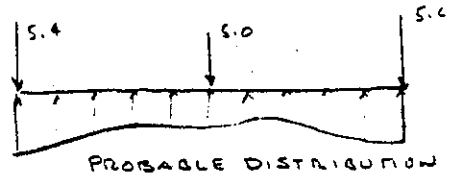
HAVE ADDED DEAD WEIGHT @ EACH END: USE: 3.0' MAT



w = 0.75 − 0.45 = 0.30 ksf
W = wL = 0.30 × 50 = 15.0 K



0.30 × 25 × 5/8 = 4.7 K
0.30 × 25 × 3/8 = 2.8 K



PROBABLE DISTRIBUTION

V_{MAX} = 4.70 K

$v = \frac{4700}{12 \times 33}$ = 12.0 psi (ok)

w = (5.4 + 5.0 + 5.4)/50 = 0.32 ksf

$- M = \frac{wL^2}{8} = \frac{0.32(25.0)^2}{8} = 24.7$ 'K

$A_s = \frac{M}{ad} = \frac{24.7}{1.44 \times 31.5} = 0.55$ in.² (4" Cover)

$A_{s_{MIN}} = 0.002 \times 36 \times 12/2 = 0.43$ in.²

USE: #7 @ 12" T&B (As = 0.60)
#6 @ 12" T&B DIST. (A4)

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE H-1 Con't

LABORATORY TESTING DATA SUMMARY

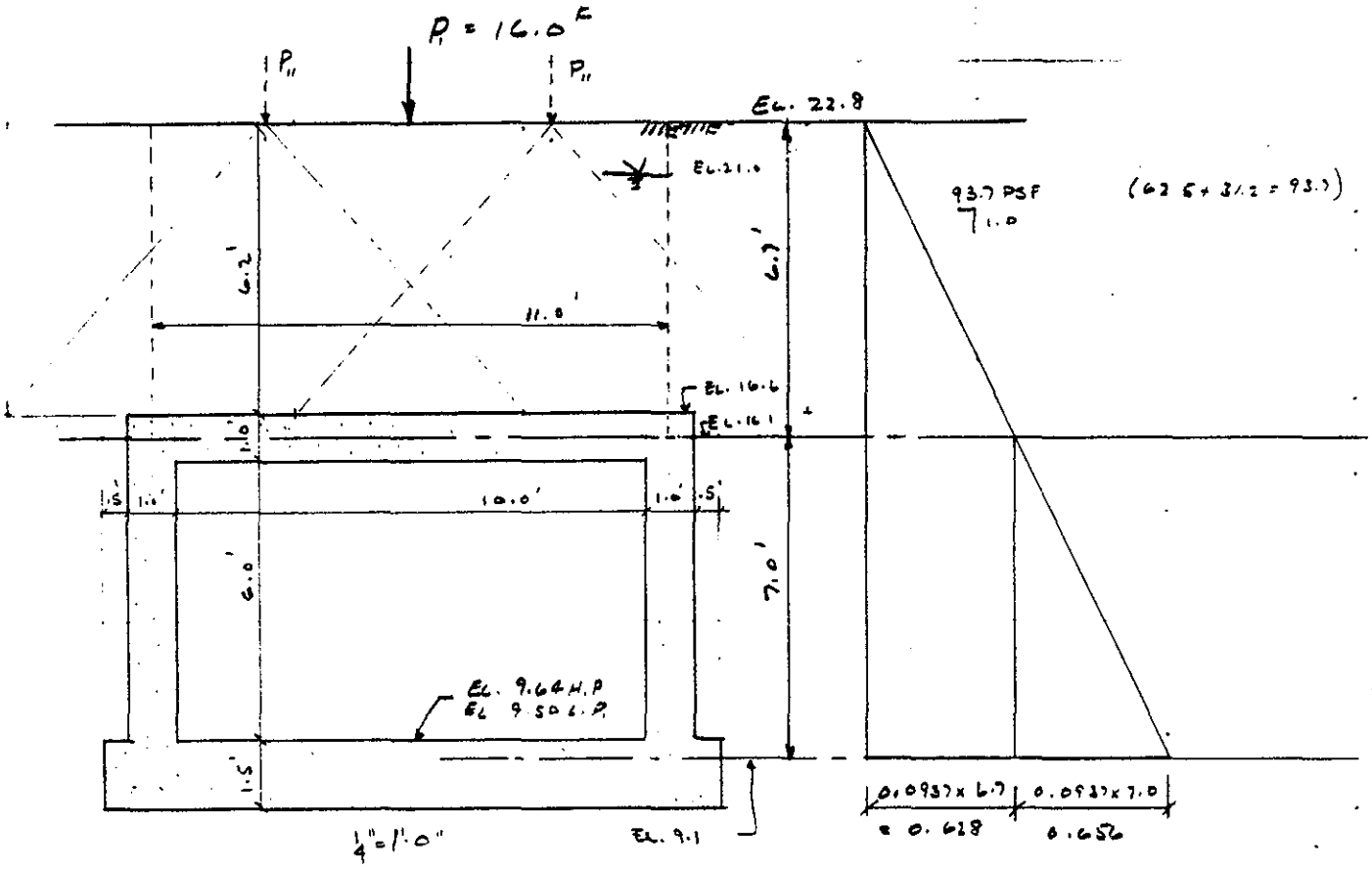
Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
T-10	S-15	31.0-33.0					44											f-m. SAND and Silt, trace Gravel (SM)
	S-16	33.0-35.0					37											f.-m. SAND and Silt, little Gravel (SM)
	S-17	35.0-37.0					42											f.-m. SAND and Silt, trace Gravel (SM)
	S-18	38.0-38.3					48											SILT, some Sand and Gravel (ML)
	S-21	44.5-45.5					42											SAND and Silt, little Gravel (SM)

SUMMARY OF LAB TESTS
TABLE NUMBER ____

Project MDC - TOWN BROOK (QUINCY) Acct. No. 6041 Page 54 of 121
Subject 10' x 6' CULVERT Comptd. By J. T. HOWARD Date 8-08-84
Detail CONFIGURATION / LOADING Ck'd. By A Volpini Date 8/9/84

BC -1



FLOATATION: -

UPLIFT = 12.0 x 8.5 x 0.0624 = 6.4 K

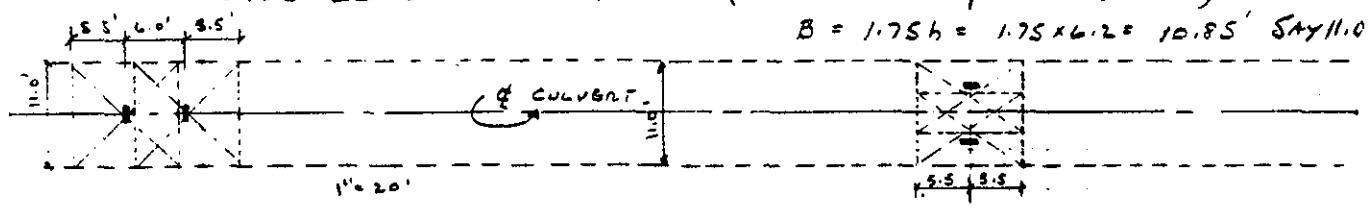
F.S. = 11.0 / 6.4 = 1.7

DEAD LOAD = (12.0 x 8.5 - 10.0 x 6.0) 0.15 = 6.3 K
(SOIL) 12.0 x 6.2 (.125.0 - 0.062) = 4.7
11.0 K

LOADING:

LIVE LOAD H20 TRUCK (NO IMPACT w/COVER > 3.0')

B = 1.75h = 1.75 x 6.2 = 10.85' SAY 11.0'



CONDITION I. TRUCK ⊥ CULVERT

$w_{LL_1} = \frac{2P}{A_1} = \frac{2 \times 16.0}{11.0(5.5+6.0+5.5)} = 0.17\ ^{K}/_{S_F}$

CONDITION II. TRUCK ∥ CULVERT

$w_{LL_{11}} = \frac{2P}{A_{11}} = \frac{2 \times 16}{11 \times 11} = 0.265\ ^{K}/_{S_F}$

DEAD LOAD = 0.125 x 6.2 + 0.15 x 1.0 = 0.925

TOTAL LOAD = LL + DL = 0.265 + 0.925 = 1.19 $^{K}/_{S_F}$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

BC - 2

FIXED-END MOMENTS

TOP SLAB $FEM = \frac{wL^2}{12} = 1.19(11.0)^2/12 = 12.00^{k}$ $M_s = 1.5 \times FEM = 18.0^{k}$

TOP / WALL $FEM = \frac{0.628(7.0)^2}{12} + \frac{0.656(7.0)^2}{30} = 3.64$ 5.5

BOT / WALL $FEM = \frac{0.628(7.0)^2}{12} + \frac{0.656(7.0)^2}{20} = 4.17$ 6.3

BOT SLAB $FEM = \frac{1.34(11.0)^2}{12} = 13.50$ 20.2

DISTRIBUTION $K = \frac{I}{L}$ $I \propto d^3$

$K_T = \frac{1.0^3}{11.0} = 0.091 \div 0.234 = 0.39$

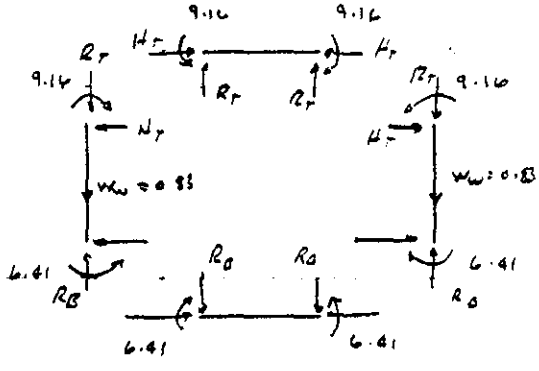
$K_W = \frac{1.0^3}{7.0} = \frac{0.143}{0.234} \div 0.234 = \frac{0.61}{1.00}$

$K_W = \frac{1.0^3}{7.0} = 0.143 \div 0.450 = 0.32$

$K_B = \frac{1.5^3}{11.0} = \frac{0.307}{0.450} \div 0.450 = \frac{0.68}{1.00}$

.61	.39	.39	.61
+ 3.64	-12.00	+ 12.00	- 3.64
+ 5.10	+ 3.26	- 3.26	- 5.10
- 1.50	- 1.63	+ 1.63	+ 1.50
+ 1.91	+ 1.22	- 1.22	- 1.91
- 0.92	- 0.61	+ 0.61	+ 0.92
+ 0.93	+ 0.60	- 0.60	- 0.93
+ 9.16	- 9.16	+ 9.16	- 9.16

.32	.68	.68	.32
- 4.17	+ 13.50	- 13.50	+ 4.17
- 2.99	- 6.34	+ 6.34	+ 2.99
+ 2.55	+ 3.17	- 3.17	- 2.55
- 1.83	- 3.89	+ 3.89	+ 1.83
+ 0.96	+ 1.94	- 1.94	- 0.96
- 0.93	- 1.97	+ 1.97	+ 0.93
- 6.41	+ 6.41	- 6.41	+ 6.41



$R_T = \frac{1.19 \times 11}{2} = 6.54^{k}$

$R_B = \frac{1.34 \times 11}{2} = 7.37^{k}$

$H_T = \frac{0.628 \times 7.0}{2} + \frac{0.656 \times 7.0}{6} + \frac{9.16 - 6.41}{7.0} = 3.3$

$H_B = \frac{0.628 \times 7.0}{2} + \frac{0.656 \times 7.0}{3} - \frac{9.16 - 6.41}{7.0} = 3.$

BOT SLAB. $w = \left(6.3 + 4.7 \times \frac{135}{62}\right)\frac{1}{12} - 1.5 \times 0.15 + 0.265 = 1.34\ ^{k}/_{ft}$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

BC-3

TOP SLAB

$R_T = 6.54^k$

$V = 6.54 - 1.19\left(\frac{9.5+6.0}{12}\right) = 5.0^k$

$v = \frac{V}{bd} = \frac{5000}{12 \times 9.5} = 44 \text{ psi}$

$-M = 9.16^{'k}$

$-A_s = \frac{-M}{ad} = \frac{9.16}{1.44 \times 9.5} \quad 0.67 \text{ in}^2$

$+M = 6.54 \times 5.5 - 1.19(5.5)^2/2 - 9.16 = 8.81^{'k}$

$+A_s = \frac{8.84}{1.44 \times 9.5} = 0.65 \text{ in}^2/2$

BOT. SLAB

$R_B = 7.37^k$

$V = 7.37 - 1.34\left(\frac{13.5+6.0}{12}\right) = 5.19^k$

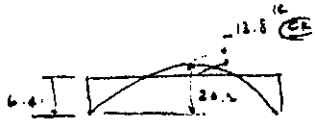
$v = \frac{5190}{12 \times 13.5} = 32.0 \text{ psi}$ < 40 psi NO KEY

$-M = 6.41^{'k}$

$-A_s = \frac{-M}{ad} = \frac{6.41}{1.44 \times 9.5} = 0.47 \text{ in}^2$

$+M = 7.37 \times 5.5 - 1.34(5.5)^2/2 - 6.41 = 13.9^{'k}$

$+A_s = \frac{+M}{ad} = \frac{13.9}{1.44 \times 13.5} = 0.72 \text{ in}^2$



USE: #6 @ 8" IN CORNERS A_s : 0.66
#7 @ 8" POS. STEEL A_s : 0.90

WALLS

$0.628x + \frac{0.0937}{2}x^2 = 3.36$

$x^2 + 13.40x = 71.72$

$x^2 + 13.40x + 44.92 = 71.72 + 44.92$

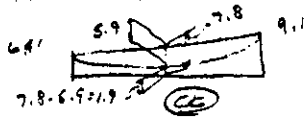
$(x + 6.70)^2 = 116.64$

$x + 6.70 = 10.80$

$x = 4.10'$

$+M = 3.36 \times 4.10 - 0.628(4.10)^2/2 - 0.0937(4.10)^3/6 - 9.16 = -1.74^{'k}$

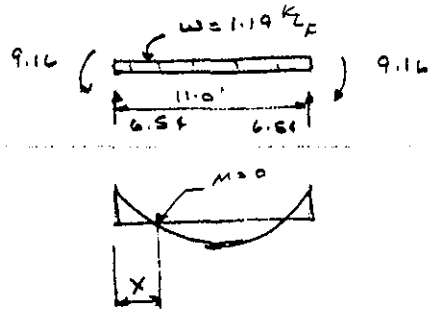
NO POSITIVE MOMENT



USE #5 @ 8" VIF. $A_s = 0.47$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

BC-4



$$9.16 + 1.19\frac{(x)^2}{2} - 6.54x = 0$$

$$x^2 - 10.99x = -15.39$$

$$x^2 - 10.99x + 30.20 = 30.20 - 15.39$$

$$(x - 5.50)^2 = 14.81$$

$$x - 5.50 = \pm 3.85$$

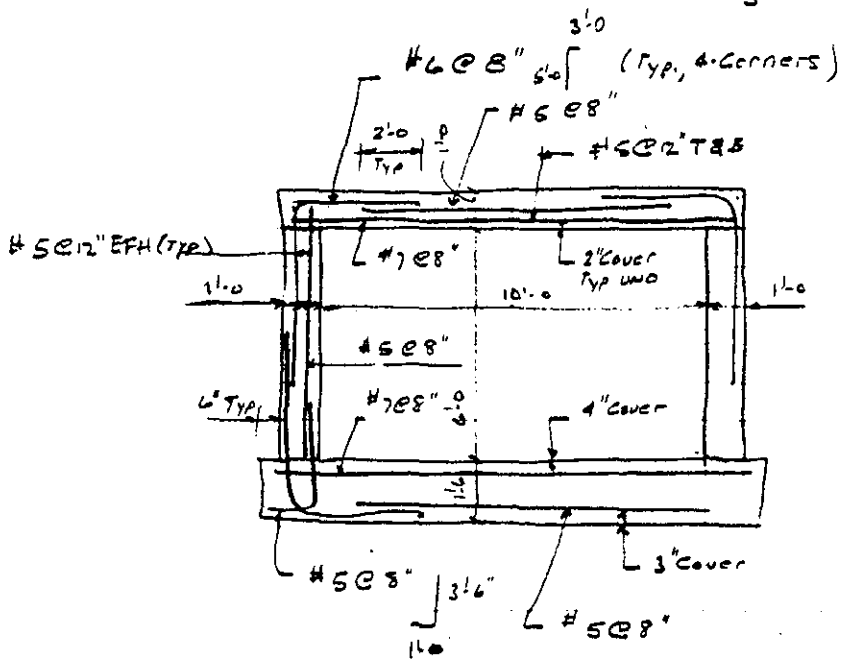
$$x = 5.50 - 3.85 = 1.65'$$

SAY 1'-8

EXTENSION BEYOND 0" MOM. + 1-8

3'-4"

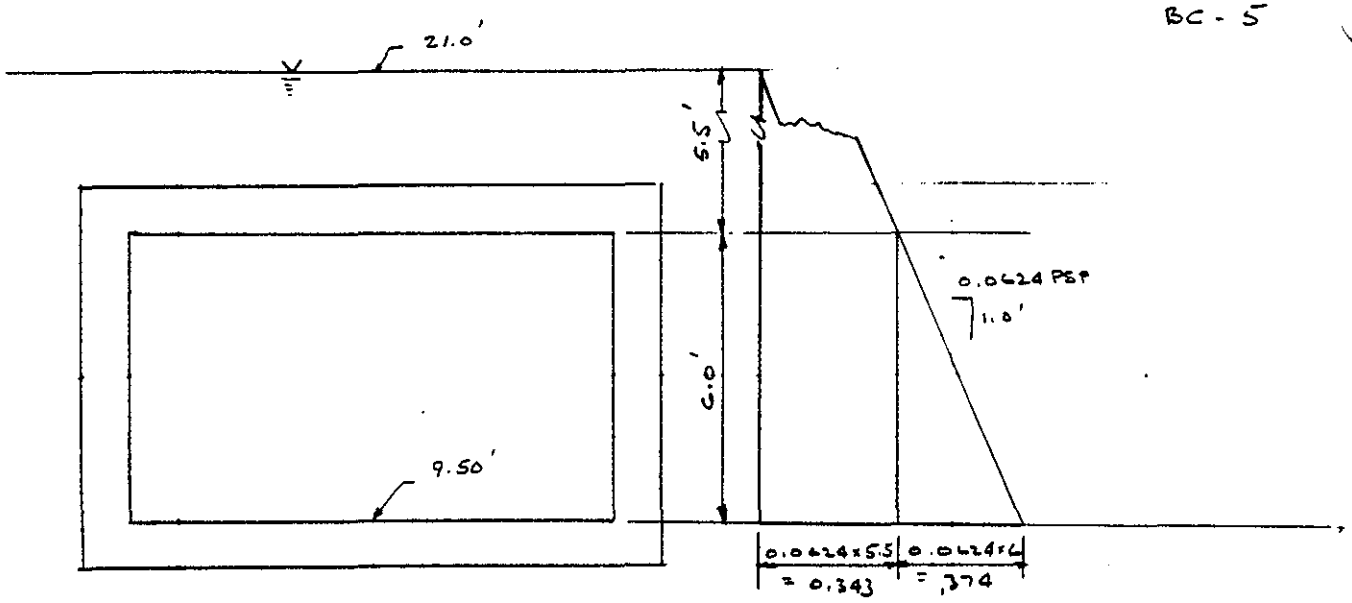
$$\frac{L}{5} + 16'' = \frac{11}{5} + 0.83 = 3.03'$$



NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

Project MDC - TOWN BROOK (QUINCY) Acct. No. 6041 Page 58 of 121
Subject 10' x 6' CULVERT Comptd. By J.T. HOWARD Date 8.09.84
Detail INTERNAL PRESSURE Ck'd. By G. Volpini Date 9/9/84

BC - 5



TOP SLAB

w = INTERNAL PRESSURE − SLAB (NEGLECT SOIL ABOVE SLAB)

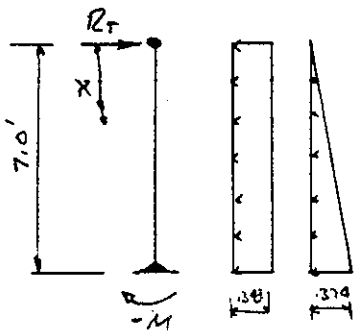
$= 0.343 - 0.15 = 0.193\ ^k/_F$ SAY $w = 0.20\ ^k/_F$; PINNED ENDS

$M = \frac{wl^2}{8} = \frac{0.20(11.0)^2}{8} = 3.0^{'k}$

$A_s = \frac{3.0}{1.44 \times 9.5} = 0.22\ in^2$ REQ'D. OK

HAVE #5 @ 8" = 0.47 in^2

WALLS



$0.343x + \frac{0.0624x^2}{2} = 1.16^k$

$x^2 + 11.0x = 37.18$

$x^2 + 11.0x + 30.25 = 37.18 + 30.25$

$(x + 5.5)^2 = 67.43$

$x + 5.5 = 8.21$

$x = 2.71'$

$-M = \frac{0.343(7.0)^2}{8} + \frac{0.374(7.0)^2}{15} = 2.10 + 1.22 = 3.32^{'k}$

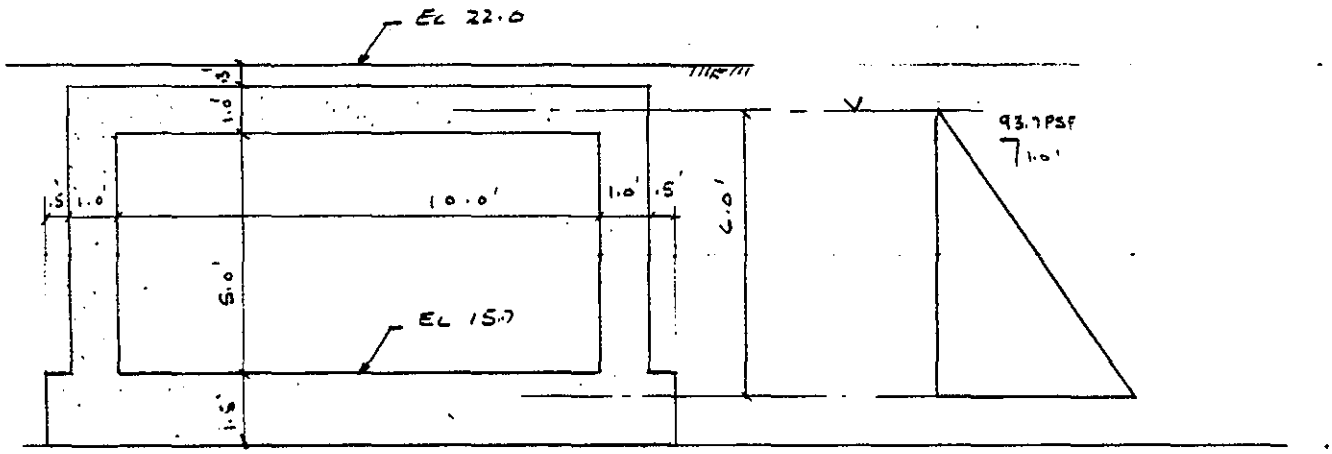
$-A_s = \frac{-M}{ad} = \frac{3.32}{1.44 \times 9.5} = 0.24\ in^2$

$R_T = \frac{0.343 \times 7.0}{2} + \frac{0.374 \times 7.0}{2 \times 3} - \frac{3.32}{7.0} = 1.20 + 0.44 - 0.47 = 1.16^k$

$+M = 1.16 \times 2.71 - 0.343(2.71)^2/2 - 0.0624(2.71)^3/6 = 1.71^{'k}$

$+A_s = \frac{+M}{ad} = \frac{1.71}{1.44 \times 9.5} = 0.12\ in^2$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS



SCALE 1/4" = 1'-0"

FLOATION:-

UPLIFT = 12.0 x 7.5 x 0.0624 = 5.62^k

DEAD LD = (12.0 x 7.8 − 10.0 x 5) 0.15 = 6.54

$F.S = \frac{6.54}{5.62} = 1.16$

LOADING:-

LIVE LOAD = H20 TRUCK + 30% IMPACT

CASE A. REBARS ⊥ TRAFFIC

$M_s = \frac{11.042}{32}(16.0)(1.3) = 8.45^{'k}$ (AASHTO)

CASE B. REBARS ∥ TRAFFIC

$M_s = 0.9 \times 11 \times 1.3 = 12.87^{'k}$ (AASHTO)

EK $M_s = \frac{PL}{4} \times \frac{I}{E} = \frac{16.0 \times 11.0}{4} \cdot \frac{1.3}{4.44} = 12.88^{'k}$

$FEM = M_s/2 = 12.88/2 = 6.44^{'k}$

DEAD LOAD $w = 0.15 \times 1.3 = 0.20^{k}/_{sf}$ (TOP)

LIVE LOAD $w = 16.0/_{4.4 \times 11.0} = 0.328^{k}/_{sf}$ (BOT) NO IMPACT

DEAD LOAD $w = \frac{6.64}{12.0} - 1.5 \times .16 = 0.320^{k}/_{sf}$ (BOT)

NONREPRODUCIBLE GRID FORM 145
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BC-7

FIXED END MOMENTS

TOP SLAB:- $FEM = \frac{0.20(11.0)^2}{12} + 6.44 = 8.46^{1K}$ $M_s = 15.90$

TOP WALL:- $FEM = \frac{0.0937(6.0)^3}{30} = 0.67$

BOT WALL:- $FEM = \frac{0.0937(6.0)^3}{20} = 1.00$ $M_s = 1.26$

BOT. SLAB:- $FEM = \frac{(0.32+0.33)(11.0)^2}{12} = 6.55$ $M_s = 9.82$

DISTRIBUTION $K = \frac{I}{L}$; $I \propto d^3$

$K_T = \frac{1.0^3}{11.0} = 0.091 \div 0.258 = 0.35$ (SLAB)

$K_W = \frac{1.0^3}{6.0} = \frac{0.167}{0.258} \div 0.258 = 0.65$ (WALL)

$K_W = \frac{1.0^3}{6.0} = 0.167 \div 0.474 = 0.35$ (WALL)

$K_B = \frac{1.5^3}{11.0} = \frac{0.307}{0.474} \div 0.474 = 0.65$ (SLAB)

.65	.35	.35	.65
+0.67	-8.46	+8.46	-0.67
+5.06	+2.73	-2.73	-5.06
-0.97	-1.36	+1.36	+0.97
+1.51	+0.82	-0.82	-1.51
-0.76	-0.41	+0.41	+0.76
+0.76	+0.41	-0.41	-0.76
+6.27	-6.27	+6.27	-6.27

.35	.65	.65	.35
-1.00	+6.55	-6.55	+1.00
-1.94	-3.61	+3.61	+1.94
+2.53	+1.80	-1.80	-2.53
-1.52	-2.81	+2.81	+1.52
+0.75	+1.41	-1.41	-0.75
-0.76	-1.40	+1.40	+0.76
-1.94	+1.94	-1.94	+1.94

NONREPRODUCIBLE GRID FORM 145
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Subject 10'x5' CULVERT Comptd. By J.T HOWARD Date 8-16-84
Detail ALTERNATE LOADINGS Ck'd. By [signature] Date 8/16-84

BC-8

B. MAX. VERTICAL LOAD – MIN LATERAL LOAD.

FIXED END MOMENTS:

TOP SLAB: (FROM PREVIOUS WORK) = 8.46 ^{1K}

TOP WALL: 0.67 x 0.0625/0.0937 = 0.45

BOT WALL: 1.00 x 0.0625/0.0937 = 0.67

BOT SLAB: (FROM PREVIOUS WORK) = 6.55

DISTRIBUTION (FROM PREVIOUS WORK)

C. MIN. VERTICAL LOAD – MAX LATERAL LOAD

FIXED END MOMENTS:

TOP SLAB 8.46 – 6.44 = 2.02 ^{1K}

TOP WALL = 0.67

BOT WALL = 1.00

BOT SLAB $\frac{0.32(11.0)^2}{12}$ = 3.23

CASE B

.65	.35	.35	.65
+0.45	-8.46	+8.46	-0.45
+5.21	+2.80	-2.80	-5.21
-1.03	-1.40	+1.40	+1.03
+1.58	+0.85	-0.85	-1.58
-0.79	-0.43	+0.43	+0.79
+0.79	+0.43	-0.43	-0.79
+6.21	-6.21	+6.21	-6.21

.35	.65	.65	.35
-0.67	+6.55	-6.55	+0.67
-2.06	-3.82	+3.82	+2.06
+2.60	+1.91	-1.91	-2.60
-1.58	-2.93	+2.93	+1.58
+0.79	+1.47	-1.47	-0.79
-0.79	-1.47	+1.47	+0.79
-1.71	+1.71	-1.71	+1.71

$+M_T = 15.9 - 6.2 = 9.7^{1K}$

$+M_W = 1.26 - \left(\frac{6.21+1.71}{2}\right) = -2.7$

$+M_B = 9.82 - 1.71 = 8.11^{1K}$

CASE C

.65	.35	.35	.65
+0.67	-2.02	+2.02	-0.67
+0.88	+0.97	-0.97	-0.88
-0.97	-0.24	+0.24	+0.97
+0.79	+0.42	-0.42	-0.79
-0.39	-0.21	+0.21	+0.39
+0.39	+0.21	-0.21	-0.39
+1.37	-1.37	+1.37	-1.37

.35	.65	.65	.35
-1.00	+6.55	-6.55	+1.00
-1.94	-3.61	+3.61	+1.94
+0.44	+1.80	-1.80	-0.44
-0.78	-1.46	+1.46	+0.78
+0.40	+0.73	-0.73	-0.40
-0.40	-0.73	+0.73	+0.40
-3.28	+3.28	-3.28	+3.28

$+M_T = 2.02 \times 1.5 - 1.37 = 1.66^{1K}$

$+M_W = \frac{0.0937(6.0)^3}{16} - \left(\frac{3.28+1.37}{2}\right) = -1.06$

$+M_B = 3.23 \times 1.5 - 3.28 = 1.56$

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Detail RESULTS Ck'd. By [signature] Date 8-16-84

BC-9

TOP SLAB:-

$-M = 6.27^{1K}$

$-A_s = \frac{-M}{ad} = \frac{6.27}{1.44 \times 9.5} = 0.46 \text{ IN.}^2$ USE: #6 @12" ($A_s = 0.44 \text{ IN.}^2$)

$R \text{ (FOR MOM)} = 0.20 \times 11.0/2 + \frac{16.0 \times 1.3}{4.44 \times 2} = 3.44^K$

$+M = 3.44 \times 5.5 - 0.20(5.5)^2/2 - 6.27 = 9.62^{1K}$

$15.9 - 6.3 = 9.6^{1K}$ (CK)

$+A_s = \frac{+M}{ad} = \frac{9.62}{1.44 \times 9.5} = 0.70 \text{ IN.}^2$ USE: #8@12" ($A_s = 0.79 \text{ IN.}^2$)

$V_{MAX} = 0.20(8.42)/2 + \frac{16.0 \times 1.3}{4.44} \times \frac{9.71}{11.00} = 0.84 + 4.14 = 4.98^K$

$v = V/bd = \frac{4980}{12 \times 9.5} = 44 \text{ psi}$

BOT SLAB:-

$-M = -3.28^{1K}$

$-A_s = \frac{-M}{ad} = \frac{3.28}{1.44 \times 9.5} = 0.24 \text{ IN.}^2$

$R = (0.32 + 0.33)\,11.0/2 = 3.58$

$+M = 3.58 \times 5.5 - 0.65(5.5)^2/2 - 1.94 = 7.92$

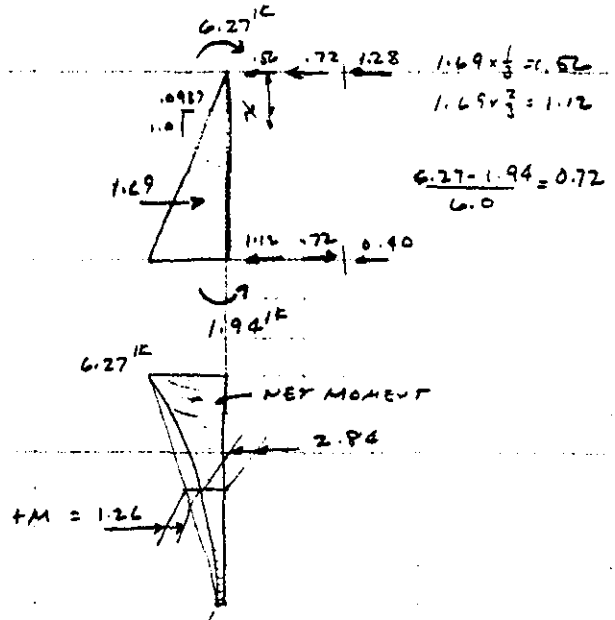
$+9.82 - 1.94 = 7.88$ (CK)

$+A_s = \frac{+M}{ad} = \frac{8.11}{1.44 \times 13.5} = 0.42 \text{ IN.}^2$ USE: #6@12" T. ($A_s = 0.44$)

$V = R - wX = 3.58 - 0.65\left(\frac{13.5 + 6.0}{12}\right) = 2.52^K$

$v = \frac{V}{bd} = \frac{2520}{12 \times 13.5} = 16 \text{ psi}$

WALLS



$1.69 \times \frac{1}{3} = 1.56$

$1.69 \times \frac{2}{3} = 1.12$

$\frac{6.27 - 1.94}{6.0} = 0.72$

$\frac{0.0937x^2}{2} = 1.28$

$x = 5.23'$

∴ NO POSITIVE MOMENT

$-\left(\frac{6.27 + 1.94}{2}\right) + 1.26 = -2.84^{1K}$

$v = \frac{1280}{12 \times 9.5} = 12. \text{ PSI}$

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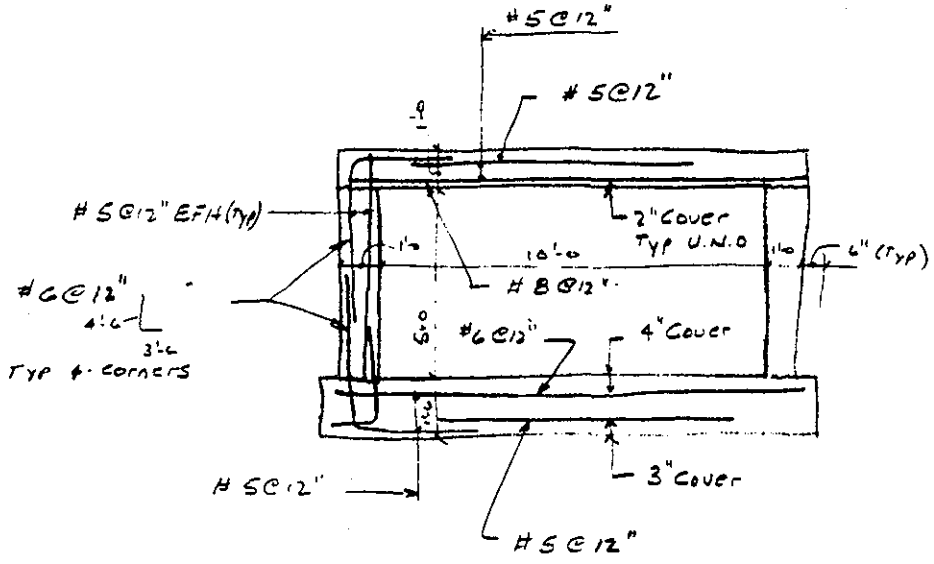
Detail TYPICAL SECTION Ck'd. By [signature] Date 9-16-84

BC-10



NONREPRODUCIBLE GRID FORM 145

METCALF & EDDY, ENGINEERS

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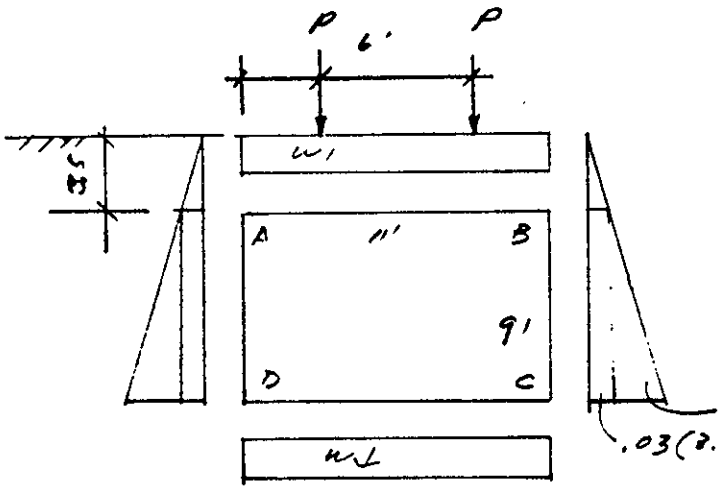
Detail Structural Ck'd. By [illegible] Date [illegible]

BC-12

Case 2A

SSS-2



$P = 20.8^k$

$P_D = \frac{20.8}{(1.75 \times 3)^2} = .75^{ksf}$

$w_1 = 3'$ soil + slab DL + HS20 dist. load

$= 3(.13) + .15 + .75 = 1.3^{ksf}$

$.03(9') = .27^{ksf}$

$.03(3.5') = .10^{ksf}$

$w_4 = \frac{w_1(12) + 2(.15 \times 8)}{12} - .15$

$= 1.3^{ksf}$

$FEM_{AB} = \frac{1.3(11)^2}{12} = 13.1^{k\text{-}ft}$

$FEM_{DC} = 13.1^{k\text{-}ft}$

$FEM_{BC} = \frac{.10(9)^2}{12} + \frac{.03(9)^3}{30} = .7 + .73 = 1.4^{k\text{-}ft}$

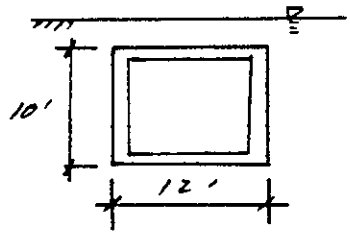
$FEM_{CB} = .70 + \frac{.03(9)^3}{20} = 1.8^{k\text{-}ft}$

From Moment distribution Results (see SSS-4)

$M^+_{AB} = \frac{1.3(11)^2}{8} - 7.8 = 11.9^{k\text{-}ft}$

$V_A = 1.3(\frac{11}{2}) = 7.2^k$, $V_{ed} = 7.2 - \frac{6+9.5}{12}(1.3) = 5.6^k$

check uplift



$U = 10 \times 12 \times .063 = 7.6^k \uparrow$

$DL = 2 \times .15 \times 12 + 2 \times .15 \times 8 = 3.6 + 2.4 = 6^k \downarrow$

$7.6 > 6^k$, need weepholes

Try 4" ø weepholes @ 4'-0" above base (same as for Type I Precast Culverts)

↙ 1' above WH

$U_{4'} = 6' \times 12 \times .063 = 4.5^k \uparrow$

$\therefore$ S.F. uplift $= \frac{6.0}{4.5} = 1.33 > 1.25$, OK.

Use weepholes @ 4'-0" o.c. Eq.s.

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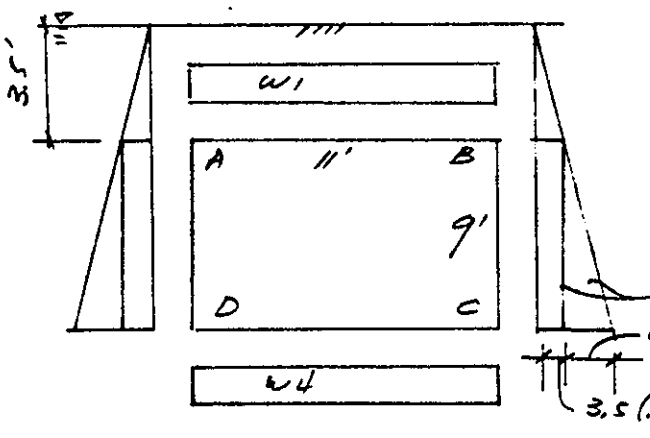
Subject School Street Structure Comptd. By J. MOOERS Date 3/31/80
Detail Structural Ck'd. By Avery Date [illegible]

BC-13

SSS-3

Case 3



w_1 = sat. soil + slab dl.

$= .065(3) + .15 = .34^{ksf}$

w_2 = soil pressure = 65 pcf

w_3 = ground water = 35 pcf

$w_2 + w_3$ = 100 pcf

$w_4 = \frac{12(w_1) + 2(.15 \times 8)}{12} - .15$

$= .39^{ksf}$

$FEM_{AB} = \frac{.34(11)^2}{12} = 3.4^{k\text{-}ft}$ $FEM_{DC} = \frac{.39(11)^2}{12} = 3.9^{k\text{-}ft}$

$FEM_{BC} = \frac{.35(9)^2}{12} + \frac{.9(9)^2}{30} = 2.4 + 2.4 = 4.8$

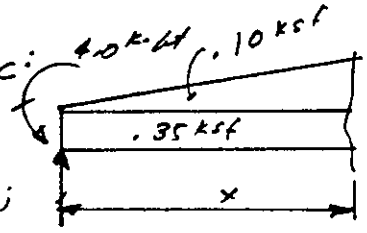
$FEM_{CB} = 2.4 + \frac{.9(9)^2}{20} = 6.0^{k\text{-}ft}$

Distribution Factors

$K_r = \frac{12^3}{11} = 157 = K_b$ $K_w = \frac{12^3}{9} = 192$

$DF_{AD} = \frac{157}{157+192} = .45, .55$

M^+_{BC}:



$2.9^k = V_{ad;BC}$

$V_{ad;CB} = 1.6 + 2(1.4) + .09 = 4.5^k$

see Moment Dist. Results in SSS-4

$V_{ad;BC} = .35\left(\frac{9}{2}\right) + \frac{.1(9)^2}{2} \times \frac{1}{3} - \frac{(4.8-4.0)}{9'}$

$= 1.6 + 1.4 - .09 = 2.9^k$

$V_x = 0: \; 2.9 - .35x - \frac{.1x^2}{2}$

$x = 4.9'$

$M^+_{4.9'} = 2.9(4.9) - 4 - \frac{.35(4.9)^2}{2} - \frac{.1(4.9)^3}{6}$

$= 14.2 - 4 - 4.2 - 2 = +4.0^{k\text{-}ft}$

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Case 2A

	A 11'		B 9'		C 11'		D 9'
DF	.55	.45	.45	.55	.55	.45	.45 .55
FEM	+1.4	-13.1	+13.1	-1.4	+1.8	-13.1	+13.1 -1.8
D,	+6.4	+5.3	-5.3	-6.4	+6.2	+5.1	-5.1 -6.2
M-		-7.8		-7.8		$\boxed{-8.0}$	-8.0
M+		$\boxed{+11.9}$				+11.9	
Vad;		7.2k				7.2k	
Vad		5.6k				$\boxed{5.6^k}$	

Case 3

DF	.55	.45	.45	.55	.55	.45	.45	.55
FEM	+4.8	-3.4	+3.4	-4.8	+6.0	-3.9	+3.9	-6.0
D,	-.8	-.6	+.6	+.8	-1.2	-.9	+.9	+1.2
M-		-4.0		-4.0		-4.8		-4.8
M+				$\boxed{+4.0}$				
Vad;					4.5k			

12" Roof | 12" Wall | 12" Base | 12" Wall

$M_{R_{12''}} = .226(9.5)^2 = 20.4^{k\text{-}ft} > 11.9^{k\text{-}ft}$, ok

$V_{wall} = .06(12 \times 9.5) = 6.8^k > 5.6^k$, ok.

$V_{base\ \&\ wall} = .04(12 \times 9.5) = 4.6^k > 4.5^k$, ok.

$A_s^+ = \frac{11.9}{1.44(9.5)} = .87\ in^2$

$A_s\ min = .002(12 \times 9.5) = .23\ in^2$ → #5 @ 12" E.F.

$A_s^+ = \frac{4.0}{1.44(9.5)} = .29\ in^2$

$A_s^- = \frac{8.0}{1.44(9.5)} = .58\ in^2$

12" Roof	12" Wall	12" Base	12" Wall
#7 @ 12" T	#5 @ 12" E.W.E.F.	#9 @ 12" T	#5 @ 12" E.W.E.F.
#8 @ 10" B		#7 @ 12" B	

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Max. Bg. Pressure

Case 2A → w_4 = 1.3 ksf (← includes periodic HS20 load)

Culvert full of water
= .163 × 8' = .50

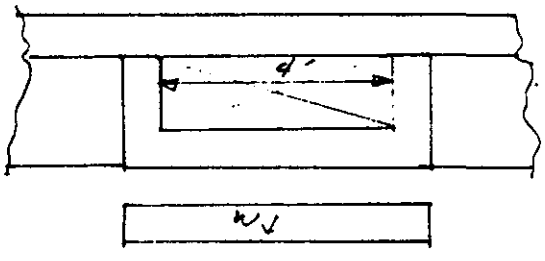
2'-0" Conc. mat = .30

1'-0" base slab = .15

2.25 ksf > 2.0 ksf , but 1.8 ksf with 1'-0" L_{20}, more common condition

∴ Bearing pressure ok.

12" Spread slab



Let w_4 = 1.3 ksf

$$M = \frac{1.3(4)^2}{8} = 2.6 \text{ k-ft}$$

$$As_{12''} = \frac{2.6}{1.44(2.5)} = .21 \text{ in}^2$$

use 12" all around w/ #5 @ 12" E.F.E.W.

RETAINING WALLS

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Subject SOUTH RETAINING WALL Comptd. By V. T HOWARD Date 9-05-84
Detail STABILITY Ck'd. By J. C. Finitz Date 9-7-84

R.W-2

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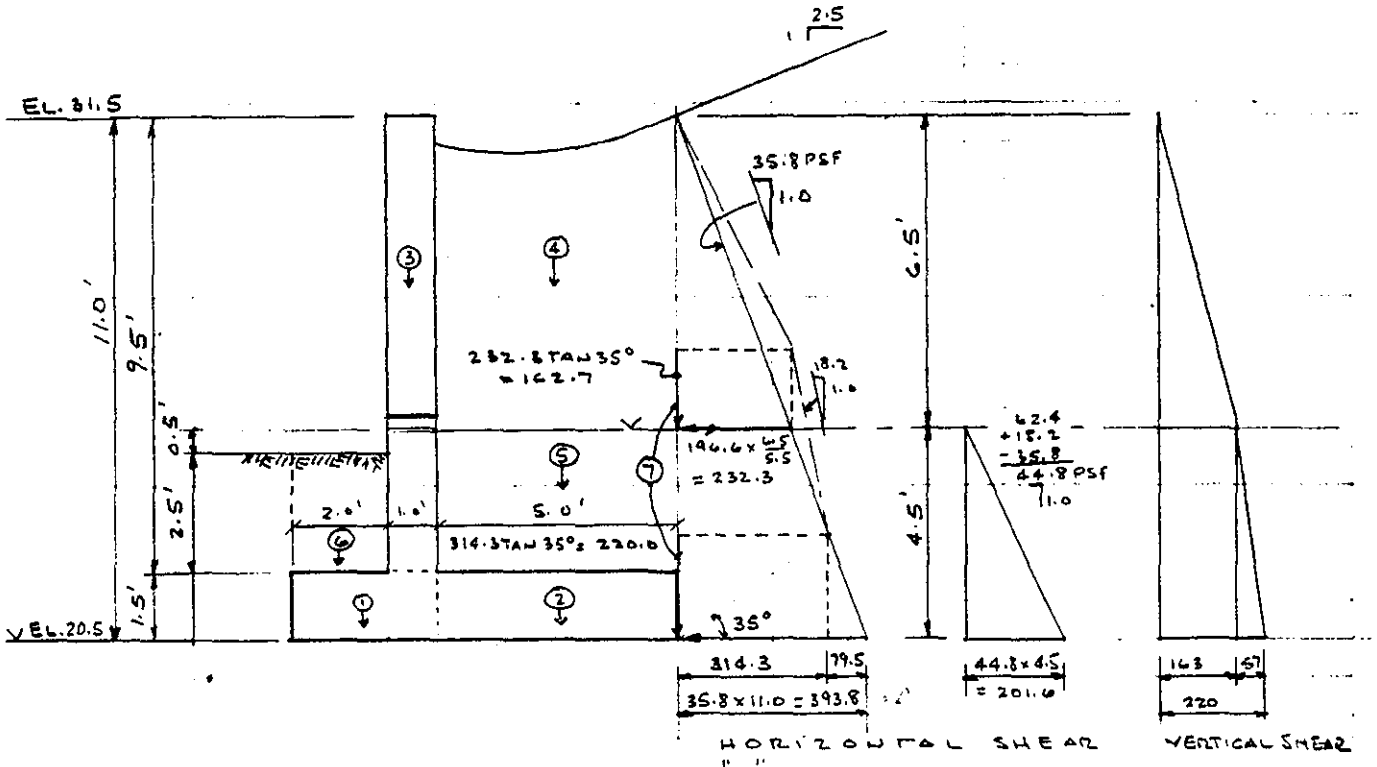


SUMMATION OF MOMENTS ABOUT "A"

$163 \times 6.5/2 = 530$
$163 \times 4.5 = 733$
$57 \times 4.5/2 = 127$
1.39

	W × H × W	= P × a = M
①	3.0 × 1.5 × 0.15	= 0.68 × 1.50 =
②	5.0 × 1.5 × (0.15 − 0.06)	= 0.67 × 5.50 =
③	1.0 × 9.5 × 0.15	= 1.42 × 2.50 =
④	5.0 × 6.0 × 0.12	= 3.60 × 5.50 =
⑤	5.0 × 3.0 × (0.12 − 0.06)	= 0.90 × 5.50 =
⑥	2.0 × 2.5 × 0.12	= 0.60 × 1.00 =
⑦	(VERTICAL SHEAR)	= 1.39 × 8.00 =
		P = 9.26 M = 44.74

⑧ $0.0358(11.0)^2/2 = 2.16$ — × $11.0/3$ = − 7.92
⑨ $0.0448(4.5)^2/2 = 0.45$ — × $4.5/3$ = − 0.68

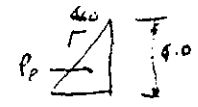
H = 2.61 P = 9.26^K M = 36.15^{1K}

$$\bar{X} = \frac{\Sigma M}{\Sigma P} = \frac{36.15}{9.26} = 3.90' \qquad e = \frac{8.0}{2} - 3.90 = 0.10'$$

Soil Pressure: $p = \frac{P}{W}\left(1 \pm \frac{6e}{W}\right) = \frac{9.26}{8.0}\left(1 \pm \frac{6 \times 0.10}{8.0}\right) = 1.16 \pm 0.09$
$= 1.25\ ^{K}/_{F}$ MAX
$= 1.07\ ^{K}/_{F}$ MIN

OVERTURNING:- $F.S = \frac{44.74}{8.60} = 5.2$

SLIDING:- $F.S = \frac{9.26 \times 0.40 + 3.68}{2.61} = 2.83$



$P_p = 0.46(4.0)^2/2 = 3.6$

R.W-3

STEM:-

BASE:-

$$V = 35.8\,(9.5)^2/2 + 44.8\,(3.0)^2/2 = 1615 + 202 = 1817^{\text{lb}}$$

$$v = \frac{V}{bd} = \frac{1817}{12 \times 9.5} = 16 \text{ psi} < 40 \text{ psi} \quad \text{NO KEY}$$

$$M = 1.62 \times 9.5/3 + 0.20 \times 3/3 = 5.13 + 0.20 = 5.33^{\text{'k}}$$

$$A_s = \frac{M}{ad} = \frac{5.33}{1.44 \times 9.5} = 0.39 \text{ in.}^2$$

USE: # 6 @ 12" V REAR FACE.

1.5' ABOVE BASE:

$$M = 0.0358(8.0)^3/6 + 0.0448(1.50)^3/6 = 3.05 + 0.03 = 3.08^{\text{'k}}$$

$$A_s = \frac{M}{ad} = \frac{3.08}{1.44 \times 9.5} = 0.23 \text{ in.}^2$$

USE: # 5 @ 12" VRF 2.5' ABOVE BASE

FOOTING:

TOE:

$$V = 1.25 \times 2.0 = 2.50^{\text{k}}$$

$$v = \frac{V}{bd} = \frac{2500}{12 \times 14} = 15 \text{ psi}$$

$$M = 2.50 \times 1.0 = 2.50^{\text{'k}}$$

$$A_s = M/ad = 2.50/1.44 \times 14 = 0.13 \text{ in.}^2$$

HEEL

$$V = 5.0 \times 9.0 \times 0.12 + 5.0 \times 1.5 \times 0.15 = 5.4^{\text{k}} + 1.1^{\text{k}} = 6.5^{\text{k}}$$

$$v = \frac{V}{bd} = \frac{6500}{12 \times 15.5} = 35 \text{ psi}$$

$$M = 6.5 \times 5.0/2 - 1.0(5.0)^2/2 = 16.25 - 12.50 = 3.75^{\text{'k}}$$

$$A_s = M/ad = \frac{3.75}{1.44 \times 15.5} = 0.17 \text{ in.}^2$$

$$A_{s\,MIN} = 12 \times 18 \times .002 = 0.43 \text{ in.}^2$$

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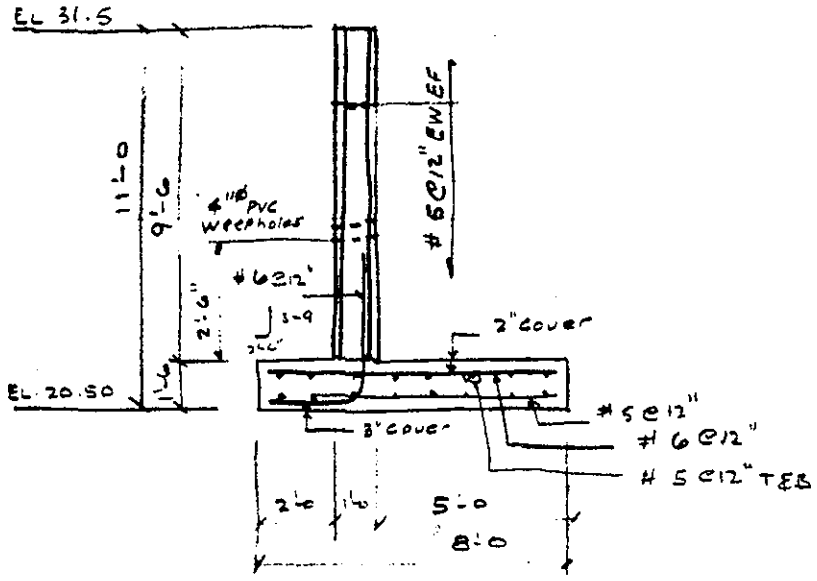
METCALF & EDDY, ENGINEERS

Project	MDC- TOWN BROOK (QUINCY)	Acct. No.	6041	Page	72 of 121
Subject	SOUTH RETAINING WALL	Comptd. By	J. THOWARD	Date	8-07-84
Detail	TYPICAL SECTION	Ck'd. By	J. Howard	Date	9/14/84

JTH REV. 9-05-84

RW-4



NONREPRODUCIBLE GRID FORM 145

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INTAKE SHAFT

LINING OF SHAFTS IN SOIL

METHOD: Perform all analyses on the Inlet Shaft, as this is the shaft with greatest depth to bedrock, and also the one with unbalanced earth levels at the top.

General approach will be to

(1) apply full external water load, and calculate concrete stresses;

(2) apply full external soil load, and resolve into radial and unbalanced pressures;

(a) calculate compressive stresses from radial forces

(b) " bending " " " unbalanced ".

(3) Add all stresses

(4) Introduce reinforcing or concrete thickness if stresses are tensile.

In all of the above, assume for a start that lining thickness = 12".

GIVEN DATA

Top of Bedrock - El -47

Top of ground adjacent to shaft (after const) El. 23

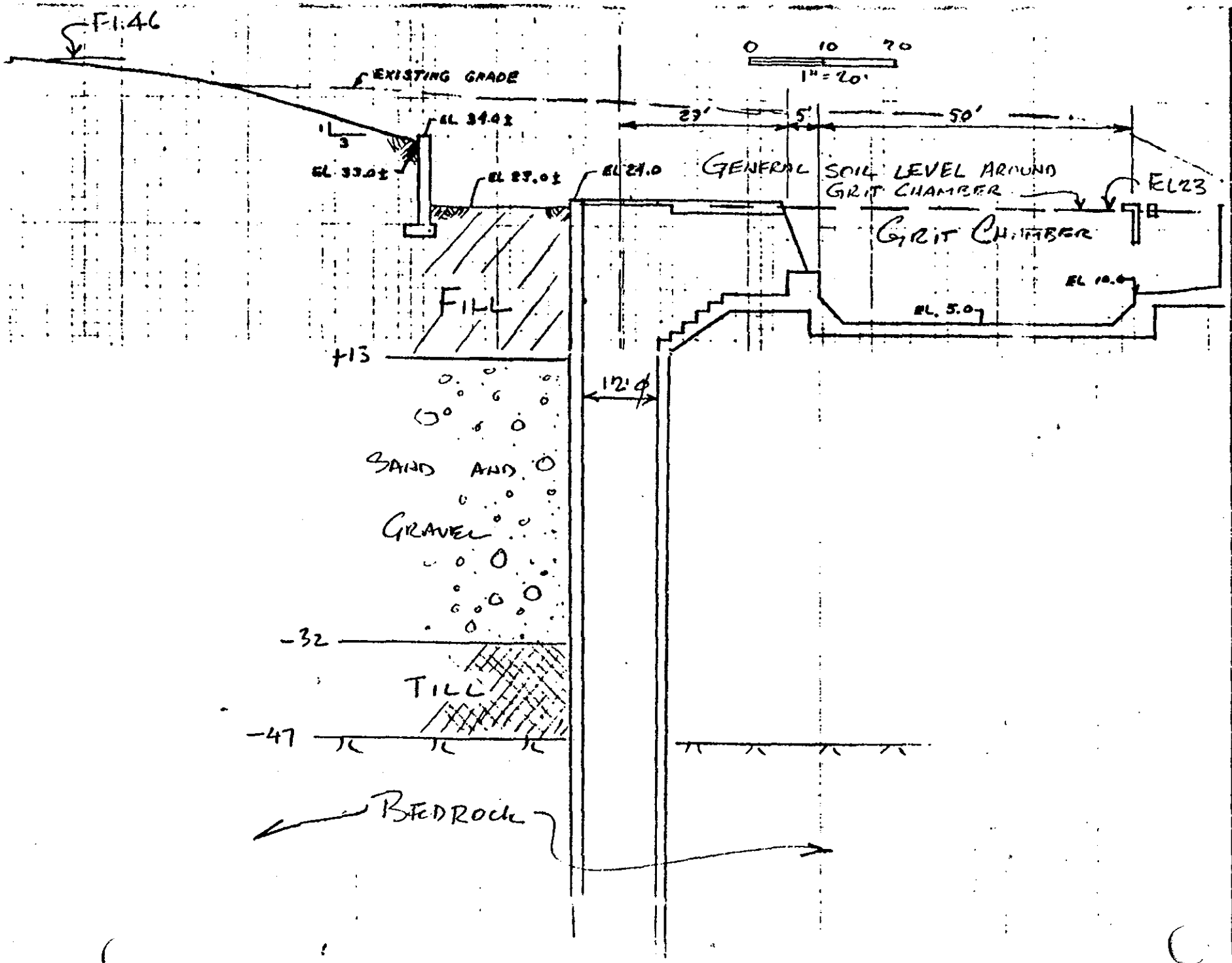
El of groundwater (assume at grnd surface) - El. 23

Ground parameters as below (See TP 25 of Design Rpt):

Depth	Elev	Material	cohesion	ϕ	γ_m	γ_{sat}	γ_{sub}
0	40	Fill	1000	0	120	130	70
27	13	Sand/G	0	30	120	150	90
72	-32	Till	4000	0	120	150	90
87	-47						

A cross section taken through the highest ground, then turning to go through the basin of the grit chamber, is as shown on p. 2

APPENDIX F



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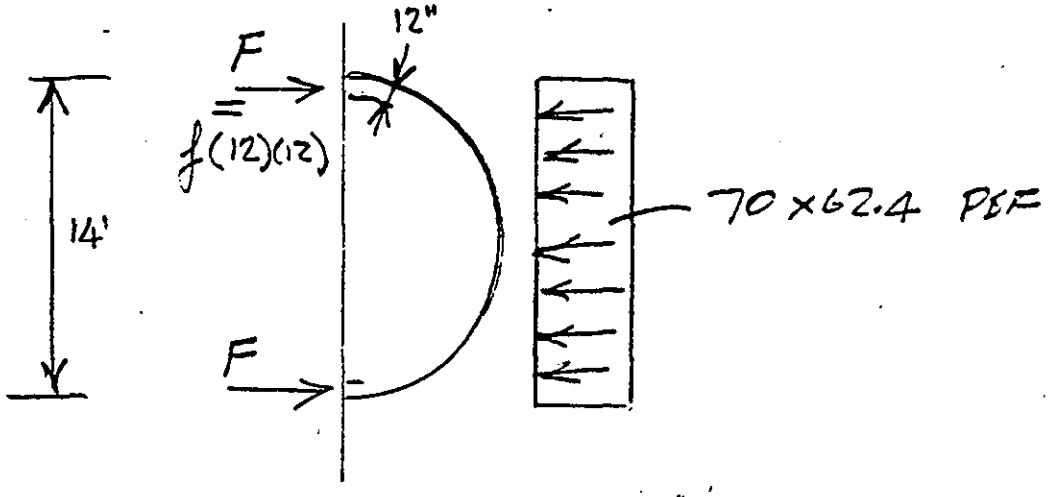
Boston, Massachusetts 02114

By FTW | Date 3/21/84 | Page 2 of 7
Project Town Brook | No. MF-6

STRESS DUE TO GROUNDWATER

Max groundwater head = -23-(-47) = 70'



$$2F = 2(f_w)(12)(12) = 70(62.4)(14)(1)$$

$$\frac{\# \ \cancel{in} \ \cancel{in}}{\cancel{in}^2} = \frac{\cancel{FT} \ \# \ \cancel{FT} \ \cancel{FT}}{\cancel{FT}^3}$$

$$\therefore f_w = \frac{70(62.4)(14)}{2(12)(12)}$$ ✓ Where f_w = pressure in psi

$$= 212 \text{ psi}$$

SOIL LOADS

Assume that max soil pressure on one coordinate axis is at-rest pressure due to soil at El +46, and minimum pressure on other axis is from soil at El. +23.0 (See sketch on p. 2). Further, assume that pressures are equal to

$$P_h = k_o \gamma Z$$

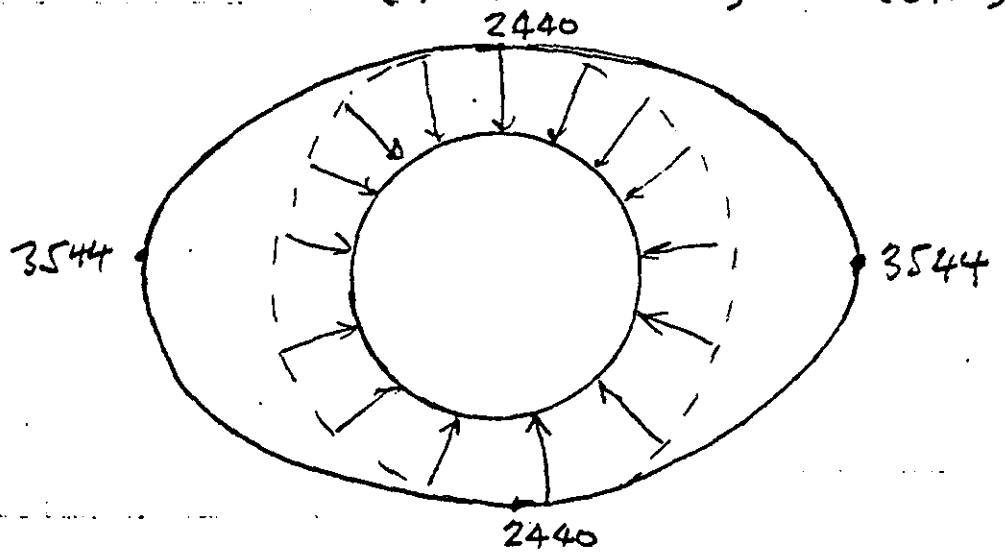
where k_o = coeff of earth pres at rest = 0.4
(See p. 417, Peck Hanson & Thornburn).

Max stress = .4[(46-23)(120) + (23-13)(70) + (13+32)(90) + (47-32)(

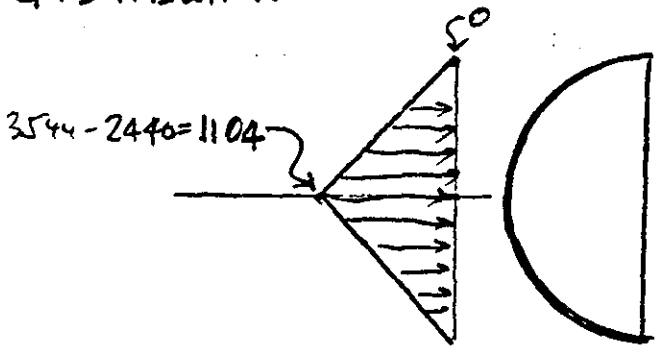
= .4(2760 + 700 + 4050 + 1350) = .4(8860) = 3544 psf

Min stress = .4[(23-13)70 + (13+32)90 + (47-32)90]

= .4(700 + 4050 + 1350) = .4(6100) = 2440 psf



The total load on the lining is given by the solid elliptical line. Consider a radial stress of 2440 psf to be applied, plus another stress given approximately by the following triangular stress distribution:



STRESS DUE TO RADIAL SOIL LOAD

As for groundwater on p. 3,

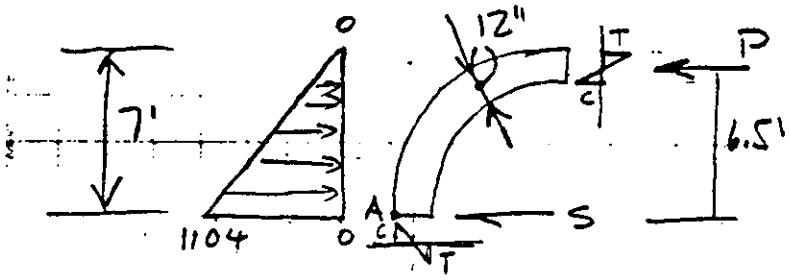
$$2(f_r)(12)(12) = 2440(14)(1)$$

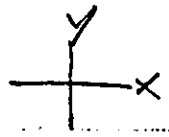
f_r = stress due to radial soil load

$$= \frac{2440(14)}{2(12)(12)} = 119 \text{ psi}$$

STRESS DUE TO TRIANGULAR SOIL LOAD

Consider free body at points of maximum bending stress, as follows:





Assume that couples at 1/4 points are equal.

$$\Sigma F_x = 0 \quad \therefore \quad \frac{1104+0}{2}(7)(1) = P + S$$

$$\Sigma M_A = 0 \quad \therefore \quad \frac{1104+0}{2}(7)\left(\frac{7}{3}\right) = 6.5(P)$$

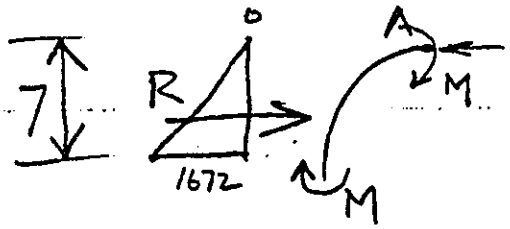
$$P = \frac{1104(7)(7)}{2(3)(6.5)} = 1387 \#$$

$$S = \frac{(1104)7}{2} - 1387 = 3864 - 1387 = 2477 \#$$

Compressive stress $= \frac{P}{A} = \frac{1387}{12 \times 12} = 10$ psi

Shear stress $= \frac{S}{A} = \frac{2477}{144} = 17$ psi

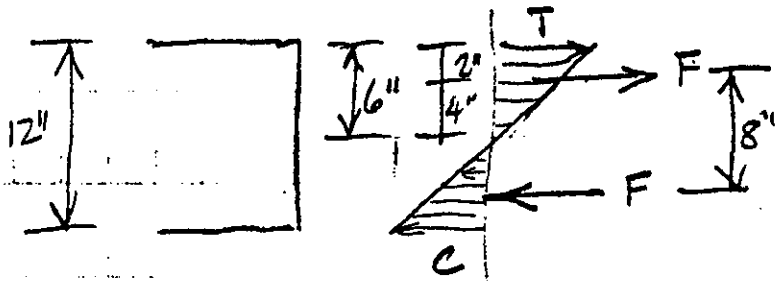
To evaluate moment, assume that section cracks, and that equilibrium is provided by moments at quarter points



$R = \frac{1104 + 0}{2}(7)(1) = 3864$ #

$\Sigma M_A = 0 \quad \therefore M + M = 3864 \times 2/3 \times 7 = 18{,}032$ ~~#~~ ft

$M = 18{,}032/2 = 9{,}016$ ft ~~#~~



$8F = 9{,}016 \times 12$

$F = 13{,}524$ #
(per foot of shaft)

$\frac{T + 0}{2} \times 6 \times 12 = 13{,}524$

$\therefore T = C = \frac{13{,}524 \times 2}{6 \times 12} = \underline{\underline{376 \text{ psi}}}$

MAXIMUM STRESSES

groundwater, radial soil, Triangular, bending

Maximum Compressive Stress = 212 + 119 + 10 + 376

= 717 psi OK ✓

Minimum Compressive Stress:

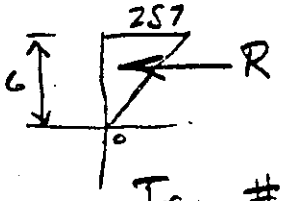
Occurs when the shaft is filled with water, and there is no net external water load; at axis where max soil load occurs.

Min. comp stress = 119 + (-376) = 257 psi (Tension)
(reinf needed)

Max Shear Stress = 17 psi OK ✓

REINFORCEMENT

Introduce principal (circumferential) reinforcement at 2" from both inside and outside faces. Put in sufficient reinforcement to take tension of 257 psi:



$\frac{257+0}{2} \times 6 \times 12 = R = 9252$ #/LF of tunnel

Try #6 @ 9" $A_s = .442 \text{ in}^2$

Allowable strength = .442 × 20,000 = 8840 #/bar
= 8840 × 12/9 = 11,786 #/FT

Since 11,786 > 9252 #6@9 is OK

This is not a refined design method, nor an efficient balance between thickness of lining and reinforcement. Recompute for final design.

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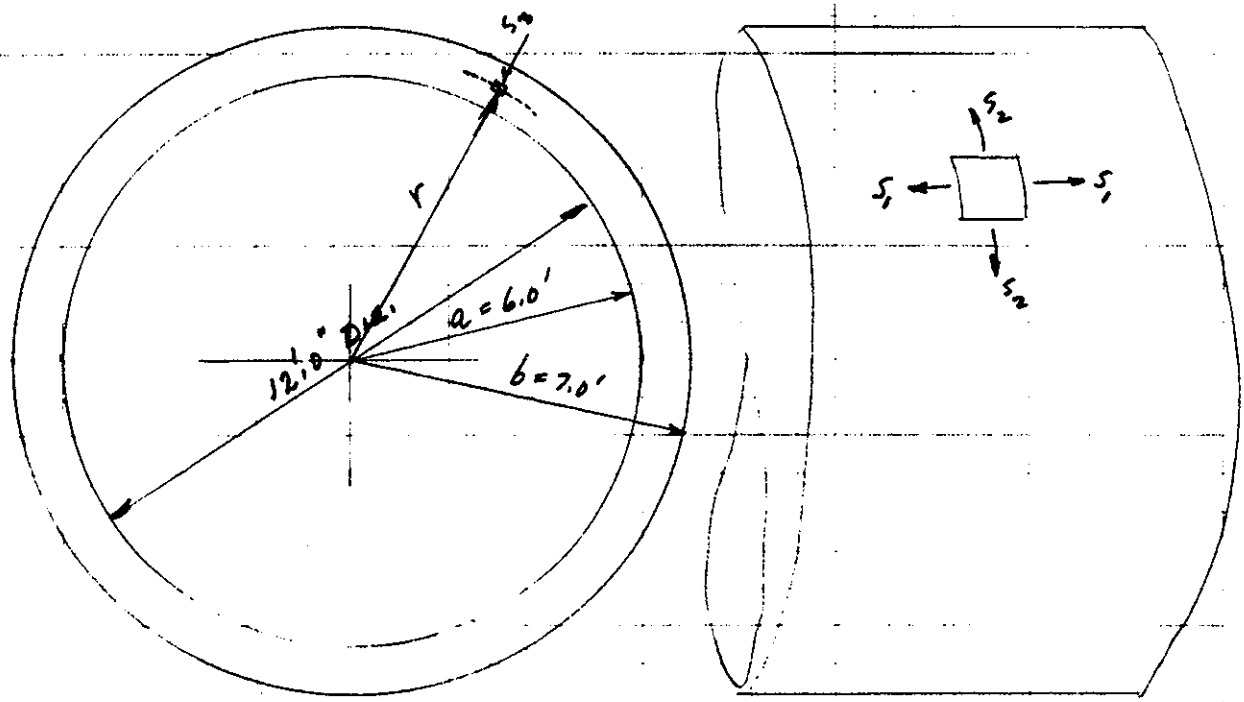
IS-1

a = inner radius of vessel
b = outer radius of vessel
r = radius to point where stress is to be found



Assume wall thickness = 1'-0"
Ratio of wall thickness to Radius = $\frac{1.0}{6.0} = 0.167 > 0.1$
∴ a "thick vessel"
(Roark pg 309)

Maximum Compression in Shaft

Maximum Uniform External Radial Earth Pressure = 4080 psf
Maximum Uniform External Radial Water Pressure = 4370 psf
8450 psf

Maximum Total Uniform External Pressure = 8450 psf = 58.68 psi

"THICK" vessel

UNIFORM EXTERNAL RADIAL PRESSURE "p"
p = 58.68 psi
Hoop wall stress = $S_2 = -p\frac{b^2(a^2+r^2)}{r^2(b^2-a^2)}$

Use Roark "Formulas for Stress and Strain TABLE XIII pg 308 Loading No 34

a = 72 b = 84

Max Hoop wall stress at r = a (INNER SURFACE) $S_2 = -p\frac{b^2(\cancel{a^2+r^2})}{\cancel{r^2}(b^2-a^2)} = -p\frac{2b^2}{(b^2-a^2)}$

$\max S_2 = -58.68\frac{2(84)^2}{(84^2-72^2)} = 442$ psi COMPRESSION

7056 5184
1872

12240
5184 7056

$\ldots b^2(a^2+b^2) \ldots (a^2+b^2) = -58.68(72^2+84^2)$ 292 psi

IS-2

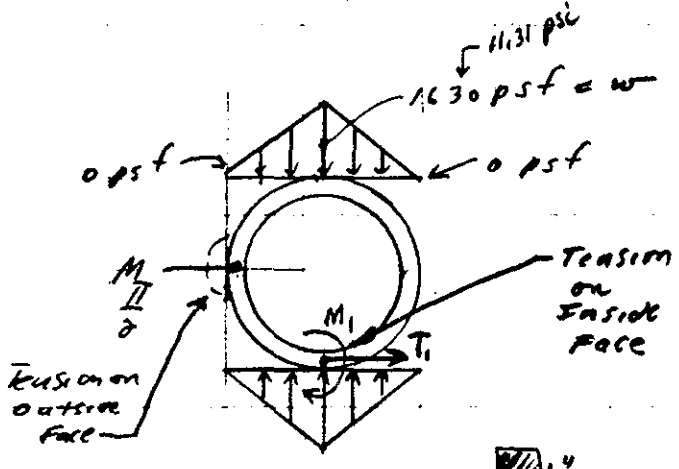
Meriodonal stress = $S_1 = 0$

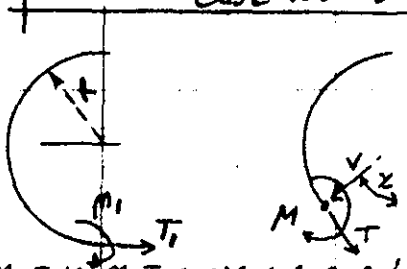
Radial wall stress = $S_3 = p\dfrac{b^2(r^2-a^2)}{r^2(b^2-a^2)}$

Max Radial wall stress at $r=b$ $S_3 = p\dfrac{b^2(b^2-a^2)}{b^2(b^2-a^2)} = p$ $\dfrac{58.68}{}$ psi

Calculate Bending Effects from triangular loading on 2 sides caused by difference in grade (see R Rusin 7/19/84 Memo)

USE ROARK TABLE VIII FORMULAS FOR CIRCULAR RINGS AND ARCHES Case No 12





M, T, V, M T and V are positive when as shown

$u = \cos x$

$z = \sin x$

1"

#/ft × " = #/in $w = 11.31 \times 1'' = 11.31$ psi w = applied Load (lbs / lin in) = 1630 / 144 = 11.31 lbs/lin. in.

$R = \dfrac{6+7}{2} = 6.5' = 78''$

$M_1 = 0.305\,wR^2 = 0.305\,(11.31)(78)^2$

$M_1 = \underline{20987}$ in-lbs / linear inch of pipe (1" Long Length of pipe) (tension on inside face)

for Negative moment → $M_{\pi/2} = M_1 - T_1 R(1-u) - \frac{1}{2} wR^2 z = 20987 - 23\overset{1794}{(78)(1-0)} - \overset{34905}{\frac{1}{2}(11.31)(78)^2(1)} = \underline{-15212''}$

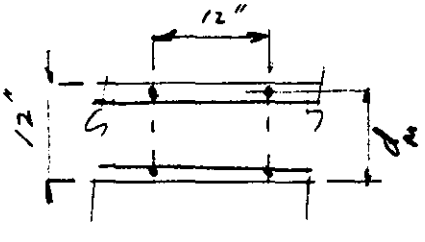
$u = \cos 90° = 0$

$z = \sin 90° = 1$

$T_1 = -wL(0.02653) = -(11.31)(78)(0.02653) = 23.40$ / lin inch.

Compression

Maximum Bending Moment = 20.987 in-lbs / linear in.
= 1.748 ft-lbs / linear in.
= 20.987 ft-lbs / foot
= 21.0 ft-k / foot



$d_m = \sqrt{\dfrac{21.0}{.226}} = 9.63 > 9.5$ provided No Good

$d = 12 - 2 - .5 = 9.5''$ $A_s = \dfrac{21.0}{1.44 \times 9.5''} = 1.53$ #" << 0.55 provided

#6C9

No Good

TRY $t = 15''$ $d = 15 - 2.0 - 0.50 = 12.50 > 9.63$ ok

Inside Face $A_s = \dfrac{21.0}{1.44 \times 12.50} = 1.16$ #" ~~provide #8C6 (1.50~~

Outside Face $A_s = 15.20$ [illegible] #" provide #9C12 (1.00

* Note: When we consider effect of Axial Compressive Load, Small IF any

* USE #9C12 IF SECS 95 & 7 provide

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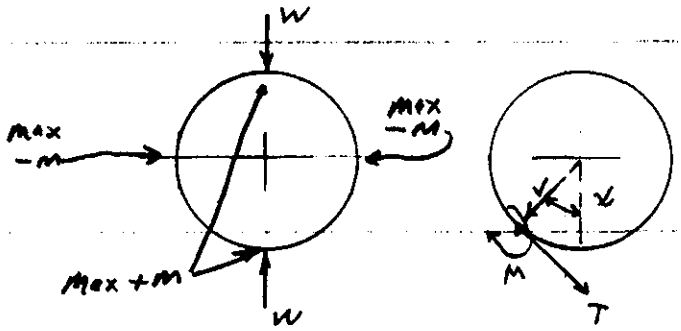
JS-3

CHECK "triangular load" analysis using "Concentrated" Loads
ROARK TABLE VIII Loading 1



at $x = 0$ (top or bottom)

$Max + M = 0.3183\,WR$

$Max + M = 0.3183(882)(78)$

$= \underline{21879}$ in-lbs/Lin in.

11.31 psi

$W = \frac{1630}{144}(78) = 882$ #/Lin in.

$R = 78''$

Comparing 21879 in-lbs with result of 20987 in-lbs from triangular load, we can conclude that distributed load has little effect on value of bending moment

$Max - M = -0.1817\,WR$ at $x = \frac{\pi}{2}$ (sides)

$= -0.1817(882)(78)$

$= -\underline{12500}$ in-lbs/Lin in.

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IS-4

using 15" wall
a = 72"
b = 87"

Check Stresses in Inlet Shaft

+ Compression stress
− Tension stress

max $S_2 = 58.68 \frac{2(87)^2}{(87^2 - 72^2)}$ 7569 5184 2385

$S_2 = 372$ #/□" See pg 1

I Maximum Compressive Stress in Liner (15" THICK)

a) Radial Earth plus Radial Water = 8450 psf (4080 psf, 4370 psf)

Max Hoop Wall Stress = 372 psi Compression (inner face)
Min Hoop Wall Stress = 313 psi Compression (outer face)

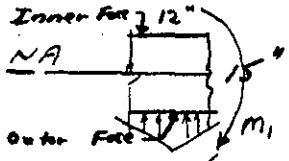
b) Triangular Load due to differences in Grade

See pg 2

Maximum Induced Moment = 21.0 ft-k / foot length

MIN $S_2 = -58.68 \frac{(72^2 + 87^2)}{87^2 - 72^2}$ 7569 5184 2385

Min S_2 = 313 #/□"



$S = \frac{bd^2}{6} = \frac{12(15)^2}{6} = 450$

20987 ≅ 21'k

Max Compressive Bending Stress = $\frac{21.0(12000)}{450}$ = 560 psi Compression

Max Tensile Bending Stress = −560 psi tension (Inner Face)

Maximum Compressive Stress:

Max Compressive Stress = 372 + 560 = 932 psi, OK
Allowable = 0.35(4000) = 1400
$f_c' = 4000$

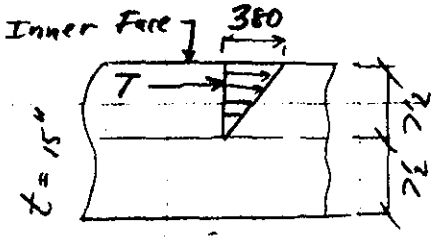
II Maximum Tensile Stress In Liner (Inside face of Liner)

a) Radial Earth ONLY (4080 psf)

Max Hoop Wall Stress = $\frac{4080}{8450}(372)$ = 180 psi Compression at Inner face

b) Triangular Load due to diff. in Grade
Maximum Tensile Bending Stress = −560 psi Tension at Inner Face

Maximum Tensile Stress = +180 − 560 = −380 psi



$T = \frac{1}{2}(380)(7\frac{1}{2})(12) = 17100$ #/foot TENSION

$f_{\text{ft}} = \frac{17100}{1.58 \text{ □"}} = 10822$ psi ok
< 20,000 allowable

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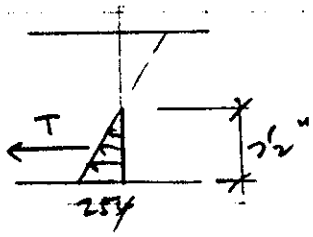
III Check Tensile Stress at Outside Face of Liner

Hoop Wall Stress at Outside Face Due to Radial Earth ONLY $= \frac{4080}{8450}(313) = 151$ psi Compression

Tensile Bending Stress at Outside Face (at $x = \frac{\pi}{2}$ (90°)) $= \frac{15.2(12000)}{450} = -405$ psi Tension

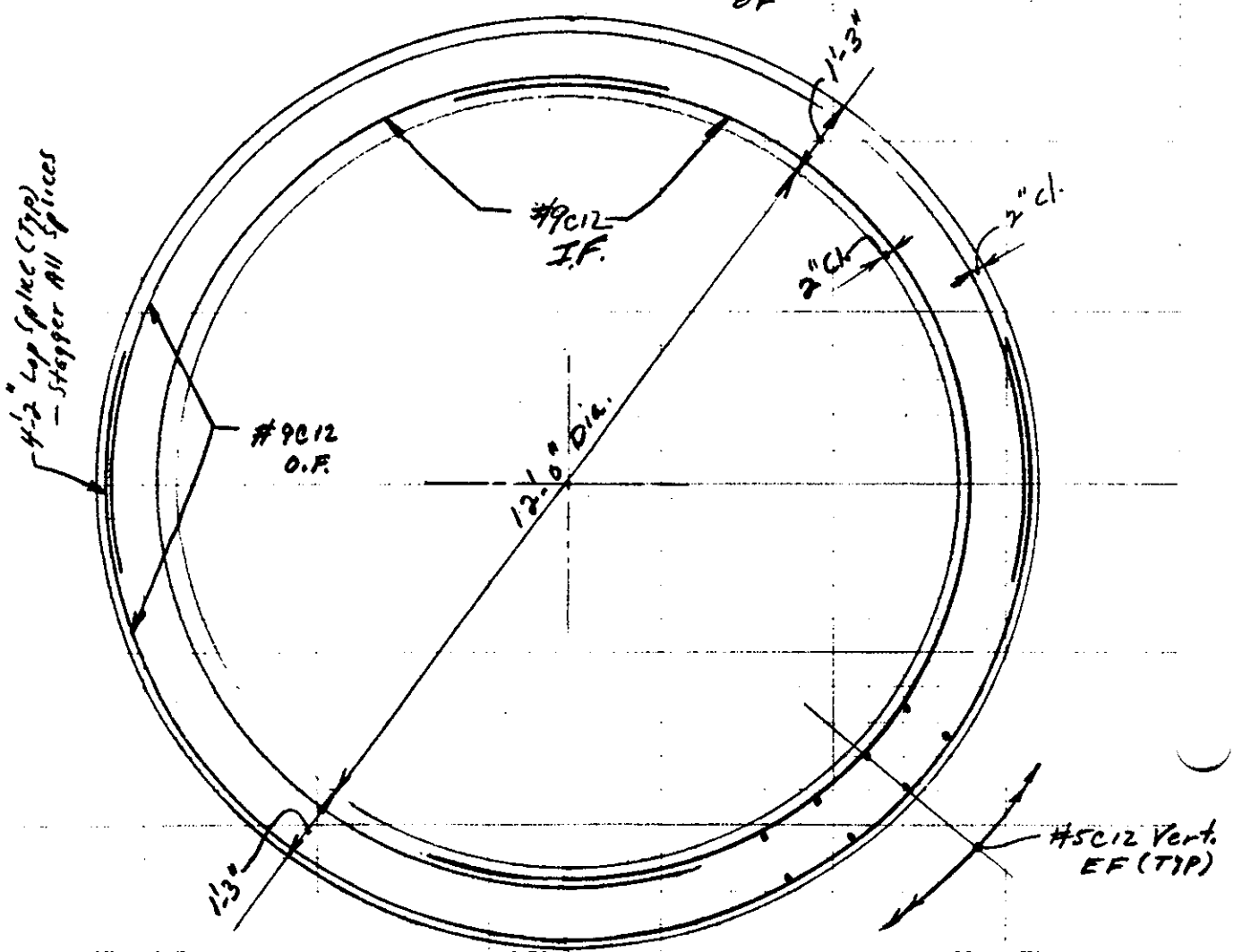
Maximum Resultant Tensile Stress $= 151 + (-405) = -254$ psi Tension at outside face



$T = \frac{1}{2}(254)(1'2)(12) = 11430$ #/foot

$f_T = \frac{11430}{1.0} = 11430$ psi < 20,000 ok

#9C12 OF



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IS-C

$\frac{6+7.25}{2} = 6.62$

$79.5''$

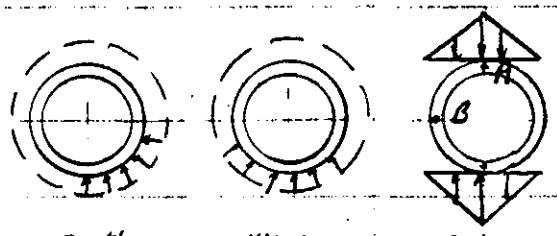
Analysis of Stresses in Shaft (wall t = 15")

actual

$* M_A = 0.305 wR^2 = .305(11.31)(79.5)^2$

15" wall $= 21802''\#$

SAY USE 20987 (12" wall) VALUE



Earth Radial Press | Hyro Radial Press | Triangular Earth press from diff in finished grade ELS

a) Consider Earth Pressure combined with Triangular Soil (No Hydro)

$S_2 = \frac{4080}{8450}\left(\frac{372+313}{2}\right) = 165$ psi

using 15" wall

Consider point "A"

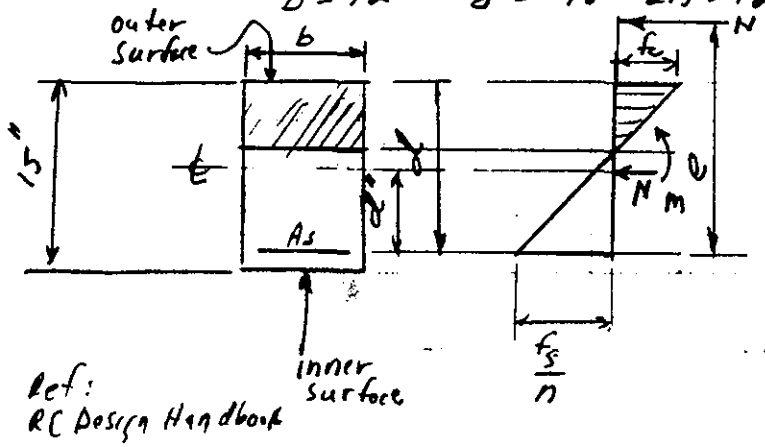
$* \quad M_A = 20987$ in-lbs/" Tension on inner face

$f_A \approx 165$ psi ——— COMPRESSION

$\therefore N$ axial Load $= 165(12 \times 15) = 29700\#$

$M = 20987''\text{lb/"} = 20.9$ 1 kips / foot length

$b = 12''$ $d = 15 - 2.5 = 12.5''$ $d'' = 5.00''$ $A_s = 1.58$ sq in. $n = 10$



$e = \frac{12m}{N} + d'' = \frac{12(20.9)}{29.7} + 5.00$

$e = 8.44 + 5.00 = 13.44''$

$e/d = \frac{13.44}{12.50} = 1.07$

assume $J = 0.85$

$i = \frac{1}{1 - \frac{Jd}{e}} = 4.86$

Ref: RC Design Handbook

$m = q = \frac{n A_s i}{bd} = \frac{10(1.58)(4.86)}{12(12.5)} = 0.51$

TABLE 11 for $m = q = 0.51 \Rightarrow K = 0.62$

TABLE 13 $z = 1/3$ $K = 0.62$: $J = 0.80$ actual

TABLE 10 $e/d = 1.07$ $J = 0.80$; $i = 1/(1 - 0.80\frac{1}{1.07}) = 3.96$

$f_s = \frac{1000N}{JA_s i} \times \frac{e}{d} = \frac{29700}{0.80(1.58)(3.96)} \times 1.07 = \underline{6348}$ psi

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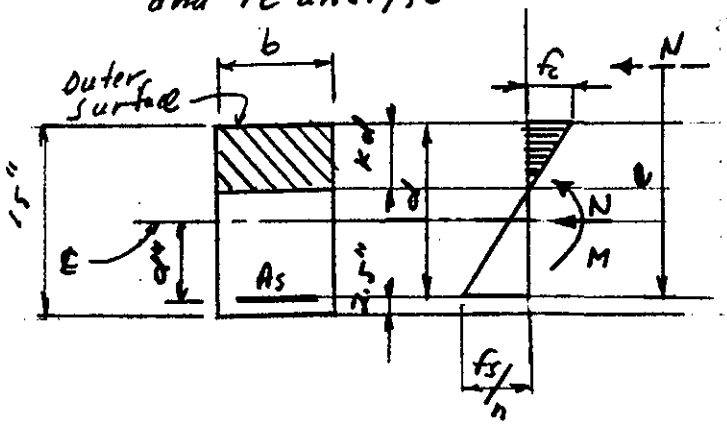
IS-7

$$f_c = \frac{f_s}{n} \times \frac{K}{1-K} = \frac{634.8}{10} \times \frac{0.62}{1-0.62} = \underline{1035 \text{ psi}}$$

Conclusion: Axial Load "N" reduces tensile stress in reinforcement. Try reducing A_s to 1.00 ☐" (#9@12) and re analyse



Given:
$b = 12''$
$d = 15 - 2.5 = 12.5''$
$d'' = 5.0''$
$A_s = 1.00$
$n = 10$
$M = 20.9^{1k}$
$N = 29.7^{k}$

$$e = \frac{12M}{N} + d'' = \frac{12(20.9)}{29.7} + 5.00'' = 13.44''$$

$$e/d = 13.44/12.50 = 1.07$$

Assume $J = 0.85$; $i = \frac{1}{1 - J\frac{d}{e}} = 4.86$ From TABLE 10 : $i = 3.83$

$$m = q = \frac{nA_s i}{bd} = \frac{10(1.00)(4.86)}{12(12.5)} = 0.32$$

From TABLE 11 $m = 0.26$ $q = 0.26$: $K = 0.51$

From TABLE 13 $z = 1/3$ $K = 0.51$: $J = 0.84$

From TABLE 10 $e/d = 1.07$ $J = 0.84$ $i = \frac{1}{1 - \frac{Jd}{e}} = \frac{1}{1 - \frac{.85(1)}{1.07}} = 4.86$

$$f_s = \frac{1000N}{jA_s i} \times \frac{e}{d} = \frac{29700}{0.84(1.00)(4.86)} \times 1.07 = \underline{7784 \text{ psi}}$$

$$f_c = \frac{f_s}{n} \times \frac{K}{1-K} = \frac{7784}{10} \times \frac{0.51}{1-.51} = \underline{810 \text{ psi}}$$

∴ use #9@12 IF

Although Conservative, use #9@12 OF also

NONREPRODUCIBLE GRID FORM 145
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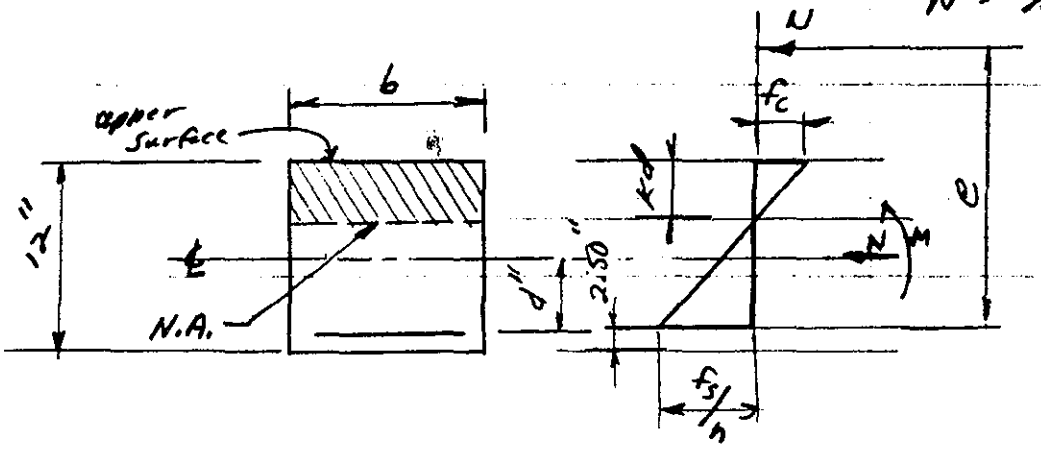
IS- 8

Try 12" Wall Combining Bending and Axial Loads

Try: $b = 12''$
$d = 12 - 2.5 = 9.5''$
$d'' = 3.50$
$\#8 @ 6 = A_s = 1.58 \quad n = 10$

$M = 20.9^{1k}$
Earth Pressure (No Hydro)
$S_2 = \frac{40\,80(4+2+3\,5\,3)}{8450} = 200 \ \#/\phi''$
$N = 200(12)(12) = 28800\#$



$$e = \frac{12M}{N} + d'' = \frac{12(20.9)}{28.8} + 3.50 = 12.20''$$

$$e/j = \frac{12.20}{9.50} = 1.29$$

Assume $j = 0.85$: $i = \frac{1}{1 - j\,d/e} = \frac{1}{1 - (0.85)\frac{1}{1.29}} = 2.93$

$$m = q = \frac{n A_s i}{bd} = \frac{10(1.58)(2.93)}{12(9.5)} = 0.41$$

FROM TABLE 11 $m = 0.41$, $q = 0.41$: $K = 0.59$

FROM TABLE 13 $z = 43$ $K = 0.59$: $J = 0.81$

FROM TABLE 10 $e/d = 1.29$ $J = 0.81$: $i = 2.68$

$$f_s = \frac{1000N}{jA_s i} \times \frac{e}{d} = \frac{28800}{0.81(1.58)(2.68)} \times 1.29 = 10831 \text{ psi}$$

$$f_c = \frac{f_s}{n} \times \frac{K}{1-K} = \frac{10831}{10} \times \left(\frac{0.59}{1-.59}\right) = 1558 \text{ psi}$$

1.43

.41

NG

$> 0.35 f'_c = 1400$

Conclusion: 12" wall insufficient, even with heavy reinforcing – Keep 15" wall

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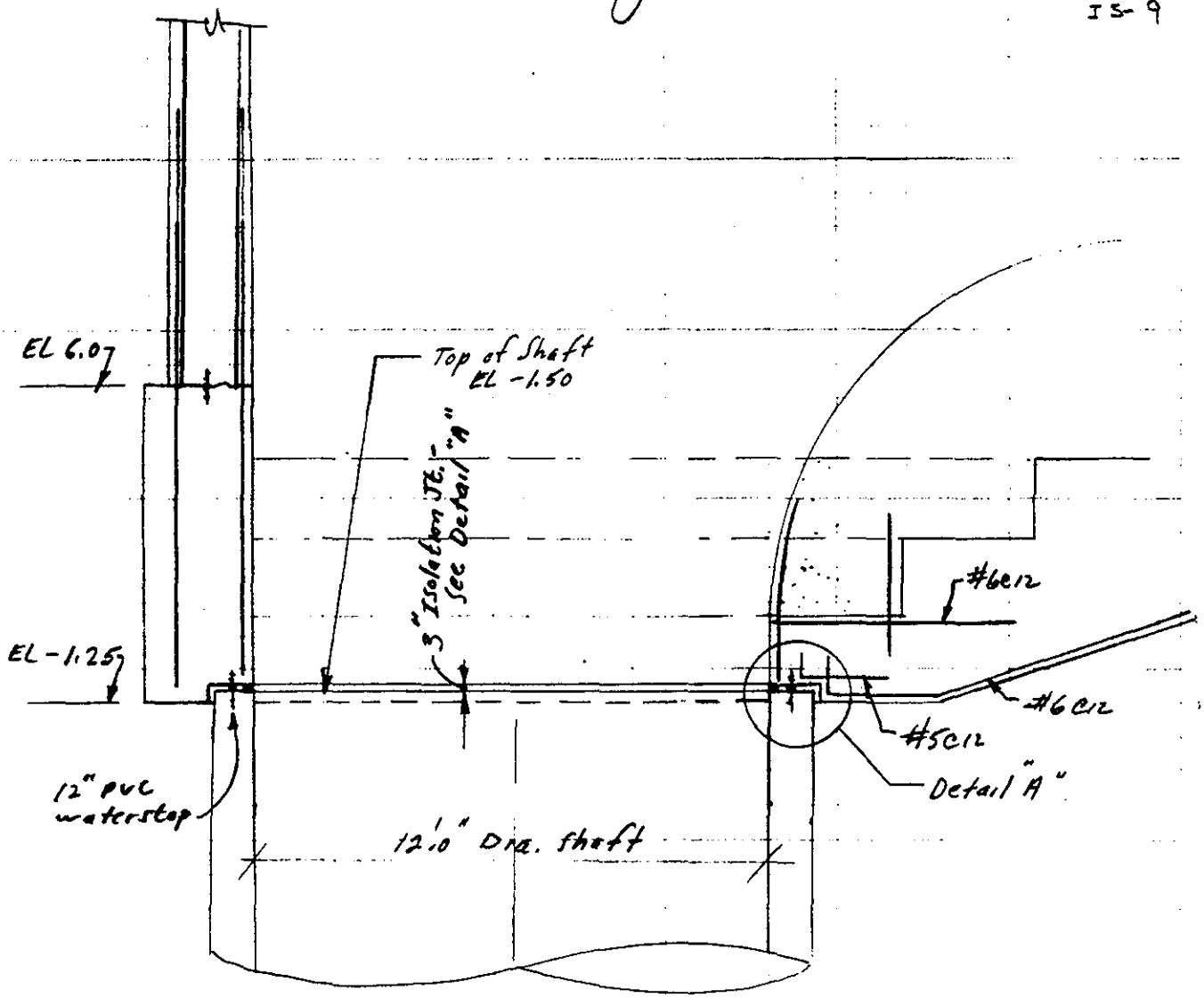
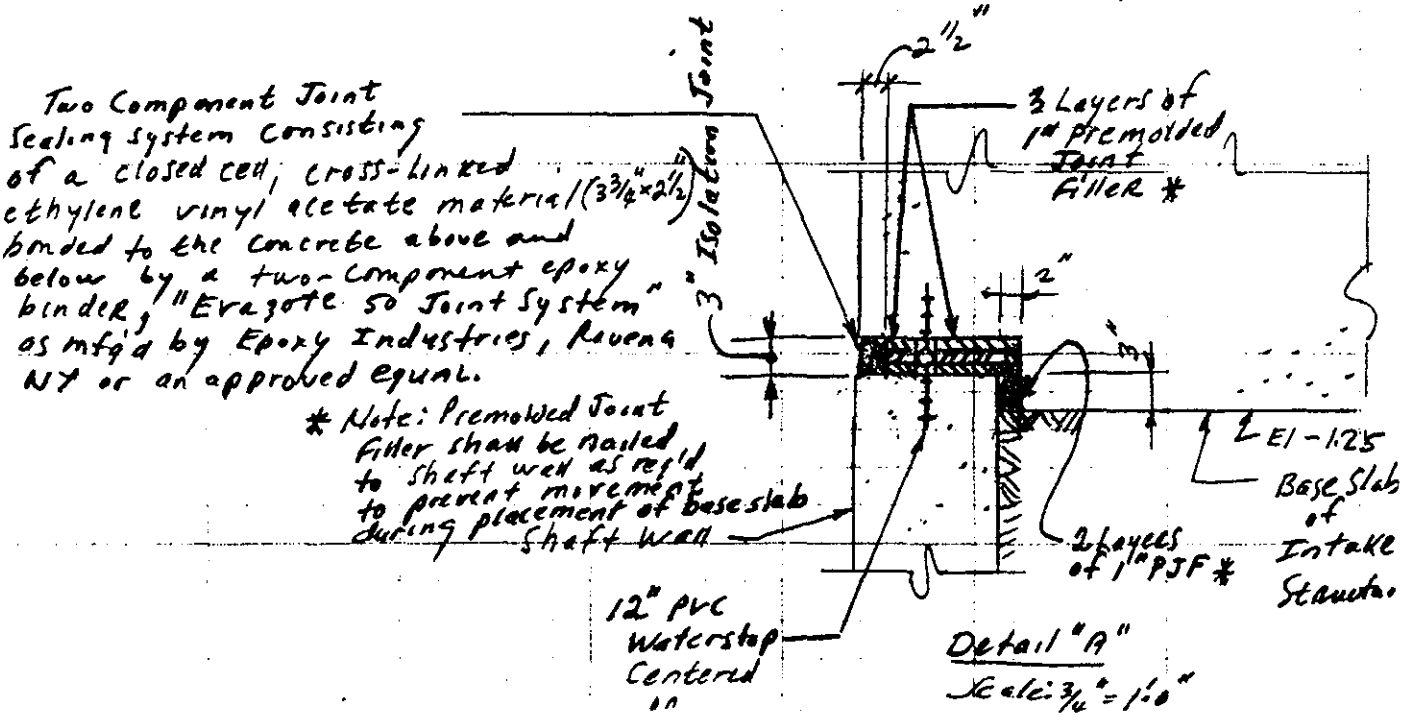
OUTLET STRUCTURE

Project	MDC - Town Brook	Acct. No.	6041	Page	89 of 121
Subject	Intake Structure	Comptd. By	Daniel Bornstein	Date	7/26/84
Detail	Intake Shaft Detail	Ck'd. By	John Howard	Date	9-05-84

IS-9





NONREPRODUCIBLE GRID FORM 145
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Project MDC- TOWN BROOK (QUINCY) Acct. No. 6041 Page 90 of 121
Subject OUTLET STRUCTURE Comptd. By J.T. HOWARD Date 7-11-84
Detail GENERAL ARRANGEMENT Ck'd. By G Volpini Date 8/21/84

OS-1

NONREPRODUCIBLE GRID FORM 145

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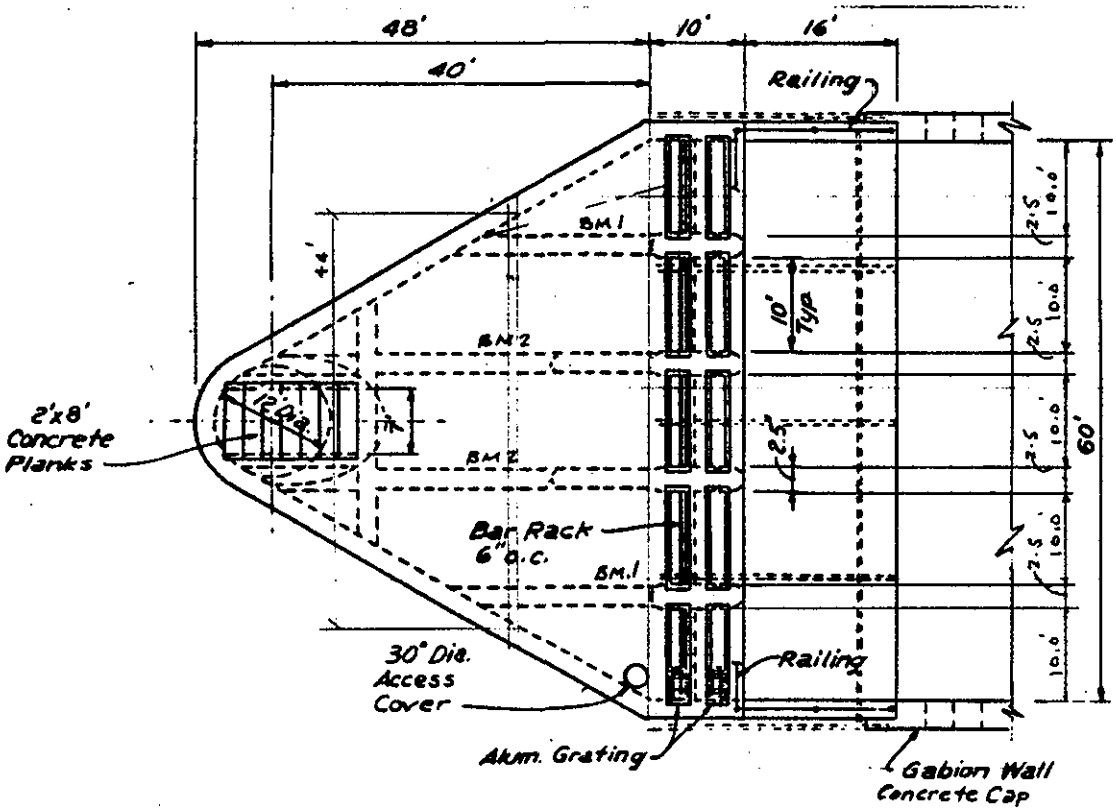
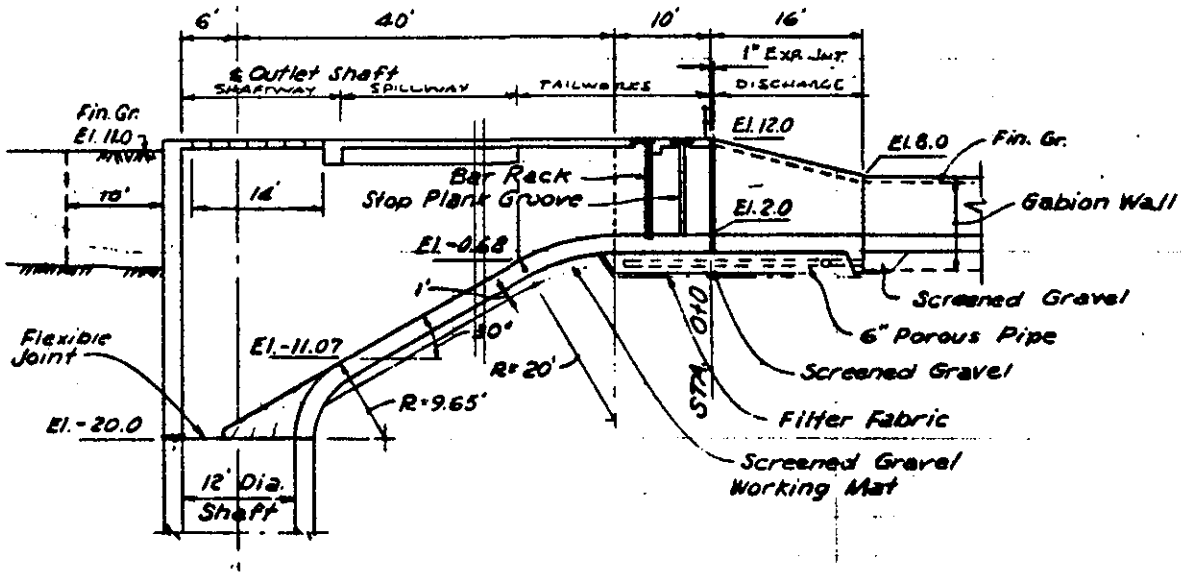


PLAN

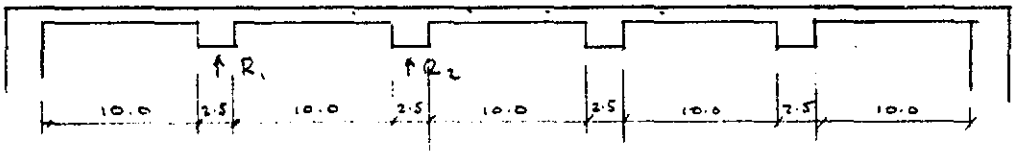


SECTION

OUTLET STRUCTURE
SCALE 1" = 20'

OS-2

LOAD CASE A: LL = 150 PSF L = 10.0' D = 8"



$w = w_{LL} + w_{DL} = 0.15 + 0.10 = 0.25 \ ^{k}/_{ft}$

$V_{MAX} = 1.15 \frac{wL}{2} = \frac{1.15 \times 0.25 \times 10.0}{2} = 1.44^{k}$

$+M_{MAX} = \frac{wL^2}{11} = 0.25(10)^2/11 = 2.27^{'k}$ (ACI)

$A_s = \frac{M}{ad} = M/1.44 \times 5.5 = 0.29 \ in.^2$

$-M_{MAX} = \frac{wL^2}{10} = 0.25(10)^2/10 = 2.50^{'k}$ (ACI)

$A_s = 2.50/1.44 \times 5.5 = 0.31 \ in^2$

$M_R = 0.221(5.5)^2 = 6.7^{'k}$

FOR CONTINUATION OF TOP SLAB DESIGN, SEE OS-3, LOAD CASE B.

NONREPRODUCIBLE GRID FORM 145
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05-3

LOAD CASE B : LL = ONE H-10 TRUCK + IMPACT

$W_R = 8.0^K$ $W_F = 2.0^K$

$E = 4.0 + 0.06L = 4.0 + 0.06 \times 10 = 4.6'$

$I = 1 + \frac{50}{L+125} = 1 + \frac{50}{10+125} = 1.37$

$P_R = \frac{WI}{E} = \frac{8.0 \times 1.3}{4.6} = 2.30^K$ $P_F = \frac{2.0 \times 1.3}{4.6} = 0.6^K$

$FEM_{DL} = \frac{wL^2}{12} = 0.10(12.5)^2/12 = 1.30^{'K}$

$FEM_{LL} = \frac{PL}{8} = 2.30 \times 12.5/8 = \frac{3.60}{4.90}$ $0.6 \times 12.5/8 = 1.00^{'K}$

-4.90	+4.90	-2.30	+2.30	-1.30	+1.30	-1.30	+1.30	-1.30	+1.30
+4.90	-1.30	-1.30	-0.50	-0.50	0.00	0.00	0.00	0.00	-1.30
-0.65	+2.45	-0.25	-0.65	0.00	-0.25	0.00	0.00	-0.65	0.00
+0.65	-1.10	-1.10	+0.32	+0.33	+0.12	+0.13	+0.32	+0.33	0.00
-0.55	+0.33	+0.16	-0.55	+0.06	+0.17	+0.16	+0.07	0.00	+0.17
+0.35	-0.24	-0.25	+0.24	+0.25	-0.17	-0.16	-0.03	-0.04	-0.17
-0-	+5.04	-5.04	+1.16	-1.16	+1.17	-1.17	+1.66	-1.66	-0-

V_S =	↑1.78	1.78↑	↑0.92	0.92↑	↑0.62	0.62↑	↑0.62	0.62↑	↑0.62	0.62↑
V_M =	↓0.40	0.40↑	↑0.31	0.31↓	---	---	↓0.04	0.04↑	↑0.13	0.13↓
V =	↑1.38	2.18↑	↑1.23	0.61↑	↑0.62	0.62↑	↑0.58	0.66↑	↑0.75	0.49↑

$-M = 5.04^{'K}$

$-A_S = \frac{-M}{ad} = \frac{5.04}{1.44 \times 5.5} = 0.64 \text{ IN.}^2$

USE: #5 @ 6" T (A_S = 0.62)

$+M = 1.38 \times 6.25 - 0.10(6.25)^2/2 = 6.67^{'K}$

$+A_S = \frac{+M}{ad} = \frac{6.67}{1.44 \times 5.5} = 0.84 \text{ IN.}^2$

USE: #6 @ 6" B. (A_S = 0.88)

USE: #5 @ 12" T & B (DIST.)

$V_{MAX} = \frac{2.30}{12.5}\left(12.5 - \frac{5.5}{12}\right) + 0.10\left(12.5 - \frac{2 \times 5.5}{12}\right)\frac{1}{2} = 2.22 + 0.58 = 2.80^K$

$v = \frac{V}{bd} = \frac{2800}{12 \times 5.5} = 43 \text{ psi}$

NONREPRODUCIBLE GRID FORM 145
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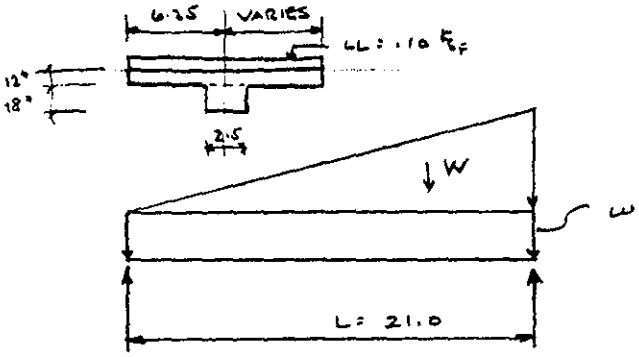
05-4

LOAD CASE A. LL = 100 PSF L = 21'



BM = 30" x 30"
d = 27.5
$M_R = 2.5(27.0)^2 \times 0.221 = 403.^{k}$
$V = 0.07 \times 30 \times 27.0 = 56.7^{k}$

LL = 0.10 KSF
DL = 0.10
$0.20 \times 6.25 \times 1.15 = 1.44$ KLF
BM = $2.5 \times 1.5 \times 0.15$ = 0.56
w = 2.00 KLF

$W = 0.20 \times \frac{6.25}{2} \times 21 = 13.1^{k}$

$$M = \frac{wL^2}{8} + \frac{WL}{8} = \frac{2.00(21.0)^2}{8} + \frac{13.1 \times 21.0}{8} = 110.25 + 34.39 = 144.64^{'k}$$

$$A_s = \frac{M}{ad} = \frac{145}{1.44 \times 27.0} = 3.73 \text{ in}^2$$

USE: 5-#8 B (A_s = 3.95 in²)
3-#8 T

$$R_R = \frac{wL}{2} + \frac{2W}{3} = \frac{2.00 \times 21}{2} + \frac{2 \times 13.1}{3} = 21.0 + 8.7 = 29.7^{k}$$

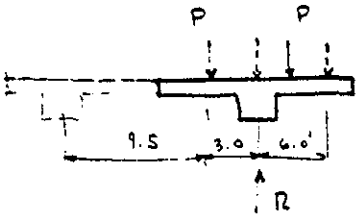
$$V = 29.7 - \frac{2.00 \times 27.0}{12} - 0.20 \times 5.91 \times \frac{27.0}{12} = 29.7 - 4.5 - 2.6 = 22.6^{k}$$

$$v = \frac{V}{bd} = \frac{22{,}600}{30 \times 27.0} = 28.0 \text{ psi}$$

USE: #4 @ 12" ∎

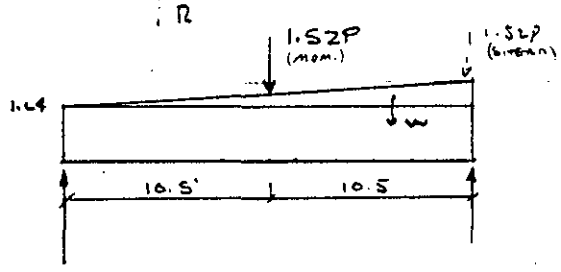
OS-5

LOAD CASE B: LL = H-10 TRUCK + 30% IMPACT



$R_1 = 2P\,\frac{9.5}{12.5} = 1.52P$

$R_2 = P\left(1.0 + \frac{6.5}{12.5}\right) = 1.52P$



$P = 8.0 \times 1.3 = 10.4^K$

$1.52\,P = 1.52 \times 10.4 = 15.8^K$

$w_{DL} = 1.80 \times \frac{0.15}{0.25} + 0.52 = 1.64^{K}/_F$

$W_{DL} = 16.4 \times \frac{0.15}{0.25} = 9.84^K$

$M = \frac{1.64\,(21.0)^2}{8} + \frac{9.84 \times 21}{8} + \frac{15.8 \times 21.0}{4} = 90.4 + 25.8 + 83.0 =$

$A_s = 180.0 / 1.44 \times 27.0 = 4.63\ in^2$

USE: 5-#9 B ($A_s = 5.00\ in^2$)
3-#9 T

$V_{MAX} = 1.50 \times 21/2 + \frac{0.10 \times 5 \times 21}{2} \times \frac{2}{3} + 15.8 = 15.7 + 3.5 + 15.8 = 35.0^K$

$v = \frac{35000}{30 \times 27} = 43.\ psi$ OK

USE: #4 ▭ @ 12"

EQUIVALENT UNIFORM LL:

$w = \frac{8M}{L^2} = \frac{8 \times 180}{(21.0)^2} = 3.26^{K}/_F$ TL

$- 1.67$ DL

$1.59 / 10 = .16^{K}/_F$ LL

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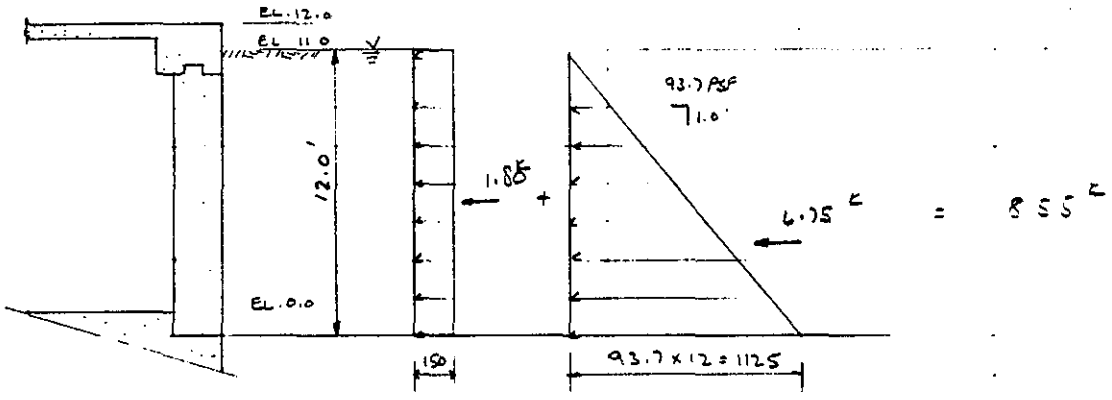
OS-11

SPAN WALLS VERTICALLY :
ASSUME FIXED AT BASE, HINGED AT TOP



EXTERNAL LOADING

$$-M_{BASE} = \frac{0.15(12.0)^2}{8} + \frac{1.125(12.0)^2}{15} = 2.70 + 10.80 = 13.5^{'K}$$

$$R_T = \left[\frac{0.15(12.0)^2}{2} + \frac{0.0937(12.0)^3}{6} - 13.5\right]\frac{1}{12} = (10.8 + 27.0 - 13.5)\tfrac{1}{12} = 2.0^K$$

$$R_B = (1.8 \times 6 + 6.75 \times 8 + 13.5)\frac{1}{12} = 78.3/12 = 6.53$$

$$8.53^K$$

$$+M_{MAX} @ x \approx 5.12$$

$$V = 2.0 - 0.15 \times 5.12' - 0.0937(5.12)^2/2 = 0$$

$$+M = 2.0 \times 5.12 - 0.15(5.12)^2/2 - 0.0937(5.12)^3/6 = 6.18^{'K}$$

$$-A_s = \frac{-M}{ad} = \frac{13.5}{1.44 \times 21.5} = 0.44 \text{ in}^2$$

USE: #6 @ 12 OF VERT
$A_s = 0.44 \text{ in}^2$

$$+A_s = \frac{+M}{ad} = \frac{6.2}{1.44 \times 19.5} = 0.22 \text{ in}^2$$

USE: #5 @ 12 IF VERT
$A_s = 0.31 \text{ in}^2$

INTERNAL LOADING

$$M_{BASE} = \frac{0.0626(12.0)^3}{15} = 7.2^{'K}$$

$$A_s = \frac{7.2}{1.44 \times 19.5} = 0.26 \text{ in}^2$$

USE: #6 @ 12" IF VERT
$A_s = 0.44 \text{ in}^2$

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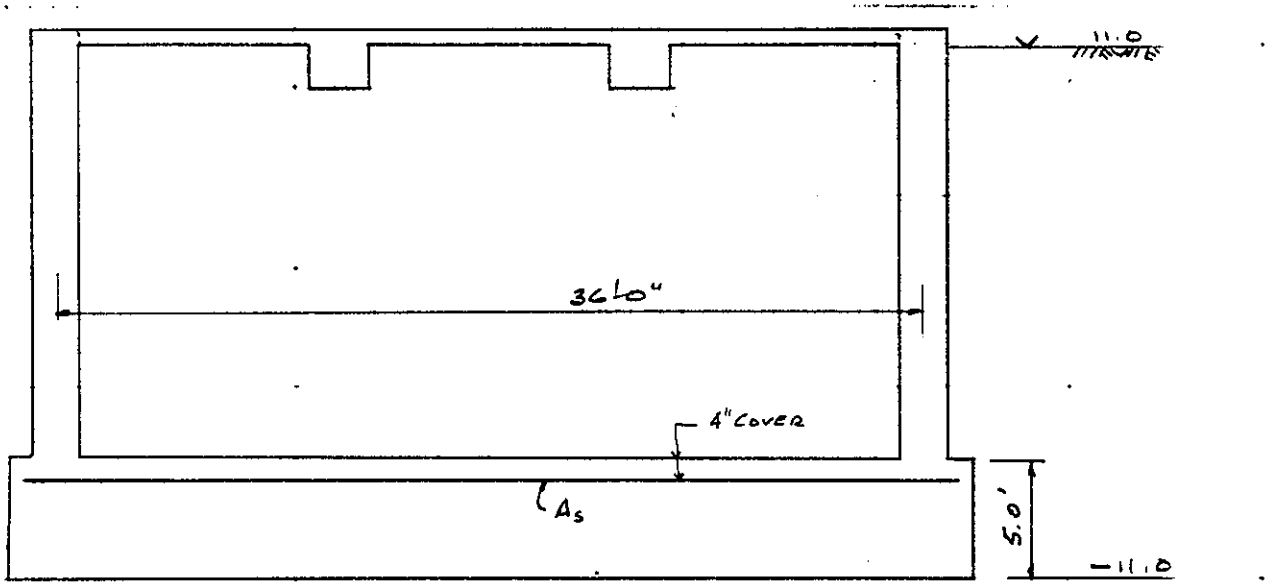
OS-28

1.0' WIDE SECTION – 18.0' FROM ℄ SHAFT



DEAD WEIGHT OF STRUCTURE:-

TOP SLAB:- 34.0 × 0.67 = 22.78 CF
BM STEM:- 5.0 × 1.83 = 9.15
WALL:- 4.0 × 18.0 = 72.00
103.93 CF × 0.15 k/CF = 15.6 k / 38 = 0.41 k/F

HYDROSTATIC HEAD

0.0624 [11.0 – (–11.0)] = 1.37 k/F
– DL SLAB 5.0 × 0.15 = –0.75
0.62 k/F

$$M = \frac{0.62(36.0)^2}{8} = 100.5^{IF}$$

$$A_s = \frac{100.5}{1.44 \times 55} = 1.27 \text{ IN}^2$$

NONREPRODUCIBLE GRID FORM 145
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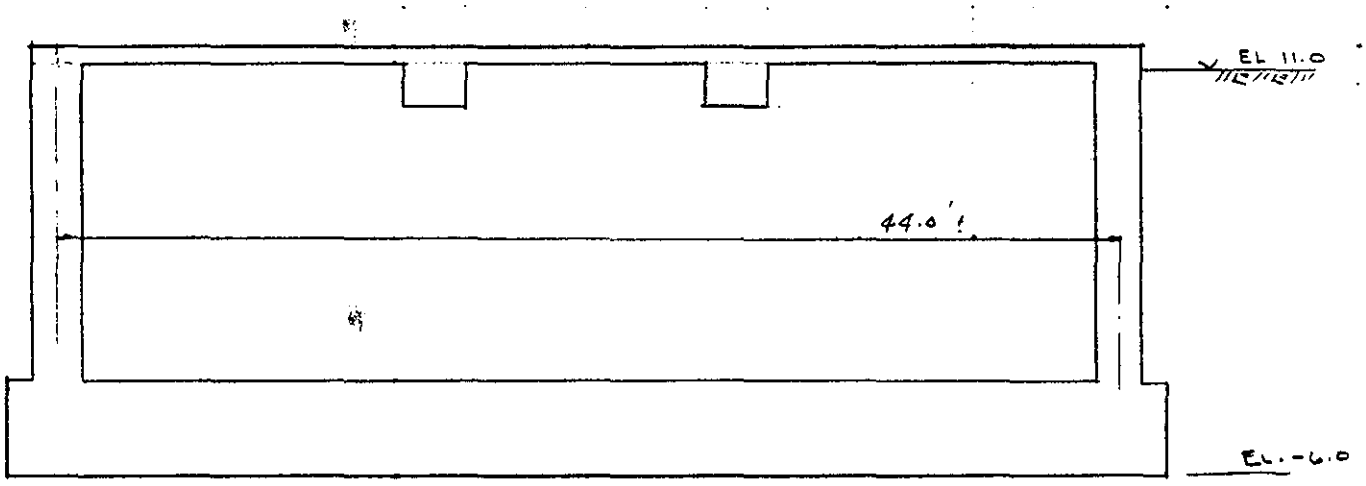
05-29

1.0' WIDE SECTION - 25.0' FROM ℄ SHAFT



DEADWEIGHT OF 1.0' STRIP:-

TOP SLAB:	46.0 x 0.67 =	30.82 CF
BM STEM:	5.0 x 1.83 =	9.15
12" WALL:	4.0 x 13.33 =	53.32

$93.29 \times 0.15 = 14.0^{K} \Big/ 46.0 = 0.30^{K}/_{F}$

HYDROSTATIC HEAD

$0.0624\,[11.0-(-6.0)] = 1.06\ ^{K}/_{F}$

$-$ DL SLAB $\quad 3.0 \times 0.15 = 0.45$

$0.61\ ^{K}/_{F}$

$$M = \frac{0.61}{8}(44.0)^2 = 147.6^{'K}$$

$$d_{REQ'D} = \left(\frac{147.6}{0.221}\right)^{\frac{1}{2}} = 25.9\ IN \qquad d_{AVAIL} = 36-5 = 31$$

$$V = \frac{0.61}{2}\left(44 - \frac{31}{12}\right) = 12.63^{K}$$

$$v = \frac{V}{bd} = \frac{12{,}630}{12 \times 31} = 34\ psi$$

$$A_s = \frac{M}{ad} = \frac{147.8}{1.44 \times 31} = 3.31\ IN^2$$

USE 4.0' SLAB

$$w = 1.06 - 4 \times 0.15 = 0.46\ ^{K}/_{F}$$

$$M = \frac{wl^2}{8} = \frac{0.46(44)^2}{8} = 111.32^{'K}$$

$$A_s = \frac{M}{ad} = \frac{111.32}{1.44 \times 43} = 1.80\ IN.^2$$

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Subject OUTLET STRUCTURE Comptd. By J. T. HOWARD Date 8-16-84
Detail BASE SLAB Ck'd. By A. Volpini Date 8/21/84

OS-30

HYDROSTATIC UPLIFT GOVERNS :-

$w_8 = 0.0624 \times 31 - 0.15 \times 8 = 0.73$ KSF

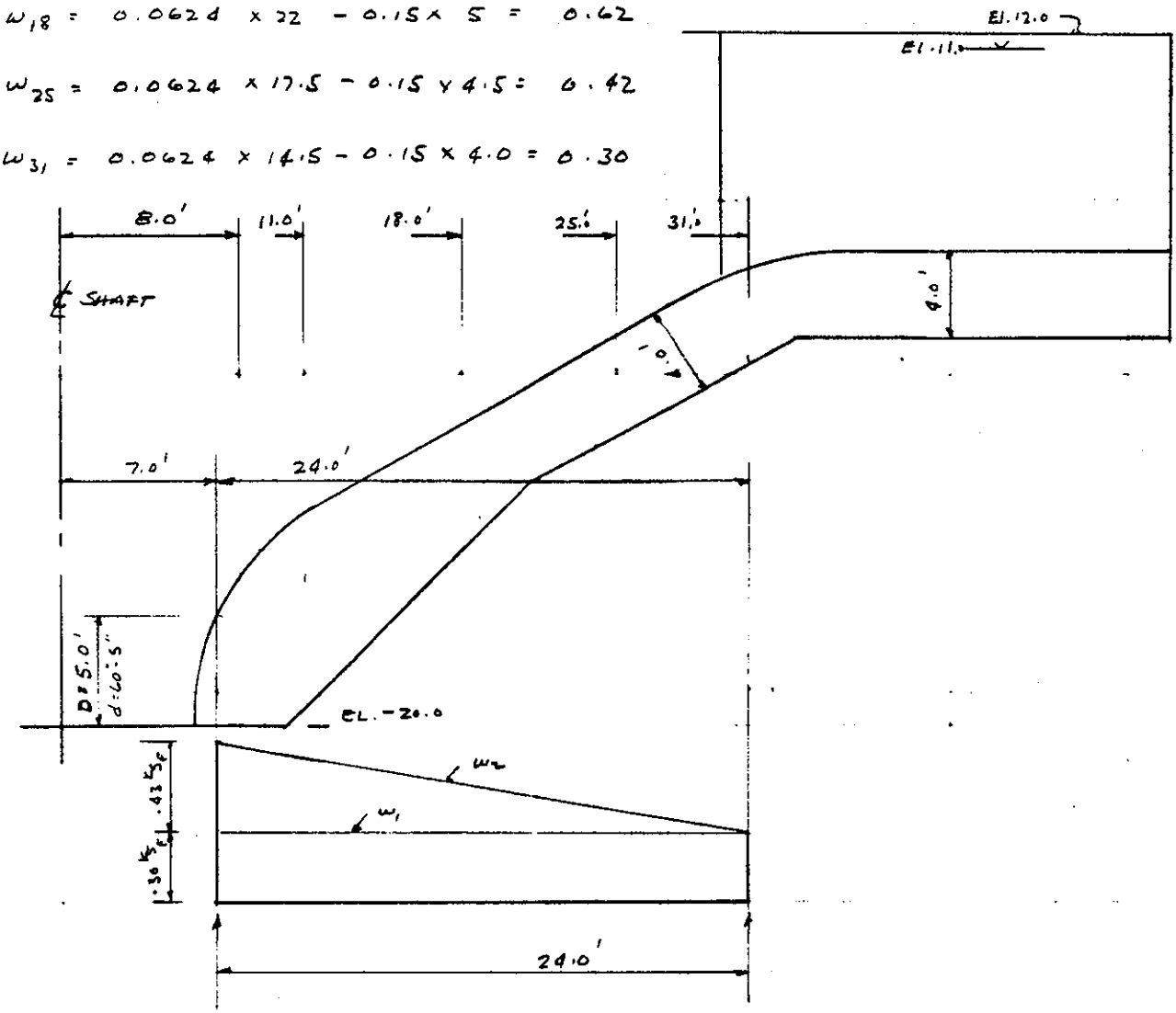
← DISTANCE FROM ℄ SHAFT.

$w_{11} = 0.0624 \times 30 - 0.15 \times 8 = 0.67$

$w_{18} = 0.0624 \times 22 - 0.15 \times 5 = 0.62$

$w_{25} = 0.0624 \times 17.5 - 0.15 \times 4.5 = 0.42$

$w_{31} = 0.0624 \times 14.5 - 0.15 \times 4.0 = 0.30$



TRY SPANNING SLAB LONGITUDINALLY (PARALLEL TO FLOW LINE)

$$M = \frac{w_1 l^2}{8} + \frac{w_2 l^2}{16} = \frac{0.30(24.0)^2}{8} + \frac{0.43(24.0)^2}{16} = 21.6 + 15.5 = 37.1^{k}$$

$$A_s = \frac{M}{ad} = \frac{37.1}{.44 \times 43} = 0.60 \text{ IN.}^2$$

USE: #7 @12" o/c EW, T&B

$R_L = 0.30 \times 24/2 = 0.43 \times 24.0/3 = 3.6^{k} + 3.4 = 7.0^{k}$

$R_R = 3.6 + 1.7 = 5.3^{k}$

$V = R - wd = 7.0 - 0.30 \times \frac{43}{12} = 6.0^{k}$

$v = 6000/12 \times 90" = 6.0$ PSI

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05-31

LATERAL BM. SUPPORTING LONGITUDINAL SLAB

AT THE SHAFT

$w = R_L = 7.0^{k}/_{F}$

$R_A = \frac{wL}{2} = \frac{7.0 \times 24}{2} = 84.0^{k}$

$V = R - wd = 84.0 - 7.0 \times \frac{55}{12} = 52.0^{k}$

$v = \frac{52000}{36 \times 55} = 26.0\ psi$

$M = \frac{wL^2}{8} = \frac{7.0(24.0)^2}{8} = 504.0^{'k}$

$M_n = 3.0\ (55)^2\ 0.221 = 2005.0^{'k}$

$A_s = \frac{M}{a\,d} = \frac{504.0}{1.44 \times 55} = 6.36\ in.^2$

USE 5-#10 ($A_s = 6.35\ in^2$)

AT THE PIERS (ASSUME 5.0' WIDE LOCAL BEAM)

$w = R_L + w_p \times 5.0 = 5.30 + 0.35 \times 5.0 = 7.05^{k}/_{F}$

$-M = 16.2 \times \frac{7.06}{0.35} = 326.3^{'k}$

$-A_s = \frac{-M}{b\,a\,d} = \frac{326.3}{5.0 \times 1.44 \times 43} = 1.05\ in^2/1.0'$

$+M = 17.5 \times \frac{7.05}{0.35} = 352.5^{'k}$

$+A_s = \frac{+M}{b\,a\,d} = \frac{352.5}{6.0 \times 1.44 \times 43} = 1.14\ in^2/1.0'$

USE #10 @ 12" ($A_s = 1.27\ in.^2$)

NONREPRODUCIBLE GRID FORM 145
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TOWN RIVER CULVERT

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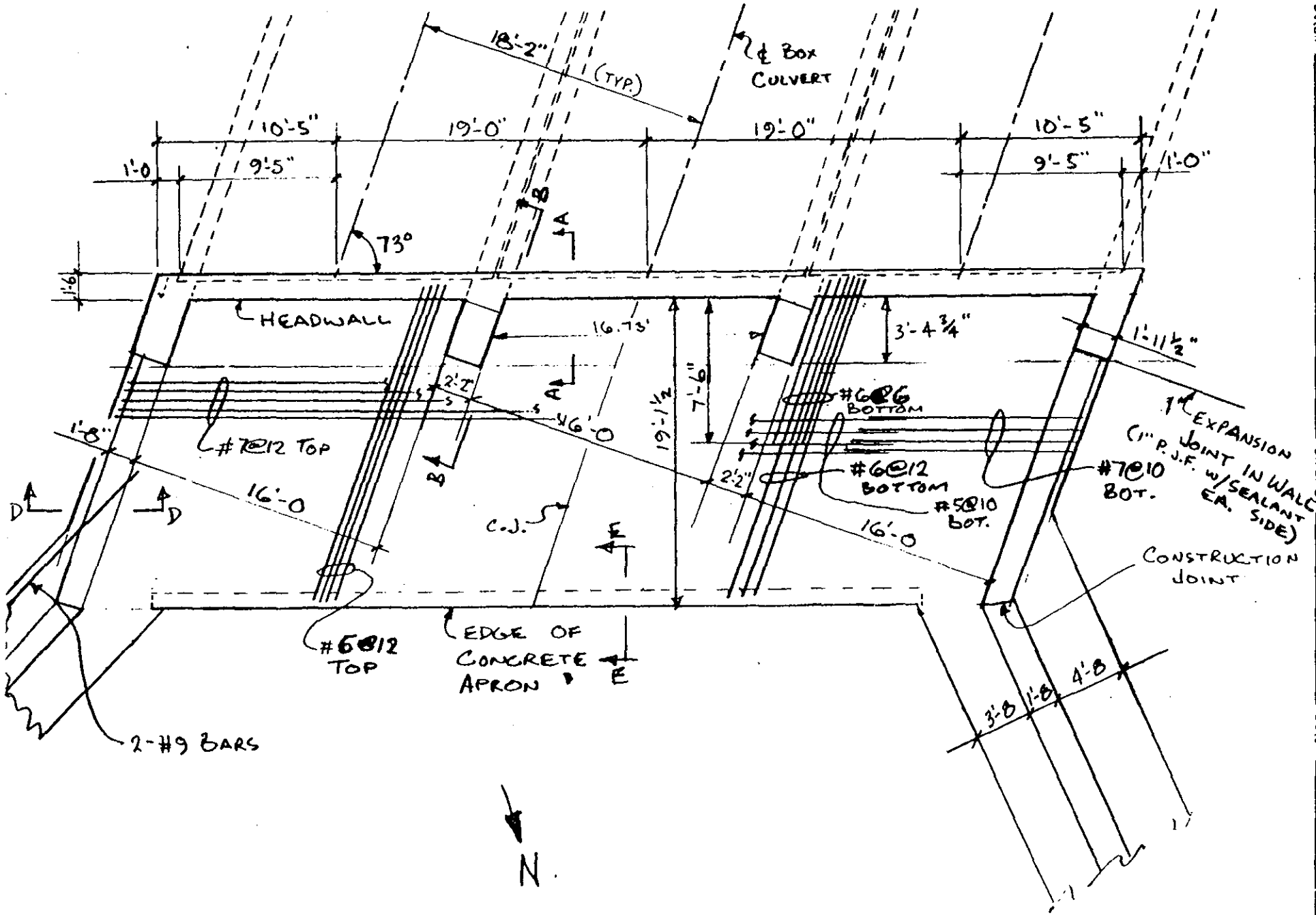
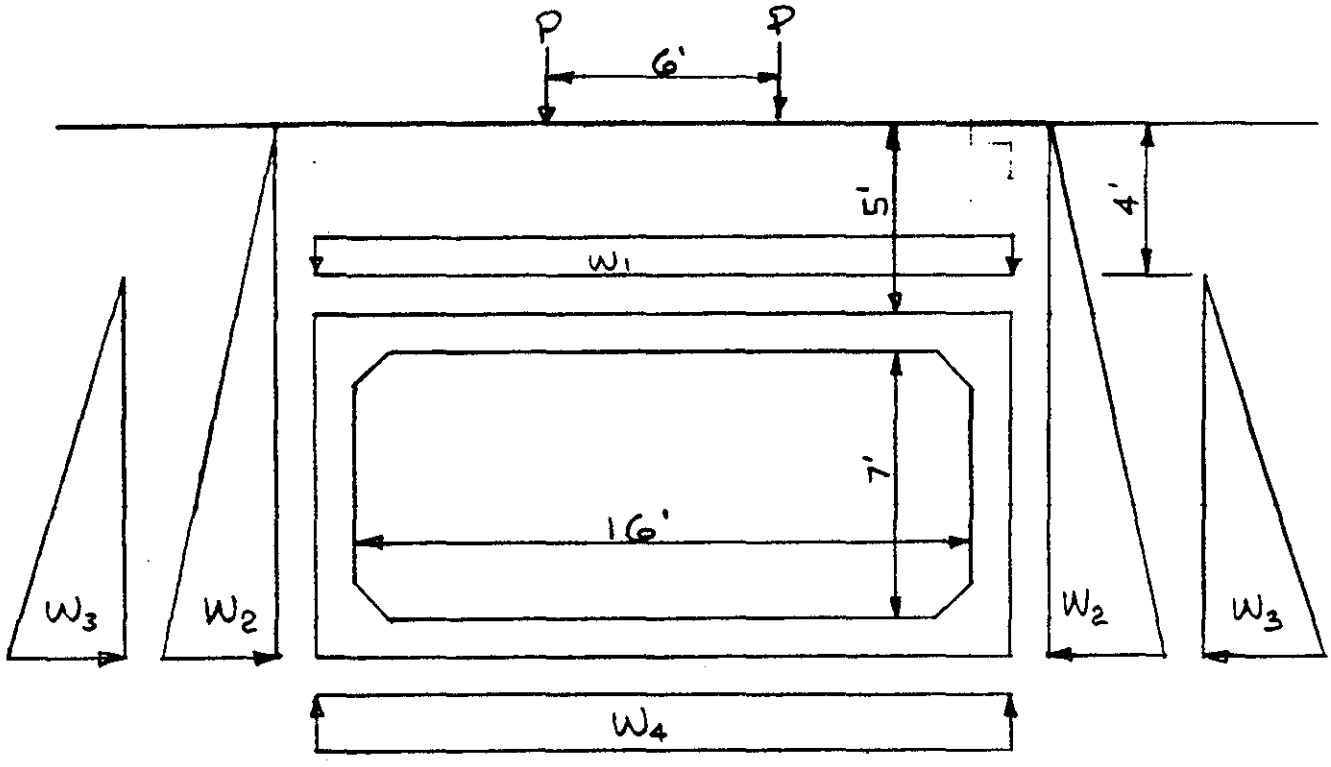


Project MDC - TOWN RIVER Acct. No. 7183 Page 1 of 54

Subject CULVERT Compd. By RITTMEYER Date 5/9/84

Detail HEADWALL PLAN @ OUTLET Ck'd. By Date

Project MDC - TOWN RIVER Acct. No. 7183 Page 12.1 of 54
Subject CULVERT Comptd. By RITTMEYER Date
Detail LOAD DIAGRAM Ck'd. By Date



W_2 = 65 PCF EQUIVALENT FLUID PRESSURE

W_3 = 30 PCF EQUIVALENT FLUID PRESSURE

ADD ABOVE INFORMATION TO STANDARD DRAWING

NONREPRODUCIBLE SHIP FORM 199
METCALF & EDDY, ENGINEERS

P.C. CONCRETE

$f_y = 60^{KSI}$ $f'_c = 5^{KSI}$ $n = 7$

$f_s = 24^{KSI}$ $f_c = 2^{KSI}$ $v_c = 67\,PSI$

$j = .876$ $R = 326$

$$RQ'D\ A_s = \frac{M_w \times 12}{24 j d} \qquad \frac{24j}{12} = 1.75$$

$$= \frac{M_w}{1.75d} \qquad M_w \text{ IN FT-K}$$

C.I.P. CONCRETE

$f_y = 60^{KSI}$ $f'_c = 3000\,PSI$ $n = 10$

$f_s = 24^{KSI}$ $f_c = 1200\,PSI$ $v_c = 52\,PSI$

$j = .889$ $\frac{24j}{12} = 1.78$ $R = 178$

NONREPRODUCIBLE GRID FORM 145
METCALF & EDDY, ENGINEERS

APPENDIX G

SUBSURFACE INVESTIGATIONS LABORATORY TEST RESULTS AND PROFILES AT INLET

CONTENTS

Figure	Description
G-1	Location of Borings
G-2	Section 2-2
G-3	Section 3-3
G-4	Boring Log T-2

TABLES

Table G-1 Summary Laboratory Soil Testing

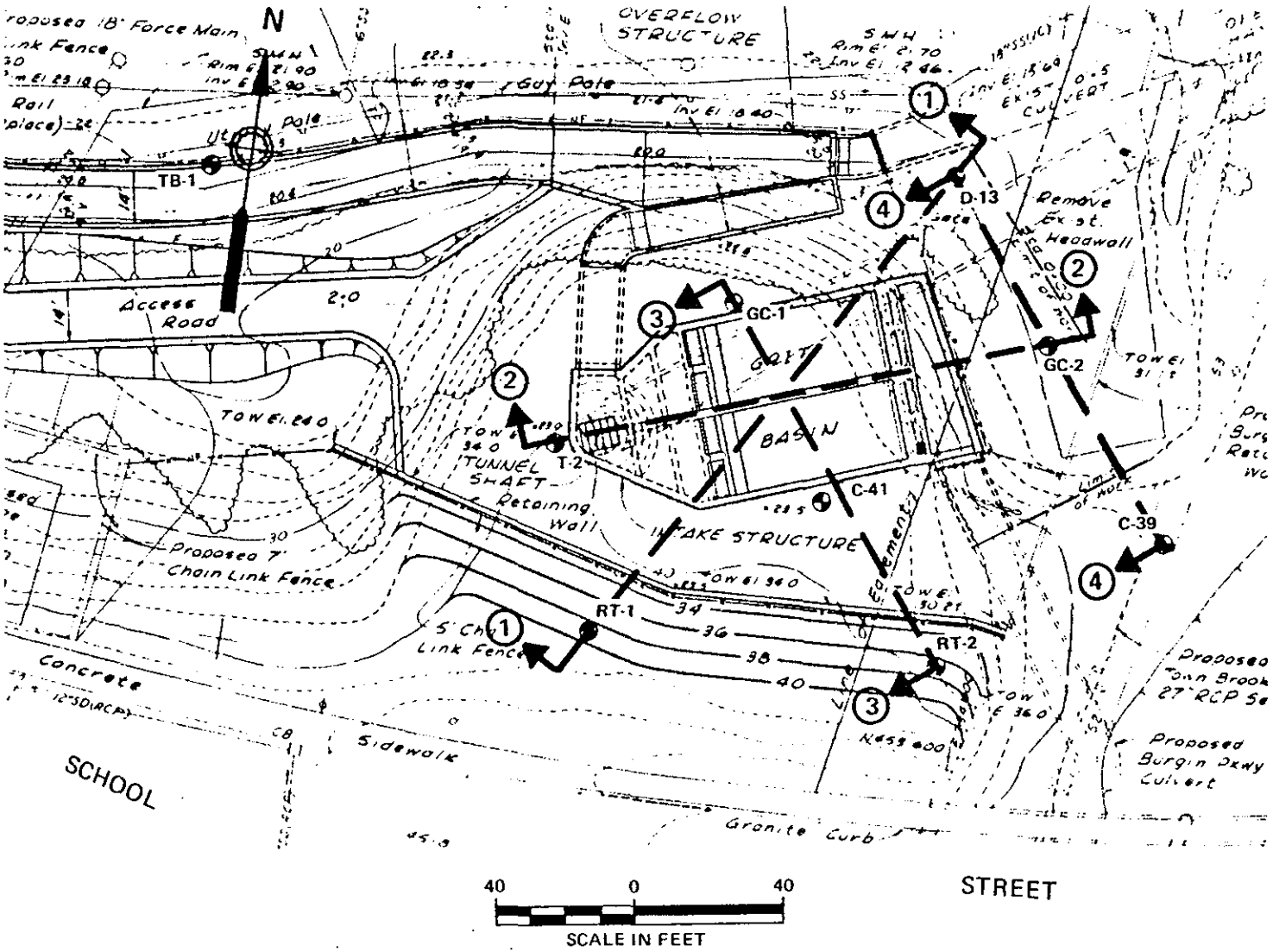


FIG. G-1 LOCATIONS OF BORINGS AND SUBSURFACE PROFILES AT INLET AREA

METCALF & EDDY

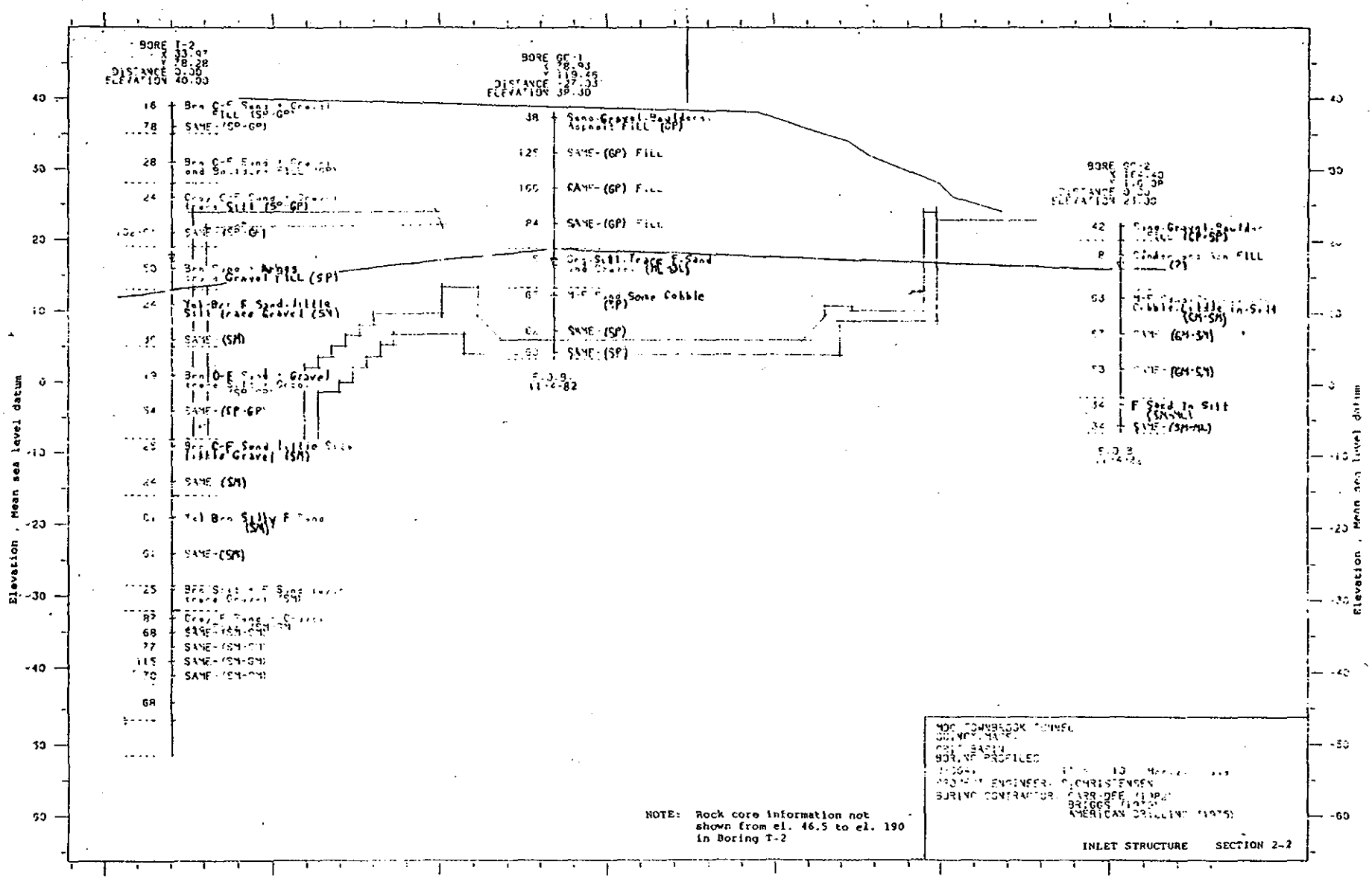


FIG. G-2 Subsurface Profile 2-2

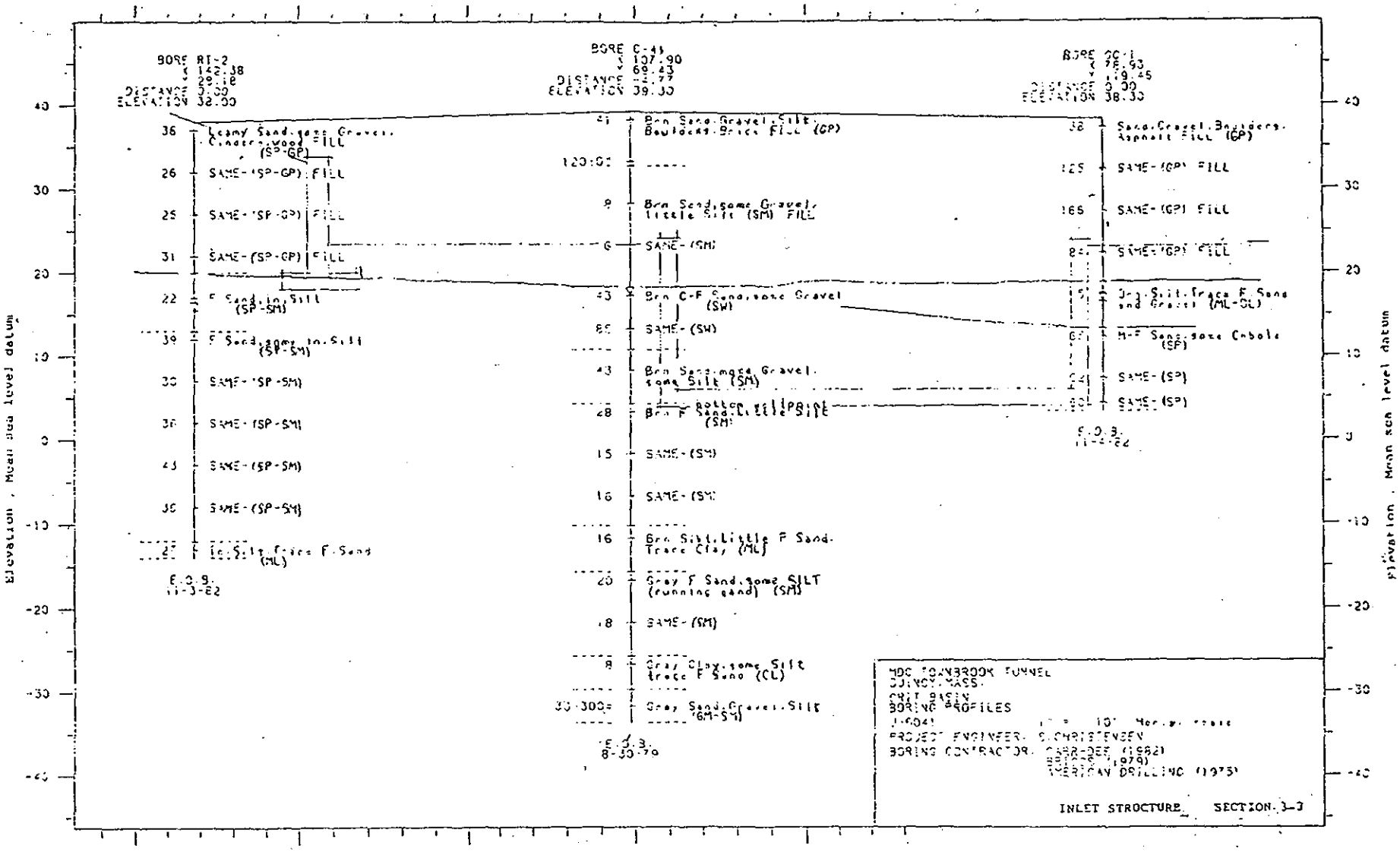


FIG. G-3 Subsurface Profile 3-3

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

TO Metcalf & Eddy, Inc ADDRESS Boston, Mass
PROJECT NAME MDC Town Brook Stormwater LOCATION Quincy, Mass
REPORT SENT TO above /Tunnel PROJ. NO.
SAMPLES SENT TO " OUR JOB NO. 83-237

SHEET 1 OF 4
DATE
HOLE NO. T-2
LINE & STA.
OFFSET
SURF. ELEV.

GROUND WATER OBSERVATIONS
At 23' after 12 Hours
At ____ after ____ Hours

Rods "NW"	CASING	SAMPLER	CORE BAR.
Type	HW NW	~~S/S~~	~~NQWL~~
Size - D	4 3	1 3/8"	
Hammer Wt	300#	140#	BIT
Hammer Fall	24"	30"	Dia.

	Date	Time
START	3/31/83	a.m. p.m.
COMPLETE	4/22/83	a.m. p.m.
TOTAL HRS.		

BORING FOREMAN A. Whitaker
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	To 12-18	Moisture Density or Consist.	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		0'-1.5'	D	6	7	9	M/m/d		Dark Brown fine to coarse Sand & gravel & FILL	1	1.5	1.0
		3.5'-5'	D	27	67	11	W/v/d	5'		2	1.5	0.8
		8.5'-10'	D	9	14	14	W/m/d	12'	Brown fine to coarse Sand & gravel & boulders - FILL	3	1.5	1.0
		13.5'-15'	D	9	12	12	M/m/d		Gray fine to coarse Sand & gravel, tr. silt - FILL	4	1.5	0.7
		18.5'-19'	D	102				21'		-	0.5	0
		23.5'-25'	D	10	10	40	W/dense	27'	Brown Sand & ashes, tr. gravel - FILL	5	1.5	0.2
		28.5'-30	D	9	12	12	W/m/d	33.5'	Yellow Brown fine SAND, (silty) tr. coarse gravel	6	1.5	1.0
		33.5'-35'	D	12	14	22	M/dense	35'	Yellow Brown fine SAND, little silt, tr. fine to med. gravel	7	1.6	0.9
		38.5-40'	D	12	9	10	W/m/d.		Brown fine to coarse SAND & Gravel, tr. silt & cobbles	8	1.5	0.7

GROUND SURFACE TO 45' USED 4" CASING: THEN 3" to 89.5 & cored

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler
Cohesionless Density: 0-10 Loose; 10-30 Med. Dense; 30-50 Dense; 50 + Very Dense
Cohesive Consistency: 0-4 Soft; 4-8 M/Stiff; 8-15 Stiff; 15-30 V-Stiff; 30 + Hard

SUMMARY:
Earth Boring 86.5
Rock Coring 143.5
Samples 20

HOLE NO. T-2

TOWN PRESS — EAST PROV.

FIG. G-4 BORING LOG T-2

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

SHEET 2 OF 4
DATE
HOLE NO. T-2
LINE & STA.
OFFSET
SURF. ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO

ADDRESS
LOCATION
PROJ. NO.
OUR JOB NO. 83-237

GROUND WATER OBSERVATIONS
At after Hours
At after Hours

	CASING	SAMPLER	CORE BAR.
Type			
Size - O.			
Hammer Wt			BIT
Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	12-18	Moisture Density or Consist.	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		43.5'-45'	D	21	30	24	M/v/d		Brown fine to coarse SAND & fine to medium gravel, tr. silt	9	1.5	1.1
								48'				
		48.5'-50'	D	12	12	13	W/m/d		Brown fine to coarse SAND, lit fine to medium gravel & silt	10	1.5	0.5
		53.5'-55'	D	10	12	12	"			11	1.5	1.0
								56'				
									Yellow Brown silty fine SAND			
		58.5'-60'	D	13	24	37	V/d/m			12	1.5	1.4
		63.5'-65'	D	19	31	30	W/v/d			13	1.5	0.7
								68.5				
		68.5'-70'	D	7	9	16	W/v/s		Brown layered SILT & fine Sand, tr. fine gravel	14	1.5	1.0
								72'				
		72'-74'	D	21	40	47	M/v/d		Gray fine SAND & gravel, embedded in silt (Till)	15	2.0	1.0
						49						
		74'-76'	D	30	31	37				16	2.0	0.7
						39						
		76'-78'	D	30	37	40				17	2.0	1.0
						31						
		78'-80'	D	27	61	54			" & fine to coarse gravel	18	2.0	0.8
						49						

GROUND SURFACE TO USED "CASING: THEN

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thin-wall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring
Rock Coring
Samples

HOLE NO T-2

FIG. G-4 CONT'D

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

SHEET 3 OF 4
DATE
HOLE NO. T-2
LINE & STA.
OFFSET
SURF. ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO

ADDRESS
LOCATION
PROJ. NO.
OUR JOB NO. 83-237

GROUND WATER OBSERVATIONS

At _____ after _____ Hours

At _____ after _____ Hours

	CASING	SAMPLER	CORE BAR.
Type			
Size - D.			
Hammer Wt			BIT
Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		

BORING FOREMAN
INSPECTOR
SOILS ENGR.

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	To 6-12	12-18	Moisture Density or Consist	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		80'-82	D	15	31	39	M/v/d		TILL	19	2.0	0.8
						40						
		83-85'	D	24	30	38				20	2.0	1.0
						—						
				Coring Time								
		86.5-94.5	C	5	6	7		86.5		C1	8'	6.8
				6	5	3			Pink GRANITE			
				3	3			91.5				
		94.5-100	C	3	3	5			Gray ARGILLITE	2	5.5	5.5
				6	6			97'				
									Pink GRANITE			
		100-108	C	5	6	5				3	8'	8'
				4	4	5						
				4	4							
		108-114	C	5	5	5				4	6'	6'
				5	5	5						
		114-121.5	C	5	6	6				5	7.5	7.5
				6	6	6						
				6	6							
		121.5-131.5	C	6	6	6				6	10	10
				6	6	6						
		131.5-141.5	C	6	6	7				7	10	10
				7	7	7						
			7	7	7	8						
		141.5-151.5	C	8	8	8				8	10	10
				8	8	8						
			8	8	8	8						
		151.5-161.5	C	8	8	8				9	10	10
				8	8	8						
			8	8	8	8						
		161.5-171	C	8	7	7				10	9.5	9.5
				7	7	7						
			7	7	7	7						
		171-181	C	7	7	7				11	10	10
				7	7	7						
			7	7	7	7						

GROUND SURFACE TO _____ USED _____ "CASING: THEN _____

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thin Wall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density	Cohesive Consistency
0-10 Loose	0-4 Soft
10-30 Med. Dense	4-8 M/Stiff
30-50 Dense	8-15 Stiff
50 + Very Dense	15-30 V-Stiff
	30 + Hard

SUMMARY:
Earth Boring
Rock Coring
Samples

HOLE NO. T-2

FIG. G-4 CONT'D

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE G-1

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_0	Laboratory Log and Soil Description
C39	S-1	0.0-1.5					18											Dark Gray, fine to medium SAND, some fine Gravel, little Silt (SM)
	S-2	5.0-6.5					15											Brown, fine SAND, trace fine Gravel, little Silt (SM)
	S-3	10.5-12.0					22											Brown, fine SAND, some Silt (SM)
	S-4	15.5-17.0					57											Brown SILT and fine Sand (ML)
	S-5	20.5-22.0					13											Brown fine SAND, little Silt (SM)
	S-6	25.5-27.0					39											Brown fine SAND and Silt (SM)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE G-1 Con't

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS							Permeability	STRENGTH TESTS					CONSOL.	Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf		Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	C_c / e_o	
C39	S-7	30.5-32.0					31											Brown, fine SAND, some Silt (SM)

SUMMARY OF LAB TESTS
TABLE NUMBER ____

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE G-1 Con't

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
T-2	S-15	72.0-74.0					55											Gray SILT, some Sand and Gravel (ML)
	S-16	74.0-76.0					58											Gray SILT, some Sand, little Gravel (ML)
	S-17	76.0-78.0					63											Gray SILT, some Sand, trace Gravel (ML)
	S-18	78.0-80.0					12											GRAVEL and Sand, little Silt (GP-GM)
	S-20	83.0-85.0					47											Gray SILT, some Sand and Gravel (ML)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

APPENDIX H

SUBSURFACE INVESTIGATIONS LABORATORY TEST RESULTS AND PROFILES AT OUTLET

CONTENTS

Figure	Description
H-1	Location of Borings
H-2	Section 1-1
H-3	Section 2-1
H-4	Boring Log T-10

TABLES

Table H-1 Summary Laboratory Soils Testing

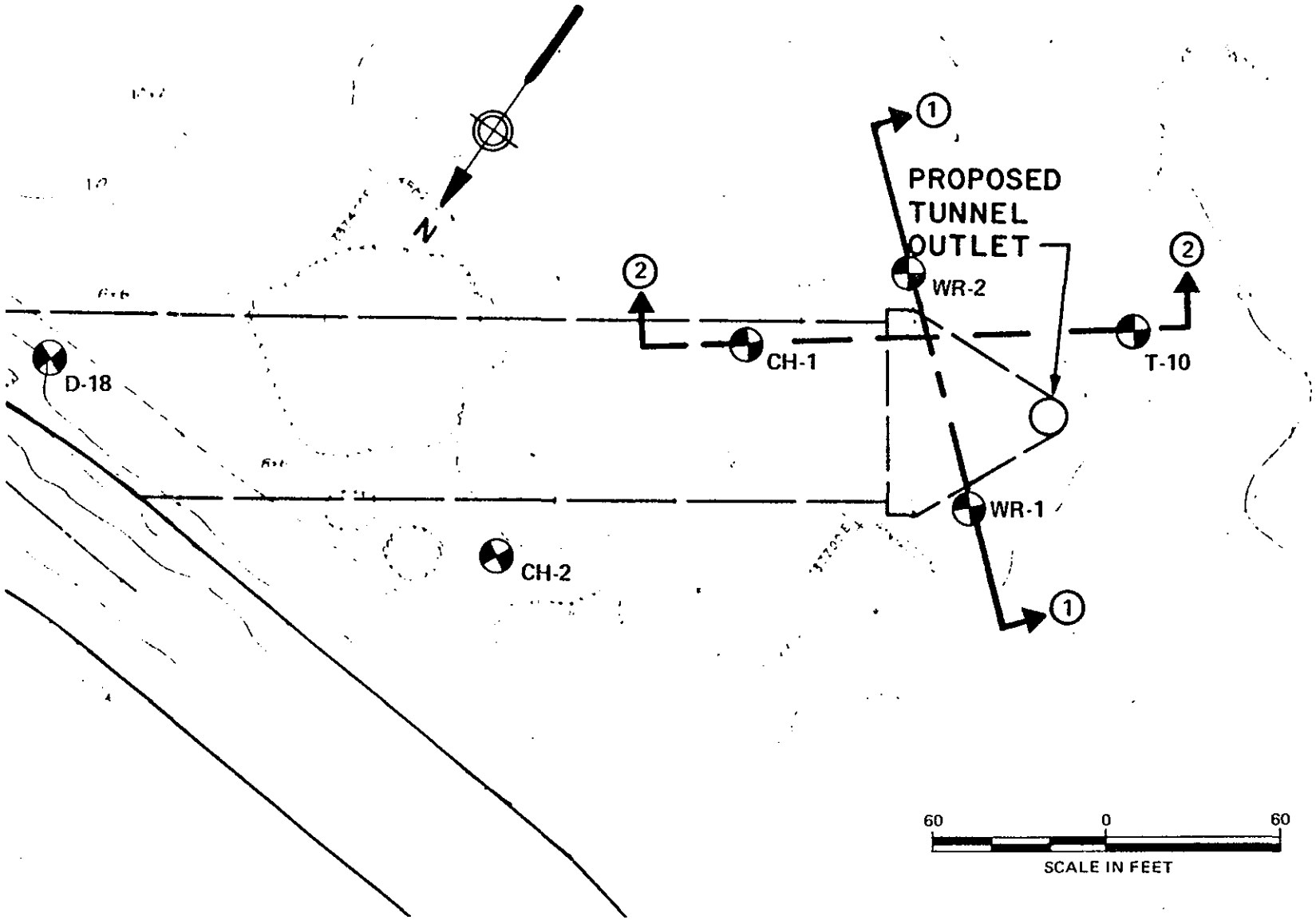


FIG. H-1 LOCATIONS OF BORINGS AND SUBSURFACE PROFILES AT OUTLET AREA

METCALF & EDDY

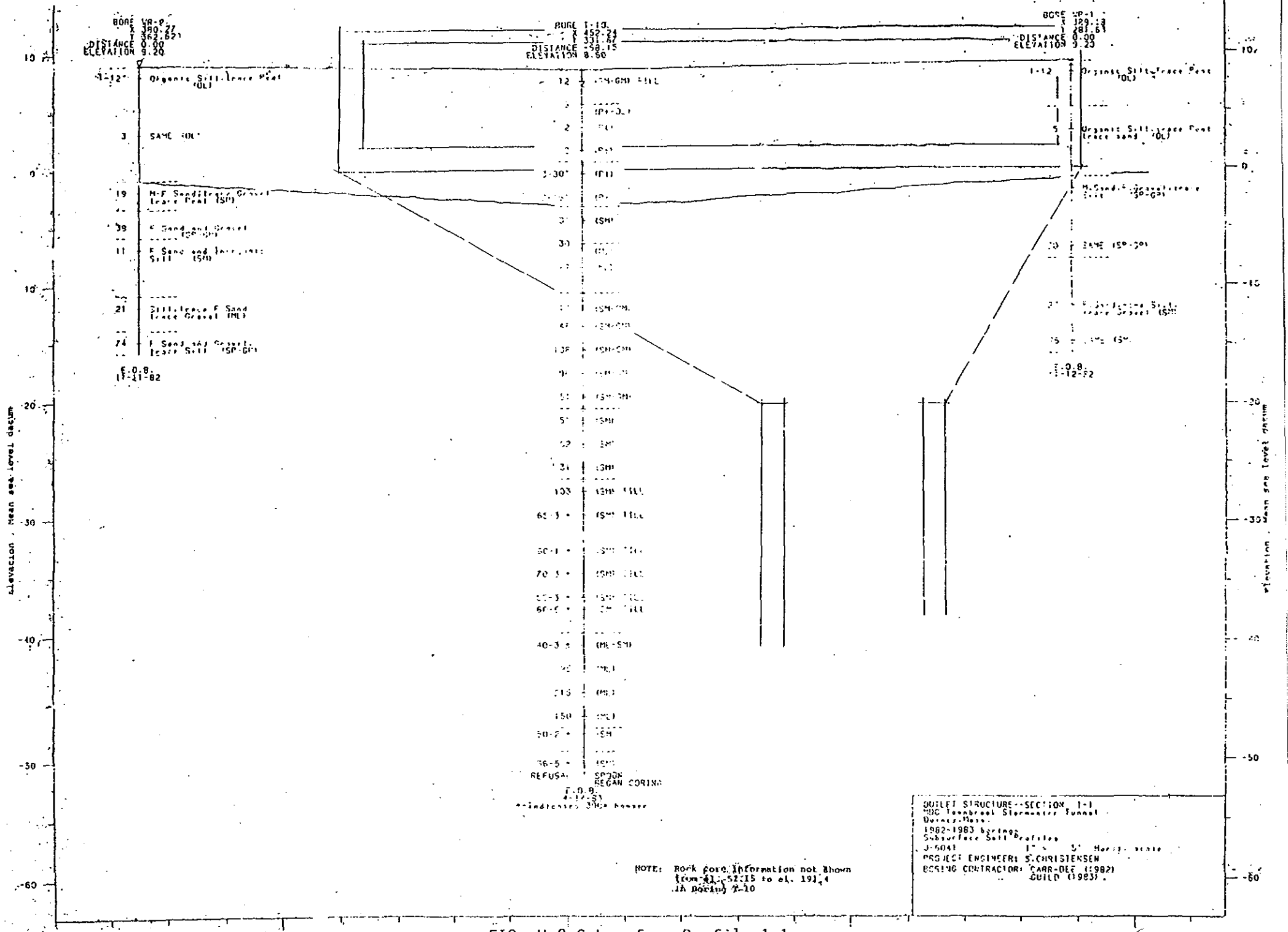


FIG. II-2 Subsurface Profile 1-1

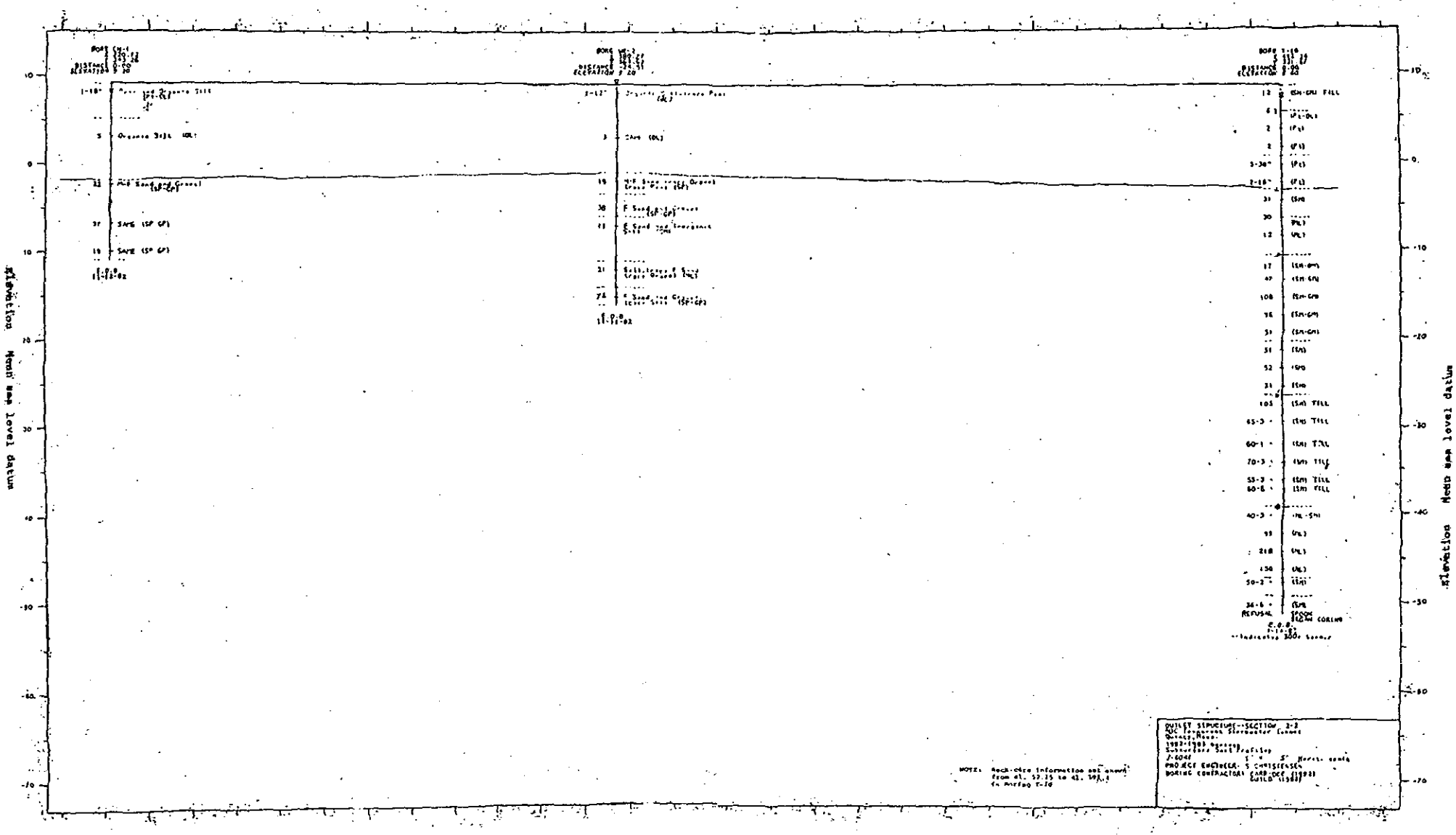


FIG. H-3 Subsurface Profile 2-2

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

SHEET 1 OF 3
DATE
HOLE NO. T-10
LINE & STA.
OFFSET
SURF. ELEV.

TO Metcalf & Eddy, Inc. ADDRESS Boston, Mass
PROJECT NAME MDC Town Brook Stormwater LOCATION Quincy, Mass
REPORT SENT TO above /Tunnel PROJ. NO.
SAMPLES SENT TO " OUR JOB NO. 83-237

GROUND WATER OBSERVATIONS
At 1'6" after Hours
At after Hours

	CASING	SAMPLER	CORE BAR.
Type	HW NW	S/S	NQ2WL
Size O.D.	4" 3"	1 3/8"	2"
Hammer Wt	300#	140#	BIT Dia.
Hammer Fall	24"	30"	

	Date	Time
START	4/5/83	a.m. p.m.
COMPLETE	4/14/83	a.m. p.m.
TOTAL HRS.		
BORING FOREMAN	K. Allen	
INSPECTOR	Tracy	
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	6-2	To 2-8	Moisture Density or Consist.	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		0'-2'	D	30	30	6	Dense		Brown fine to coarse Sand & gravel, little silt - FILL	1	24'	2"
						6						
		2'-4'	D	4	5	1	Soft	3'		2	24'	2"
						1			Root mat.& organic silt. Difficulty picking up sample-pushing gravel Fill)			
		4'-6'	D	1	1	1					24'	0
						1						
		6'-8'	D	1	1	1	"			3	24'	2"
						1		8'				
		8'-10'6"	D	3	/30"		"		Dark Brown PEAT	4	30"	4"
		10'6"-11'10"	D	2/16"			"	11'10"		5	16"	10"
		11'10"-12'	D	4/2"			"	12'	Gray clayey SILT	5A	2"	2"
		12'-14'	D	8	19	12	Dense	13'	Gray silty fine SAND	6	24"	16"
						18		14'	Gr.silty F-C SAND & Gravel			
		14'-16'	D	12	14	16	"	15'	Gr.F-C SAND&Gravel, tr. silt	7	24"	15"
						21						
		16'-18'	D	12	7	5	Stiff		Gray fine sandy SILT	8	24"	10"
						4						
		18'-19'2"	UF					19'2"		P1	14"	11"
		19'6"-21'	D	18	10	7	M/dense		Gray fine to coarse SAND & Gravel, some silt	9	18"	8"
		21'-23'	D	7	14	33	V/dense			10	24"	9"
						43						
		23'-25'	D	51	42	66	"		" little silt	11	24"	15"
						78						
		25'-27'	D	21	36	60	"		" Yellow Brown	12	24"	10"
						36						
		27'-29'	D	20	26	25				13	24"	7"
						18		29'				
		29'-31'	D	24	24	27	"		Brown silty fine SAND, some coarse sand & fine to coarse gravel	14	24"	9"
						22						
		31'-33'	D	26	30	22	Dense			15	24"	16"
						20		33'				
		33'-35'	D	21	15	16	"		Brown silty fine SAND	16	24"	10"
						16		35'				
		35'-37'	D	19	48	55	Very		Gray silty fineSAND, little c. sand & fine gravel	17	24"	11"
						105	dense					
		37'	D	Ref.								
		38'-38'3"	D	100/0"			"		" & fine to coarse grave	18	3"	3"
					65/3"	(300#)						

GROUND SURFACE TO 50 USED 4 "CASING: THEN 3" to 62'6"

Sample Type
D=Dry C=Cored W=Washed
UP=Undisturbed Piston
TP=Test Pit A=Auger V=Vane Test
UT=Undisturbed Thinwall

Proportions Used	
trace	0 to 10%
little	10 to 20%
some	20 to 35%
and	35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density		Cohesive Consistency	
0-10	Loose	0-4	Soft
10-30	Med. Dense	4-8	M/Stiff
30-50	Dense	8-15	Stiff
50 +	Very Dense	15-30	V-Stiff
		30 +	Hard

SUMMARY:
Earth Boring 60'9"
Rock Coring 139'3"
Samples 29

HOLE NOT-10

TOWN PRESS — EAST PROV.

FIG. H-4 BORING LOG T-10

GUILD DRILLING CO., INC.

100 WATER STREET EAST PROVIDENCE, R. I.

SHEET 2 OF 3
DATE
HOLE NO. T-10
LINE & STA.
OFFSET
SURF. ELEV.

TO
PROJECT NAME
REPORT SENT TO
SAMPLES SENT TO

ADDRESS
LOCATION
PROJ. NO.
OUR JOB NO. 83-237

GROUND WATER OBSERVATIONS

At _____ after _____ Hours

At _____ after _____ Hours

	CASING	SAMPLER	CORE BAR.
Type			
Size I.D.			
Hammer Wt			BIT
Hammer Fall			

	Date	Time
START		a.m. p.m.
COMPLETE		a.m. p.m.
TOTAL HRS.		
BORING FOREMAN		
INSPECTOR		
SOILS ENGR.		

LOCATION OF BORING

DEPTH	Casing Blows per foot	Sample Depths From - To	Type of Sample	Blows per 6" on Sampler From 0-6	6-12	To 12-18	Moisture Density or Consist	Strata Change Elev.	SOIL IDENTIFICATION Remarks include color, gradation, Type of soil etc. Rock-color, type, condition, hardness, Drilling time, seams and etc	SAMPLE No	Pen	Rec
		38'3"-41'3"	C					41'3"	Drilled ahead w/R.B. Still in			
		41'3"-41'4"	D	100/0"	*60	0/1"	V/dense	42'6"	refusal material-Gr.fine to	19	1"	1"
		42'6"-43'9"	D	100	*55	*70/3"	"		coarse GRAVEL	20	15"	15"
		44'6"-45'6"	D	100/3"	*55	/3"	"		Gray silty fine SAND, some coa. sand & fine to coarse gravel	21	12"	12"
						*74						
		46'6"-47'6"	D	110	*60		"			22	12"	12"
		48'-49'6"	D	51	48	100/3"	Hard	48'		23	18"	16"
						*40/3"			Gray clayey SILT, little fine to medium SAND & fine gravel			
		50'-52'	D	21	45	50	"			24	24"	18"
						115						
		52'-53'6"	D	42	85	130	"			25	18"	15"
		54'-56'	D	21	45	105	"			26	18"	16"
								56'				
		56'-56'8"	D	100/4"	*15/2"		Very		Gray silty fine SAND, some	27	8"	8"
						*50/2"	dense	58'	coarse sand & fine-med. gravel			
		58'-60'	D	100/1"	*36	/5	"		Gray clayey fine SAND &	28	24"	20"
			*21	*23	*45		"	60'9"	weathered Argillite			
		60'-60'9"	D	125	*75/3"		"		Gray weathered ARGILLITE	29	9"	6"
							Min/ft					
		60'9"-66'	C				7			C1	63"	52"
		66'-76'	C				9			C2	120	120
		76'-86'	C				6			3	120	118
		86'-94'6"	C				4			4	102	98
		94'6"-104'	C				3			5	114	108
		104'-108'	C				3			6	48	60
		108'-116'6"	C				3			7	120	107
		116'6"-120'3"	C				5			8	45	52
		120'3-130'3"	C				4			9	120	118

GROUND SURFACE TO _____ USED _____ "CASING: THEN _____

Sample Type
D: Dry C: Cored W: Washed
UP: Undisturbed Piston
TP: Test Pit A: Auger V: Vane Test
UT: Undisturbed Thinwall

Proportions Used
trace 0 to 10%
little 10 to 20%
some 20 to 35%
and 35 to 50%

140 lb Wt. x 30" fall on 2" O.D. Sampler

Cohesionless Density
0-10 Loose
10-30 Med. Dense
30-50 Dense
50 + Very Dense

Cohesive Consistency
0-4 Soft
4-8 M/Stiff
8-15 Stiff
15-30 V-Stiff
30 + Hard

SUMMARY:
Earth Boring _____
Rock Coring _____
Samples _____

HOLE NO T-10

TOWN PRESS — EAST PROV.

FIG. H-4 CONT'D

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE H-1

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
CH-1	S-2	5.0-7.0	8	63.2	3.3	2900	98			4.9								Dark gray, clayey Organic SILT of high plasticity (OH)
	S-3	10.0-11.0	9	54.7	3.1	3000	98			4.4								Dark gray, clayey Organic SILT of high plasticity (OH)
	S-3A	11.0-12.0	10		3.5	950	8			1.3								Gray-brown, fine to coarse GRAVEL and fine to coarse SAND, trace (+) Silt (GP-GM)
CH-2	S-1	0.0-2.0	11				6											Brown, fine to medium SAND, some fine to coarse Gravel, trace Silt (SP)
	S-2	5.0-7.0	12				22											Gray, fine to medium SAND, some (-) Silt & Clay (overall low plasticity) (SM)
CH-2	S-3	10.0-12.0	13	56.6	3.3	3750	94			5.8								Dark gray, clayey Organic SILT of high plasticity, trace fine Sand (OH)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE H-1 Con't

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
T-10	S-23	48.0-49.5			21.5	16.1	82											Silty CLAY, little Sand, trace Gravel (CL-ML)
	S-24	50.0-52.0					68											Silty CLAY, some Sand, little Gravel (CL)
	S-26	54.0-56.0			26.7	17.3	62											Silty CLAY, some Sand, trace Gravel (CL)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

TOWN BROOK
QUINCY, MASSACHUSETTS

TABLE H-1 Con't

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____
Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
T-10	S-15	31.0-33.0					44											f-m. SAND and Silt, trace Gravel (SM)
	S-16	33.0-35.0					37											f.-m. SAND and Silt, little Gravel (SM)
	S-17	35.0-37.0					42											f.-m. SAND and Silt, trace Gravel (SM)
	S-18	38.0-38.3					48											SILT, some Sand and Gravel (ML)
	S-21	44.5-45.5					42											SAND and Silt, little Gravel (SM)

SUMMARY OF LAB TESTS
TABLE NUMBER ____

APPENDIX I

SUBSURFACE INVESTIGATIONS, LABORATORY TEST RESULTS, AND PROFILES AT SOUTHERN ARTERY CULVERT AND CHANNEL

CONTENTS

Subject	Page
Test Boring	I-1
Results of Subsurface Investigations	I-4
Laboratory Analysis	I-8
Laboratory Test Procedures	I-9

FIGURES

Figure	Description	
I-1	Typical Boring Log	I-2
I-2	Culvert Boring Plan	I-5
I-3	Soil Profile at Proposed Culvert, Borings BG-2, 4, 6 and 8	I-6
I-4	Soil Profile at Proposed Culvert Borings BG 2, 3, 5 and 7	I-7

TABLE

I-1	Summary of Lab Tests	I-11

TEST BORINGS

In order to determine subsurface conditions at the proposed culvert site and along the upstream and downstream reaches, 29 test borings were conducted between the proposed Town Brook Relief Tunnel confluence with Town River and the Town River Bay. Boring locations are shown in Plates 4-23 and 4-24.

CARR-DEE TEST BORING AND CONSTRUCTION CORPORATION

37 LINDEN STREET P.O. BOX 321 MEDFORD, MASSACHUSETTS 02155 Telephone 391-4500

To METCALF & EDDY, INC., ENGINEERS, BOSTON, MA. Date NOV. 29, 1982 Job. No. 82274

Location MDC TOWNBROOK STORMWATER TUNNEL, QUINCY, MA. Scale 1" = 4 ft.

BORING BC-1

GROUND SURFACE EL 10.1

0'3" ASPHALT

TOP SOIL GRAVEL, CINDERS, SOME WOOD FILL	14 12 9 8	S#1, FROM 0'6" TO 2'6" RECOVERED 18"
	9 6 5 5	S#2, FROM 5'0" TO 7'0" RECOVERED 11"
ORGANIC SILT, SOME FIBRES, SOME SHELLS		UP#1, FROM 11'0" TO 13'0" RECOVERED 18"
	1 2 2 2	S#3, FROM 13'0" TO 15'0" RECOVERED 3"
	2 2 2 2	S#4, FROM 15'0" TO 17'0" RECOVERED 14"
VERY DENSE CLAYEY FINE SAND, GRAVEL, STONES, SOME COBBLES, SOME INORGANIC SILT	30 30 46 57	S#5, FROM 20'0" TO 22'0" RECOVERED 11"
	48 36 40 49	S#6, FROM 25'0" TO 27'0" RECOVERED 15"
	49 68	S#7, FROM 30'0" TO 31'0" RECOVERED 8"

10'0"

18'6"

32'0"

CONTINUED ON SHEET NO. 2

FIG. I-1 TYPICAL BORING LOG

CARR-DEE TEST BORING AND CONSTRUCTION CORPORATION

37 LINDEN STREET P.O. BOX 321 MEDFORD, MASSACHUSETTS 02155 Telephone 391-4500

To METCALF & EDDY, INC., ENGINEERS, BOSTON, MA. Date NOV. 29, 1982 Job. No. 82274

Location MDC TOWNBROOK STORMWATER TUNNEL, QUINCY, MA. Scale 1" = 4 ft.

BORING BC-1 (CONTINUED)

32'0"

REFUSAL - (UNABLE TO ADVANCE ROLLER-BIT)

WATER LEVEL 6'0"

SIZE OF CASING NW, LENGTH 20'0"

NUMBER OF DRIVE SAMPLES (S), 7

NUMBER OF 3" PISTON SAMPLES (UP), 1

DRILLER: J. CENTRELLA, INSPECTOR: C. SWEET

DATE STARTED 11-2-82, 11-16-82

All samples have been visually classified by AJD Unless otherwise specified, water levels noted were observed at completion of borings, and do not necessarily represent permanent ground water levels. Figures in right hand column indicate number of blows required to drive _____ inches using 140 lb. weight falling 30 inches ±. Figures in column to left (if noted) indicate number of blows to drive casing one foot, using 300 lb. weight falling 24 inches ±.

SWEET

RESULTS OF SUBSURFACE INVESTIGATIONS

Soil profiles at the proposed culvert location were developed based on the 8 boring logs in this area. Figure I-2 shows the location of these borings. The profiles (Figures I-3 and I-4) were used in the preliminary design to compute foundation requirements for the proposed culvert.

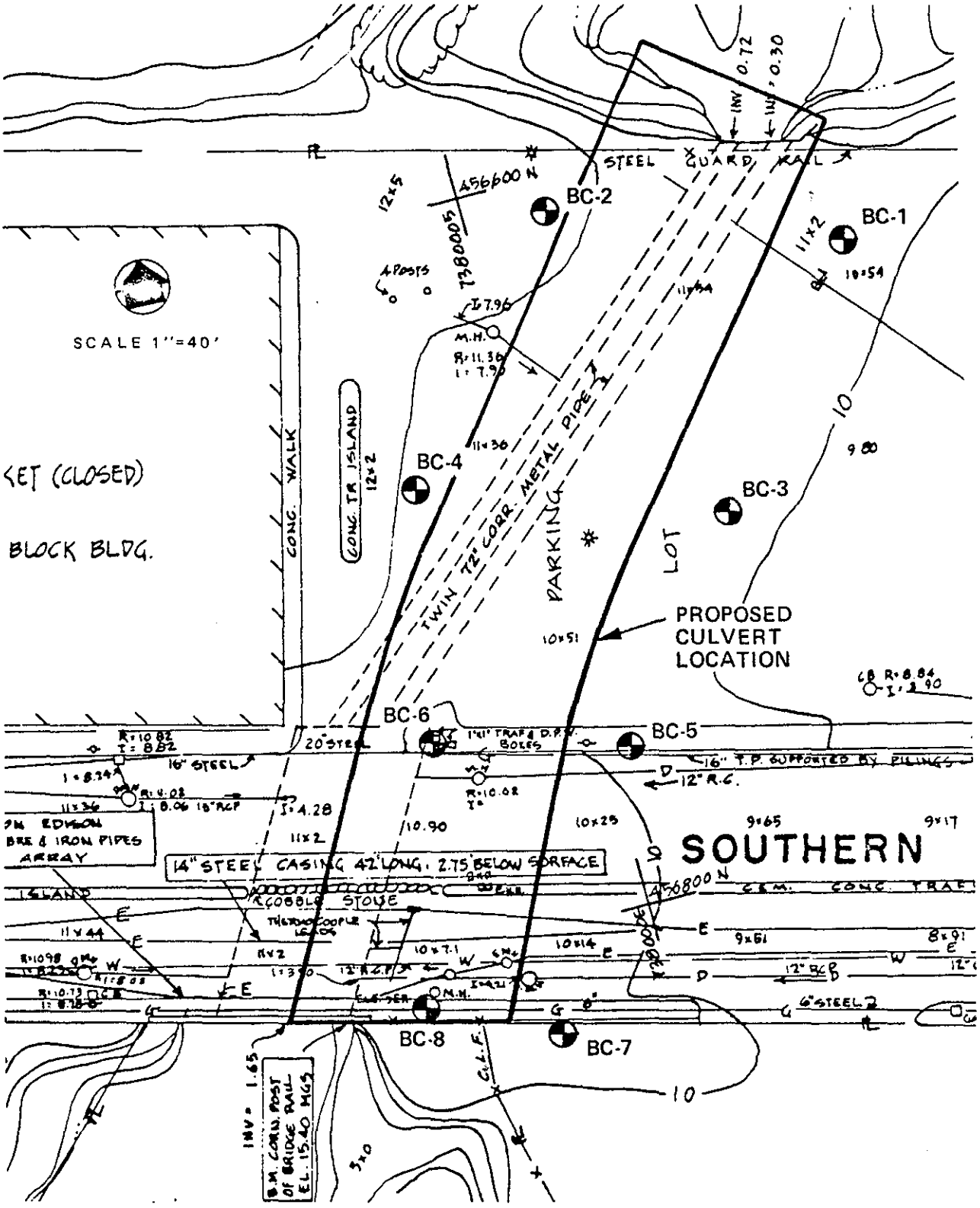


FIG. 1-2 CULVERT BORING PLAN

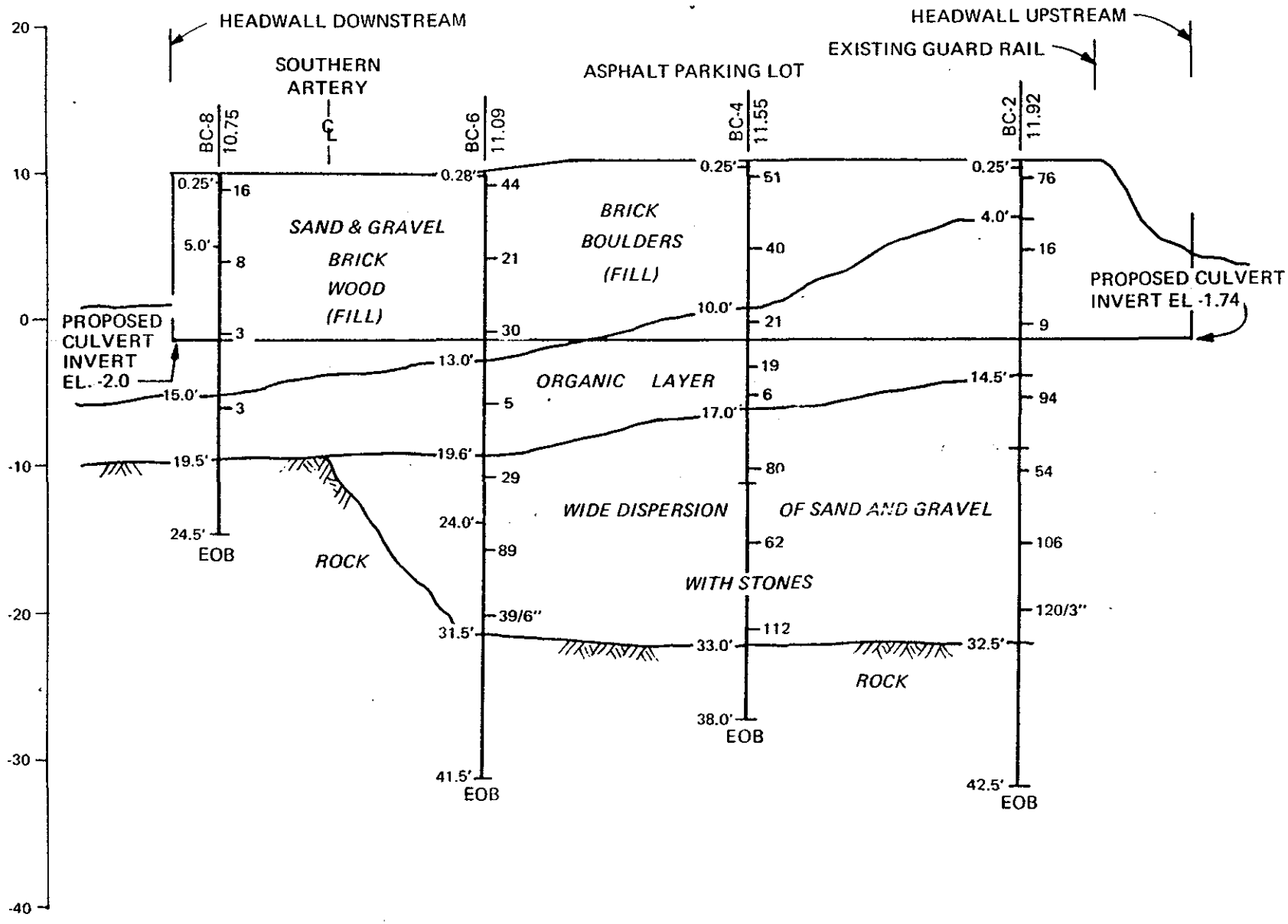


FIG. I-3 SOIL PROFILE AT PROPOSED CULVERT, BORINGS BC-2,4,6 AND 8

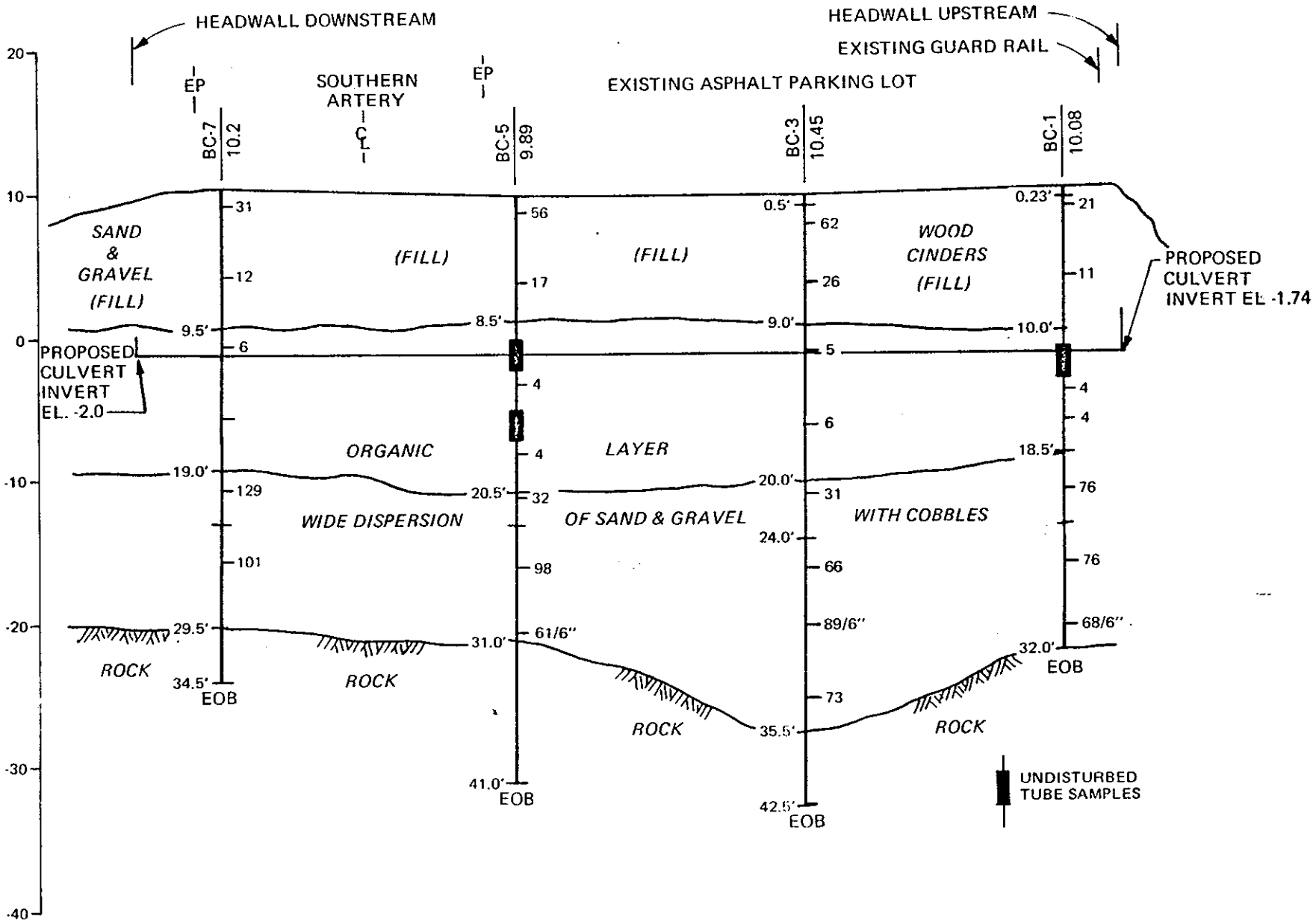


FIG. I-4 SOIL PROFILE AT PROPOSED CULVERT, BORINGS BC-1,3,5 AND 7

METCALF & EDDY

LABORATORY ANALYSIS

A laboratory analysis was conducted on several of the soil samples generated from some of the 29 boring locations. This was done to determine the following soil parameters: grain size, moisture content, organic content, pH of soil, and conductivity of soil. These parameters aided in the preliminary design of the culvert foundation bedding, the channel improvements, and soil support systems.

LABORATORY TEST PROCEDURES

TOWN BROOK
QUINCY, MASSACHUSETTS
FILE NO. L-3816

1. The following tests were performed with the noted ASTM test designation:

TEST	ASTM DESIGNATION
Grain Size	D1140-54 (Sieve only)
Moisture Content	D2216-80
Organic Content	D2974-7
pH of Soil	(See Item 2)
Conductivity of Soil	(See Item 3)

2. Test Procedures for pH Determinations

pH determinations were performed in accordance with "Methods of Soil Analysis", by the American Society of Agronomy, Part 2, pps. 922-923, "Method of pH of Soil in Water". The procedure is as follows:

To 40 g. of soil in a 250-ml. beaker, add 100 ml. of distilled water, and stir the suspension several times during the next 30 minutes. Let the soil suspension stand for about 1 hour to allow most of the suspended clay to settle out from the suspension. Adjust the position of the electrode in the clamps of the electrode holder so that, upon lowering the electrode into the beaker, the glass electrode will be immersed well into the clear supernatant solution to establish a good electrical contact through the ground-glass joint or the fiber-capillary hole.

The soil pH in water was measured with a digital pH meter (ORION Model 701A). Prior to testing the pH of the soil slurry, the meter was calibrated using standard buffer solutions of 7.00, 4.01, and 10.00 respectively. The pH of the distilled water used to dilute the soil sample was recorded as 6.7.

3. <u>Test Procedures for Conductivity Measurements</u>

Conductivity tests were performed using a Yellow Springs (Model 33) Conductivity Meter in the following manner:

> To 40 g. of soil in a 250 ml. beaker, add 150 ml. of distilled water and stir the suspension several times. Let the soil suspension stand for one hour to allow most of the suspended particles to settle out. Totally immerse the conductivity probe in the solution and read direct in μMHOS/cm from the instrument.
>
> Prior to testing the soil/water suspension, the calibration of the instrument was checked using conductivity standards of 1000 and 10,000 μMHOS/cm.

TABLE I-1

TOWN BROOK
QUINCY, MASSACHUSETTS

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____

Project No. L-3816 Project Engr. D.Schulze Assigned By J.Stangenberg Date Assigned 8/22/83 Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	pH	CONDUCTIVITY µMHOS/cm	Sieve -200 %	Hyd -2µ %	G_s	ORGANIC MATTER (%)	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
CH-3	S-2	5.0-7.0	14				13											Yellow-brown, fine to medium SAND, trace fine Gravel, little Silt, trace Organics (SM)
CH-5	S-2	5.0-7.0	15				7											Brown, fine to coarse GRAVEL and fine to coarse SAND, trace Silt (GP-GM)
	S-3	10.0-12.0	16				8											Gray, fine to coarse SAND and fine GRAVEL, trace (+) Silt (SP-SM)
	S-4	15.0-17.0	17	44.3	3.5	3400	97			4.4								Dark gray, clayey Organic SILT of high plasticity (OH)
Ch-6	S-1	0.0-2.0	18		3.4	2400	18			4.0								Gray-brown, fine to coarse GRAVEL and (-) fine to medium SAND, little (+) Organic Silt, trace Roots, Glass [illegible] (GM)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

GOLDBERG-ZOINO & ASSOCIATES, INC.
GEOTECHNICAL-GEOHYDROLOGICAL CONSULTANTS

TOWN BROOK
QUINCY, MASSACHUSETTS

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____

Project No. L-3816 Project Engr. D.Schulze Assigned By J.Stangenber Date Assigned 8/22/83 Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	pH	Conductivity µMHOS/cm	Sieve -200 %	Hyd -2µ %	G_s	Organic Matter (%)	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
CH-6	S-2	5.0-7.0	19	/			27											Brown, fine to medium SAND, little (+) fine Gravel, some Silt (SM)
CH-7	S-4	12.0-14.0	21	45.4	3.6	5500	87			3.9								Dark gray, clayey Organic SILT of high plasticity, trace (-) fine Gravel, trace fine Sand (OH)
CH-8	S-1	0.0-2.0	22	40.4	3.4	3200	30			6.8								Dark gray, fine to medium SAND Sizes, some (+) clayey Organic Silt, trace Coal pieces, Cinders (SM)
	S-2	5.0-7.0	23	42.1	4.3	3550	53			4.7								Dark gray, clayey Organic SILT of high plasticity and fine to medium SAND (OH)
CH-9	S-2	2.0-4.0	24				24											Brown, fine to coarse SAND, some fine Gravel some (-) Silt (SM)
	S-3	5.0-7.0	25				30											Brown, fine to medium SAND, some (-) fine Gravel, some (+) Silt (SM)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

GOLDBERG-ZOINO & ASSOCIATES, INC.
GEOTECHNICAL-GEOHYDROLOGICAL CONSULTANTS

TOWN BROOK
QUINCY, MASSACHUSETTS

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____

Project No. L-3816 Project Engr. D. Schulze Assigned By J. Stangenberg Date Assigned 8/22/83 Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	pH	Conductivity µmhos/cm	Sieve -200 %	Hyd -2µ %	G_s	Organic Matter (%)	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
CH-10	S-2	5.0-7.0	26	68.8			85											Dark gray, clayey Organic SILT of high plasticity, little fine Sand (OH)
CH-11	S-2	2.0-4.0	27	7.2	5.4	290	22			0.8								Gray, fine to coarse GRAVEL and fine to medium SAND, some (-) slightly organic Silt (GM)
	S-3	5.0-7.0	28				17											Gray, fine GRAVEL and fine to medium SAND, little Silt (GM)
SP-1	S-1	0.0-2.0	29				13											Dark brown, fine to coarse SAND, some (+) fine to coarse Gravel, little (-) Silt (SM)
	S-2	5.0-7.0	30	50.7			93											Dark gray, clayey Organic SILT of high plasticity, trace fine Sand (OH)
SP-2	S-1	0.0-2.0	31				8											Brown, fine to coarse SAND and fine GRAVEL, trace (+) Silt (SP-SM)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

GOLDBERG-ZOINO & ASSOCIATES, INC.
GEOTECHNICAL-GEOHYDROLOGICAL CONSULTANTS

TOWN BROOK
QUINCY, MASSACHUSETTS

LABORATORY TESTING DATA SUMMARY

Reviewed by _____
Date _____

Project No. L-3816 Project Engr. D.Schulze Assigned By J.Stangenber Date Assigned 8/22/83 Required _____

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS: Water Content %	pH	Conductivity µmhos/cm	Sieve -200 %	Hyd -2µ %	G_s	Organic Matter (%)	Permeability	STRENGTH TESTS: Torvane or Type Test	σ_c or $\bar{\sigma}_c$ or σ psf	Failure Criteria	$\sigma_1 - \sigma_3$ or τ psf	Strain %	CONSOL. C_c / e_o	Laboratory Log and Soil Description
SP-2	S-3	10.0-12.0	33	48.6	4.9	4700	98			3.3								Dark gray, clayey Organic SILT of high plasticity (OH)

SUMMARY OF LAB TESTS
TABLE NUMBER _____

GOLDBERG-ZOINO & ASSOCIATES, INC.
GEOTECHNICAL-GEOHYDROLOGICAL CONSULTANTS

APPENDIX J

COST ESTIMATES

Table	Subject	Page
J-1	Town River Improvements Costs From GDM Table C-4	J-1
J-2	Town River Culvert and Channel Improvements	J-1
J-3	Relief Tunnel, Shafts, Inlet and Outlet Works Costs From GDM Table C-4	J-2
J-4	Relief Tunnel, Shafts, Inlet and Outlet Works, Drill and Blast Tunnel Excavation	J-3
J-5	Relief Tunnel, Shafts, Inlet and Outlet Works, TBM Excavation, Cast-in-Place Lining	J-4
J-6	Relief Tunnel, Shafts, Inlet and Outlet Works, TBM Excavation, Precast Lining	J-5

TABLE J-1
TOWN RIVER IMPROVEMENTS COSTS
FROM GDM TABLE C-4

.09 Town River Improvements				
Site Preparation	1	JOB	LS	25,000
Water Control	1	JOB	LS	50,000
Traffic Control	1	JOB	LS	75,000
Demolition	1	JOB	LS	60,000
Support of Utilities	1	JOB	LS	25,000
Excavation	27,500	CY	5.40	148,500
Sheetin (left in place)	5,000	SY	14.00	70,000
Sheeting (removed)	20,000	SY	7.00	140,000
Screened Gravel	450	CY	12.00	5,400
Compacted Fill	4,500	CY	10.00	45,000
Gravel Base	4,000		12.00	48,000
Filter Fabric	18,000	SY	1.50	27,000
Gabions	3,000	CY	106.00	318,000
Reinforcement Steel	37,600	lbs.	.50	18,800
Tremie Concrete	3,500	CY	125.00	437,500
Structural Concrete	300	CY	188.00	56,400
Precast Concrete Culverts	280	LF	1,600.00	448,000
Bituminous Concrete	2,000	Ton	35.00	70,000
Granite Curb	1,400	LF	16.00	22,400
				$ 2,089,500
		Contingencies		210,500
		Total (Town River)		$ 2,300,000

TABLE J-2 DETAILED COST ESTIMATE
TOWN RIVER CULVERT AND CHANNEL IMPROVEMENTS
(OCT 1985 PRICE LEVEL)

Description	Estimated Quantity	Unit	Unit Prices ($)	Estimated Amount ($)
.09 Town River Improvements				
Site Preparation	1	JOB	LS	25,000
Water Control	1	JOB	LS	50,000
Traffic Control	1	JOB	LS	75,000
Demolition	1	JOB	LS	60,000
Support of Utilities	1	JOB	LS	25,000
Excavation	24,000	CY	6.00	144,000
Sheeting (left in place)	23,600	SF	22.00	519,200
Sheeting (removed)	3,800	SF	10.00	38,000
Screened Gravel	650	CY	15.00	9,750
Compacted Fill	4,500	CY	10.00	45,000
Gravel Base	4,000	CY	12.00	48,000
Gravel Bedding	4,500	CY	12.00	54,000
Filter Fabric	15,000	SY	1.50	22,500
Gabions	3,000	CY	106.00	318,000
Reinforcement Steel	60,000	lb.	.50	30,000
Tremie Concrete	4,000	CY	125.00	500,000
Structural Concrete	400	CY	250.00	100,000
Precast Concrete Culverts	280	LF	1,600.00	448,000
Bituminous Concrete	2,000	Ton	45.00	90,000
Granite Curb	1,400	LF	18.00	25,200
Fences, Guard Rails, Lighting	1	JOB	LS	40,000
				$2,666,650
			Contingency	263,000
			Total	$2,929,650

TABLE J-3
RELIEF TUNNEL, SHAFTS, INLET AND OUTLET WORKS
FROM GDM COST TABLE C-4
DRILL AND BLAST TUNNEL EXCAVATION

Table C-4 (Cont.)

Description	Estimated Quantity	Unit	Unit Prices($)	Estimated Amount($)
15.1 Relief Tunnel				
Mobilization	1	JOB	LS	1,000,000
General Excavation	12,000	CY	5.40	64,800
Intake and Outlet Shafts	1	JOB	LS	3,500,000
Sheeting	4,000	SF	15.00	60,000
Screened Gravel	5,000	CY	12.00	60,000
Pervious Fills	5,500	CY	12.00	66,000
Compacted Backfill	5,000	CY	12.00	60,000
Gravel Base Course	1,100	CY	12.00	13,200
Bituminous Concrete	265	Tons	35.00	9,275
Reinforcement Steel	562,400	lbs.	.50	281,200
Waterstops	900	LF	21.00	18,900
Structural Concrete	4,500	CY	188.00	846,000
Concrete-Shaft Lining	600	CY	260.00	156,000
Tunnel Excavation	4,052	LF	1,860.00	7,536,720
Rock Bolts	7,000	ea	120.00	840,000
Rock Pins	4,900	ea	30.00	147,000
Wire Mesh	90,000	SF	1.50	135,000
Steel Ring Beams	300,000	lb.	1.10	330,000
Wood Lagging and Blocking	3,500	CF	10.00	35,000
Tunnel Lining Concrete	6,130	CY	400.00	2,452,000
Drilling Grout Holes	6,600	LF	25.00	165,000
Drilling Exploratory Holes	600	LF	50.00	30,000
Pipe & Fittings for Grouting	6,100	lb.	5.00	30,500
Portland Cement in Grout	6,600	CF	12.00	79,200
Mineral Filler in Grout	6,600	CF	12.00	66,000
Sand in Grout	6,600	CF	10.00	16,500
Fluidifier in Grout	5,500	lbs.	3.00	16,500
Connections to Grout Holes	220	ea	10.00	2,200
Shotcrete, Mobilization	1	JOB	LS	100,000
Shotcrete, Applicatoin	2,400	SY	30.00	72,000
Miscellanous Metals & Stoplogs	1	JOB	LS	152,000
Topsoil & Seeding	6,000	SY	1.65	9,900
				$18,413,595
		Contingencies		1,856,405
	Total Relief Tunnel			$20,270,000

TABLE J-4 DETAILED COST ESTIMATE
RELIEF TUNNEL, SHAFTS, INLET AND OUTLET WORKS
DRILL AND BLAST TUNNEL EXCAVATION
(OCT 1985 PRICE LEVEL)

Description	Estimated Quantity	Unit	Unit Prices ($)	Estimated Amount ($)
15.1 Relief Tunnel				
Mobilization	1	JOB	LS	3,000,000
Tunnel Excavation	4,052	LF	1,860.00	7,536,720
Rock Bolts	6,000	ea	200.00	1,200,000
Rock Pins	4,200	ea	40.00	168,000
Wire Mesh	85,000	SF	1.50	127,500
Steel Ring Beams	300,000	lb.	1.20	360,000
Wood Lagging and Blocking	3,500	CF	10.00	35,000
Tunnel Lining Concrete	10,530	CY	300.00	3,159,000
Reinforcement Steel	45,000	lb.	.50	22,500
Waterstops	5,800	LF	21.00	121,800
Drilling Grout Holes	6,600	LF	25.00	165,000
Drilling Exploratory Holes	600	LF	50.00	30,000
Pipe and Fittings for Grouting	6,200	lb.	5.00	31,000
Portland Cement in Grout	6,600	CF	12.00	79,200
Mineral Filler in Grout	6,600	CF	12.00	79,200
Sand in Grout	6,600	CF	10.00	66,000
Fluidifier in Grout	5,500	lb.	3.00	16,500
Connections to Grout Holes	440	ea	10.00	4,400
Shotcrete, Mobilization	1	JOB	LS	100,000
Shotcrete, Application	2,600	SY	30.00	78,000
Tunnel Instrumentation	1	JOB	LS	350,000
Geologic Mapping	1	JOB	LS	120,000
Aeration System	1	JOB	LS	434,000
Intake Shaft	1	JOB	LS	557,000
Outlet Shaft	1	JOB	LS	1,209,000
Inlet and Outlet Works				
General Excavation	50,000	CY	6.00	300,000
Gabions	1,800	CY	106.00	190,800
Filter Fabric	5,000	SY	1.50	7,500
Sheeting	8,000	SF	22.00	176,000
Screened Gravel	2,500	CY	15.00	37,500
Pervious Fills	7,500	CY	12.00	90,000
Compacted Backfill	3,000	CY	12.00	36,000
Gravel Base Course	1,100	CY	12.00	13,200
Gravel Bedding	500	CY	12.00	6,000
Bituminous Concrete	265	Tons	45.00	11,925
Reinforcement Steel	450,000	lb.	.50	225,000
Waterstops	900	LF	21.00	18,900
Structural Concrete	3,000	CY	250.00	750,000
Topsoil & Seeding	6,000	SY	3.00	18,000
Miscellaneous Metal & Stoplogs	1	JOB	LS	165,000
				21,095,645
			Contingency	2,110,000
			Total	$23,205,645

TABLE J-5 DETAILED COST ESTIMATE
RELIEF TUNNEL, SHAFTS, INLET AND OUTLET WORKS
TBM EXCAVATION, CAST-IN-PLACE LINING
(OCT 1985 PRICE LEVEL)

Description	Estimated Quantity	Unit	Unit Prices ($)	Estimated Amount ($)
Relief Tunnel				
Mobilization	1	JOB	LS	5,000,000
Tunnel Excavation	4,052	LF	1,600.00	6,483,200
Rock Bolts	7,000	ea	200.00	1,400,000
Rock Pins	4,900	ea	40.00	196,000
Wire Mesh	75,000	SF	1.50	112,500
Steel Ring Beams	66,000	lb.	1.20	79,200
Wood Lagging and Blocking	1,000	CF	10.00	10,000
Tunnel Lining Concrete	6,440	CY	300.00	1,932,000
Reinforcement Steel	45,000	lb.	.50	22,500
Waterstops	5,800	LF	21.00	121,800
Drilling Grout Holes	6,600	LF	25.00	165,000
Drilling Exploratory Holes	600	LF	50.00	30,000
Pipe and Fittings for Grouting	6,200	lb.	5.00	31,000
Portland Cement in Grout	500	CF	12.00	6,000
Mineral Filler in Grout	500	CF	12.00	6,000
Sand in Grout	500	CF	10.00	5,000
Fluidifier in Grout	880	lb.	3.00	2,640
Connections to Grout Holes	440	ea	10.00	4,400
Shotcrete, Mobilization	1	JOB	LS	100,000
Shotcrete, Application	2,300	SY	30.00	69,000
Tunnel Instrumentation	1	JOB	LS	350,000
Geologic Mapping	1	JOB	LS	120,000
Aeration System	1	JOB	LS	434,000
Intake Shaft	1	JOB	LS	586,000
Outlet Shaft	1	JOB	LS	1,209,000
Inlet and Outlet Works				
General Excavation	50,000	CY	6.00	300,000
Gabions	1,800	CY	106.00	190,800
Filter Fabric	5,000	SY	1.50	7,500
Sheeting	8,000	SF	22.00	176,000
Screened Gravel	2,500	CY	15.00	37,500
Pervious Fills	7,500	CY	12.00	90,000
Compacted Backfill	3,000	CY	12.00	36,000
Gravel Base Course	1,100	CY	12.00	13,200
Gravel Bedding	500	CY	12.00	6,000
Bituminous Concrete	265	Tons	45.00	11,925
Reinforcement Steel	450,000	lb.	.50	225,000
Waterstops	900	LF	21.00	18,900
Structural Concrete	3,000	CY	250.00	750,000
Topsoil & Seeding	6,000	SY	3.00	18,000
Miscellaneous Metals & Stoplogs	1	JOB	LS	165,000
				20,521,065
			Contingency	2,052,000
			Total	$22,573,065

TABLE J-6 DETAILED COST ESTIMATE
RELIEF TUNNEL, SHAFTS, INLET AND OUTLET WORKS
TBM EXCAVATION, PRECAST LINING
(OCT 1985 PRICE LEVEL)

Description	Estimated Quantity	Unit	Unit Prices ($)	Estimated Amount ($)
Relief Tunnel				
Mobilization	1	JOB	LS	5,000,000
Tunnel Excavation	4,052	LF	1,600.00	6,483,200
Rock Bolts	400	ea	200.00	80,000
Wire Mesh	15,000	SF	1.50	22,500
Steel Ring Beams	66,000	lb.	1.20	79,200
Wood Lagging and Blocking	1,000	CF	10.00	10,000
Precast Lining Concrete	4,740	CY	215.00	1,019,100
Reinforcement Steel	508,000	lb.	.50	254,000
PeaGravel (Liner)	1,150	CY	25.00	28,750
Grout (Liner)	10,250	CF	17.50	179,375
Drilling Grout Holes	6,600	LF	25.00	165,000
Drilling Exploratory Holes	600	LF	50.00	30,000
Pipe and Fittings for Grouting	30,000	lb.	5.00	150,000
Portland Cement in Grout	500	CF	12.00	6,000
Mineral Filler in Grout	500	CF	12.00	6,000
Sand in Grout	500	CF	10.00	5,000
Fluidifier in Grout	880	lb.	3.00	2,640
Connections to Grout Holes	5,300	ea	10.00	53,000
Shotcrete, Mobilization	1	JOB	LS	100,000
Shotcrete, Application	500	SY	30.00	15,000
Tunnel Instrumentation	1	JOB	LS	350,000
Geologic Mapping	1	JOB	LS	120,000
Aeration System	1	JOB	LS	434,000
Intake Shaft	1	JOB	LS	586,000
Outlet Shaft	1	JOB	LS	1,209,000
Inlet and Outlet Works				
General Excavation	50,000	CY	6.00	300,000
Gabions	1,800	CY	106.00	190,800
Filter Fabric	5,000	SY	1.50	7,500
Sheeting	8,000	SF	22.00	176,000
Screened Gravel	2,500	CY	15.00	37,500
Pervious Fills	7,500	CY	12.00	90,000
Compacted Backfill	3,000	CY	12.00	36,000
Gravel Base Course	1,100	CY	12.00	13,200
Gravel Bedding	500	CY	12.00	6,000
Bituminous Concrete	265	Tons	45.00	11,925
Reinforcement Steel	450,000	lb.	.50	225,000
Waterstops	900	LF	21.00	18,900
Structural Concrete	3,000	CY	250.00	750,000
Topsoil & Seeding	6,000	SY	3.00	18,000
Miscellaneous Metals & Stoplogs	1	JOB	LS	165,000
				18,433,590
			Contingency	1,843,000
			Total	$20,276,590